

Jackson Crossing

Alexandria, Virginia



Final Report

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Structural Option

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Building Statistics

Building Height: 54' 7 1/4"
Number of Floors: 5
Gross Square Foot: 107,740 sq. ft.
Type of Building: Multi-Family Residential
Total Project Cost: \$16 Million
Construction Dates: 4/4/2014-12/17/2015

Project Team

Owner: AHC, Inc.
Construction Management: Harkins Builders, Inc.
Architect: Bonstra | Haresign Architects, LLP
Civil Engineer: VIKI, Virginia, LLC
Structural Engineer: Rathgeber Goss Associates
MEP Engineer: Metropolitan Engineering, Inc.
Landscape Architect: Landscape Architectural Bureau
Specifications Cons.: Bethel Specifications Consulting

Mechanical

- All apartment units have operable windows
- Typical floor houses a mounted vertical heat pump (DX Split System) and is provided with vibration isolation
- Roof houses condensing units
- Upper garage exhausts 12,000 CFM of air and supplies 17,250 CFM of air
- Lower Garage exhausts 5250 CFM of air



Electrical

- Dominion Virginia Power Service supplies power into one pad mounted transformer
- 2 1600A, 208/120V Feeders run from the transformer
- All units are individually metered

Structural System

Gravity System

- 18" deep wood trusses spaced at 24" o.c.
- Wood bearing walls
- 12" Reinforced two-way concrete slab
- 24"x16" Concrete columns typical

Lateral System

- Reinforced Concrete Shear Wall
- Reinforced Masonry Shear Wall
- Wood Shear Wall

Thesis Advisor: Linda M. Hanagan, PhD, P.E.
Website: <http://www.engr.psu.edu/ae/thesis/portfolios/2016/mab6150/index.htm>

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Executive Summary

Jackson Crossing is a low income residential apartment building in Alexandria, Virginia. The project site is located just south of Washington, D.C. directly adjacent to a highway, Jefferson Davis Highway. The highway divides the area surrounding the site into a commercial zone on one side and a residential zone on the Jackson Crossing side. The building is five stories above grade with a lower and upper garage level below grade.

For this report, a redesign will be considered to replace the existing wood framed structure on a concrete podium. The redesign will be a concrete system with two-way reinforced flat plate slabs supported by concrete columns along with concrete moment frames to resist lateral forces.

Relating to the gravity system, a column dimension of 14" by 14" was found to be adequate based off the compressive strength of the section. Two columns were designed; one for columns with factored axial loads above 260k and one for columns with factored axial loads below 260k. The two-way slabs were initially designed with a 9" thickness but were ultimately detailed as 8" thick. This thickness was determined to provide a reasonable amount of reinforcement along with meeting deflection requirements.

The lateral system was redesigned as moment frames around the exterior of the structure. Eccentricities between the Center of Rigidity and Center of Mass were minimum with the frame layout. Both the columns and beams in the frames were designed with a 16" by 24" section to provide a greater moment of inertia in the direction of loading. Two separate frame reinforcement details were designed; one for the north-south direction and one for the east-west direction. This is because the north-south direction frames experience larger forces due to their shorter lengths.

A construction management study was done to determine the redesigned structure's impact on the project's cost and schedule. From this breadth, it was determined that the redesigned increased the structure cost by 22% and increased the duration along the critical path by 24 days.

The mechanical breadth focused on the acoustical performance of the redesigned structure. A concrete slab was found to have an adequate STC rating along with an IIC rating assuming a carpet and pad is installed.

A two-way reinforced concrete slab system is a viable alternative for Jackson Crossing. While it adds cost and time to the construction, the redesigned structure is durable and has a smaller floor depth compared to the existing wood framed structure.

Acknowledgments

For all of their help and support this semester, I would like to thank the following people:

- All of the engineers at Rathgeber and Goss Associates for their guidance and for allowing me to use Jackson Crossing as a thesis project. I would especially like thank Justin Domire for the help and advice he gave me throughout this project.
- The owners at AHC, Inc. for providing me permission for this project.
- Harkins Builders and especially Russell Tipton for providing information necessary for my construction breadth.
- My classmates at Penn State along with my friends and family at home.
- Professors in the AE Department, especially my advisor Dr. Linda Hanagan.

1 Introduction

1.1 General Building Information

Jackson Crossing is a five story above grade residential apartment for low income residents in Alexandria, Virginia. The apartment complex will include a community space for educational programing and an underground parking garage for residents of the building.

The location of Jackson Crossing’s site is just south of Washington D.C. as Figure 1 depicts. On a micro-level, the project site is accessed by highway (Jefferson Davis Hwy) directly adjacent to Jackson Crossing. From Figure 2, there is both single family homes and multifamily apartments surround Jackson Crossing on one side of the highway and commercial developments on the opposite side.



Figure 1 (Courtesy of Google Maps) – Location of Jackson Crossing in relation to Washington D.C.

The following is Jackson Crossing’s project team:

Owner:	AHC, Inc.	CM:	Harkins Builders, Inc.	Architect:	Bonstra Haresign Architects, LLP
Civil:	VIKA, Virginia, LLC	Structural:	Rathgeber/Goss Associates	MEP:	Metropolitan Engineering, Inc.



Project Site | Highway(Jefferson Davis Hwy) | Residential | Commerical

Figure 2 (Courtesy of Google Maps) – Micro site of Jackson Crossing with the surround area types

The site of Jackson Crossing is gently sloped in elevation with the highest point at the east end of the building as in Figure 3. From Figure 4, the first level below ground is the lower garage level and from this level a ramp elevates cars up to the next garage level. The first residential level starts at the first floor with additional residential floors continuing up to the fifth floor. The height of the building from the average grade to the top of the parapet is 55' 5".



Figure 3 - North Elevation of Jackson Crossing

Figure 5 is a typical floor plan of a residential level along with the major dimensions of the building. Jackson Crossing is much longer in the east-west direction than in the north-south direction. Stairwells and elevators are highlighted in green at either end and provide vertical circulation.

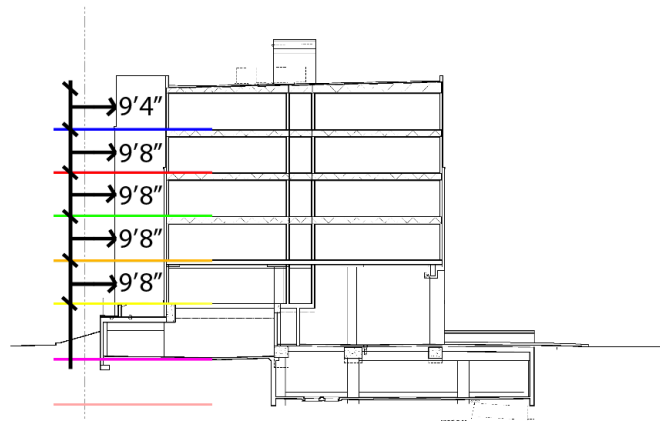


Figure 4 - Building Section with Floor to Floor Heights

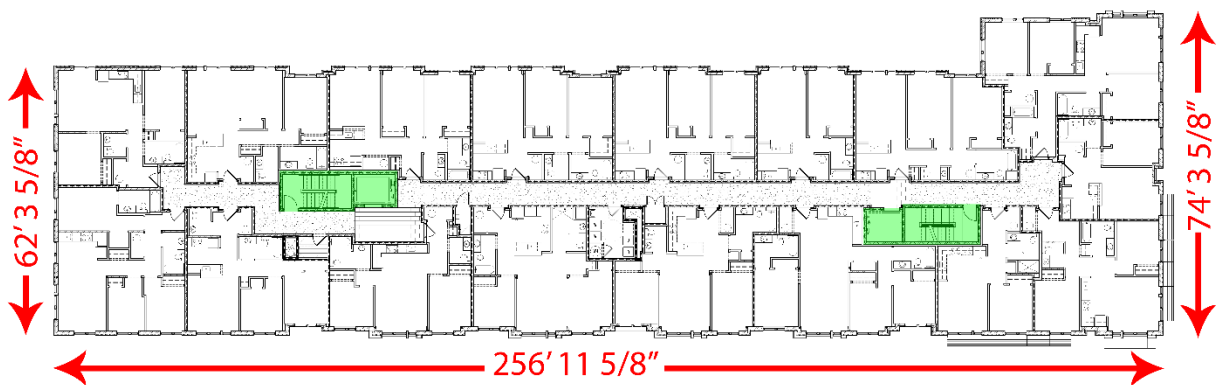


Figure 5 - Typical Residential Floor Plan

1.2 Existing Structural System

1.2.1 Existing Gravity System

In broad terms, the gravity system of Jackson Crossing is wood framing on a concrete podium. The concrete slab or podium acts as a transfer slab by directing the load from the four wood frame levels to the columns supporting the slab then eventually the foundation. In Figure 4 the concrete podium is at the second floor while the wood framing is at the third, fourth, fifth, and roof level.

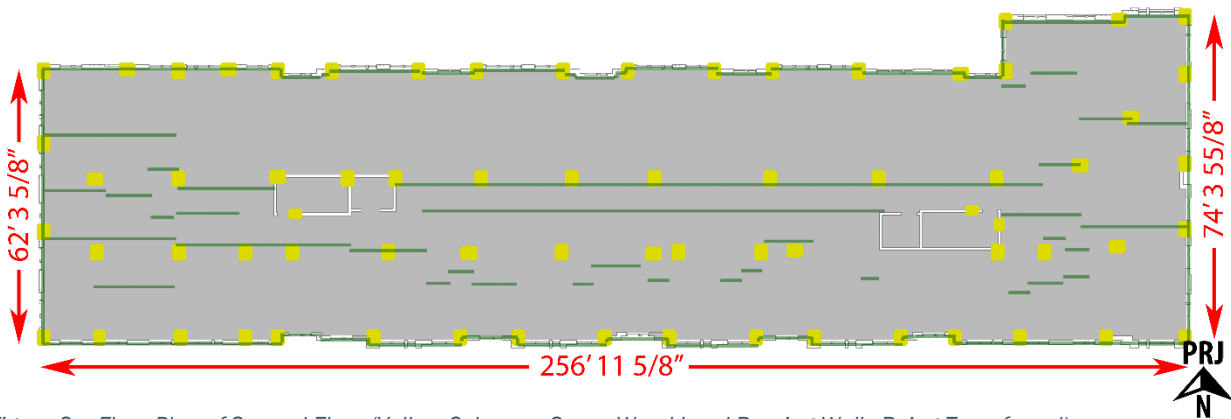


Figure 6 – Floor Plan of Second Floor (Yellow-Columns, Green-Wood Load Bearing Walls Being Transferred)

The structure at the second floor is detailed in Figure 4. The load from the four wood frame levels is carried to the second floor slab through wood load bearing walls highlighted green in Figure 4. The concrete podium was designed as a two-way slab and is supported by columns highlighted in yellow. The transfer forces caused by the wood bearing walls required the slab's depth to be twelve inches.

Loads are again transferred at the first level as in Figure 7 (on the next page). Columns colored in yellow transfer loads from the green columns while the purple columns support both the first and second floor. The slab at the first floor was designed as a one way slab with beams. These beams are colored in blue in Figure 7. The slab's depth is also twelve inches as with the second floor.

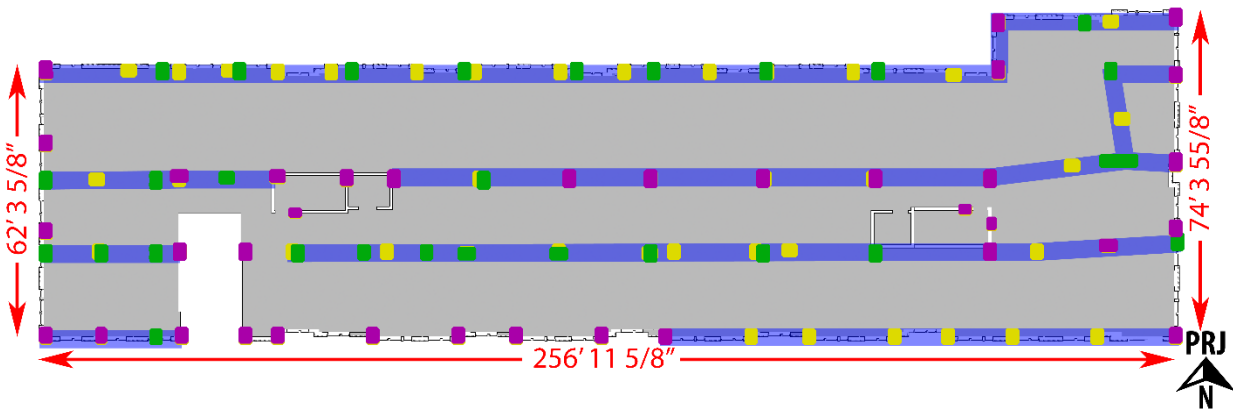


Figure 7 – Floor Plan of First Level (Yellow-Supporting First Floor, Green-Supporting Second Floor, Purple-Supporting First and Second Floor, Blue-Concrete Beams)

1.2.2 Existing Lateral System

The lateral system of Jackson Crossing consists entirely of shear walls. Figure 8 is an overview of the lateral system. The shear walls up to the second floor transfer slab are reinforced concrete walls located around the elevator core. Masonry shear walls rest on the concrete walls at the second floor and extend up through the height of the building. Masonry shear walls are also located around the stairwell corridors.

In addition to the masonry shear walls above the second floor, wood shear walls also contribute to the lateral resistance but only in the north and south direction.

These wood shear walls are anchored into the transfer slab at the second floor. The locations of the wood shear walls are outlined on top of a typical floor plan in Figure 9 (on the next page).

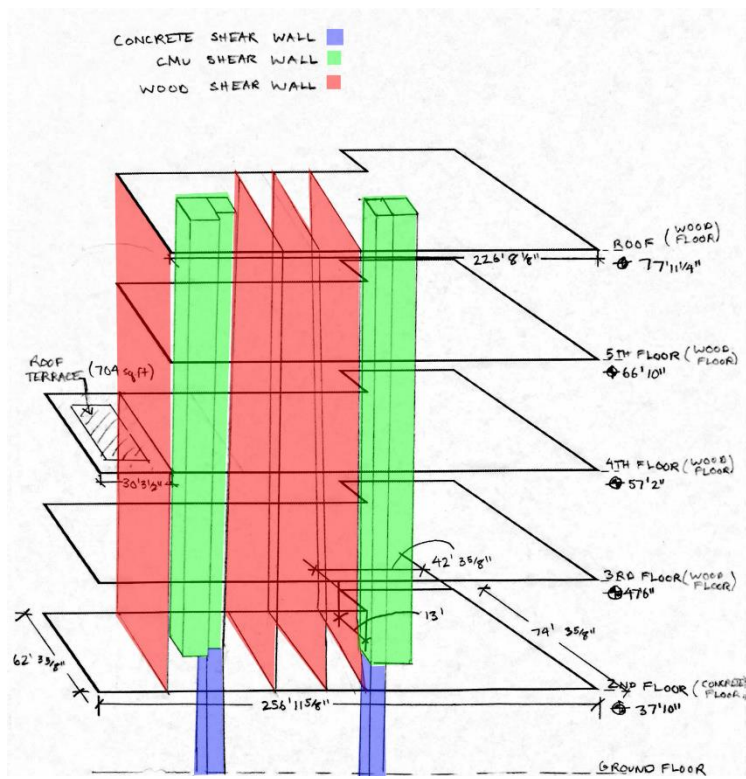


Figure 8 – Overview of Jackson Crossing's Lateral System

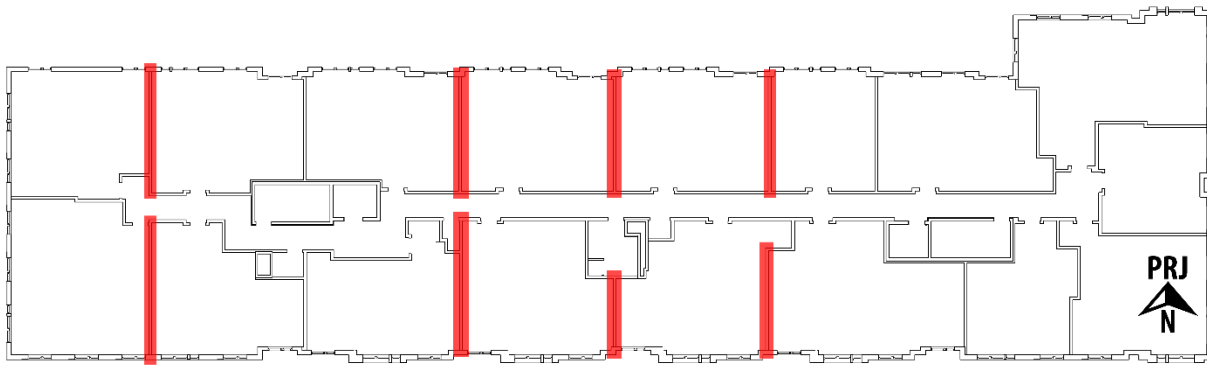


Figure 9 – Location of Wood Shear Walls on a Typical Floor Plan

1.3 M.A.E. Requirements

To incorporate graduate level coursework, the alternative design will include skills and concepts learned from classes in the Integrated B.A.E./M.A.E. program. The modeling of the lateral system will require knowledge in lateral frame design acquired from AE 530, Computer Modeling of Building Structures. The design of concrete moment frames will use lessons learned in AE 538, Earthquake Resistant Design of Buildings, including the code requirements for the reinforcing of moment frame members.

2 Redesigned Structure

The purpose for the redesigned structure is to explore the possibility of converting Jackson Crossing to a two-way flat plate system and a reinforced concrete moment frame. This system should allow a smaller structural floor depth while being more durable.

Throughout this report, sections of code will be referenced; these code texts will include:

- ACI 318-11: Building Code Requirements for Structural Concrete and Commentary
- ASCE 7-10: Minimum Design Loads for Buildings and Other Structures
- IBC 2009: International Building Code

2.1 Design Loads

2.1.1 Dead Load

The following section will focus on the superimposed dead load that will be considered for the redesigned structure. The self-weight of the structure will be considered in the computer modelling of the slab and as appropriate in hand calculations. Values for weights are referenced from Table 17-13 of the *Steel Construction Manual* and Appendix B of *Design of Wood Structures*.

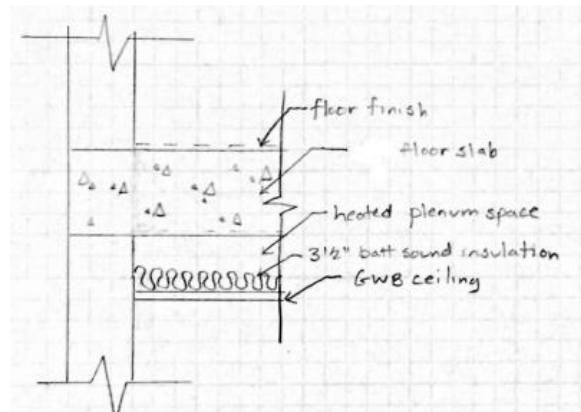


Figure 10 – Section Detail of Typical Residential Level

2.1.1.1 Typical Residential Level

For a typical residential level the weight of a floor finish, sound insulation, and GWB ceiling was considered. In addition the weight of unmovable wood stud partitions was included. Figure 10 is a section detail of a typical residential level while Table 1 is a summary of the individual weights. From Table 1, a conservative superimposed dead load of 22 psf will be used in the redesign.

Table 1 – Summary of Weights for Typical Residential Level

	Weight (psf)
Floor Finish	1
Batt Insulation	1.75
GWB Ceiling	2.5
Partitions	12.5
Mechanical Allowance	4
Total SDL (psf):	21.75

2.1.1.2 Roof Level

At the roof level, the superimposed dead load will include the weight of a single ply roof membrane, rigid insulation, wood subfloor, sound insulation, and a GWB ceiling. The summary of these weights are listed in Table 2. From Table 2, a conservative superimposed dead load of 16 psf will be used for the roof level.

Table 2 – Summary of Weights for Roof Level

	Weight (psf)
Single Ply Roof Membrane	1
Rigid Insulation	3
Batt Insulation	1.75
GWB Ceiling	2.5
Subfloor	2.875
Mechanical Allowance	4
<hr/>	
Total SDL (psf):	15.125

2.1.1.3 Exterior Wall

Figure 11 is a section detail of the exterior wall while Table 3 is a summary of the individual weights. From Table 3, conservative weight of 56 psf will be used for the exterior wall.

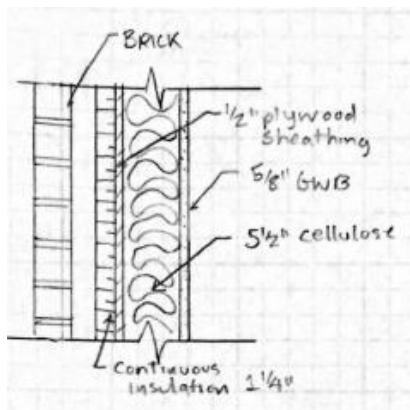


Figure 11 – Section Detail of Exterior Wall

Table 3 – Summary of Loads on Exterior Wall

	Weight (psf)
4" Brick	40
Plywood	1.5
Continuous Insulation	1.88
GWB	2.5
Cellulose Insulation	10
<hr/>	
Total SDL (psf):	55.88

2.1.2 Live Load

Live loads from the IBC-2009 are listed in Table 4. Live Load Reduction per IBC-2009 Section 1607.9 will be applied where appropriate.

Table 4 – Summary of Live Loads

Area	Load
Living Units	40 PSF
Lobbies/Stairs/Exits	100 PSF
Corridors Above 1st Floor	40 PSF
Parking Decks	40 PSF
Roof Terrace	100 PSF

2.1.3 Non-Typical Loads

2.1.3.1 Roof Terrace

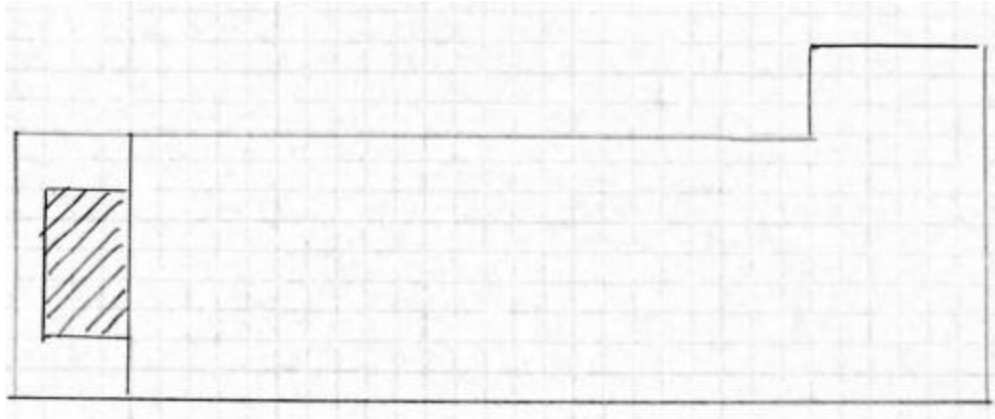


Figure 12 - Location of Roof Terrace on Floor 4

A roof terrace is located on the fourth floor as in Figure 12. A higher superimposed dead load and live load must be considered for this area. Figure 13 is a section detail of the roof terrace while Table 5 summarizes the weights. From Table 5, a conservative superimposed dead load of 40 psf will be considered.

In addition a live load of 100 psf will be considered for the roof terrace area as required by IBC 2009.

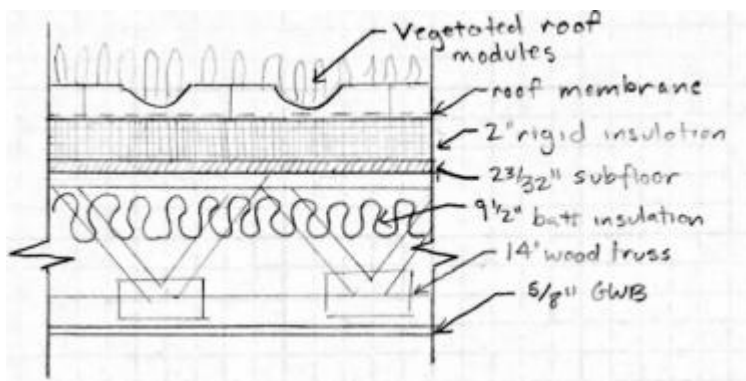


Figure 13 - Section Detail of Roof Terrace

Table 5 - Summary of Loads for Roof Terrace

	Weight (psf)
Roof Membrane	1
Rigid Insulation	3
Subfloor	3
Batt Insulation	1.75
GWB Ceiling	2.5
Vegetated Modules	20
Misc	5
<hr/>	
Total SDL (psf):	36.25

2.1.3.2 Mechanical Equipment on Roof

The weight of the mechanical equipment on the roof is summarized in Figure 14. The equipment on the roof includes condensers, fans, and roof top units.

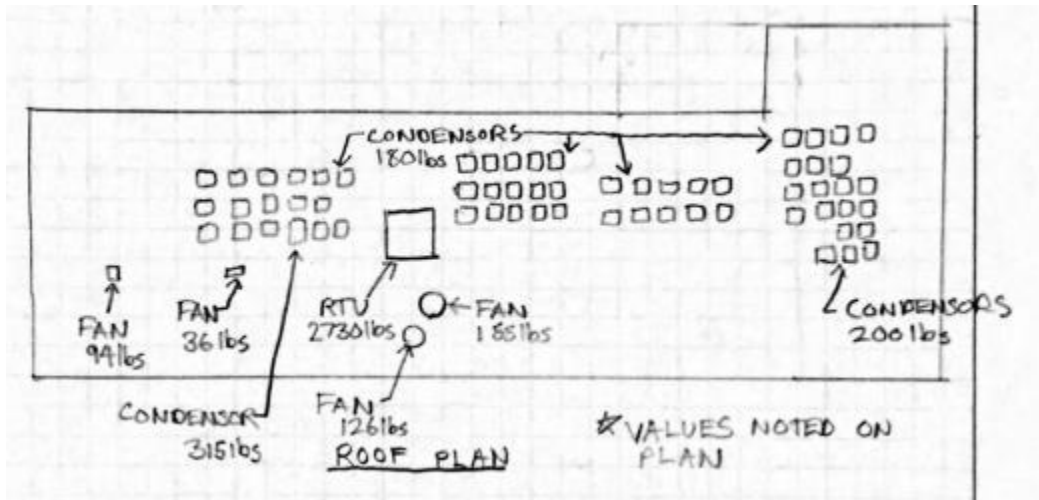


Figure 14 - Non Typical Loads on Roof

2.1.4 Snow Load

Snow load on the roof was determined using section 7 of ASCE 7-10. The ground snow load, p_g , was found in Figure 7-1 from Chapter 7. The different directions of snow drift considered are detailed in Figure 15. Both leeward and windward snow drift was considered around the stairwell and elevator core walls that extend beyond the roof. Only the maximum pressure from the leeward or windward drift was considered. Table 6 lists the design drift values.

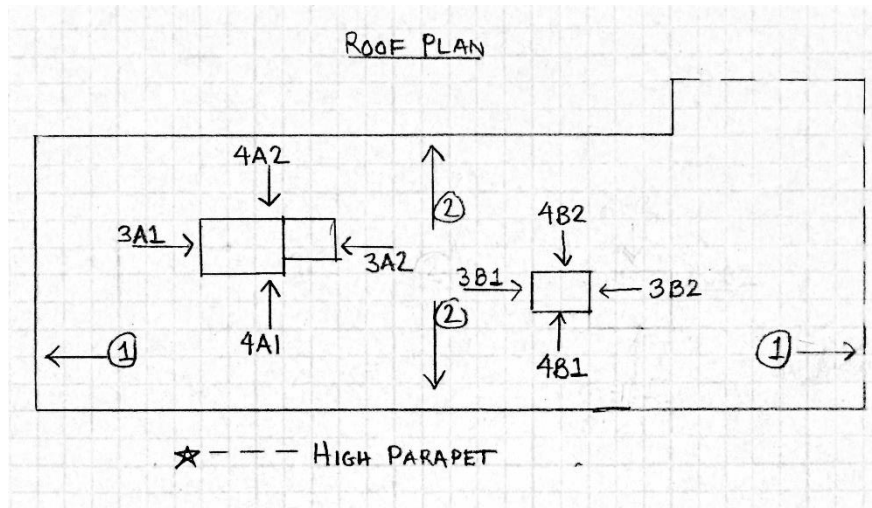


Figure 15 - Snow Drift Directions Considered for Design

Table 6 – Snow Drift Values

	p_g (psf)	25.0	Fig. 7-1	
	p_f (psf)	17.5	(7.3-1)	
	γ	17.3	Eq. 7.7-1	
	h_b (ft)	1.0		
High Parapet				
	h_c (ft)	1.05		
	h_c/h_b	1.04	>0.2	
*Drift Required (Sec. 7.7.1)				
Low Parapet				
	h_c (ft)	0.38		
	h_c/h_b	0.37	>0.2	
*Drift Required (Sec. 7.7.1)				
Drift Calculations				
Windward				
Direction	l_u (ft)	h_d (ft)	w (ft)	p_d (psf)
1	226.7	3.7	16.5	18.1
2	82.3	2.3	10.2	18.1
4A1	27.5	1.2	5.0	21.5
4A2	23.6	1.1	4.5	19.4
4B2	32.0	1.4	5.5	23.6
3A1	51.6	1.8	7.2	31.1
3A2	174.5	3.3	13.1	56.2
3B1	184.7	3.3	13.4	57.7
3B2	59.0	1.9	7.7	33.3
Leeward				
4B1	20.0	1.3	5.4	23.1

2.2 Gravity System

2.2.1 Gravity Columns

The general process for designing the gravity columns will be as follows:

- Determine initial assumed column dimension from ϕP_n (ACI Eq. 10-2)
- Model gravity columns and slabs in ETABS
- Run analysis to output column forces and then group into design sections
- Design reinforcement

The locations of the columns that will be designed to support the two-way flat plate slabs are overlaid on a floor plan in Figure 16 and Figure 17. Figure 16 is a floor plan corresponding to floors two, three, and four. Some of the columns terminate at the fourth floor as the fifth floor and roof slab is recessed in from the east side. Figure 17 is a floor plan of the fifth floor and roof that labels the columns that continue to the roof. All of the column labels in Figure 16 and Figure 17 will be referenced throughout this report.

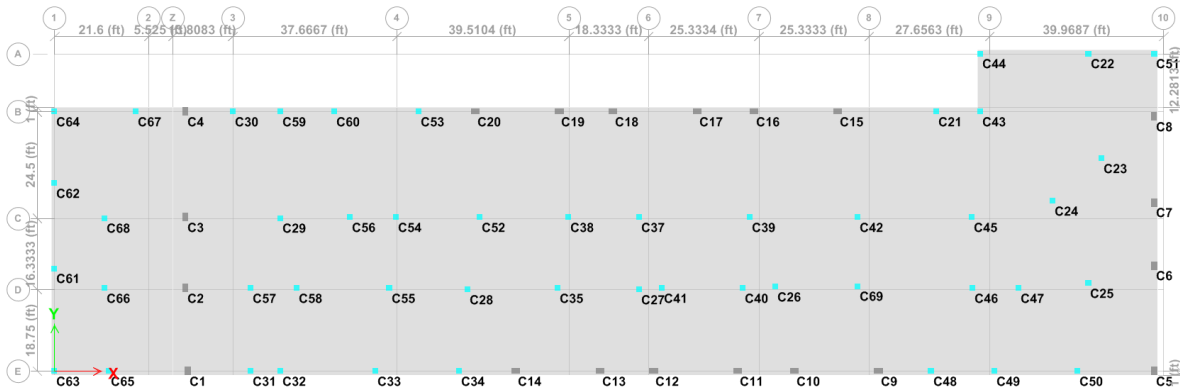


Figure 16 – Column Locations for Second, Third, and Fourth Floor

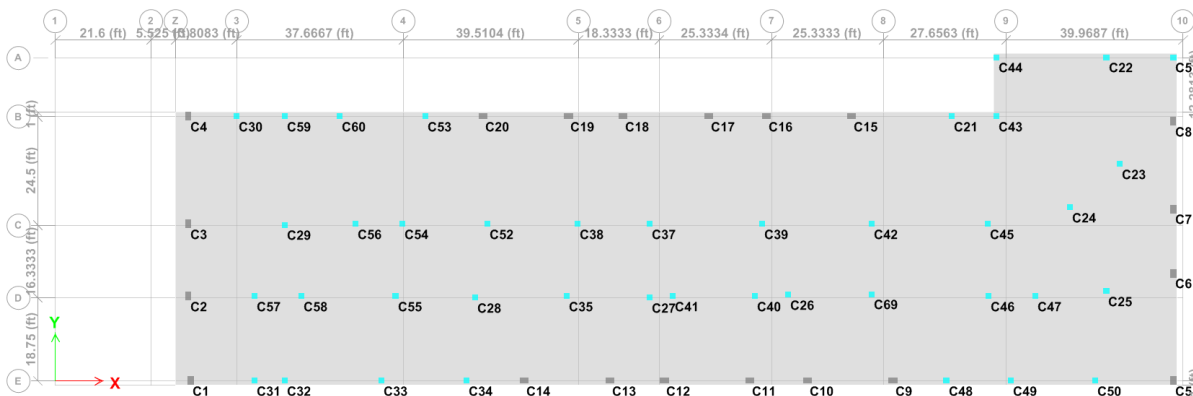


Figure 17 – Columns Locations for Fifth Floor and Roof

2.2.1.1 Initial Column Dimension

An initial column dimension will be determined by estimating the maximum axial load based of the column with the largest tributary area. The column that will be analyzed is column C42; this column had a tributary area of 512.82 ft². Figure 18 is a blown up plan of column C42 with its surrounding dimensions.

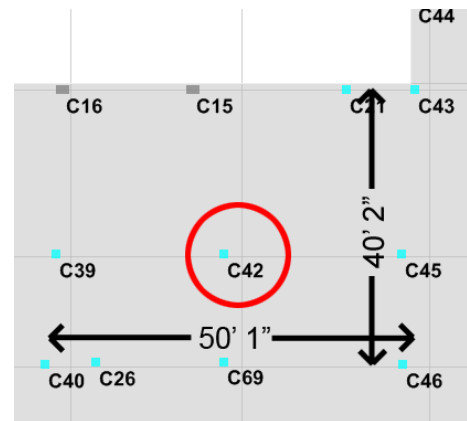


Figure 18 – Location of C42 and its surrounding dimensions

Table 7 summarizes a load takedown of column C42 to estimate its maximum axial load. A size of 16” by 16” was used to estimate the self-weight of the column. In addition the controlling load combination from the IBC was assumed to be 1.2D+1.6L+0.5S for this initial sizing of the columns.

Table 7 – Load Takedown of Column C42

Tributary Area (ft ²):		512.82						
Floor	Snow (psf)	Dead (psf)	Live (psf)	Slab S.W. (psf)	Column S.W. (k)	Total Snow (k)	Total Dead (k)	Total Live (k)
Roof	18.1	16	0	100	3.0	9.3	62.4	0.0
Fifth	0	22	40	100	2.6	0.0	65.1	20.5
Fourth	0	22	40	100	2.6	0.0	65.1	20.5
Third	0	22	40	100	2.6	0.0	65.1	20.5
Second	0	22	40	100	3.9	0.0	66.4	20.5
Total Factored* Load (k):							525.1	
*1.2D+1.6L+0.5S								

According to ACI 10.3.6 the design axial strength of a compression member, ϕP_n , shall not be taken greater than $\phi P_{n,max}$. In the case of a nonprestressed member with tied reinforcement, $\phi P_{n,max}$ is given by Eq. 10-2:

$$\phi P_{n, \max} = 0.80\phi[0.85f'_c(A_g - A_{st}) + f_y A_{st}]$$

For this initial design, an $f'c$ of 5,000 psi and a f_y of 60,000 psi will be used along with a reinforcement ratio of 1.5%. By setting $\phi P_n, \max$ equal to the total factored load from Table 7, A_g can be solved for as shown below:

$$525.1 \text{ k} = 0.80 * 0.65 * [0.85 * 5000 \text{ psi} * (A_g - 0.015 * A_g) + 60000 \text{ psi} * 0.015 * A_g]$$

$$A_g = 198.5 \text{ in}^2$$

Given that A_g is equal to 198.5 in^2 from the above equation, a column size of 14" by 14" will be used. Although a column of this size has an A_g of 196 in^2 , this column dimension is reasonable given that the estimate of the maximum column load considered a self-weight given a 16" by 16" column and a low reinforcement ratio of 1.5%.

The slenderness of a 14" by 14" section is also important to consider for the given height of the columns. According to ACI Section 10.10 slenderness effects can be neglected if meeting the conditions in Section 10.10.1. If the gravity columns are considered braced against sideway due to the rigidity of the lateral moment frames, the columns must meet the requirements of Eq. 10-7:

$$\frac{kl_u}{r} \leq 34 - 12 \left(\frac{M_1}{M_2} \right) \leq 40$$

To be conservative, a k equal to 1 and M_1/M_2 equal to +0.5 will be considered. Given a column height of 9'8":

$$\frac{1 * 9'8''}{0.3 * 14''} \leq 34 - 12 * 0.5 \leq 40$$

$$27.62 \leq 28 \leq 40 \quad * \text{OK}$$

Given the above equation is valid, slenderness effects are not significant for the gravity columns and can be neglected.

2.2.1.2 Computer Modeling of Gravity Columns

For a more complete analysis all of the columns were modeled in ETABS 2015. The two-way slabs were modelled as a membrane and assigned a rigid diaphragm to brace the columns at each level. In addition the columns were assigned fixed joints at their ground floor base. As ETABS does not recognize a column to slab connection, fake beams were modeled to connect the columns as in Figure 19 (on the next page). The fake columns were given a small dimension and no moment of inertia as to ensure they do not take any force.

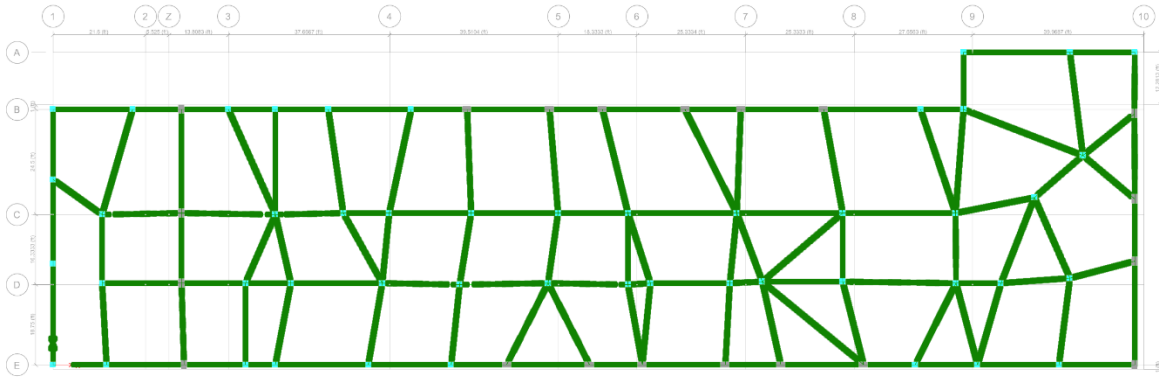


Figure 19 – Fake Beam Members (Green)

The following are the section properties given to the columns and slabs that were modelled:

Column:

Material: 5000Psi
 Depth: 14"
 Width: 14"

Fake Beam:

Depth: 5"
 Width: 5"
 Inertia Modifier: 0

Slab:

Material: 5000Psi
 Thickness: 8"

Dead, live, and snow patterns were defined along with load combinations for strength design as defined by the IBC. Uniform shell loads were applied to the slab sections for each load pattern with values from Section 2.1 of this report.

2.2.1.3 Computer Analysis Output for Gravity Columns

From the ETABS output, the worst case combinations for the columns was 1.2D+1.6L+0.5S. Column C42 had the largest axial force with a factored load of 524.70k. A complete list of forces on the gravity columns in Section A.1 of the Appendix. From this list, two design columns will be detailed from the forces of column C42 and column C47. The forces on these columns are in Table 8.

As a hand check of these forces, the factored axial load calculated for C42 from Table 7 was 525.1k. This force is less than 0.1% greater than the ETABS axial force.

Table 8 – Forces on Design Column Sections

Column	Load Combination	Axial	Shear		Moment	
		P(k)	V2 (k)	V3 (k)	M2 (k-ft)	M3 (k-ft)
Column 1 (C42)	1.2D+1.6L+0.5S	524.70	0.35	4.24	30.14	10.04
Column 2 (C47)	1.2D+1.6L+0.5S	259.96	0.05	0.31	1.33	10.60

2.2.1.4 Design of Reinforcement

The design of the longitudinal reinforcement for the two column sections was done with the aid of spColumn. Figures 20 and 21 are interaction diagrams. The columns were considered to have biaxial bending with no slenderness effects considered as was discussed in Section 2.2.1.1 of this report. In addition the columns were considered confined by #3 bars with a clear cover to the transverse reinforcement of 1.5".

Based off of the spColumn design, the longitudinal reinforcing for the columns is required to be as follows:

- Column 1: (4) #8 – A_s : 3.16in²
- Column 2: (4) #7 – A_s : 2.4in²

According to ACI Section 10.9.1, longitudinal reinforcement in a tied column shall be at least 0.01 but no more than 0.08 times the net area of a concrete section; in considering where longitudinal bars overlap the max of 0.08 is in reality 0.04. Both designed columns meet this requirement:

- Column 1: $1.93\text{in}^2 \leq 3.16\text{in}^2 \leq 7.72\text{in}^2$
- Column 2: $1.94\text{in}^2 \leq 2.4\text{in}^2 \leq 7.75\text{in}^2$

Note that the area of steel for Column 2 is the minimum allowable as a square column is required by ACI to have longitudinal bars in all four corners; therefore #7 bars are the smallest bars to meet this requirement and the minimum 1% reinforcement.

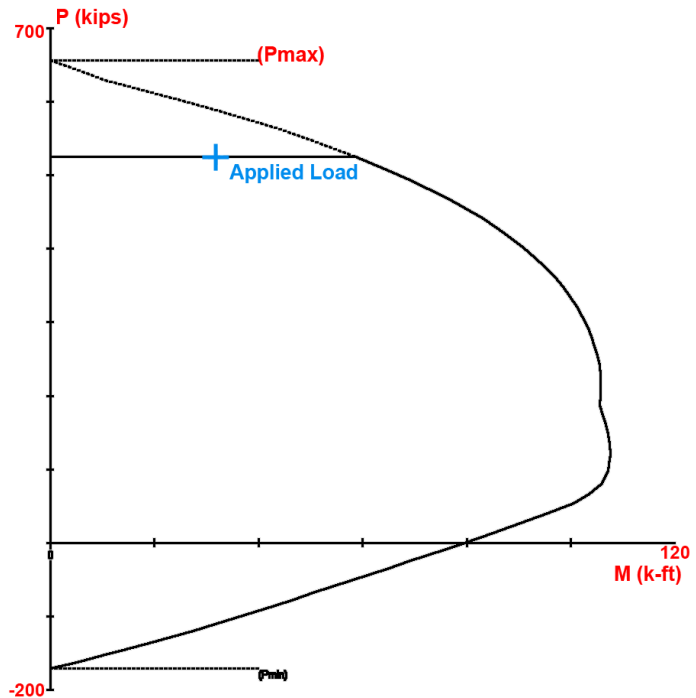


Figure 20 – Interaction Diagram of Column 1

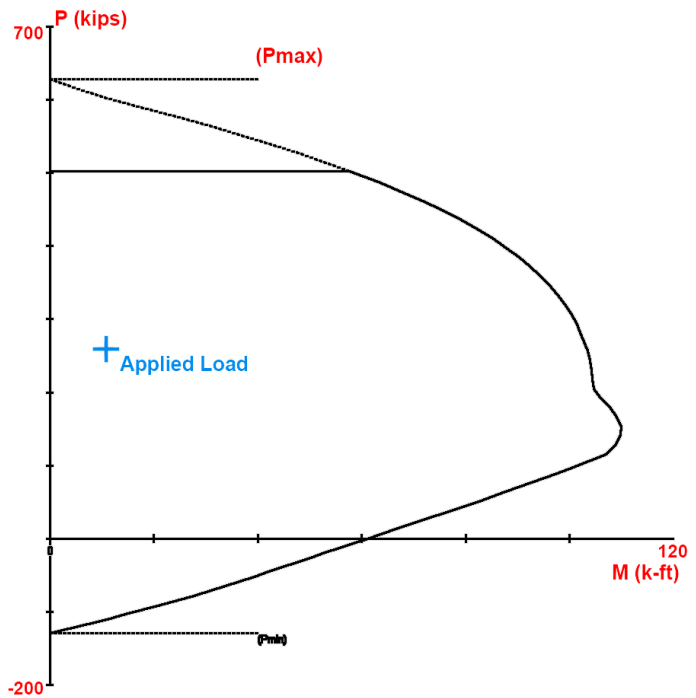


Figure 21 – Interaction Diagram of Column 2

Unless a more detailed calculation is made, the shear strength of the column or a member subjected to axial compression is given by ACI Eq. 11-4:

$$V_c = 2\left(1 + \frac{N_u}{2000A_g}\right)\lambda\sqrt{f'_c}b_wd$$

For Column 1 and Column 2 respectively ϕV_c is equal to:

$$\phi V_{c1} = 0.75\left[2\left(1 + \frac{524.7k * 1000}{2000 * 14" * 14"}\right) * 1 * \sqrt{5000psi} * 14" * 11.625"\right]$$

$$\phi V_{c2} = 0.75\left[2\left(1 + \frac{260.0k * 1000}{2000 * 14" * 14"}\right) * 1 * \sqrt{5000psi} * 14" * 11.688"\right]$$

$$\phi V_{c1} = 40.4k$$

$$\phi V_{c2} = 28.9k$$

None of the shear forces in Appendix A.1 are near $0.5\phi V_{c1}$ or $0.5\phi V_{c2}$; therefore, the spacing of the stirrups will be dictated by ACI Section 7.10.5.2. For Column 1 and Column 2 respectively the minimum stirrup spacing is:

$$s_{min1} = \min \begin{cases} 16 * \text{longitudinal bar diameters} = 16" \\ 48 * \text{tie bar diameters} = 18" \\ \text{least dimension of member} = 14" \end{cases} = 14"$$

$$s_{min2} = \min \begin{cases} 16 * \text{longitudinal bar diameters} = 14" \\ 48 * \text{tie bar diameters} = 18" \\ \text{least dimension of member} = 14" \end{cases} = 14"$$

The distance the first stirrup is placed away from the slab face is dictated by ACI Section 7.10.5.5. For both Column 1 and Column 2, the first stirrup will be placed at 6". Figure 22 and Figure 23 are detailed cross sections of Column 1 and Column 2 respectively. Figure 24 and Figure 25 are vertical details of Column 1 and Column 2 respectively.

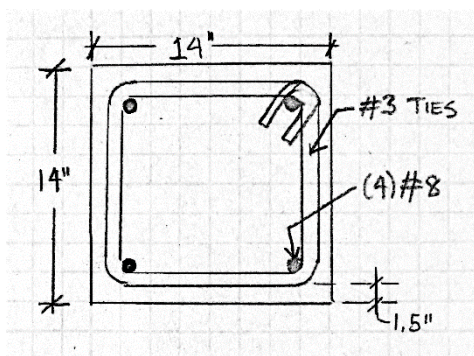


Figure 22 – Cross Section Detail of Column 1

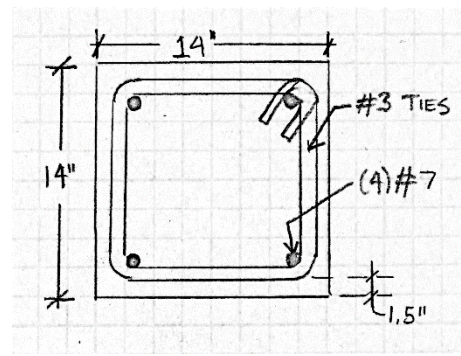


Figure 23 – Cross Section Detail of Column 2

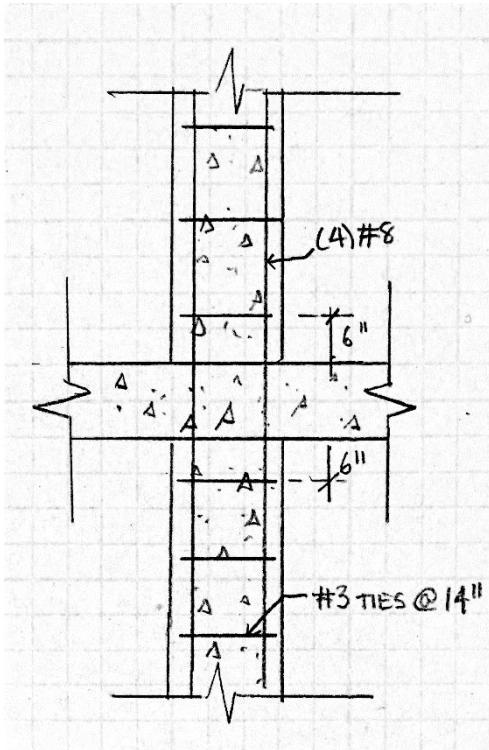


Figure 24 – Vertical Detail of Column 1

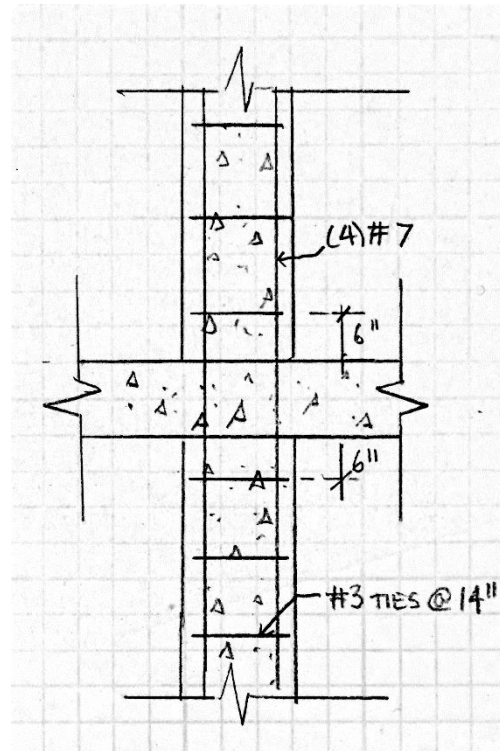


Figure 25 – Vertical Detail of Column 2

2.2.2 Two-Way Flat Plate Slab

The general process for designing the two-way flat plate slabs will be as follows:

- a) Model slab with initial thickness in RAM Concept and apply slab loads
- b) Layout design strips and analysis for stresses in slab
- c) Reiterate to optimal slab thickness
- d) Draft rebar placement plan based on analysis output

The initial thickness of the slabs was based on the minimum thickness requirements in ACI Table 9.5(c). The longest span for Jackson Crossing is an interior span with a clear distance of 23' 7". Given that the equation for an interior panel without drop panels is $l_n/33$, the initial thickness of the slab was 9". From this initial assumption, the thickness will be reduced to meet the requirements in ACI Table 9.5(b).

2.2.2.1 RAM Concept Model

Slabs for each level were initially modelled with two-way slab behavior, a 9" thickness and a 5000 psi concrete mix. 14" by 14" columns were also modelled to act as support for the slab. In addition slab openings were modeled for the stairwell and elevator cores. Figure 26 is an example screenshot of the mesh input standard plan from RAM Concept.

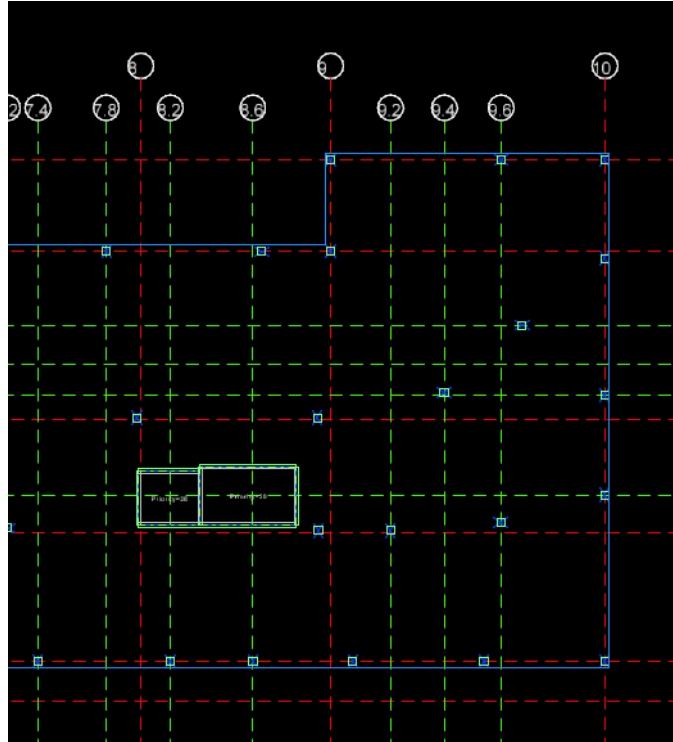


Figure 26 – Example Screenshot of RAM Concept Model

The grid system used to model the slab in addition to the columns was drafted in AutoCAD and then imported into RAM Concept. The grid was based on the grid system in the existing Jackson Crossing architectural plans. The imported grid is also shown in Figure 26.

Dead and live loads were applied to the slabs as area loads using the values detailed in Section 2.1 of this report. For the snow drift load on the roof, the load was applied in increments as in Figure 26. This was because RAM Concept does not allow area loads with different values in the four corners to create a varying load.

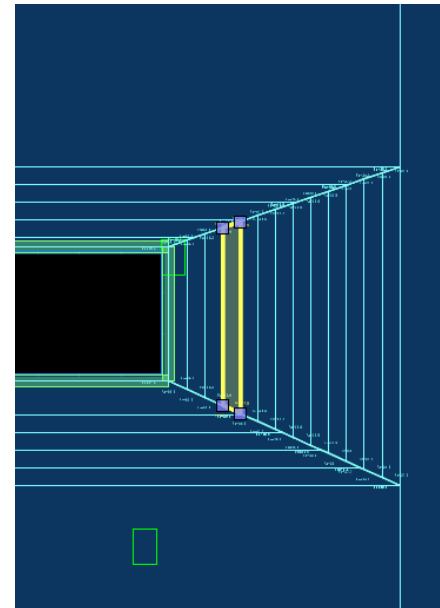


Figure 27 – Example of Snow Loading

For the latitude and longitude design strips, the following settings were specified for the column and middle strip:

Top Cover: 1.5" **Bottom Cover:** 1.5"
Top Bar: #5 **Bottom Bar:** #4
Design System: Two-Way Slab

Although these settings will design the needed steel with #4 and #5 bars, a more appropriate bar configuration can be determined by hand calculations. Figure 28 and 29 is an example of the design strip layout.

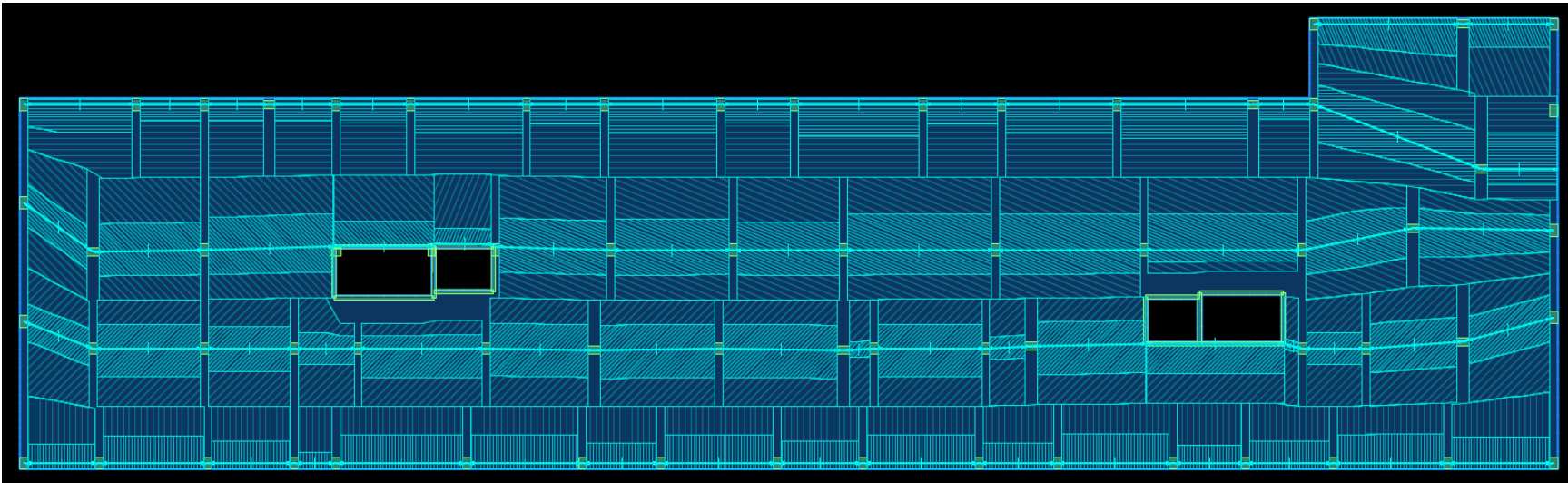


Figure 29 – Example Layout of Latitude Design Strips for Floors 2 and 3. (Darker Hatching: Column Strip, Lighter Hatching: Middle Strip)

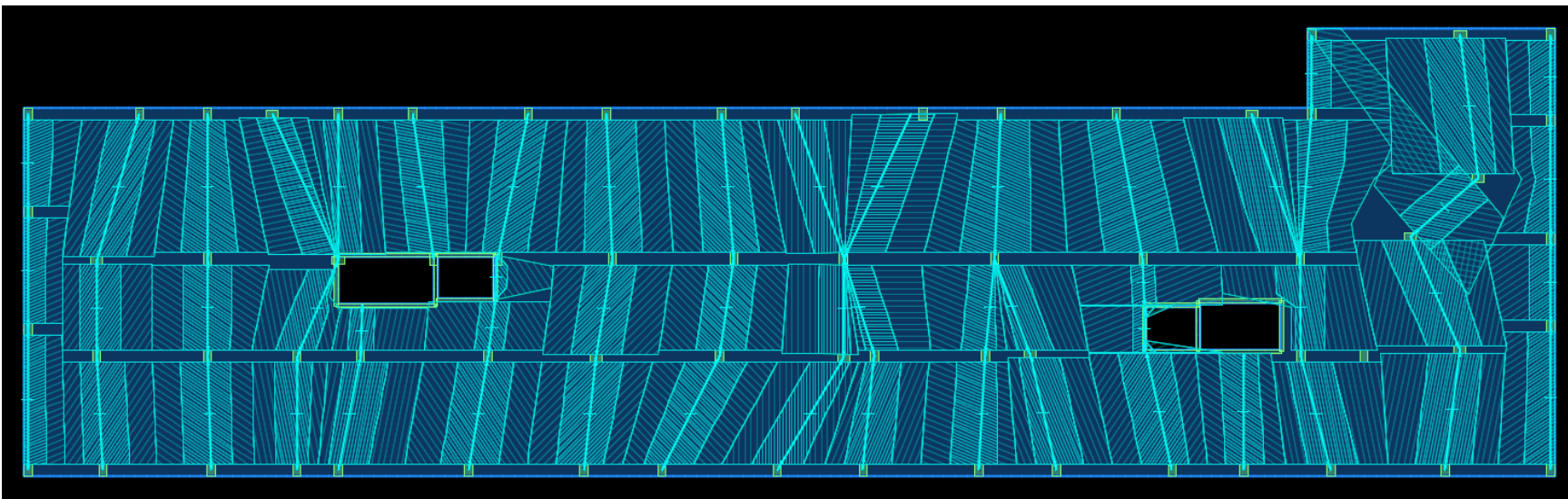


Figure 28 – Example Layout of Longitude Design Strips for Floors 2 and 3. (Darker Hatching: Column Strip, Lighter Hatching: Middle Strip)

2.2.2.2 Punching Shear Failure

Two-way punching shear was a problem for columns on every floor. Figure 30 is an example from the second and third floor slab that highlights the columns the fail in punching. The columns the fail were generally around the exterior and interior columns with a larger span adjacent to a shorter span.

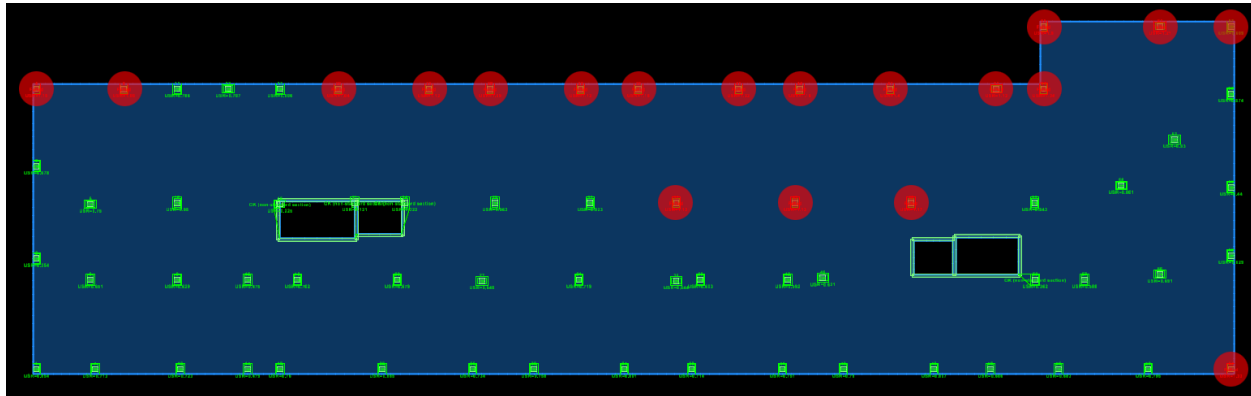


Figure 30 – Locations of Punching Shear Failures on Second and Third Floor

A closer inspection of one of the failing columns from Figure 30 was necessary to verify the validity of the RAM Concept model. The column chosen for this purpose was column C42. The dimensions around column C42 are in Figure 31. Column C42 has uneven spans in the north-south plan dimension.

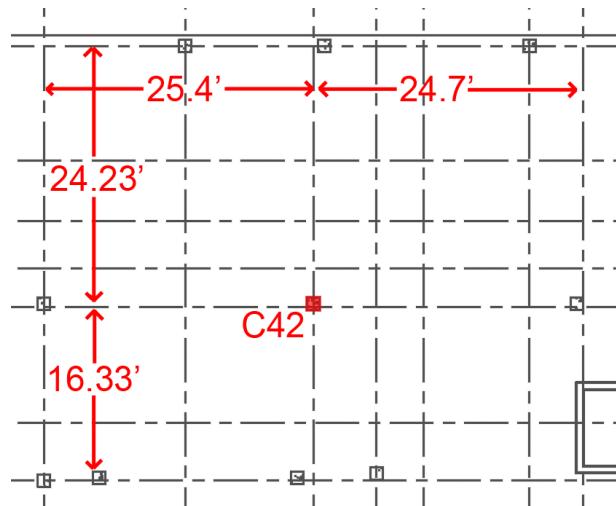


Figure 31 – Dimensions Near Column C42

An initial check was made of the punching shear without considering the added stress from the unbalanced moment. The summary of the calculations are in Table 9 (on the next page).

The capacity of the slab to resist punching, ϕV_c , was not enough to resist the punching force from the column, V_u . In addition the shear stress from the unbalanced moment worsened the punching shear even more.

From Table 10 (on the next page), the maximum punching shear stress including the unbalanced moment on C42 was 358.82 psi. A b_o of 134.5" is required to resist this punching in the concrete alone. Because of this a 28" by 28" and 12" deep shear cap will be formed in all columns failing in punching shear.

Table 9 – Summary of Punching Shear on C42

Punching Shear of Column C42			
	d (in)	5.88	
	d/2 (in)	2.94	
Area enclosed (ft ²):		2.74	
	b _o (in)	79.50	
DL (psf)	LL (psf)	Slab S.W. (psf)	Factored Load (psf)
22	40	100	210.4
Vu (k):		106.31	
	ACI Eq.	φVc (k)	
	(11-31)	148.62	
	(11-32)	122.76	
	(11-33)	99.08	
Min. φVc (k):		99.08	

Table 10 - Summary of Shear Stress from Unbalanced Moment on C42

Shear from Unbalanced Moment	
Unbalanced Moment, Mr (k-ft):	66.9
γf:	0.6
A _s needed for Mr (in ²):	1.60
Try (9)#4 A _s (in ²):	1.80
a (in)	0.64
φMn (k-ft):	45.01
γvMu (k-ft):	21.89
Jc (in ⁴)	19895.95
±(γvMuc)/Jc (psi)	131.21
v_u (psi)	358.82
bo req. (in)	134.47
width of shear cap req. (in)	28

The punching shear failures on the exterior columns will be alleviated by the addition of edge beams around the perimeter of each slab. The reinforcement and design of these edge beams will be explored in a later section of this report when the moment frame is discussed.

Torsion on the beam is necessary to consider for the edge beams. Torsion effects are permitted to be neglected if less than a calculated torsion force from ACI Section 11.5.1. For a nonprestressed member this threshold is equal to:

$$\phi \lambda \sqrt{f'c} \left(\frac{A_{cp}^2}{p_{cp}} \right)$$

Table 11 is a summary of the torsion on an edge beam at the plan north side of the slab. From this table, a 16"x24" edge beam is sufficient to neglect torsion forces. The allowable torsion on the edge beam is 8.15 k-ft while it will only experience a force of 6.63 k-ft at the worst case.

Table 11 – Torsion on Edge Beam

Torsion on North Edge of Slab for a 16" x24" Edge Beam	
Span to nearest column (ft):	24.23
DL (psf)	22
LL (psf)	40
Factored Load (psf)	90.4
Torsion (k-ft per foot)	6.63
Allowable Torsion (k-ft)	8.15

2.2.2.3 Long Term Deflection

To keep the amount of reinforcement needed within reason, a slab thickness of 8" was settled on for all floors. This 8" structural thickness is 10" thinner than the previous 18" wood truss joists. Because 8" is smaller than the minimum thickness from ACI Table 9.5(c), it is necessary to reference ACI Table 9.5(b) for maximum permissible computed deflections.

To be conservative the deflection limit for, "roof or floor construction supporting or attached to nonstructural elements likely to be damaged by large deflections" of 1/480 was used. This deflection considers the sum of the long term deflection due to all sustained loads and the immediate deflection due to any additional live load.

RAM Concept allows for the input of duration for the sustained load deflection. As the code does not explicitly specify the duration for long term deflection a duration of 5000 days or approximately 13 and a half years was considered. This sustained load deflection is in addition to a maximum short term load with a duration of 30 days for a total sustained load duration of 5033 days.

Table 12 - Summary of Long Term Deflections Compared to the Code Allowable Limit

Sustained Load Deflection		
Level	Sustained Load Δ (in)	Code Allowable Δ (in)
Floor 2	0.307	0.590
Floor 3	0.307	0.590
Floor 4	0.232	0.590
Floor 5	0.225	0.590
Roof	0.214	0.590

Table 12 lists the sustained load deflection for each floor obtained from the analysis model in addition to the code allowable deflection. The code allowable deflection is conservative as it only considered the maximum slab span of 23' 7" to determine the deflection.

2.2.2.4 Rebar Placement Plan

RAM Concept calculates required reinforcement from a finite analysis of stresses in the slab. From this analysis a more constructible reinforcement plan was drafted in AutoDesk Revit.

Also for constructability, a bottom mat of #5 bars at 12 inches on center was specified. A bottom mat provides the strength need for flexure at midspan while requiring very little additional bottom bars other than the mat itself. Figure 32(on next page) is an initial bottom reinforcement plan while Figure 33(on next page) is a bottom reinforcement plan with the addition of a bottom mat.

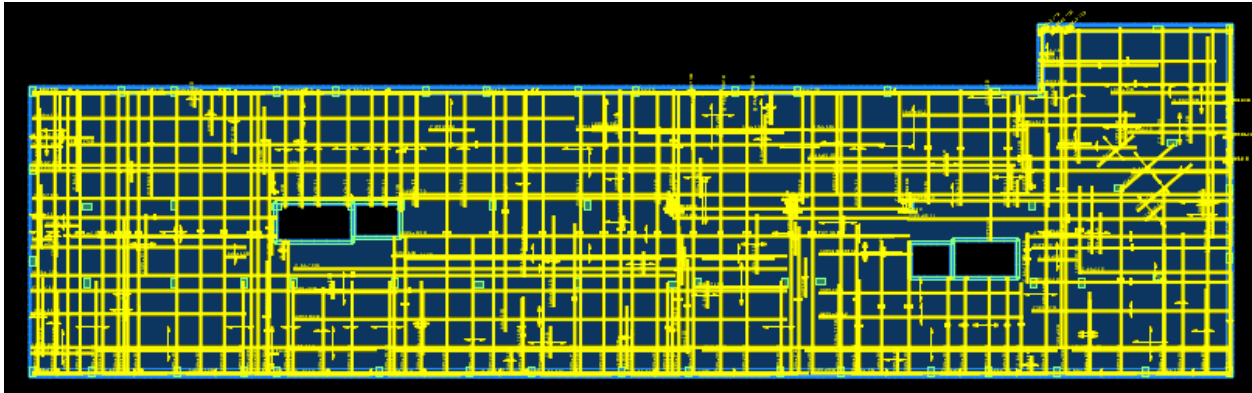


Figure 33 - Bottom Reinforcement Plan Before Bottom Mat Was Specified (Yellow Lines Represent Rebar)

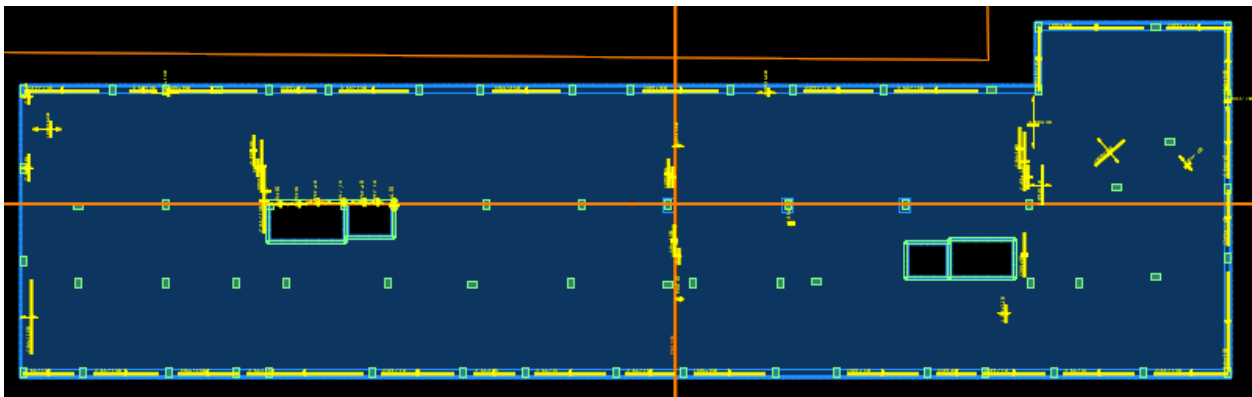


Figure 32 - Bottom Reinforcement Plan With Bottom Mat of #5 Bars at 12" o.c. (Yellow Lines Represent Rebar)

Reinforcement plans for every slab are in the Appendix under Section A.2. Figure 34 is a snapshot of a one of these reinforcement plans. The rebar is colored as follows:

- Green:** Latitude Column Strip Top Bars
- Orange:** Latitude Middle Strip Top Bars
- Red:** Longitude Column Strip Top Bars
- Blue:** Longitude Middle Strip Top Bars

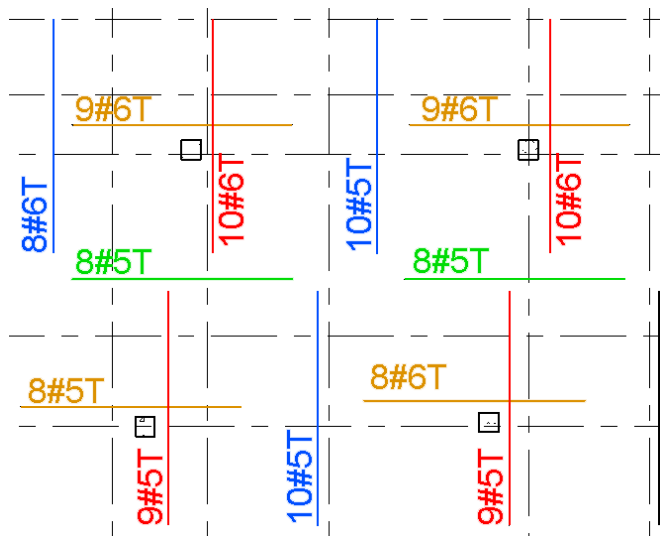


Figure 34 - Example Bay with Reinforcement Layout

The placement of this rebar is detailed in Figure 35 (on next page) and Figure 36 (on next page) for the column strip and middle strip respectively. The details are based on minimum extensions for reinforcement in slabs without beams from ACI Fig. 13.3.8 as Section 13.3.8.2 requires. For unequal adjacent spans, the larger clear span is taken as l_n .

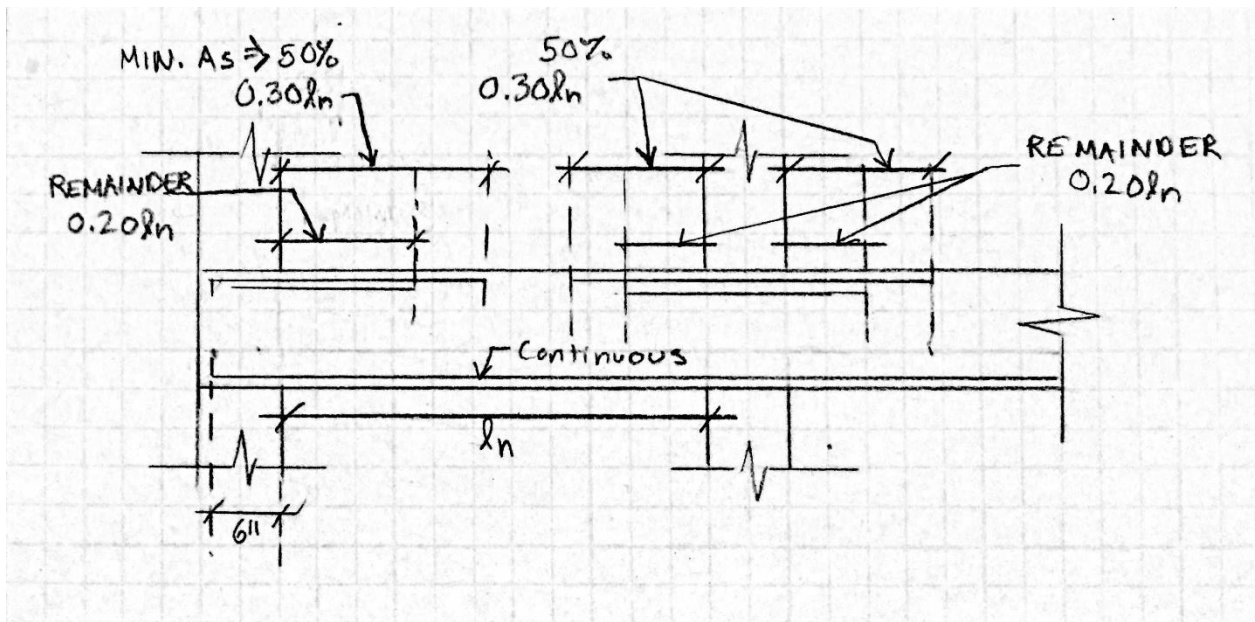


Figure 35 - Reinforcement Placement Detail for Column Strip

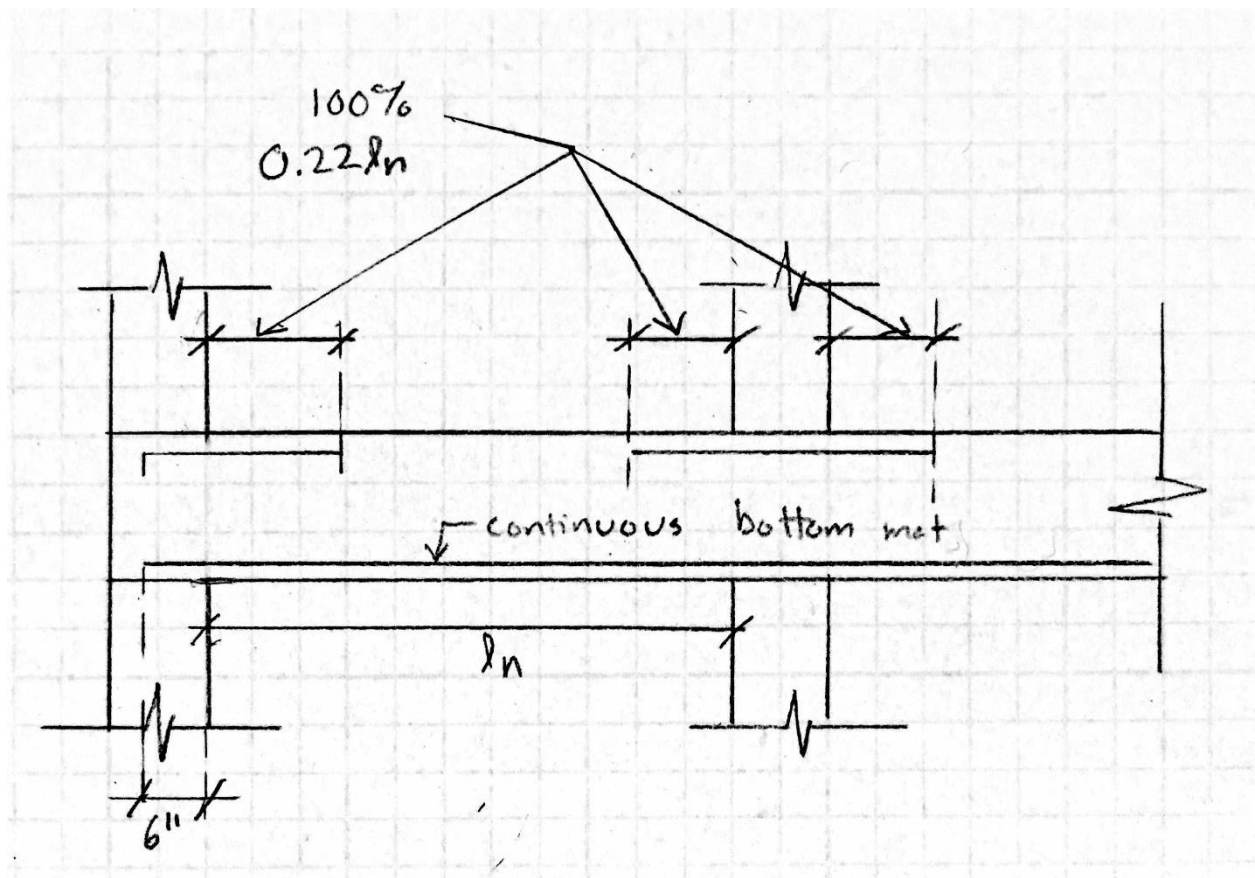


Figure 36 - Reinforcement Placement Detail for Middle Strip

2.2.3 Transfer Beams

Due to the change in column layout for the below grade parking garage, transfer beams at the ground level must handle the load of the five stories above. The depth of these transfer beams are critical because there must be enough clearance from the bottom face of the beams to the floor of the garage level slab for the movement of cars.

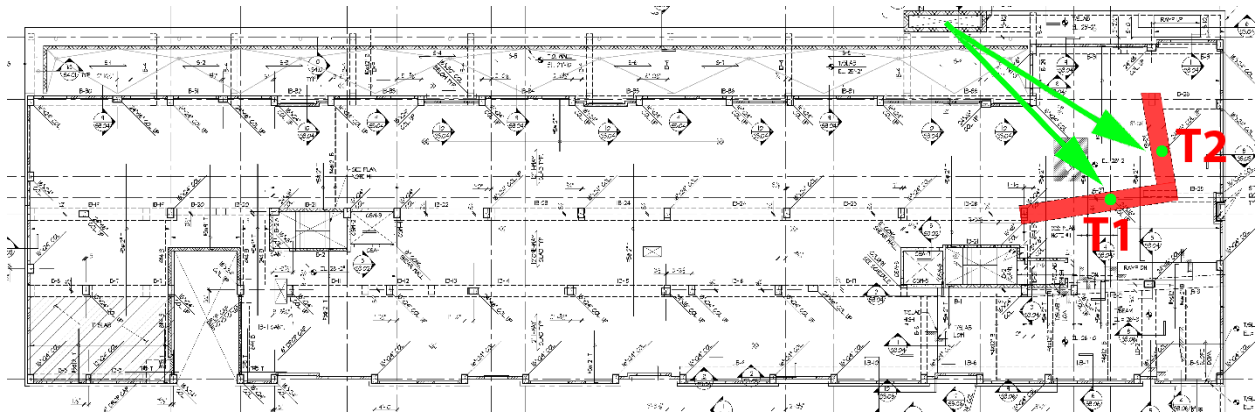


Figure 37 – Location of Critical Transfer Beam

The location of two critical transfer beams is in Figure 37; the beams are highlighted in red while the discontinuous columns are represented by a green dot. The existing transfer beams have the following dimensions and reinforcing:

T1

Width x Depth: 48" x 36"

Bottom Bars: 14#10

Top Bars: 8#9 at Full Length

Stirrups: 4 Leg #4, 1 at 2" and rest at 8" on center

T2

Width x Depth: 48" x 36"

Bottom Bars: 12#9

Top Bars: 6#9 at Full Length

Stirrups: 4 Leg #4, 1 at 2" and rest at 12" on center

To determine the forces on the transfer beams, they were modeled in ETABS along with two additional beams to the left of beam T1 for any moment transfer between the continuous span. Loads from the ground floor were applied along with the point loads from the discontinuous columns. The point loads were referenced from the list of gravity column forces in Appendix A.1.

Table 13 is the maximum shear and moment forces on transfer beams T1 and T2. Beam T1 experiences a larger moment due to its longer span. The calculations to determine what reinforcing needs changed in the transfer beams to accommodate this new loads are in Appendix A.6. The final dimensions and reinforcing for the redesigned transfer beams are:

Table 13 – Forces on Transfer Beams

Transfer Beam	+Mu (k-ft)	-Mu (k-ft)	Vu (k)
T1	1659.2	-1985.2	335.6
T2	998.2	-1070.8	264.5

Redesigned T1

Width x Depth: 48" x 36"

Bottom Bars: 14#10

Top Bars: 12#10 at Full Length

Stirrups: 4 Leg #4, 1 at 2" and rest at 6" on center

Redesigned T2

Width x Depth: 48" x 36"

Bottom Bars: 12#9

Top Bars: 8#9 at Full Length

Stirrups: 4 Leg #4, 1 at 2" and rest at 12" on center

With the additional weight from the redesigned structure, the transfer beams are able to keep their dimensions. Only the top beams needed additional reinforcement along with closer spacing of the stirrups for beam T1. The ability to keep the beam dimensions is important because the clearance height below the transfer beams will not be impacted by the redesign of the structure.

2.3 Lateral System

For the redesign, a concrete moment frame was considered around the exterior of the building except for the west moment frame as the building recesses in for the roof terrace on the fourth floor. Figure 38 is a typical floor plan with the moment frame locations highlighted in purple. The labels on the frames will be used throughout this section.

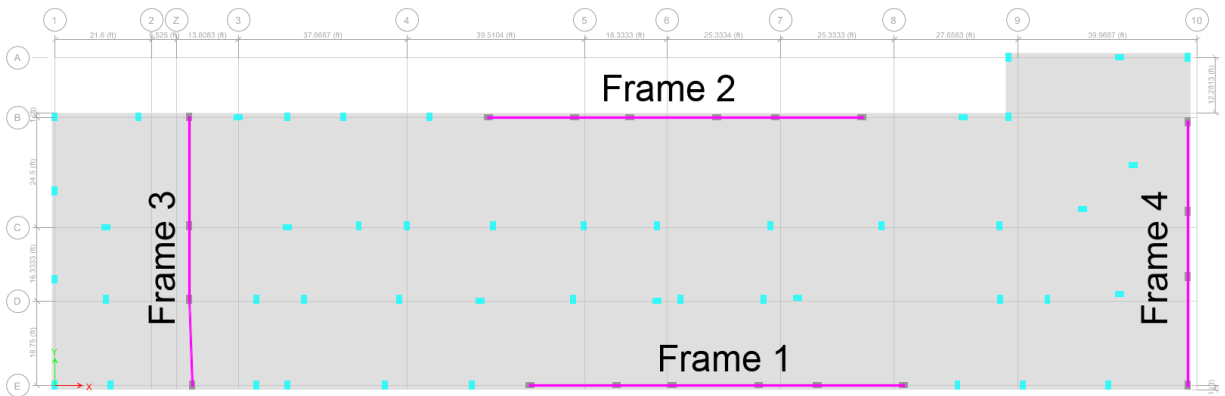


Figure 38 – Floor Plan of Moment Frames

The location of the moment frames produces minimal torsional shear. From Table 14, the greatest difference between the Center of Mass and Center of Rigidity is only 2.5% of the total length of the building.

Table 14 – Comparison of Center of Rigidity and Center of Mass

Level	Center of Mass (ft)		Center of Rigidity (ft)		Difference (ft)		% of Total Wall Length	
	x	y	x	y	x	y	x	y
Roof	144.59	31.06	141.16	29.52	3.43	1.54	1.51	2.24
Floor 5	143.94	31.04	140.96	29.66	2.98	1.39	1.32	2.02
Floor 4	139.18	31.00	140.69	29.75	-1.51	1.24	0.59	1.81
Floor 3	136.69	30.97	138.80	29.80	-2.10	1.17	0.82	1.70
Floor 2	135.25	30.96	138.41	29.87	-3.15	1.09	1.23	1.59

All of the column and beam members in the moment frame have the following properties:

Width: 16"
Depth: 24"

f'_c : 5,000psi
 f_y : 60,000 psi

2.3.1 Lateral Loads

To determine the controlled lateral load, the moment frames were modeled in ETABS along with the two-way slabs and gravity columns. Similar to the gravity column model, fake beams were again modeled to act as a connection for the columns as in Figure 19. Only the moment frames were assigned a rigid base to ensure the moment frames take the majority of the lateral load. The lateral loads were program determined for wind and seismic.

Table 15 – Lateral Load Base Shears

Base Shear (k)		
Direction of Loading	Wind	Seismic
NS	272.46	401.66
EW	65.78	401.66

Table 15 is the base shear in both directions for both wind and seismic. Seismic was the controlling case in both directions with a base shear of 402 k. This base shear was verified by hand calculations using Section 17.5 of ASCE-7. The summary of these calculations are in Table 16. The seismic base shear is for both directions.

Table 16 – Summary of Seismic Loads

Seismic Load								
Ground Accelerations								
	Ss	0.118	Sms	0.189	Sds	0.126		
	S1	0.051	Sm1	0.122	Sd1	0.082		
	Ie	1			Ta	0.586		
	R	3			Tl	8		
	SDC	B			k	1.043		
			Cs	0.047				
Level	Story Height (ft)	Elevation from Base(ft)	Sq. Ft.	Slab Weight (k)	Dead Load (k)	$w_x h_{xk}$	C_{vx}	F_x
Roof	11.1	54.6	14672.38	1467.24	310.98	115236.33	0.31	137.81
5th Floor	9.7	43.5	14672.38	1467.24	293.45	90016.66	0.24	107.65
4th Floor	9.7	33.8	16559.62	1655.96	378.37	80028.74	0.21	95.70
3rd Floor	9.7	24.2	16559.62	1655.96	331.19	55039.00	0.15	65.82
2nd Floor	14.5	14.5	16559.62	1655.96	331.19	32308.99	0.09	38.64
					SUM	9547.54	V (k)	445.62

2.3.2 Moment Frame Columns

For clarity the moment frame columns are labeled in Figure 39. Forces in these columns are listed in Table 16. 0.9D+1.0E was the controlling load combination for each of the columns.

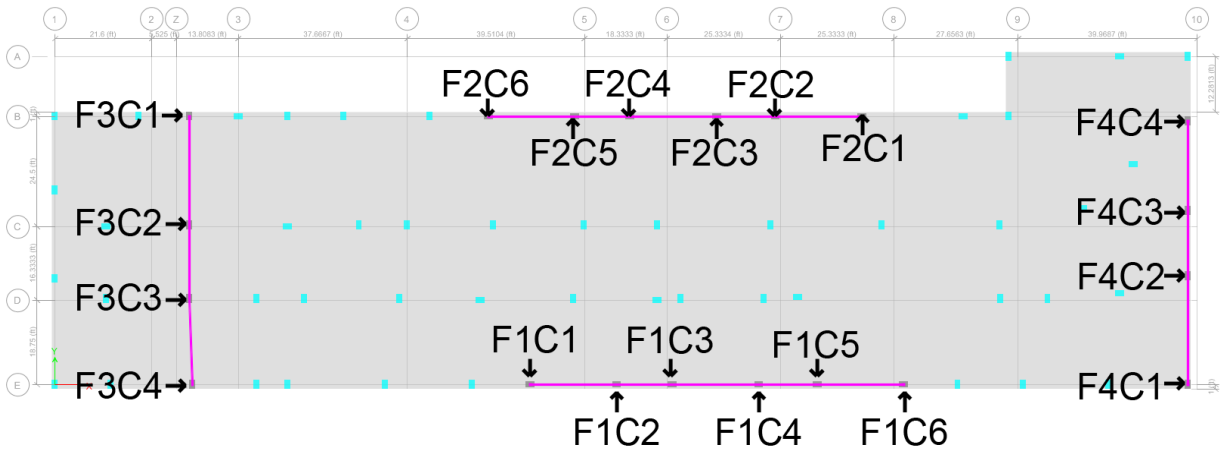


Figure 39 – Moment Frame Column Labels

Table 17 – Moment Frame Column Forces

Column	Combo	Axial (K)	M1 (k-ft)	M2 (k-ft)	Vu (k)
F1C1	0.9D+E	89.428	220.0426	2.2502	37.801
F1C2	0.9D+E	183.104	212.3095	2.5668	35.244
F1C3	0.9D+E	94.885	218.4238	2.3529	37.266
F1C4	0.9D+E	163.094	213.8092	2.3566	35.74
F1C5	0.9D+E	69.139	184.7505	2.2748	26.129
F1C6	0.9D+E	206.98	194.1208	3.0609	29.228
F2C1	0.9D+E	187.156	214.867	1.9381	35.838
F2C2	0.9D+E	122.426	219.7743	2.2003	37.461
F2C3	0.9D+E	204.248	213.9049	2.4737	35.52
F2C4	0.9D+E	118.728	222.4997	2.514	38.363
F2C5	0.9D+E	245.231	195.2924	2.7696	29.364
F2C6	0.9D+E	96.391	185.6097	2.6498	26.162
F3C1	0.9D+E	197.157	350.5509	5.5244	53.479
F3C2	0.9D+E	292.737	371.1651	0.7407	60.297
F3C3	0.9D+E	175.2994	387.6746	3.3348	65.757
F3C4	0.9D+E	83.43399	334.8581	6.6203	48.336
F4C1	0.9D+E	42.24	291.1087	4.7919	38.883
F4C2	0.9D+E	80.817	350.4922	0.7431	58.523
F4C3	0.9D+E	182.28	347.89	2.1545	57.662
F4C4	0.9D+E	200.124	310.3013	6.044	45.231

To determine whether the moment frame columns will receive any considerable slenderness effects, column F1C5 was chosen as an example to determine its slenderness from ACI Eq. (10-6). This equation was used because the moment frame is not braced against sidesway with a more rigid shear wall.

Table 18 – Slenderness Check

Bottom Column F1C5					
Properties					
b (in)	16	Cover (in)	2.5	r (0.3h)	7.2
h (in)	24	As	6		
		I (in⁴)	I Modifier	I (ft)	
	Column 1	18432	0.7	9.67	
	Column 2	18432	0.7	9.67	
	Beam 1	18432	0.35	13.00	
	Beam 2	18432	0.35	19.37	
Slenderness					
	Ψ_A	3.22			
	Ψ_B	0.00			
	k	1.36	ACI Fig. R10.10.1.1		
	kl/r:	21.91	<22	*OK	

Table 18 is a check of the slenderness of column F1C5 for a sway frame. From ACI Eq. (10-6) the kl/r of a sway frame must be less than 22 for slenderness effects to be considered negligible. For column F1C5, the kl/r was determined to be less than 22 and therefore will not be considerably influenced by slenderness effects.

Two columns will be design considering the forces from Table 17. All columns in Frame 1 and Frame 2 will be reinforced according to forces from Column F1C1. All columns in Frame 3 and Frame 4 will be reinforced according to forces from Column F3C3. Table 19 is a summary of the final reinforcement design of the columns. The calculations for the designs are in Appendix A.4.

Table 19 – Final Reinforcement of Moment Frame Columns

Summary of Moment Frame Column Design		
Column	Longitudinal Reinforcement	Stirrups
F1C1	2 Layers of 3#8	2 Leg #4 @ 16" o.c.
F3C3	2 Layers of 3#8	2 Leg #4 @ 16" o.c.

2.3.3 Moment Frame Beams

The controlling load case for the moment and shear in the moment frame beams was 1.2D+1.0E+0.5L+0.7S. This load combination controls over the 1.2D+1.6L+0.5S due to the high seismic forces.

Table 20 – Forces in Frame 1 and Frame 2 Beams

Forces in Frame 1 and Frame 2							
Beam	Mu (k-ft)-	Mu (k-ft)+	Vu (k-ft)	Beam	Mu (k-ft)	+	Vu (k-ft)
F1B1	-67.73	106.97	15.98	F2B1	-52.97	24.04	13.23
F1B2	-102.15	94.31	19.64	F2B2	-117.36	45.47	22.96
F1B3	-129.32	88.19	23.12	F2B3	-143.94	60.67	25.92
F1B4	-143.06	56.46	24.76	F2B4	-160.56	73.29	27.74
F1B5	-142.49	42.26	24.97	F2B5	-165.03	75.99	28.16
F1B6	-45.74	13.40	11.79	F2B6	-58.28	20.47	15.11
F1B7	-89.21	64.04	21.13	F2B7	-103.56	70.84	24.01
F1B8	-125.31	98.15	27.93	F2B8	-132.94	100.10	29.31
F1B9	-146.30	119.39	32.01	F2B9	-150.67	118.68	32.59
F1B10	-150.39	120.81	32.55	F2B10	-152.16	119.93	32.84
F1B11	-74.16	38.85	17.91	F2B11	-80.22	37.76	19.67
F1B12	-114.16	51.43	22.39	F2B12	-118.57	52.05	24.16
F1B13	-137.20	68.63	25.09	F2B13	-139.17	68.35	26.55
F1B14	-148.32	78.97	26.34	F2B14	-149.98	78.74	27.77
F1B15	-143.35	74.78	25.81	F2B15	-146.03	75.57	27.36
F1B16	-59.68	19.68	10.71	F2B16	-54.78	29.01	14.71
F1B17	-92.85	66.02	20.84	F2B17	-95.63	67.85	22.62
F1B18	-127.61	99.52	27.04	F2B18	-127.79	99.95	28.84
F1B19	-146.51	119.39	30.57	F2B19	-147.60	119.70	32.67
F1B20	-147.54	120.15	30.73	F2B20	-151.76	121.06	33.20
F1B21	-62.88	29.99	13.26	F2B21	-57.90	36.89	11.64
F1B22	-100.31	44.54	17.85	F2B22	-112.75	59.54	23.08
F1B23	-132.38	68.70	21.38	F2B23	-136.89	80.19	26.00
F1B24	-148.67	83.27	23.16	F2B24	-150.31	95.00	27.64
F1B25	-151.65	84.49	23.40	F2B25	-150.52	100.07	27.95

Two beams were designed. One for moment frames 1 and 2; one for moment frames 3 and 4. Table 20 and Table 21 list the forces in the beam members for the east-west frames and north-south frames respectively. The maximum forces used to design the beams are highlighted and bolded. The beam labels are listed in Figures 40 through 43.

F1B1	F1B6	F1B11	F1B16	F1B21
F1B2	F1B7	F1B12	F1B17	F1B22
F1B3	F1B8	F1B13	F1B18	F1B23
F1B4	F1B9	F1B14	F1B19	F1B24
F1B5	F1B10	F1B15	F1B20	F1B25

Figure 41 – South Elevation of Frame 1

F2B1	F2B6	F2B11	F2B16	F2B21
F2B2	F2B7	F2B12	F2B17	F2B22
F2B3	F2B8	F2B13	F2B18	F2B23
F2B4	F2B9	F2B14	F2B19	F2B24
F2B5	F2B10	F2B15	F2B20	F2B25

Figure 40 – North Elevation of Frame 2

Table 21 – Forces in Frame 3 and Frame 4 Beams

Forces in Frame 3 and Frame 4							
Beam	Mu (k-ft)	+	Vu (k-ft)	Beam	Mu (k-ft)	+	Vu (k-ft)
F3B1	-193.76	111.61	32.41	F4B1	-147.85	85.92	25.05
F3B2	-254.99	100.96	36.62	F4B2	-206.40	109.77	30.04
F3B3	-280.53	114.55	41.91	F4B3	-228.05	132.72	32.21
F3B4	-286.69	115.62	42.35	F4B4	-237.93	142.53	33.09
F3B5	-287.81	115.32	42.40	F4B5	-229.80	139.45	32.59
F3B6	-202.66	89.97	40.69	F4B6	-111.76	70.78	22.30
F3B7	-208.98	151.48	37.46	F4B7	-198.76	166.10	37.11
F3B8	-240.24	161.53	42.14	F4B8	-228.44	194.55	41.76
F3B9	-251.36	174.05	43.79	F4B9	-244.82	211.60	44.43
F3B10	-258.30	176.56	44.45	F4B10	-233.08	198.25	42.43
F3B11	-141.58	90.83	-25.65	F4B11	-140.89	83.61	20.93
F3B12	-198.88	127.68	-32.19	F4B12	-212.37	149.01	28.79
F3B13	-235.01	121.82	-41.50	F4B13	-238.47	172.97	31.59
F3B14	-247.16	127.85	-42.83	F4B14	-250.14	183.30	32.82
F3B15	-254.19	139.50	-44.11	F4B15	-241.16	172.02	31.69

F3B1	F3B6	F3B11
F3B2	F3B7	F3B12
F3B3	F3B8	F3B13
F3B4	F3B9	F3B14
F3B5	F3B10	F3B15

Figure 42 – West Elevation of Frame 3

F4B1	F4B6	F4B11
F4B2	F4B7	F4B12
F4B3	F4B8	F4B13
F4B4	F4B9	F4B14
F4B5	F4B10	F4B15

Figure 43 – East Elevation of Frame 4

The final reinforcement for longitudinal bars and stirrups for the beams are in Table 22; the calculations for the reinforcement are in Appendix A.5. The moment frame beams resisting seismic loads in the north-south direction have greater reinforcement as they experience the same amount of seismic force as the east-west direction but have a shorter length.

Table 22 – Reinforcement in Moment Frame Beams

Final Moment Frame Beam Reinforcement			
Frame	Bottom Bars	Top Bars	Stirrups
1&2	(4) #6 Full Length	(3) #6 Full Length	2 leg #4 @10" o.c.
3&4	(6) #6 Full Length	(4) #8 Full Length	2 leg #4 @10" o.c.

2.3.4 Deflections

Because the moment frames were controlled by seismic forces the inter story drift of the frames must meet the limitations of Section 12.12 of ASCE 7. The drift was also compared to a common standard of 1/400. Drift values were determined using deflection data from the ETABS lateral system model.

Table 23 compares the drift values from the ETABS lateral system model and the maximum allowable drift from ASCE 7 in addition to 1/400. Drift in both directions on the frame did not exceed either of the allowable values.

Table 23 – Inter Story Drift of Moment Frame

Deflections					
Story	Story Height (ft)	Δx (in)	Δy (in)	$0.020h_{sx}$ (in)	1/400 (in)
Roof	10.60	0.095	0.183	2.545	0.318
5th	9.67	0.115	0.199	2.320	0.290
4th	9.67	0.141	0.226	2.320	0.290
3rd	9.67	0.147	0.224	2.320	0.290
2nd	9.67	0.115	0.166	2.320	0.290

3 Construction Breadth

The purpose of the construction breadth is to compare the redesigned structure and the existing structure in terms of the total cost of the systems per floor and in addition the impact on the critical path.

3.1 Cost Analysis

The cost estimate for the redesign structure will take into account the following:

- Material costs of concrete mix for the slab, beams, columns, and shear caps
- Labor and equipment costs for placing the concrete mix with pumping
- Material and labor costs for formwork
- Material and labor costs for reinforcement of slabs and columns.

Cost multiplier data was referenced from the *2014 RSMeans Building Construction Cost Data*. The final total cost was adjusted for the location in Alexandria, VA by a factor of 93.5. Table 24 is a summary of the cost estimate.

Table 24 – Summary of Cost Analysis

Heavyweight Concrete, Ready Mix									
<i>Slab</i>									
	Unit	Amount	Multiplier			Material	Labor	Equipment	Total
			Material	Labor	Equipment				
2nd Floor	C.Y.	392.50	110.00	0.00	0.00	43175.07	0.00	0.00	43175.07
3rd Floor	C.Y.	392.50	110.00	0.00	0.00	43175.07	0.00	0.00	43175.07
4 Floor	C.Y.	392.50	110.00	0.00	0.00	43175.07	0.00	0.00	43175.07
5 Floor	C.Y.	346.65	110.00	0.00	0.00	38131.53	0.00	0.00	38131.53
Roof	C.Y.	346.65	110.00	0.00	0.00	38131.53	0.00	0.00	38131.53
<i>Beams</i>									
2nd Floor	C.Y.	59.38	110.00	0.00	0.00	6531.99	0.00	0.00	6531.99
3rd Floor	C.Y.	59.38	110.00	0.00	0.00	6531.99	0.00	0.00	6531.99
4 Floor	C.Y.	59.38	110.00	0.00	0.00	6531.99	0.00	0.00	6531.99
5 Floor	C.Y.	53.40	110.00	0.00	0.00	5873.80	0.00	0.00	5873.80
Roof	C.Y.	53.40	110.00	0.00	0.00	5873.80	0.00	0.00	5873.80
<i>Columns</i>									
2nd Floor	C.Y.	64.92	110.00	0.00	0.00	7141.40	0.00	0.00	7141.40
3rd Floor	C.Y.	64.92	110.00	0.00	0.00	7141.40	0.00	0.00	7141.40
4 Floor	C.Y.	64.92	110.00	0.00	0.00	7141.40	0.00	0.00	7141.40
5 Floor	C.Y.	57.28	110.00	0.00	0.00	6301.23	0.00	0.00	6301.23
Roof	C.Y.	62.84	110.00	0.00	0.00	6912.35	0.00	0.00	6912.35

Con'd Table 24

Shear Caps									
2nd Floor	C.Y.	0.78	110.00	0.00	0.00	85.56	0.00	0.00	85.56
3rd Floor	C.Y.	0.78	110.00	0.00	0.00	85.56	0.00	0.00	85.56
4 Floor	C.Y.	0.78	110.00	0.00	0.00	85.56	0.00	0.00	85.56
5 Floor	C.Y.	0.78	110.00	0.00	0.00	85.56	0.00	0.00	85.56
Roof	C.Y.	0.78	110.00	0.00	0.00	85.56	0.00	0.00	85.56
									SUM \$ 272,197.38
Placing Concrete									
Slab, Pumped									
2nd Floor	C.Y.	392.50	0.00	15.75	4.85	0.00	6181.89	1903.63	8085.51
3rd Floor	C.Y.	392.50	0.00	15.75	4.85	0.00	6181.89	1903.63	8085.51
4 Floor	C.Y.	392.50	0.00	15.75	4.85	0.00	6181.89	1903.63	8085.51
5 Floor	C.Y.	346.65	0.00	15.75	4.85	0.00	5459.74	1681.25	7140.99
Roof	C.Y.	346.65	0.00	15.75	4.85	0.00	5459.74	1681.25	7140.99
Beams, Large Beams Pumped									
2nd Floor	C.Y.	59.38	0.00	28.00	8.65	0.00	1662.69	513.65	2176.34
3rd Floor	C.Y.	59.38	0.00	28.00	8.65	0.00	1662.69	513.65	2176.34
4 Floor	C.Y.	59.38	0.00	28.00	8.65	0.00	1662.69	513.65	2176.34
5 Floor	C.Y.	53.40	0.00	28.00	8.65	0.00	1495.15	461.89	1957.04
Roof	C.Y.	53.40	0.00	28.00	8.65	0.00	1495.15	461.89	1957.04
Columns, 18" Pumped									
2nd Floor	C.Y.	64.92	0.00	28.00	8.65	0.00	1817.81	561.57	2379.38
3rd Floor	C.Y.	64.92	0.00	28.00	8.65	0.00	1817.81	561.57	2379.38
4 Floor	C.Y.	64.92	0.00	28.00	8.65	0.00	1817.81	561.57	2379.38
5 Floor	C.Y.	57.28	0.00	28.00	8.65	0.00	1603.95	495.51	2099.46
Roof	C.Y.	62.84	0.00	28.00	8.65	0.00	1759.51	543.56	2303.07
Shear Caps									
2nd Floor	C.Y.	0.78	0.00	15.75	4.85	0.00	12.25	3.77	16.02
3rd Floor	C.Y.	0.78	0.00	15.75	4.85	0.00	12.25	3.77	16.02
4 Floor	C.Y.	0.78	0.00	15.75	4.85	0.00	12.25	3.77	16.02
5 Floor	C.Y.	0.78	0.00	15.75	4.85	0.00	12.25	3.77	16.02
Roof	C.Y.	0.78	0.00	15.75	4.85	0.00	12.25	3.77	16.02
									SUM \$ 60,602.42
Formwork									
Slab, Flat Plate 4 use									
2nd Floor	SFCA	15029.96	1.25	4.33	0.00	18787.45	65079.72	0.00	83867.17
3rd Floor	SFCA	15029.96	1.25	4.33	0.00	18787.45	65079.72	0.00	83867.17
4 Floor	SFCA	15029.96	1.25	4.33	0.00	18787.45	65079.72	0.00	83867.17
5 Floor	SFCA	13275.13	1.25	4.33	0.00	16593.91	57481.30	0.00	74075.21
Roof	SFCA	13275.13	1.25	4.33	0.00	16593.91	57481.30	0.00	74075.21
Beams, Exterior Spandrel job-built plywood 18" wide 4 use									
2nd Floor	SFCA	2805.78	0.88	6.80	0.00	2469.09	19079.34	0.00	21548.43
3rd Floor	SFCA	2805.78	0.88	6.80	0.00	2469.09	19079.34	0.00	21548.43
4 Floor	SFCA	2805.78	0.88	6.80	0.00	2469.09	19079.34	0.00	21548.43
5 Floor	SFCA	2523.06	0.88	6.80	0.00	2220.30	17156.83	0.00	19377.12
Roof	SFCA	2523.06	0.88	6.80	0.00	2220.30	17156.83	0.00	19377.12

Con'd Table 24

<i>Columns, job-built plywood, 24"x24" Column 4 use</i>									
2nd Floor	SFCA	4382.22	0.93	5.85	0.00	4075.47	25636.00	0.00	29711.47
3rd Floor	SFCA	4382.22	0.93	5.85	0.00	4075.47	25636.00	0.00	29711.47
4 Floor	SFCA	4382.22	0.93	5.85	0.00	4075.47	25636.00	0.00	29711.47
5 Floor	SFCA	3866.67	0.93	5.85	0.00	3596.00	22620.00	0.00	26216.00
Roof	SFCA	4241.67	0.93	5.85	0.00	3944.75	24813.75	0.00	28758.50
<i>Shear Caps</i>									
2nd Floor	SFCA	31.66	1.25	4.33	0.00	39.57	137.09	0.00	176.66
3rd Floor	SFCA	31.66	1.25	4.33	0.00	39.57	137.09	0.00	176.66
4 Floor	SFCA	31.66	1.25	4.33	0.00	39.57	137.09	0.00	176.66
5 Floor	SFCA	31.66	1.25	4.33	0.00	39.57	137.09	0.00	176.66
Roof	SFCA	31.66	1.25	4.33	0.00	39.57	137.09	0.00	176.66
									SUM \$ 648,143.67
Rebar									
<i>Elevated Slabs, #4-#7</i>									
2nd Floor	lb	32637.02	0.50	0.28	0.00	16318.51	9138.37	0.00	25456.87
3rd Floor	lb	32637.02	0.50	0.28	0.00	16318.51	9138.37	0.00	25456.87
4 Floor	lb	32637.02	0.50	0.28	0.00	16318.51	9138.37	0.00	25456.87
5 Floor	lb	28824.49	0.50	0.28	0.00	14412.24	8070.86	0.00	22483.10
Roof	lb	26629.31	0.50	0.28	0.00	13314.65	7456.21	0.00	20770.86
									SUM \$ 119,624.58
<i>Columns, #8-#18</i>									
2nd Floor	lb	8939.73	0.50	0.35	0.00	4469.87	3128.91	0.00	7598.77
3rd Floor	lb	8939.73	0.50	0.35	0.00	4469.87	3128.91	0.00	7598.77
4 Floor	lb	8939.73	0.50	0.35	0.00	4469.87	3128.91	0.00	7598.77
5 Floor	lb	7888.00	0.50	0.35	0.00	3944.00	2760.80	0.00	6704.80
Roof	lb	8653.00	0.50	0.35	0.00	4326.50	3028.55	0.00	7355.05
									SUM \$ 36,856.17
									Total Cost \$ 1,137,424.21
									Adjusted Total Cost \$ 1,063,491.64
									\$ 212,698.33

The total estimated cost for the redesigned structure was \$1,064,000. This comes to a cost of \$212,700 per floor. The cost of the existing wood frame structure was about \$700,000 in total or \$175,000 per floor as there were four wood framed levels. This means the redesigned structure costs 22% more.

3.2 Schedule Analysis

The wood framing from the existing structure of Jackson Crossing had a duration of 45 days along the critical path. The schedule analysis will explore how redesigning the structure with concrete slabs and columns effects the duration along the critical path.

Table 25 is list of critical path activities related to the construction of the wood frame trusses and framing for the original Jackson Crossing structure. Both the wood framing and floor trusses were constructed in two zones. Each zone, east and west, were scheduled for durations of five days. The original total duration for these events was 45 days.

Table 25 – Wood Framing Critical Path Events

Activity	Duration	Start	Finish
Wood Frame 2nd Floor West	5	12/9/2014	12/16/2014
Wood Frame 2nd Floor East	5	12/18/2014	12/24/2014
Floor Trusses 3rd Floor West	5	12/18/2014	12/24/2014
Floor Trusses 3rd Floor East	5	12/29/2014	1/5/2015
Wood Frame 3rd Floor West	5	12/29/2014	1/5/2015
Wood Frame 3rd Floor East	5	1/6/2015	1/12/2015
Floor Trusses 4th Floor West	5	1/6/2015	1/12/2015
Floor Trusses 4th Floor East	5	1/13/2015	1/20/2015
Wood Frame 4th Floor West	5	1/13/2015	1/20/2015
Wood Frame 4th Floor East	5	1/22/2015	1/28/2015
Floor Trusses 5th Floor West	5	1/22/2015	1/28/2015
Floor Trusses 5th Floor East	5	1/29/2015	2/4/2015
Wood Frame 5th Floor West	5	1/29/2015	2/4/2015
Wood Frame 5th Floor East	5	2/5/2015	2/12/2015
Roof Trusses West	5	2/5/2015	2/12/2015
Roof Trusses East	5	2/13/2015	2/20/2015
		Total Duration:	45 Days

The redesigned structure schedule considered the duration of construction for columns, elevated slabs, and wall framing. The wall framing was assumed to take the same duration as with the original schedule. For the elevated slabs and columns, the duration was considered as follows:

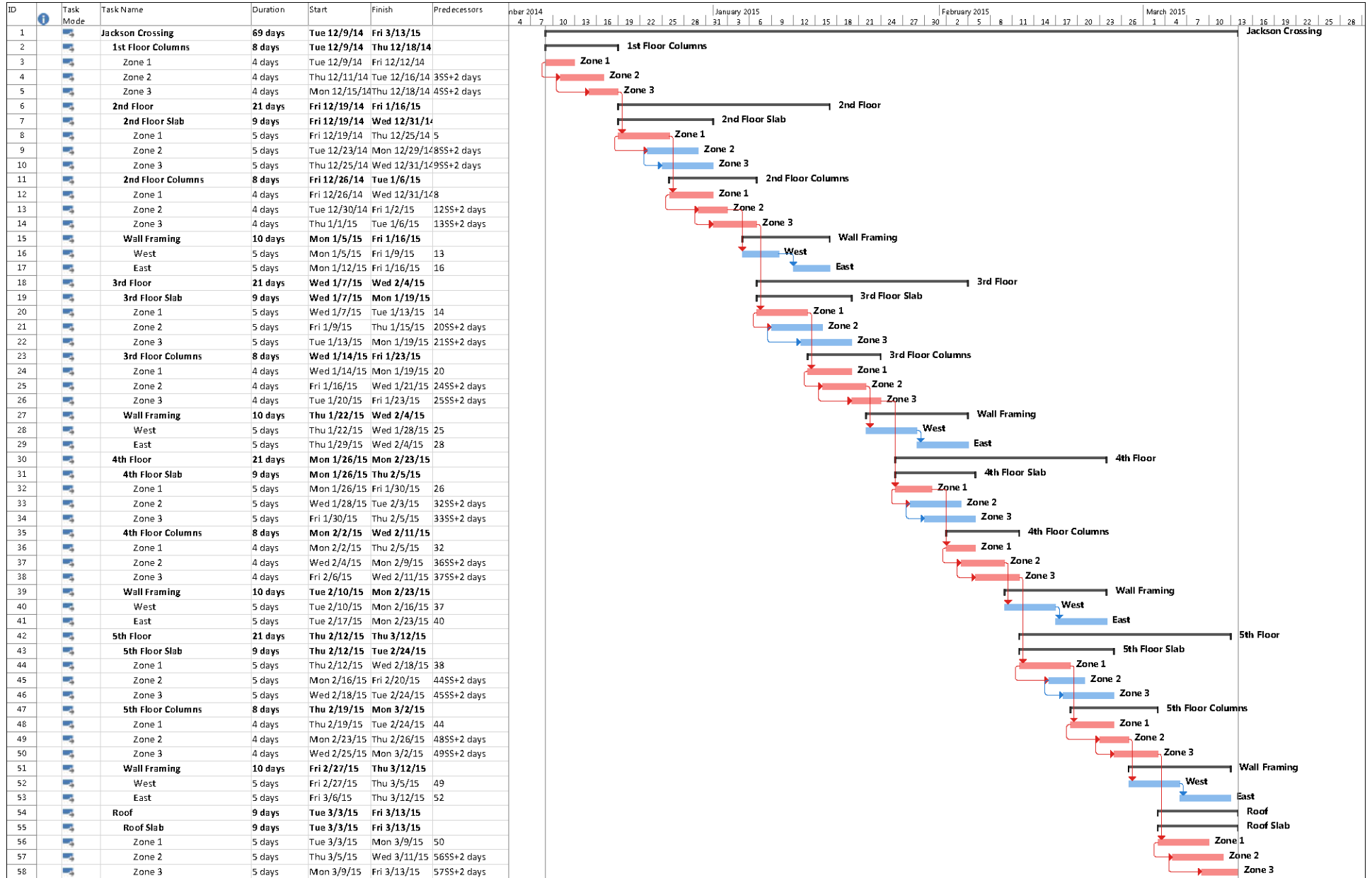
Formwork: 2 days

Reinforcement: 2 days

Pour: 1 days

According to RSMMeans the daily output for placing concrete is 160 cubic yards. Considering that the 2nd, 3rd, and 4th floors have a total volume of 392.5 cubic yards, the placing of the concrete was split into 3 zones. The forming of each new zone started after the previous zone finished. The construction of the columns were also split into three zones as their progress depends on the progress of the slabs.

Figure 44 – Schedule of Redesigned Structure



Project: Schedule
Date: Tue 4/5/16

Task	Summary	Inactive Milestone	Duration-only	Start-only	External Milestone	Critical Split
Split	Project Summary	Inactive Summary	Manual Summary Rollup	Finish-only	Deadline	Progress
Milestone	Inactive Task	Manual Task	Manual Summary	External Tasks	Critical	Manual Progress

Figure 44 is a schedule of the redesigned structure detailing the duration to construct the elevated slabs above the ground floor. The total duration of this schedule was 69 days; this is an increase of 24 days along the critical path compared to the wood framing. The tasks from Figure 44 colored in red impact the critical path.

4 Mechanical Breadth

The mechanical breadth will investigate the impact of the redesigned structure on the acoustical performance between floors. The Sound Transmission Class, STC, of a floor and ceiling system is required by code to meet a minimum value to ensure the comfort of the occupants. In addition, a minimum Impact Insulation Class, IIC, is required by code to minimize the impact of sounds such as footsteps between structural floor systems.

Jackson Crossing was originally designed to meet the standards of the *2009 Virginia Uniform Statewide Building Code*. Even though this is not the current adopted building code in Alexandria, the current adopted code, 2012, does not change the STC and IIC required values.

According to Section 1207.2 of the *2012 Virginia Construction Code*, any floor and ceiling assembly must have an STC value greater than or equal to 50. If the STC is field tested, it is only required to equal or exceed 45. In Marshall Long's book, *Architectural Acoustics*, STC ratings can be compared as follows:

Minimum Code – 50 STC
Minimum Quality – 55 STC
Medium Quality – 60 STC
High Quality – 65 STC

The STC rating of the existing floor system was estimated with the aid of the book, *Architectural Acoustics: Principles and Design*. This STC rating is in Figure 45(on next page); a wood truss joist system has an STC rating of 52 which meets code requirements.

The STC rating of an 8" slab is in Figure 46(on next page); this STC rating of 58 is beyond the minimum code requirement and can be considered minimum quality. With the addition of 2x2 wood furring and 5/8" GWB, the rating can be increased to medium quality with an STC of 63.

Figure 45 – STC Rating of Wood Truss Joists

1-1/2" gyp. Conc. On 3/4" plywood on 11" wood truss joists, 5/8" GB ceiling on resilient channels					
STC	52				
1/3 Octave-Band Frequency (Hz)	Adjustment for Contour Level (dB)	Contour Level (dB)	TL (dB)	Deficiency (dB)	Max Deficiency ≤ 8 dB?
125	-	36	30	6	OK
160	-	39	37	2	OK
200	-	42	43	0	OK
250	-	45	44	1	OK
300	-	48	48	0	OK
400	-	51	51	0	OK
500	-	52	52	0	OK
630	-	53	48	5	OK
800	-	54	49	5	OK
1000	-	55	52	3	OK
1250	-	56	55	1	OK
1600	-	56	57	0	OK
2000	-	56	58	0	OK
2500	-	56	55	1	OK
3150	-	56	61	0	OK
4000	-	56	64	0	OK
			Total	24	
			Wall is STC:		Yes

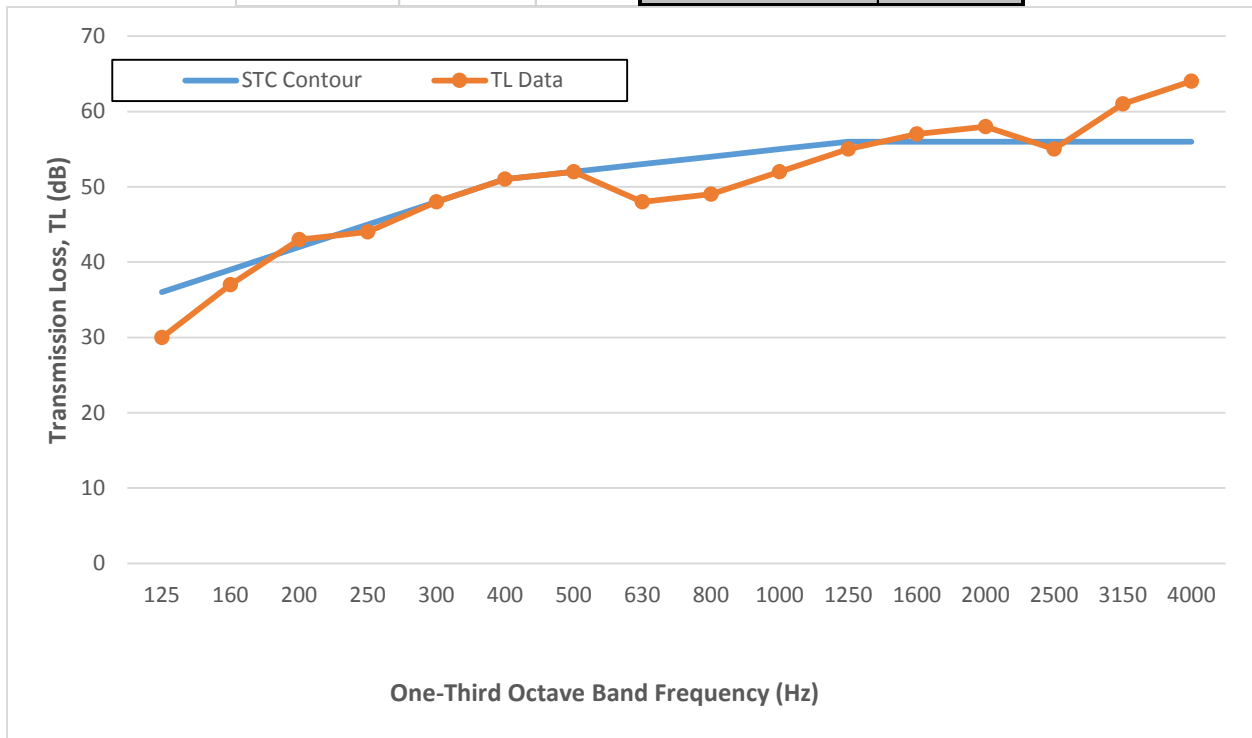
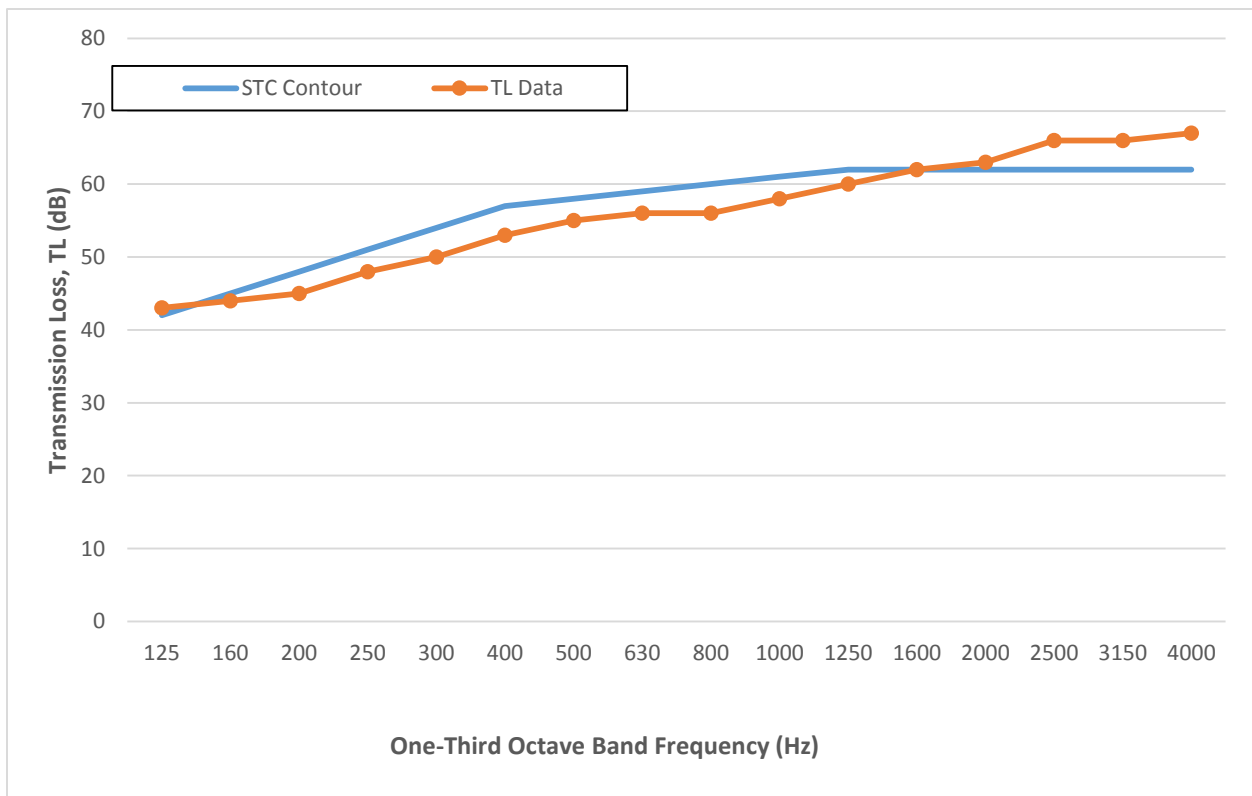


Figure 46 – STC Rating of Concrete Slab

8' Concrete Slab					
STC	58				
1/3 Octave-Band Frequency (Hz)	Adjustment for Contour Level (dB)	Contour Level (dB)	TL (dB)	Deficiency (dB)	Max Deficiency ≤ 8 dB?
125	-	42	43	0	OK
160	-	45	44	1	OK
200	-	48	45	3	OK
250	-	51	48	3	OK
300	-	54	50	4	OK
400	-	57	53	4	OK
500	-	58	55	3	OK
630	-	59	56	3	OK
800	-	60	56	4	OK
1000	-	61	58	3	OK
1250	-	62	60	2	OK
1600	-	62	62	0	OK
2000	-	62	63	0	OK
2500	-	62	66	0	OK
3150	-	62	66	0	OK
4000	-	62	67	0	OK
			Total	30	
			Wall is STC:		Yes



According to Section 1207.3 of the *2012 Virginia Construction Code*, any floor and ceiling assembly must have an IIC value greater than or equal to 50. If the IIC is field tested, it is only required to equal or exceed 45. This requirement is important as the sound of impacts on the floor system can be a nuisance to the occupants below the floor.

Architectural Acoustics: Principles and Design states that a 6 in. thick slab with no floor covering has an IIC rating of 25. This is well below the code required rating of 50. The IIC rating can be easily increased to 85 with the installation of carpet and padding; a rating of 85 meets and exceeds code requirements.

There are other methods besides carpet and padding than can bring the IIC rating of a concrete slab above the code minimum. A floating floor resting on wood furring strips and a continuous layer of compressed fiber glass has an IIC rating of 63. An additional 2 inch concrete layer floated on a layer of compressed fiber glass has an even higher IIC rating of 71.

5 Closing Remarks

The redesigned structure is an alternative to the existing wood framing on podium slab of Jackson Crossing. While wood framing is very constructible and cost effective, an alternate structure incorporating two-way reinforced concrete slabs and lateral moment frames is worth exploring.

For the gravity system, 14" by 14" columns were designed with at most 4 #8 bars for longitudinal reinforcing. The two-way slabs these columns support were determined to be 8" thick with a bottom mat of #5 bars at 12" each way in addition to top reinforcing. The existing transfer beams at the ground floor level were determined to be adequate with their current dimensions even with the additional weight of a concrete structure; additional top bars and stirrups were needed to maintain the same cross section.

The lateral system was designed with 16" by 24" beams and columns. Seismic was ultimately the controlling lateral force. The frames in the north-south direction required greater reinforcement than in the other direction because of their shorter length. The moment frames were found to deflect within acceptable limits.

Though the redesign increases structural costs by 22% and adds 24 days to the schedule, the concrete slab provides a respectable STC rating and IIC rating provided that a carpet and pad is installed.

A two-way reinforced concrete slab system is a reasonable alternative for the structure of Jackson Crossing. Although it increases cost and time to the construction, the redesigned system is durable and has a smaller floor depth when compared to the existing wood framed structure.

Works Cited

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Appendix

A.1 Gravity Column Loads

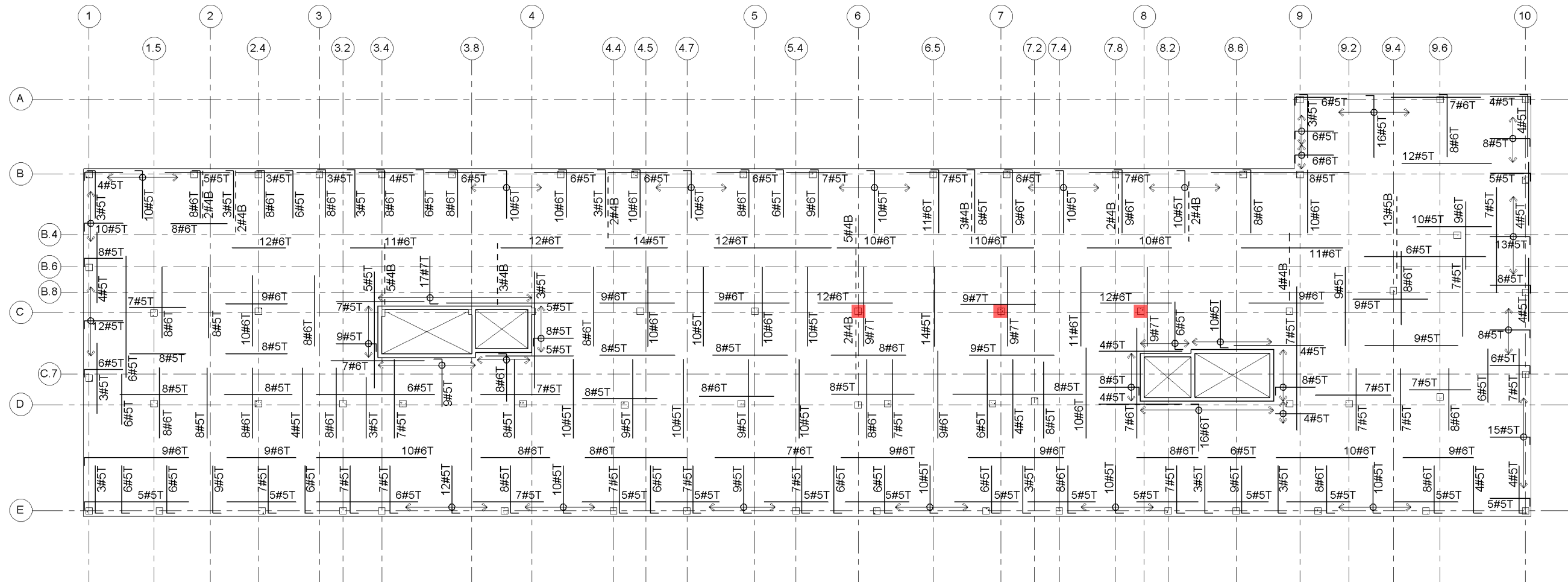
Column	Load Combination	Axial	Shear		Moment	
		P(k)	V2 (k)	V3 (k)	M2 (k-ft)	M3 (k-ft)
C42	1.2D+1.6L+0.5S	524.70	-0.35	-4.24	30.14	10.04
C39	1.2D+1.6L+0.5S	516.78	-0.75	-0.30	6.11	10.85
C45	1.2D+1.6L+0.5S	465.11	-0.81	-0.73	8.70	14.53
C37	1.2D+1.6L+0.5S	430.29	-0.44	0.41	1.77	7.34
C52	1.2D+1.6L+0.5S	397.67	-0.53	-0.01	4.32	5.51
C23	1.2D+1.6L+0.5S	386.28	0.91	-0.91	17.13	-10.67
C35	1.2D+1.6L+0.5S	384.66	0.51	-0.08	3.68	0.33
C38	1.2D+1.6L+0.5S	377.27	-0.50	-0.39	6.62	6.63
C55	1.2D+1.6L+0.5S	373.61	-0.02	-0.36	5.39	1.02
C3	1.2D+1.6L+0.5S	368.86	-0.82	0.00	10.92	-7.16
C29	1.2D+1.6L+0.5S	356.19	0.51	-0.70	3.54	-7.35
C69	1.2D+1.6L+0.5S	355.20	0.16	4.85	-26.34	6.90
C24	1.2D+1.6L+0.5S	351.40	-0.88	3.69	-11.77	0.87
C28	1.2D+1.6L+0.5S	344.68	-0.09	0.14	1.25	-2.65
C46	1.2D+1.6L+0.5S	343.25	0.26	-1.01	9.38	8.02
C54	1.2D+1.6L+0.5S	340.82	-0.65	0.53	1.05	4.96
C2	1.2D+1.6L+0.5S	326.15	0.30	-0.78	12.98	-14.13
C25	1.2D+1.6L+0.5S	323.13	2.31	-3.91	35.22	-17.36
C43	1.2D+1.6L+0.5S	322.74	0.90	1.04	-0.48	4.28
C15	1.2D+1.6L+0.5S	315.22	-0.43	1.43	10.72	-30.98
C26	1.2D+1.6L+0.5S	305.15	-1.04	0.18	5.59	3.10
C58	1.2D+1.6L+0.5S	297.81	0.12	0.44	0.54	-1.20
C47	1.2D+1.6L+0.5S	259.96	-0.05	0.31	1.33	10.60
C56	1.2D+1.6L+0.5S	253.38	-0.53	-0.33	6.26	3.54
C22	1.2D+1.6L+0.5S	251.20	0.30	0.96	5.49	-8.54
C40	1.2D+1.6L+0.5S	248.19	0.28	-0.57	6.68	4.47
C68	1.2D+1.6L+0.5S	244.98	-0.77	-0.70	0.93	0.42
C20	1.2D+1.6L+0.5S	243.59	-0.44	1.29	-2.28	-30.93
C17	1.2D+1.6L+0.5S	242.05	-0.11	1.15	7.09	-32.97
C19	1.2D+1.6L+0.5S	241.85	0.30	1.27	1.08	-35.50
C57	1.2D+1.6L+0.5S	241.59	0.10	-0.13	3.99	-1.79
C18	1.2D+1.6L+0.5S	240.42	-0.70	1.28	3.06	-29.31
C9	1.2D+1.6L+0.5S	236.85	0.78	-1.17	28.18	-16.23
C33	1.2D+1.6L+0.5S	231.40	-0.75	-0.02	2.12	5.28
C53	1.2D+1.6L+0.5S	231.39	1.07	-0.40	8.31	-5.19

Gravity Column Loads con'd

Column	Load Combination	Axial	Shear		Moment	
		P(k)	V2 (k)	V3 (k)	M2 (k-ft)	M3 (k-ft)
C21	1.2D+1.6L+0.5S	228.41	0.19	0.86	3.87	-7.00
C41	1.2D+1.6L+0.5S	226.46	0.11	0.50	0.16	4.30
C60	1.2D+1.6L+0.5S	223.15	1.06	0.28	4.14	-6.38
C16	1.2D+1.6L+0.5S	221.22	-0.35	1.00	10.16	-31.49
C50	1.2D+1.6L+0.5S	212.21	-0.78	0.03	1.82	15.92
C27	1.2D+1.6L+0.5S	207.19	0.45	0.09	4.11	-5.91
C12	1.2D+1.6L+0.5S	204.22	-0.39	-0.88	17.84	-8.92
C66	1.2D+1.6L+0.5S	203.78	0.16	0.74	-1.30	-4.33
C6	1.2D+1.6L+0.5S	197.50	0.71	-0.29	10.87	66.62
C7	1.2D+1.6L+0.5S	194.09	-0.06	-0.30	13.32	71.38
C49	1.2D+1.6L+0.5S	192.31	-0.75	0.19	0.85	14.51
C4	1.2D+1.6L+0.5S	192.20	1.70	-0.20	16.16	-22.88
C14	1.2D+1.6L+0.5S	190.41	-0.51	-0.79	12.03	-8.22
C11	1.2D+1.6L+0.5S	190.36	0.37	-0.72	20.04	-13.69
C34	1.2D+1.6L+0.5S	186.87	-0.73	-0.16	2.92	6.39
C13	1.2D+1.6L+0.5S	182.15	0.53	-0.75	14.96	-14.66
C1	1.2D+1.6L+0.5S	175.08	-1.15	-0.51	8.18	-4.89
C10	1.2D+1.6L+0.5S	170.45	-0.18	-0.65	21.80	-10.26
C48	1.2D+1.6L+0.5S	164.60	-0.60	0.28	0.28	12.65
C32	1.2D+1.6L+0.5S	164.49	-0.60	0.57	-1.47	2.95
C30	1.2D+1.6L+0.5S	161.24	-0.39	0.79	-6.24	-3.48
C59	1.2D+1.6L+0.5S	158.37	0.76	-0.03	6.00	-5.33
C67	1.2D+1.6L+0.5S	155.27	1.13	-0.32	7.78	-9.76
C8	1.2D+1.6L+0.5S	145.65	0.31	-0.42	17.33	69.08
C5	1.2D+1.6L+0.5S	143.51	-0.82	-0.43	7.65	76.15
C31	1.2D+1.6L+0.5S	130.48	-0.60	-0.18	3.09	2.47
C65	1.2D+1.6L+0.5S	126.95	-0.82	0.37	-0.25	1.69
C44	1.2D+1.6L+0.5S	126.41	0.26	0.69	2.50	8.15
C61	1.2D+1.6L+0.5S	114.02	0.11	0.95	-2.30	-4.76
C62	1.2D+1.6L+0.5S	107.77	-0.04	0.43	2.18	-3.87
C51	1.2D+1.6L+0.5S	81.12	0.29	-0.28	8.39	10.55
C64	1.2D+1.6L+0.5S	76.87	0.39	0.53	2.65	-6.45
C63	1.2D+1.6L+0.5S	70.11	-0.52	-0.31	3.84	-0.90

A.2 Slab Reinforcement Plans

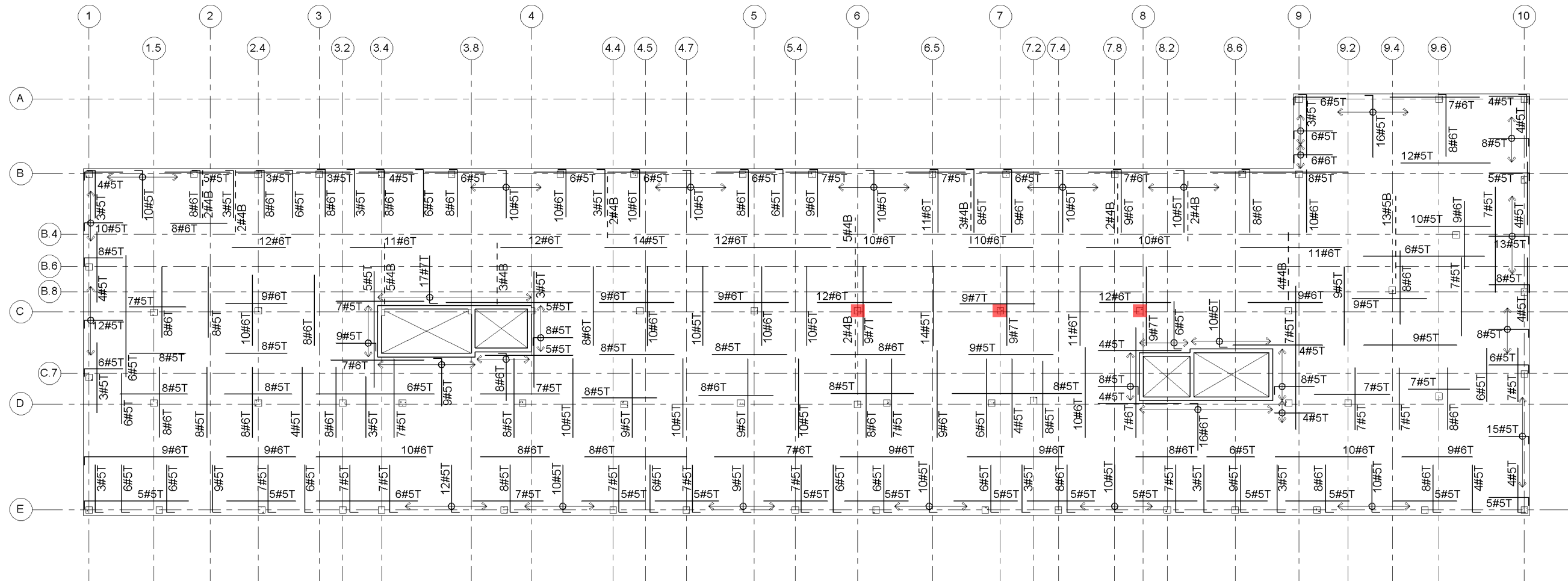
A.2.1 Second Floor Reinforcement Plan



*Reinforcement includes bottom mat of #5 at 12" o.c. in each direction

**Locations of 28"x28"x12" shear caps highlighted in red

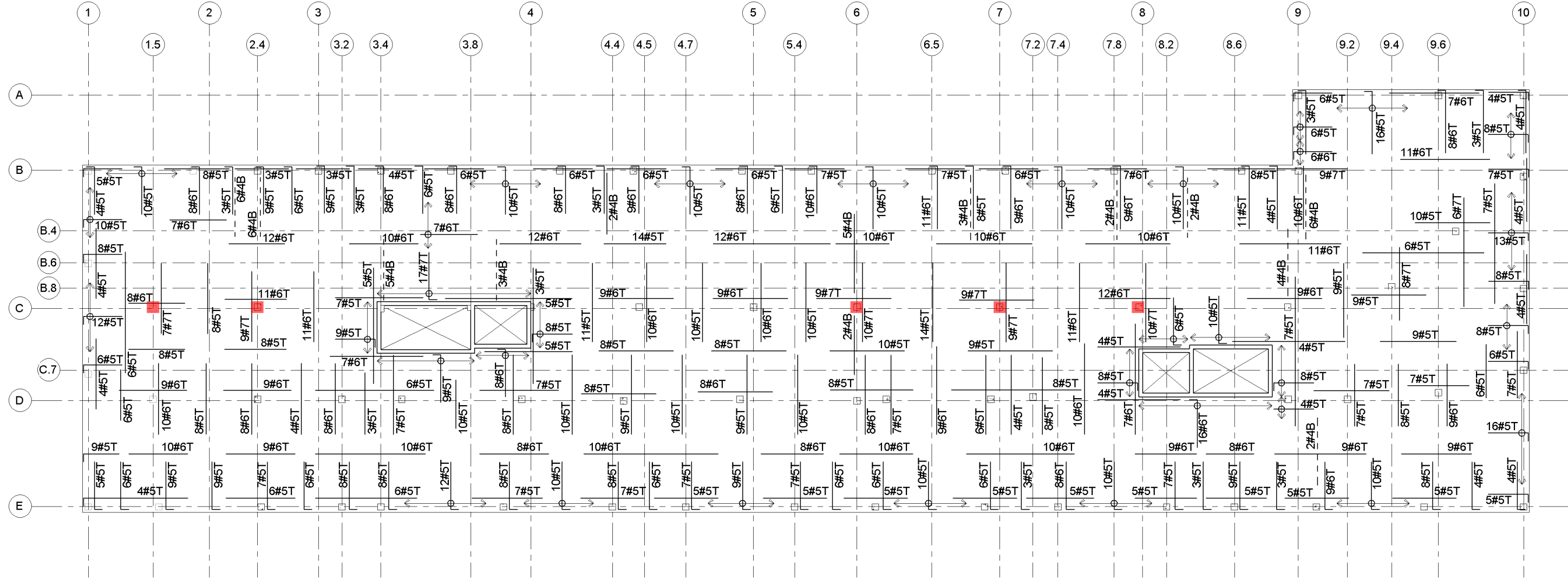
A.2.2 Third Floor Reinforcement Plan



*Reinforcement includes bottom mat of #5 at 12" o.c. in each direction

**Locations of 28"x28"x12" shear caps highlighted in red

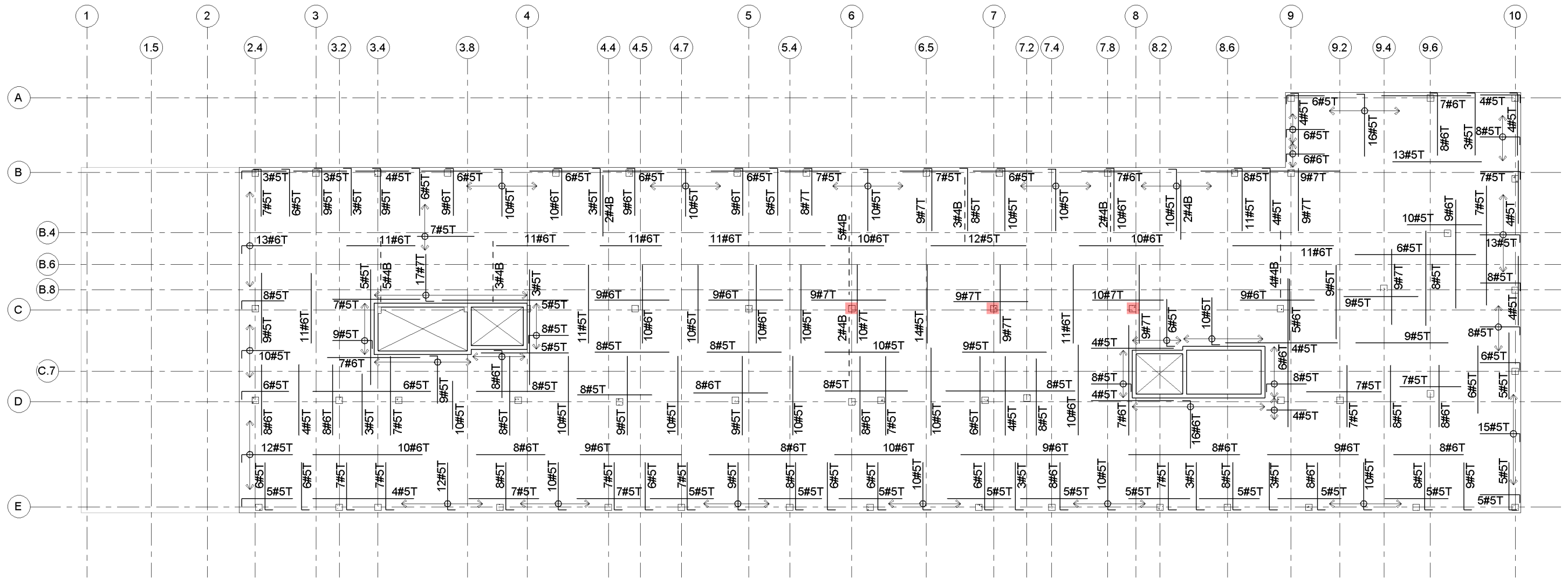
A.2.3 Fourth Floor Reinforcement Plan



*Reinforcement includes bottom mat of #5 at 12" o.c. in each direction

**Locations of 28"x28"x12" shear caps highlighted in red

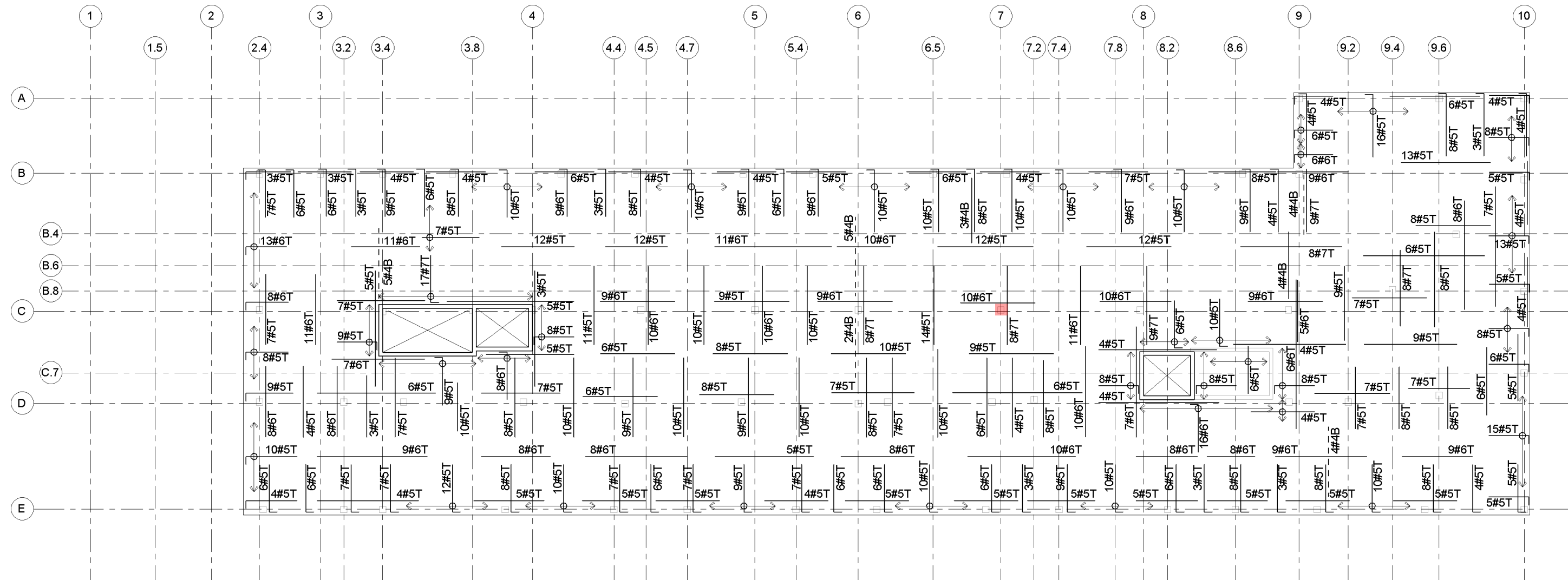
A.2.4 Fifth Floor Reinforcement Plan



*Reinforcement includes bottom mat of #5 at 12" o.c. in each direction

**Locations of 28"x28"x12" shear caps highlighted in red

A.2.5 Roof Reinforcement Plan

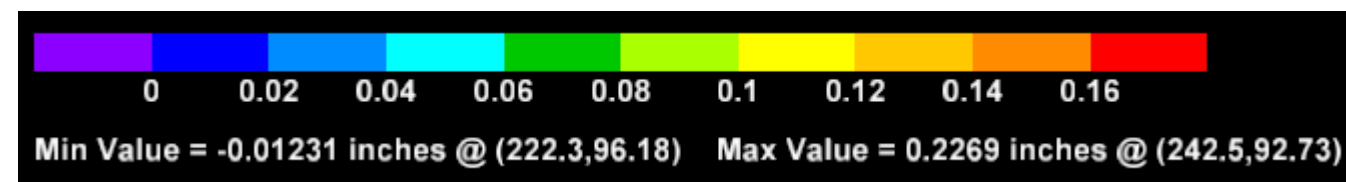
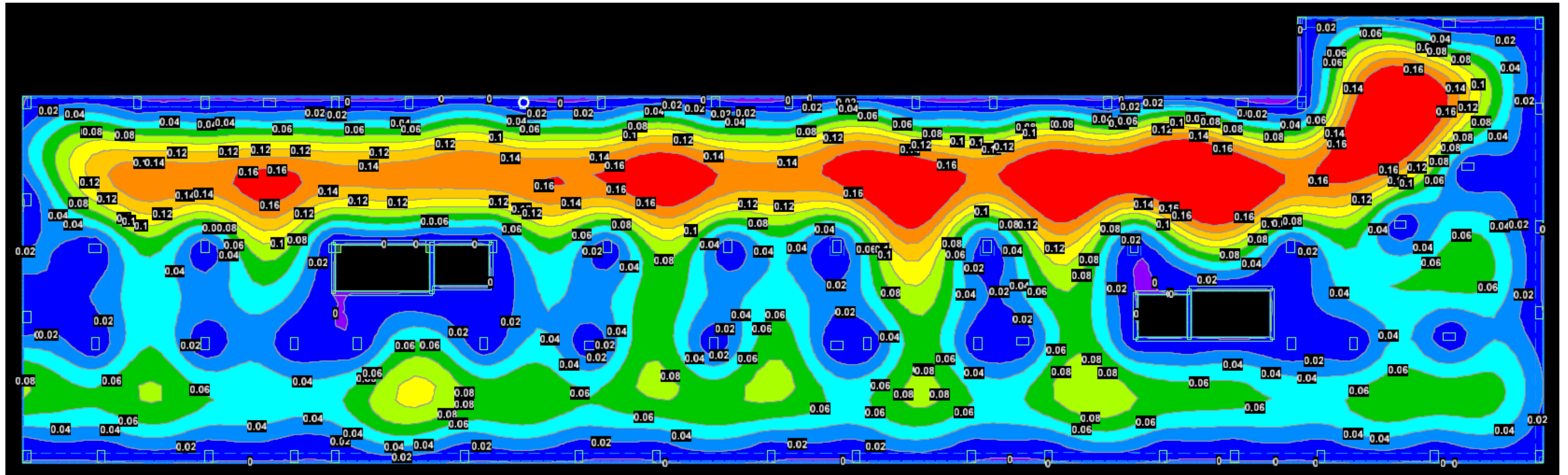


*Reinforcement includes bottom mat of #5 at 12" o.c. in each direction

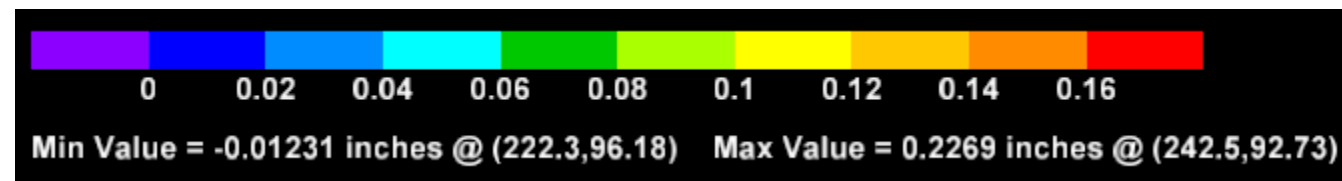
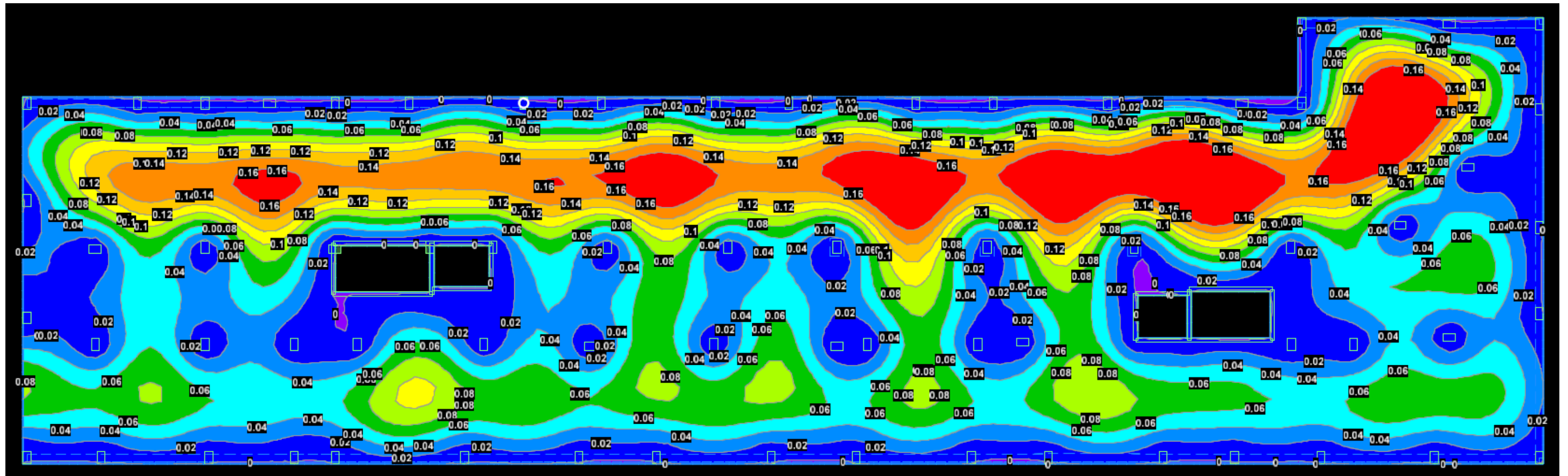
**Locations of 28"x28"x12" shear caps highlighted in red

A.3 Long Term Deflections

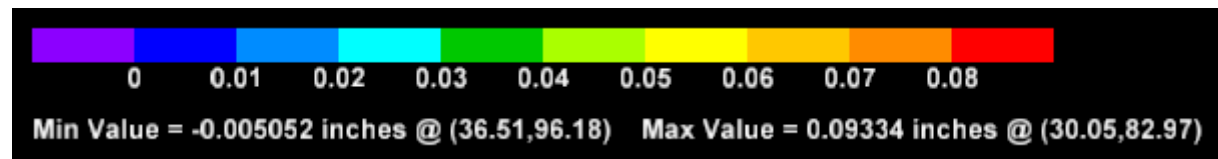
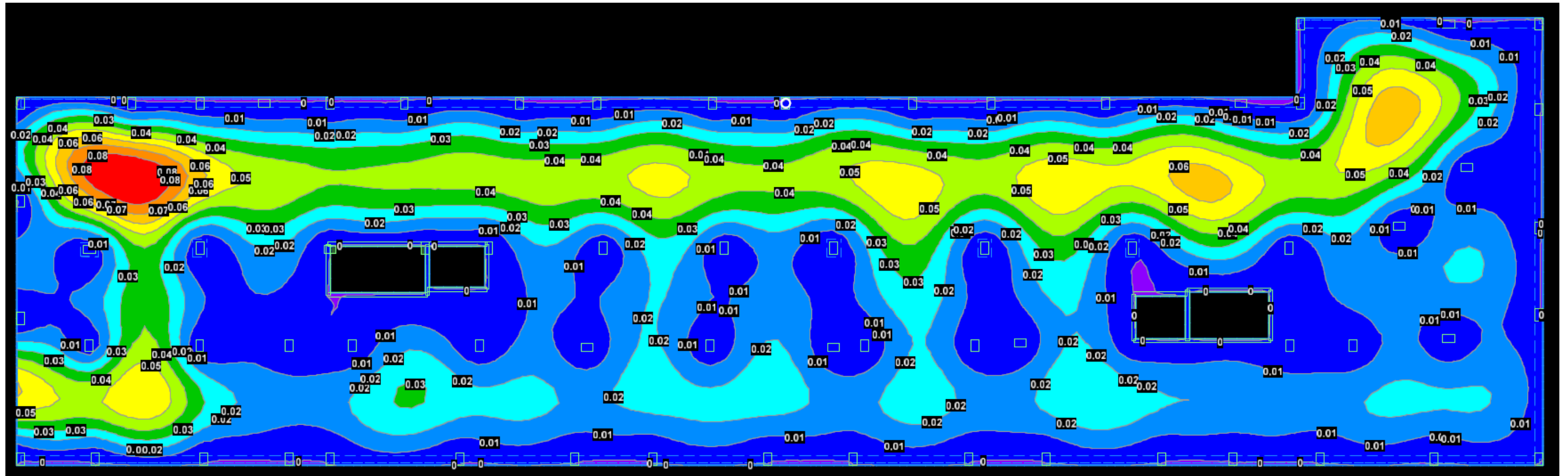
A.3.1 Second Floor Deflections



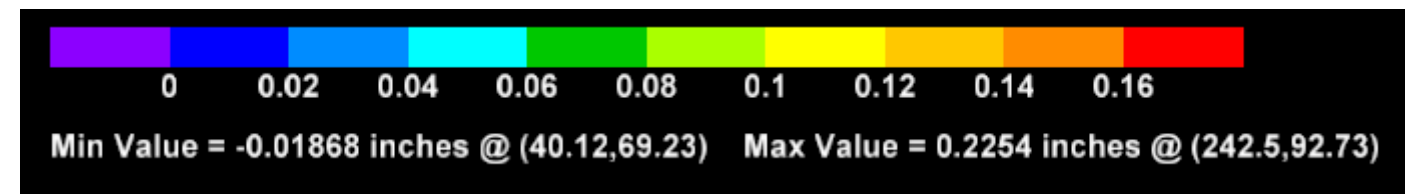
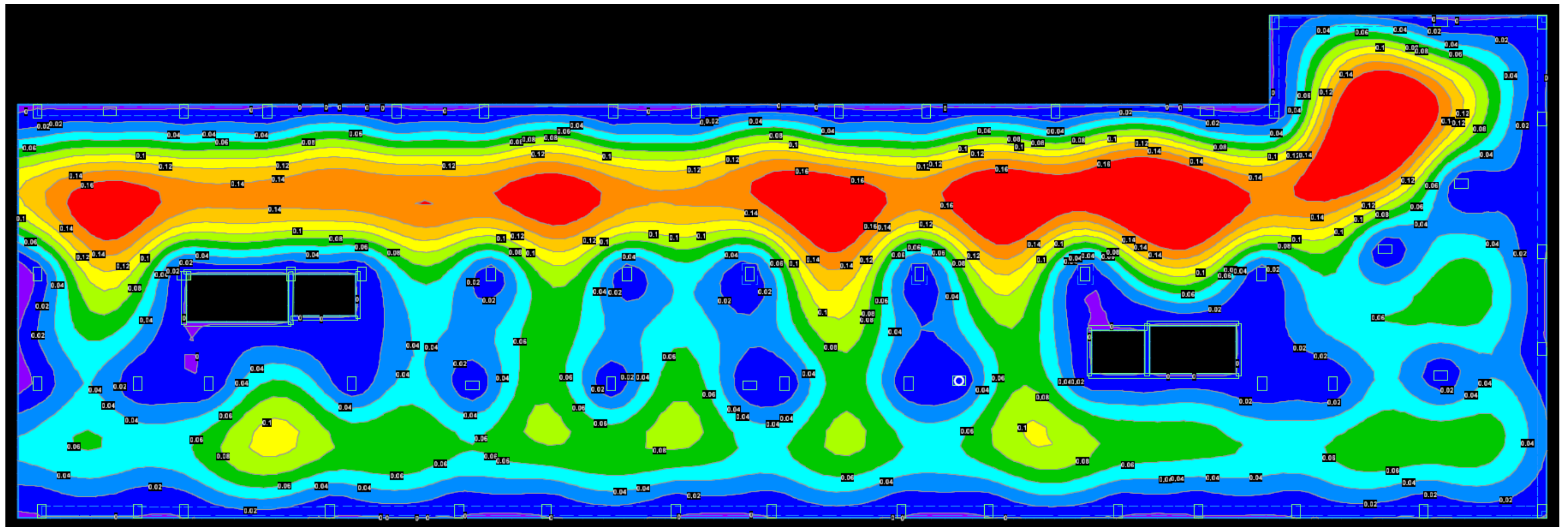
A.3.2 Third Floor Deflections



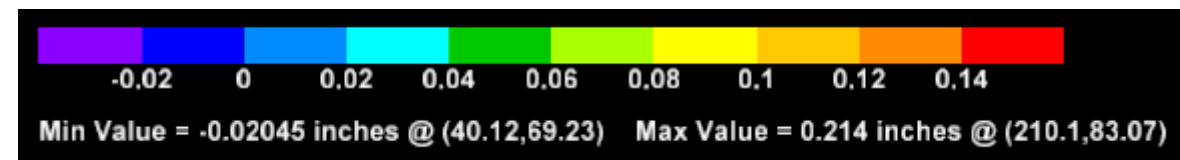
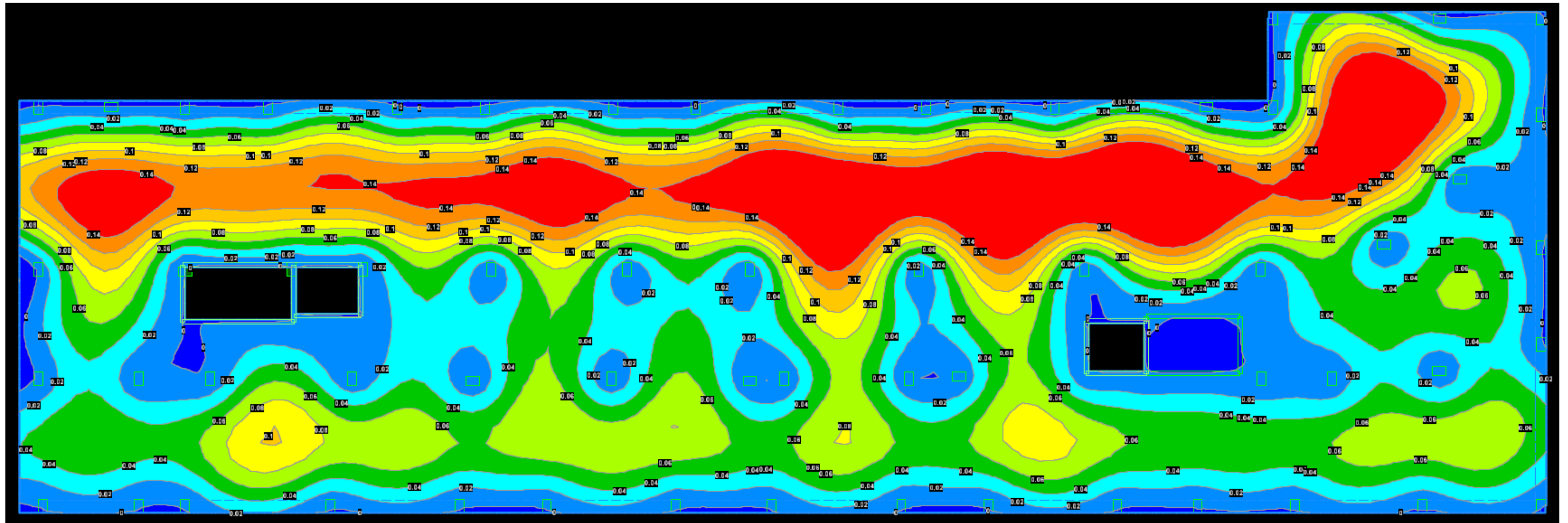
A.3.3 Fourth Floor Deflections



A.3.4 Fifth Floor Deflections



A.3.5 Roof Deflections

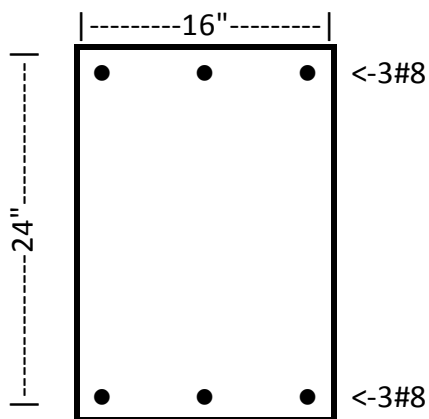


A.4 Moment Frame Column Design

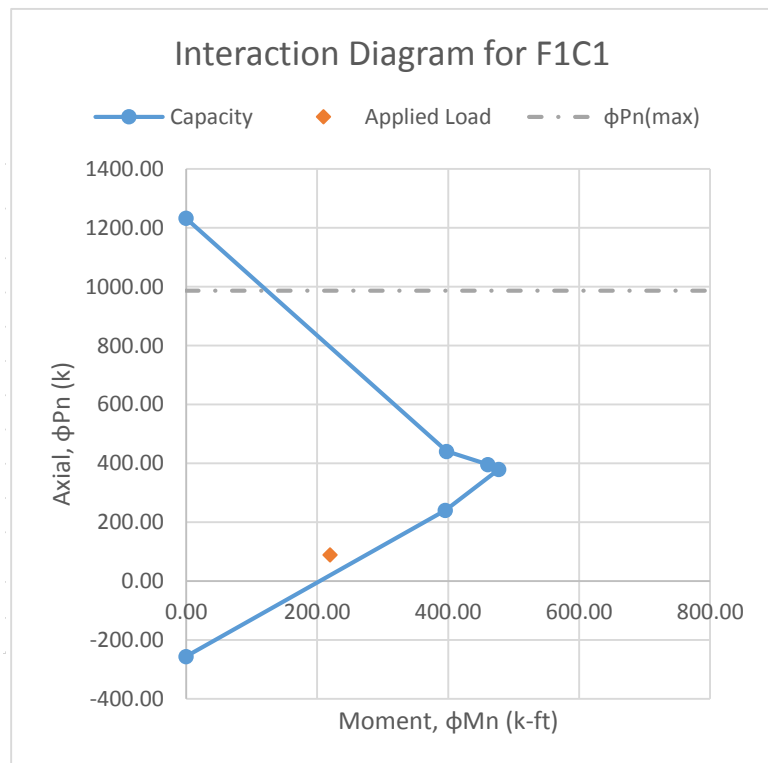
A.4.1 F1C1

Interaction Points		
Point	ϕP_n (k)	ϕM_n (k-ft)
ϕP_o (k)	1232.57	0.00
1	440.57	397.82
2	396.20	460.76
3	379.76	477.50
4	240.80	396.06
5	-255.96	0.00

Stirrups	
V_u (k)	37.8
ϕV_c (k)	40.5
Is V_u less than $0.5\phi V_c$?	No
Is V_u less than ϕV_c ?	Yes
16 longitudinal bar diameters:	16 in
48 tie diameters:	24 in
Least dimension of column:	16 in
Smin base off Avmin	
Smin1	28 in
Smin2	30 in
Final Spacing:	16 in



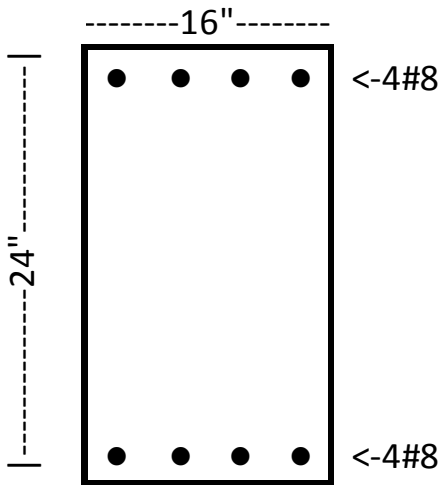
Applied Load P_u , M_u : (89.4k, 220.0k-ft)



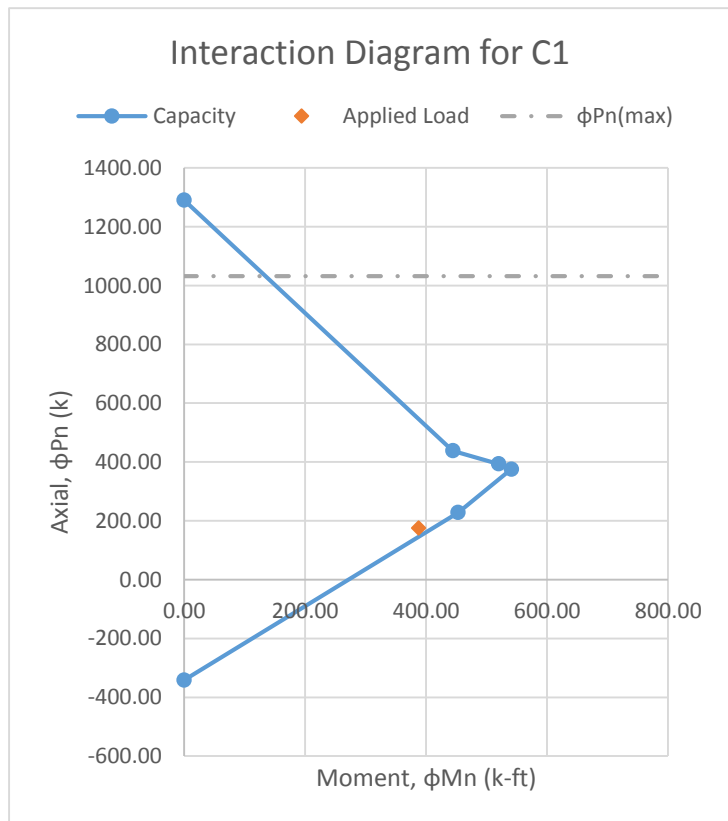
A.4.2 F3C3

Interaction Points		
Point	ϕP_n (k)	ϕM_n (k-ft)
ϕP_o (k)	1289.82	0.00
1	438.38	444.21
2	393.42	519.86
3	375.59	540.83
4	228.30	452.89
5	-341.28	0.00

Stirrups	
V_u (k)	65.8
ϕV_c	44.5
Is V_u less than $0.5\phi V_c$?	No
Is V_u less than ϕV_c ?	No
16 longitudinal bar diameters:	16 in
48 tie diameters:	24 in
Least dimension of column:	16 in
$V_u > \phi V_c$	
V_{sreq} (k)	28.30
s_{req} (in)	18.12
Final Spacing:	16 in



Applied Load P_u , M_u : (175.3k, 387.7k-ft)



A.5 Moment Frame Beam Design

A.5.1 Frame 1 and Frame 2

Top Bars				
	Mu (k-ft)	121.1		
	Vu (k)	33.2		
Dimensions				
b (in)	16		fy (ksi)	60
h (in)	24		f'c (psi)	5000
d (in)	22		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	2
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	1.287135		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	1.173333		
	a (in)	1.135707		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	1.26		
Pick	3	#6	As (in2)	1.32
	Abar (in2)	0.44		
	dbar (in)	0.75		
	Bar clr. Sp. (in)	5.375		
	dreal (in)	21.625		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	6.125		
Moment Capacity				
	a(in)	0.86		
	c (in)	1.08		
	εs	0.06		
ΦMn (k-ft)	125.89		Ok?	YES
FINAL DESIGN:		3	#6	

Bottom Bars				
	Mu (k-ft)	165.0		
	Vu (k)	33.2		
Dimensions				
b (in)	16		fy (ksi)	60
h (in)	24		f'c (psi)	5000
d (in)	22		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	2
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	1.754747		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	1.173333		
	a (in)	1.548307		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	1.73		
Pick	4	#6	As (in2)	1.76
	Abar (in2)	0.44		
	dbar (in)	0.75		
	Bar clr. Sp. (in)	3.333333		
	dreal (in)	21.625		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	4.083333		
Moment Capacity				
	a(in)	1.15		
	c (in)	1.44		
	εs	0.04		
ΦMn (k-ft)	166.72		Ok?	YES
FINAL DESIGN:		4	#6	

SHEAR			
ACI Eq. (11-3)			
	Vc (k)	48.93	
	0.5ΦVc(k)	18.35	
ACI 11.4.7.2, Vu>ΦVc			
ACI 11.4.7.9			
	Vsmax(k)	195.73	
	Vsreq (k)	-4.67	
	sreq	-111.25	
ACI 11.4.6.3, 0.5ΦVc<Vu<ΦVc			
	sreq1	28.28	
	sreq2	30	
=>	sreq	28.28427	
	smax	10.81	
	Sfinal (in)	10	

A.5.2 Frame 3 and Frame 4

Top Bars				
	Mu (k-ft)	287.8		
	Vu (k)	44.5		
Dimensions				
b (in)	16		fy (ksi)	60
h (in)	24		f'c (psi)	5000
d (in)	22		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	2
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	3.060074		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	1.173333		
	a (in)	2.700066		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	3.10		
Pick	4	#8	As (in2)	3.16
	Abar (in2)	0.79		
	dbar (in)	1		
	Bar clr. Sp. (in)	3		
	dreal (in)	21.5		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	4		
Moment Capacity				
	a(in)	2.07		
	c (in)	2.59		
	εs	0.02		
ΦMn (k-ft)	290.98		Ok?	YES
FINAL DESIGN:		4	#8	

Bottom Bars				
	Mu (k-ft)	211.6		
	Vu (k)	44.5		
Dimensions				
b (in)	16		fy (ksi)	60
h (in)	24		f'c (psi)	5000
d (in)	22		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	2
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	2.249819		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	1.173333		
	a (in)	1.985135		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	2.24		
Pick	6	#6	As (in2)	2.64
	Abar (in2)	0.44		
	dbar (in)	0.75		
	Bar clr. Sp. (in)	1.7		
	dreal (in)	21.625		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	2.45		
Moment Capacity				
	a(in)	1.72		
	c (in)	2.15		
	εs	0.03		
ΦMn (k-ft)	246.67		Ok?	YES
FINAL DESIGN:		6	#6	

SHEAR			
ACI Eq. (11-3)			
	Vc (k)	48.65	
	0.5ΦVc(k)	18.24	
ACI 11.4.7.2, Vu>ΦVc			
ACI 11.4.7.9			
	Vsmax(k)	194.60	
	Vsreq (k)	10.68	
=>	sreq	48.29	
ACI 11.4.6.3, 0.5ΦVc<Vu<ΦVc			
	sreq1	28.28	
	sreq2	30	
	sreq	28.28427	
	smax	10.75	
	Sfinal (in)	10	

A.6 Transfer Beam Design

A.6.1 Transfer Beam T1

Top Bars				
	Mu (k-ft)	1985.2		
	Vu (k)	335.6		
Dimensions				
b (in)	48		fy (ksi)	60
h (in)	36		f'c (psi)	5000
d (in)	34		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	4
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	13.65807		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	5.44		
	a (in)	4.017078		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	13.79		
Pick	12	#10	As (in2)	15.24
	Abar (in2)	1.27		
	dbar (in)	1.27		
	Bar clr. Sp. (in)	2.705455		
	dreal (in)	33.365		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$	s (in)	10		
$s \leq 12 \left(\frac{40000}{f_s} \right)$	s (in)	12		
	Sact (in)	3.975455		
Moment Capacity				
	a(in)	6.45		
	c (in)	8.06		
	es	0.01		
ΦMn (k-ft)	2067.05		Ok?	YES
FINAL DESIGN: 12 #10				

Bottom Bars				
	Mu (k-ft)	1659.2		
	Vu (k)	335.6		
Dimensions				
b (in)	48		fy (ksi)	60
h (in)	36		f'c (psi)	5000
d (in)	34		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	4
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,	As (in2)	11.41501		
$A_{s,min} = \frac{200b_w d}{f_y}$	Asmin (in2)	5.44		
	a (in)	3.357357		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$	Asreq (in2)	11.41		
Pick	14	#10	As (in2)	17.78
	Abar (in2)	1.27		
	dbar (in)	1.27		
	Bar clr. Sp. (in)	2.093846		
	dreal (in)	33.365		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$	s (in)	10		
$s \leq 12 \left(\frac{40000}{f_s} \right)$	s (in)	12		
	Sact (in)	3.363846		
Moment Capacity				
	a(in)	7.52		
	c (in)	9.40		
	es	0.01		
ΦMn (k-ft)	2368.57		Ok?	YES
FINAL DESIGN: 14 #10				

SHEAR		
ACI Eq. (11-3)		
	Vc (k)	226.49
	0.5ΦVc(k)	84.93
ACI 11.4.7.2, Vu>ΦVc		
ACI 11.4.7.9		
	Vsmax(k)	905.96
	Vsreq (k)	220.98
=>	sreq	7.25
ACI 11.4.6.3, 0.5ΦVc<Vu<ΦVc		
	sreq1	18.86
	sreq2	20
	sreq	18.85618
	smax	16.68
	Sfinal (in)	6

A.6.2 Transfer Beam T2

Top Bars				
	Mu (k-ft)	1070.8		
	Vu (k)	264.5		
Dimensions				
b (in)	48		fy (ksi)	60
h (in)	36		f'c (psi)	5000
d (in)	34		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	4
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,		As (in2)	7.367045	
$A_{s,min} = \frac{200b_w d}{f_y}$		Asmin (in2)	5.44	
	a (in)	2.166778		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$		Asreq (in2)	7.23	
Pick	8	#9	As (in2)	8
	Abar (in2)	1		
	dbar (in)	1.128		
	Bar clr. Sp. (in)	5.139429		
	dreal (in)	33.436		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	6.267429		
Moment Capacity				
	a(in)	3.38		
	c (in)	4.22		
	εs	0.02		
ΦMn (k-ft)	1142.90		Ok?	YES
FINAL DESIGN:		8	#9	

Bottom Bars				
	Mu (k-ft)	998.2		
	Vu (k)	264.5		
Dimensions				
b (in)	48		fy (ksi)	60
h (in)	36		f'c (psi)	5000
d (in)	34		Beta1	0.8
Str. Diam. (in)	0.5		# of Str. Legs	4
Str. Bar Area (in)	0.2		clear cover to Str.	1.5
Trial As				
	$A_s \geq \frac{M_u}{\phi f_y (jd)}$			
With j=0.95,		As (in2)	6.867561	
$A_{s,min} = \frac{200b_w d}{f_y}$		Asmin (in2)	5.44	
	a (in)	2.019871		
$A_s \geq \frac{M_u}{\phi f_y (d - \frac{a}{2})}$		Asreq (in2)	6.72	
Pick	12	#9	As (in2)	12
	Abar (in2)	1		
	dbar (in)	1.128		
	Bar clr. Sp. (in)	2.860364		
	dreal (in)	33.436		
ACI Eq. 10-4				
	Cc (in)	2		
$s \leq 15 \left(\frac{40000}{f_s} \right) - 2.5c_c$			s (in)	10
$s \leq 12 \left(\frac{40000}{f_s} \right)$			s (in)	12
	Sact (in)	3.988364		
Moment Capacity				
	a(in)	5.07		
	c (in)	6.33		
	εs	0.01		
ΦMn (k-ft)	1668.74		Ok?	YES
FINAL DESIGN:		12	#9	

SHEAR		
ACI Eq. (11-3)		
	Vc (k)	226.97
	0.5ΦVc(k)	85.11
ACI 11.4.7.2, Vu>ΦVc		
ACI 11.4.7.9		
	Vsmax(k)	907.88
	Vsreq (k)	125.70
=>	sreq	12.77
ACI 11.4.6.3, 0.5ΦVc<Vu<ΦVc		
	sreq1	18.86
	sreq2	20
	sreq	18.85618
	smax	16.72
	Sfinal (in)	12