III. STRUCTURAL DEPTH

i. EXISTING STEEL STRUCTURAL SYSTEM

a. Foundation System

The foundation system begins with auger cast concrete piles as per the recommendation of the geotechnical engineer, John D. Hynes & Associates, Inc. The structural engineer was presented with the choice of several different diameters and depths of piles that would perform adequately in the given soil conditions. A 16" dia., 40' long pile was selected, with a bearing capacity of 85 tons; see Figure 3: Existing Steel Structural System Foundation Plan.

On top of these piles rest the pile caps of variant cross section with a depth of 3'-1" each; see Figure 10: Existing Steel Structural System Pile Cap Configurations. Upon the pile caps rest the 24"x24" concrete piers with 18"x18" steel baseplates ranging in thickness from 1" to 2-1/4" including 4-1" dia. A325N anchor bolts. Finally, the basement slab on-grade is a 4" cast-in place concrete slab reinforced with 6x6 W2.9xW2.9 welded wire fabric; see Figure 3: Existing Steel Structural System Foundation Plan.

b. Framing System

The floor system for the Duncan Center typical on all floors is 5" composite slab with 2" 20 gage composite metal deck reinforced with 6x6 W2.0xW2.0 welded wire fabric. The deck is welded to the structural steel members beneath with composite beam action through 3/4" dia. x 4" long shear studs. The typical floor bay has spans of 27'-8"x24'-5" with the beams running in the long direction, W16x31 interior and W18x35 between columns, and girders running in the short direction, W24x55; see Figures 4, 5 & 6: Existing Steel Structural System 2^{nd}, 5^{th} & 6^{th} Floor Framing Plans.

c. Lateral Load Resisting System

The Lateral Load Resisting System is singularly comprised of the moment connected frame with flange welded/web bolted moment connections between the W18x35 beams between columns and W24x55 girder to the columns; see Figures 11 & 12: Existing Steel Structural System Column Flange & Column Web Moment Connection Details, respectively. Columns range in size from W12x45 to W12x132 and are spliced at the third and the fifth floor, see Figures 8 & 9: Existing

Steel Structural System Elevation Line A & Line 4, respectively.

d. Roof Framing

The roof framing is comprised of cold-formed steel roof trusses spaced at 24" o.c. for both the lower flat fifth floor roof and the arched sixth floor penthouse roof. The trusses rest on exterior structural steel girders, W16x26 typical at the fifth floor roof and W16x31 at the penthouse roof. Attached to trusses is 20 gage galvanized Type B roof deck; see Figure 7: Existing Steel Structural System Roof Framing Plan.

e. Foundation Plan

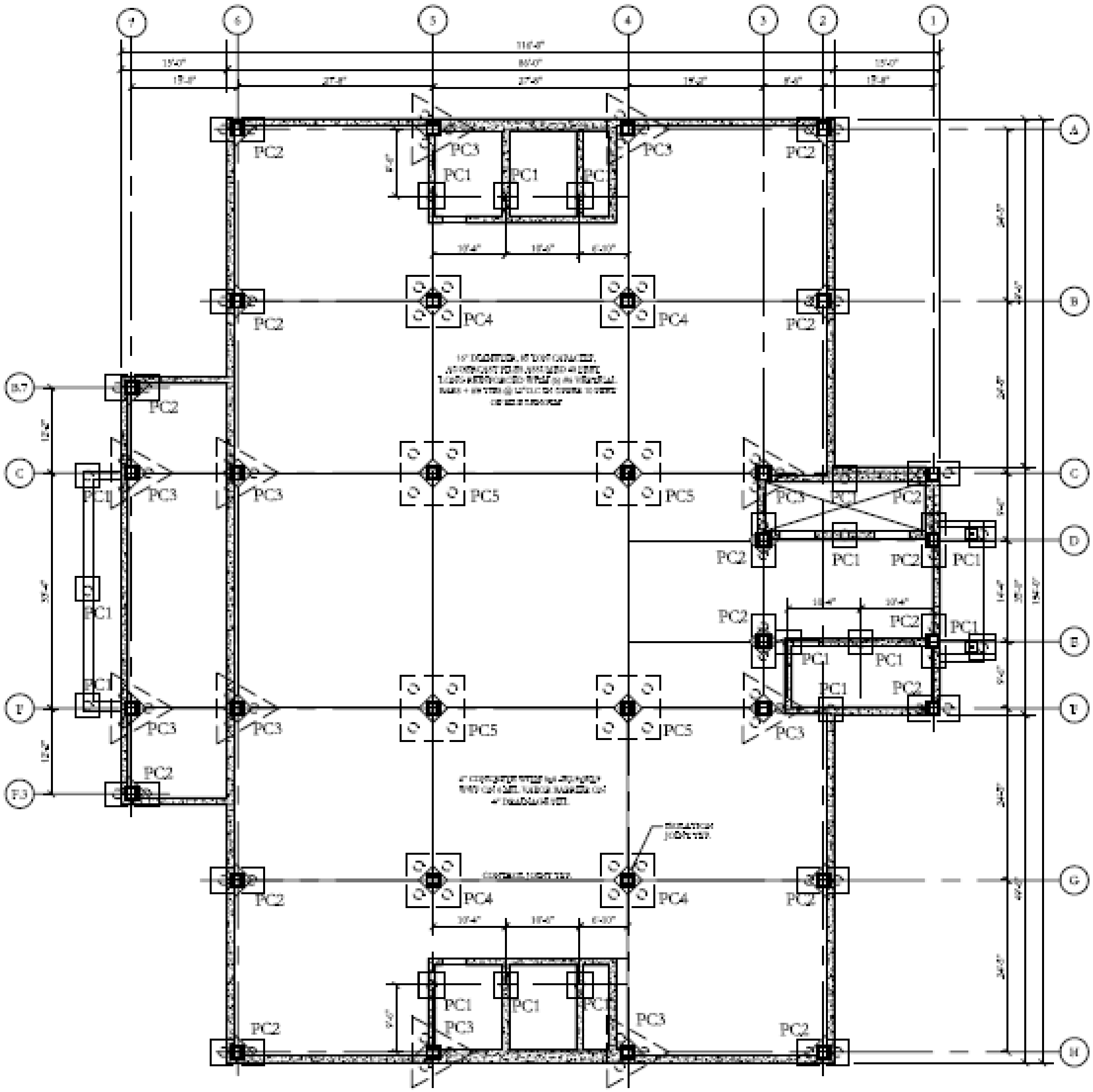


Figure 3: Existing Steel Structural System Foundation Plan

f. Framing Plans

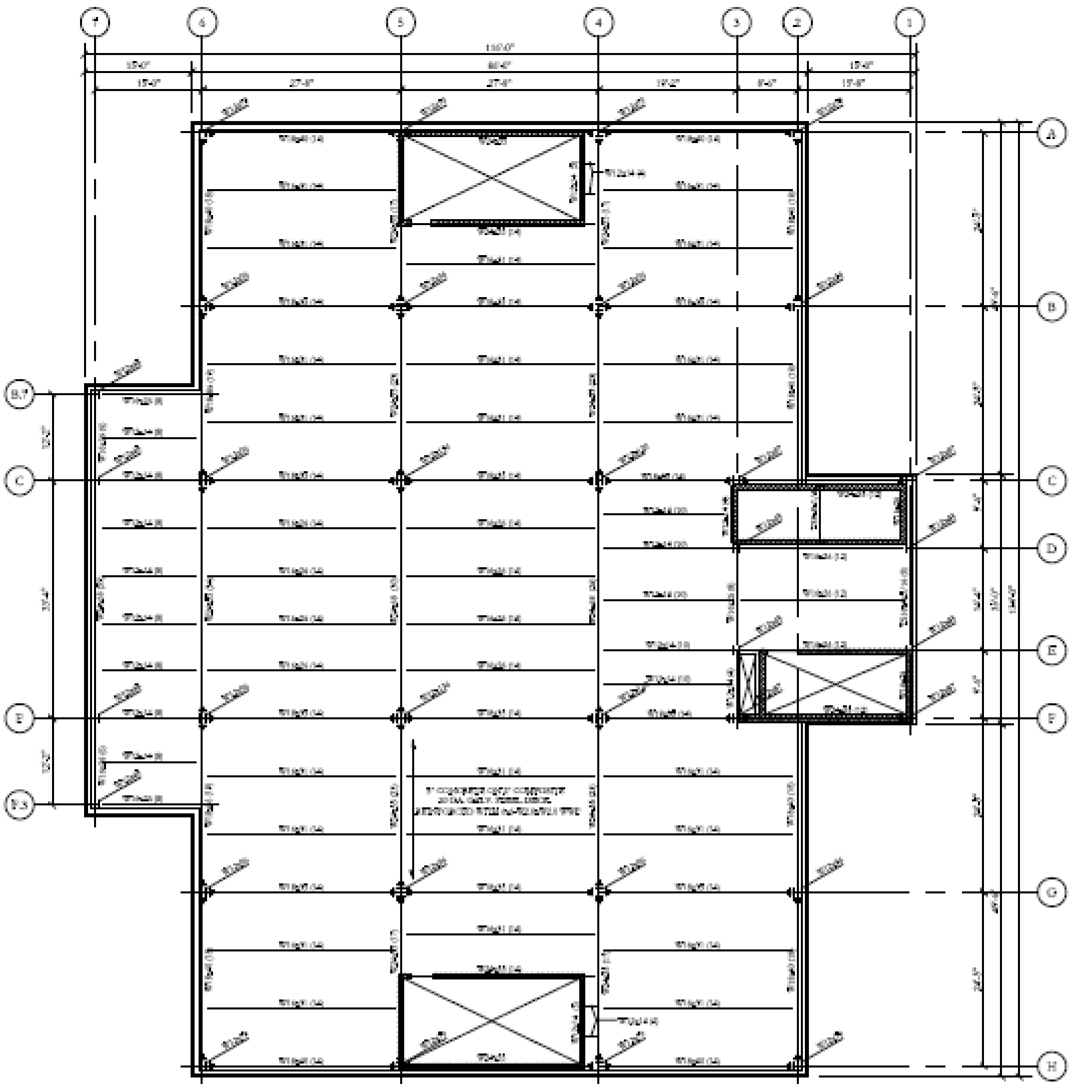


Figure 4: Existing Steel Structural System 2nd Floor Framing Plan

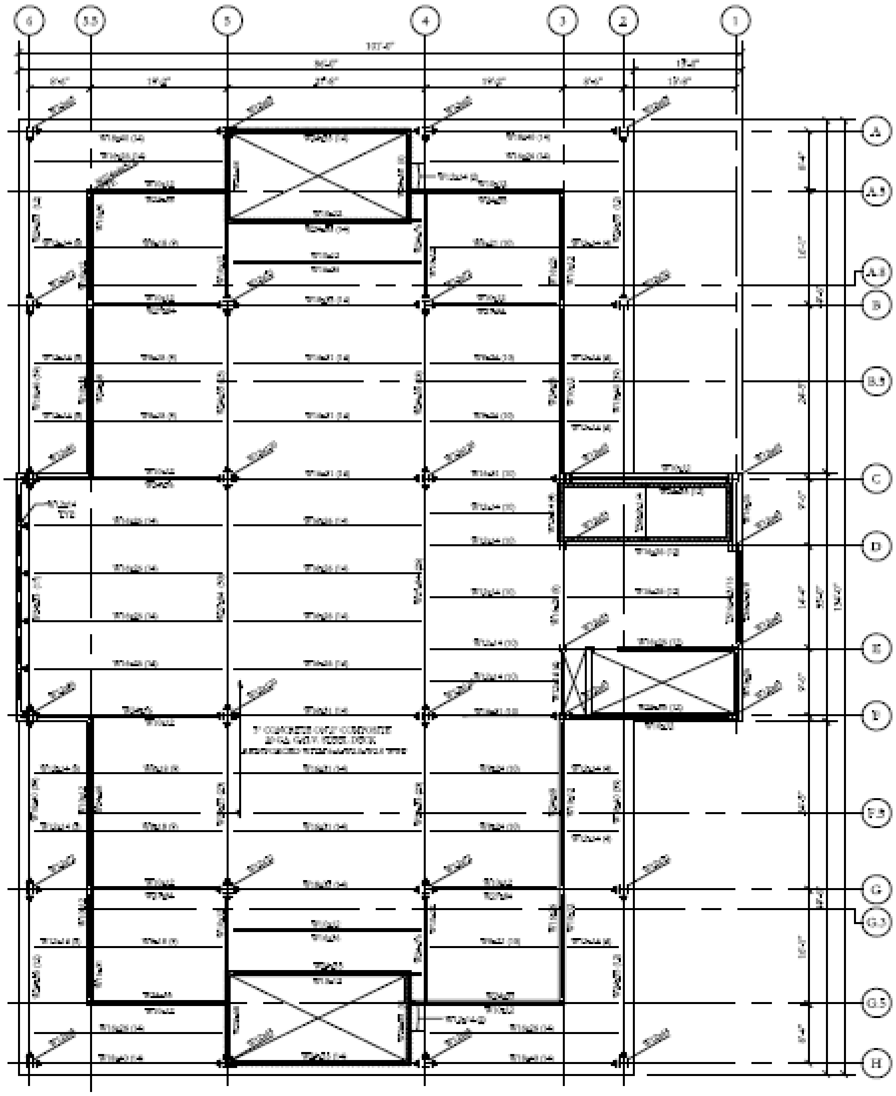


Figure 5: Existing Steel Structural System 5th Floor Framing Plan

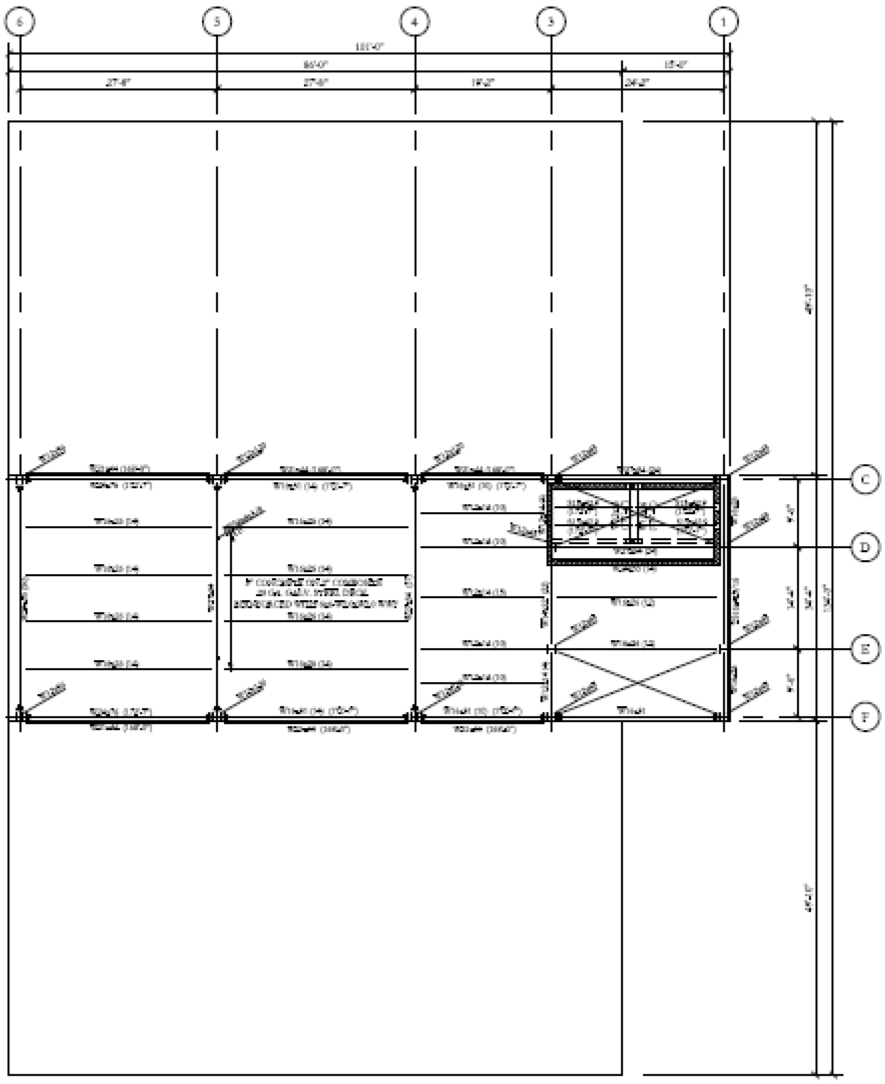


Figure 6: Existing Steel Structural System 6th Floor Framing Plan

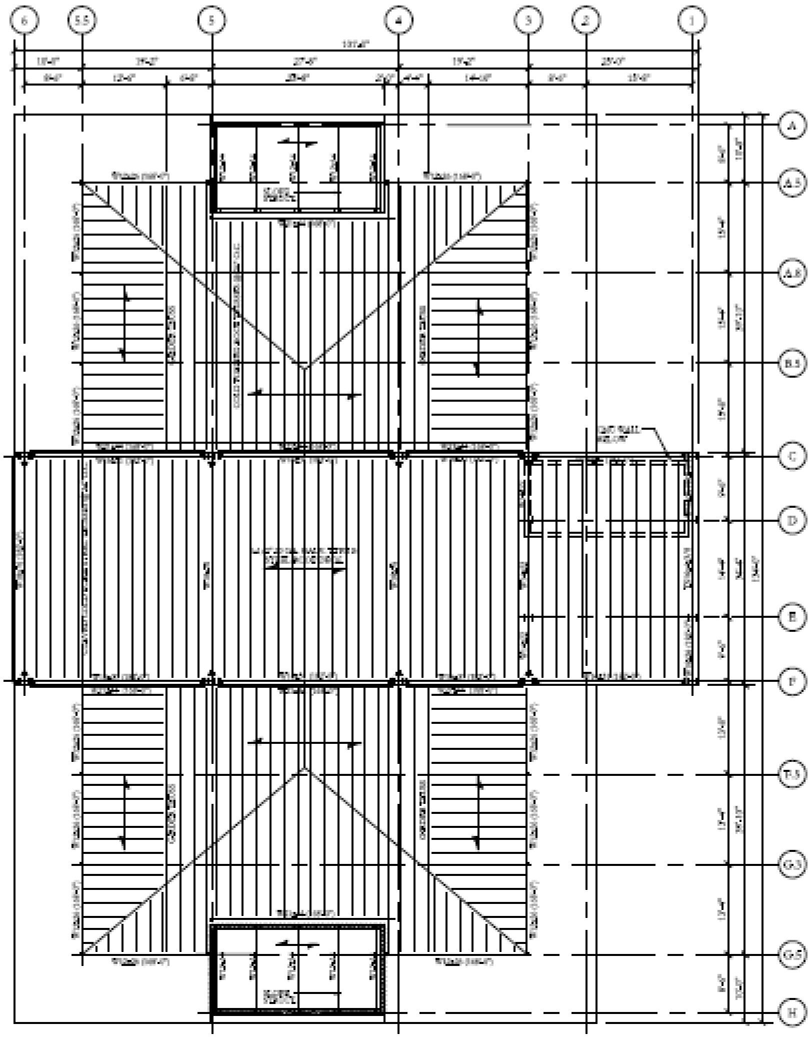


Figure 7: Existing Steel Structural System Roof Framing Plan

g. Elevations

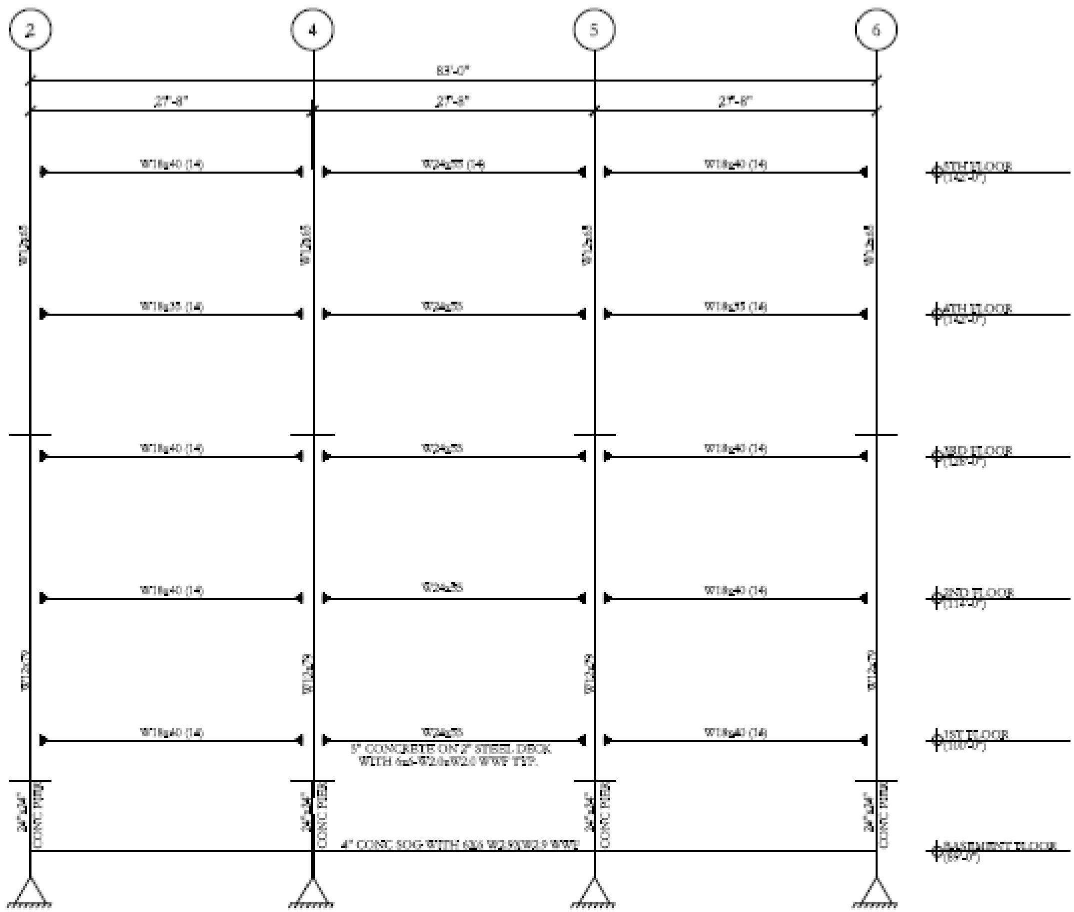


Figure 8: Existing Steel Structural System Elevation Line A

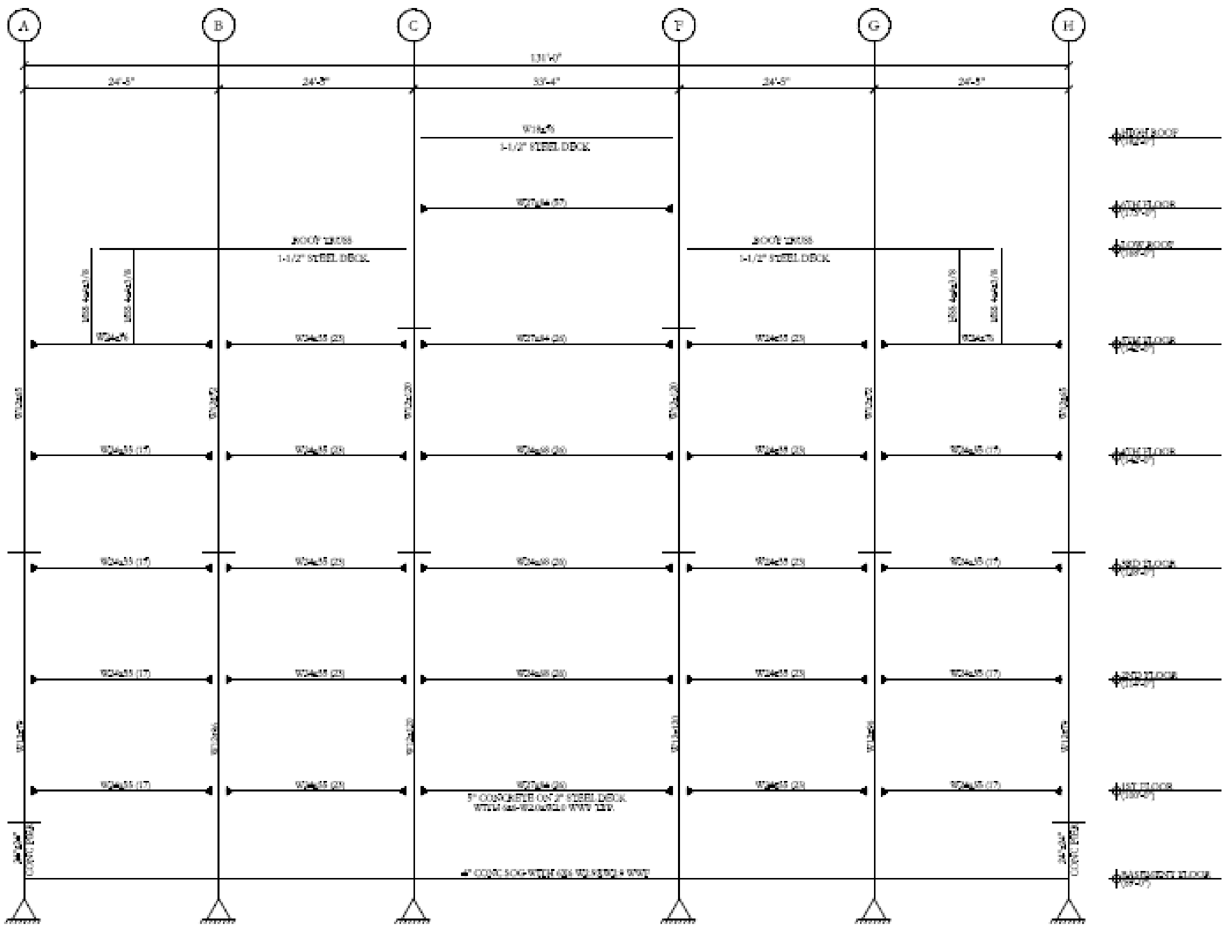


Figure 9: Existing Steel Structural System Elevation Line 4

h. Details

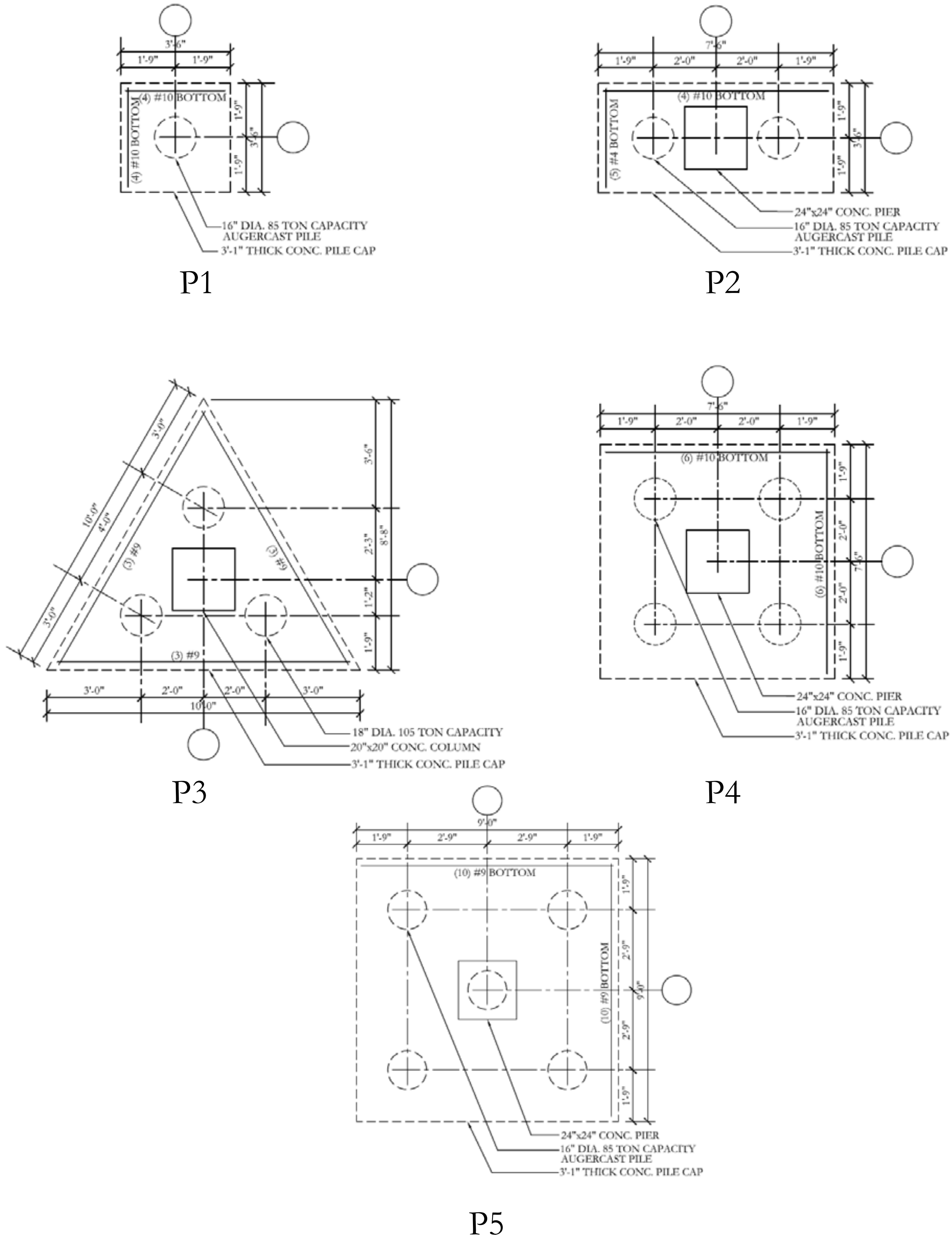


Figure 10: Existing Steel Structural System Pile Cap Configurations

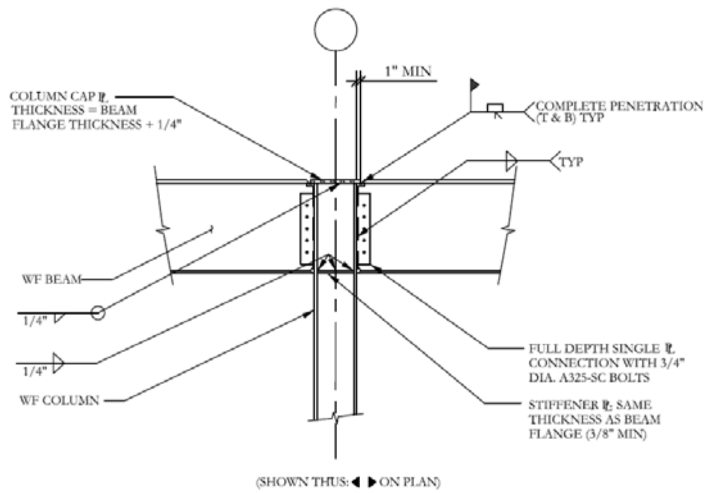


Figure 11: Existing Steel Structural System Column Flange Moment Connection Detail

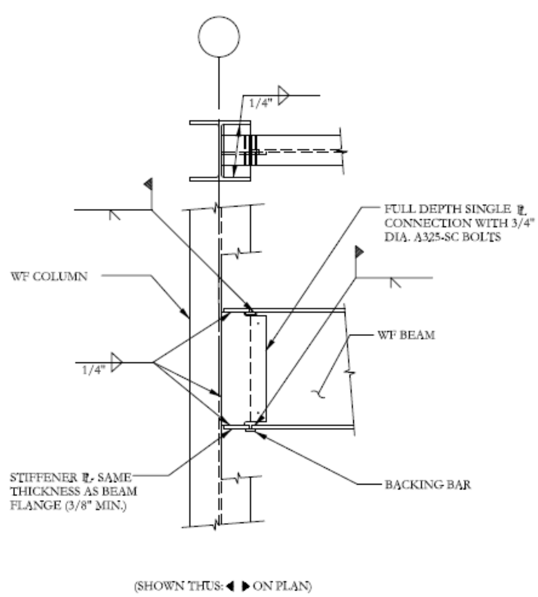


Figure 12: Existing Steel Structural System Column Web Moment Connection Detail

ii. Proposal Background

Problem Statement

When the Duncan Center was originally designed, it was decided to use a lateral load resisting system of a steel moment frame. The key advantage of using moment connected frames is that there is freedom of architectural constraints of the façade and interior space. Comparatively, braced frames and shear walls provide such constraints to the placement of doors, windows, and walls, which may play a significant contributing factor to the overall architecture of the building. Other potential deciding factors may have been the duration of construction, as an Owner would desire the building to be constructed as quickly as possible in order to turn around and lease the space, and steel is typically erected more quickly. Also, the overall weight of the building must be considered for its effect upon the foundation design, and steel is typically a lighter system than concrete.

Steel moment frames, however, are known to not always be the most cost effective lateral system that could be selected for a particular building. This is primarily due to the expense incurred by the moment connections themselves, which often incorporate multiple welds in the shop and also in the field. Thus, the current lateral system in the Duncan Center may not be the most economical and a different lateral system will be investigated to determine if steel moment frames are indeed the optimal solution.

Proposed Solution

From the preliminary study performed in the Technical Assignment #2 it was found that compared to the existing composite system, a concrete two-way flat plate conventionally reinforced system may be more cost effective, eliminate the need for spray-on fireproofing, and allow increased cavity area for MEP ductwork and equipment.

By using a concrete flat plate system, a steel framing and lateral system is no longer logical and a concrete framing and lateral system shall be put in its place. The alternative lateral system to be designed will be concrete shear walls, positioned within the building to create as little obstruction to the architecture as possible, taking into account the existing façade and typical tenant floorplan. Also, due to the significant change in weight present between the two floor systems, the foundation system will also need to be reanalyzed.

iii. Design Loads

a. Dead Loads

Summary		
Floor	20	PSF
Roof	20	PSF
Balcony	30	PSF
Exterior Wall	55	PSF
Partition Wall	20	PSF
Bearing Wall	80	PSF
Shear Wall	97	PSF

See Appendix A: pg.55 for calculations.

Note: Building dead loads do not include supporting structural member self-weights.

b. Live Loads

Space	Load	
Roof	33	PSF
Balcony	100	PSF
Stairs and Exits	100	PSF
Corridor-First Floor	100	PSF
Corridor-Other Floors	80	PSF
Lobby	100	PSF
Dance Halls and Ballrooms	100	PSF
Office Space	70	PSF

c. Snow Loads

Flat Roof Snow Load

pf=22 psf

Lower Roof Snow Drift Load

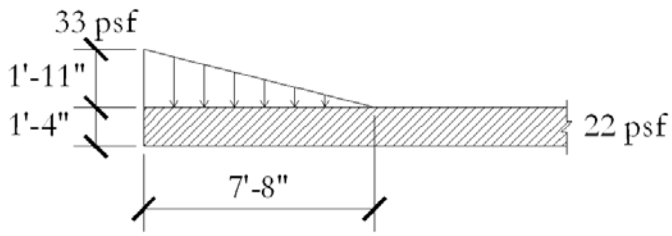


Figure 13: Snow Drift Loading Diagram

See Appendix A: pg.56-57 for calculations.

d. Wind Loads

Figure 14: North-South Direction Wind Load

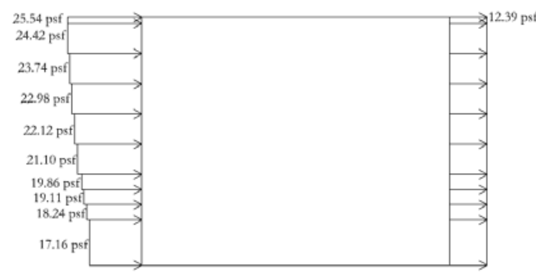


Figure 15: East-West Direction Wind Load

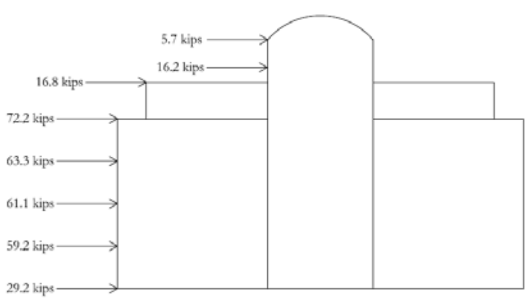


Figure 16: North-South Direction Story Shear

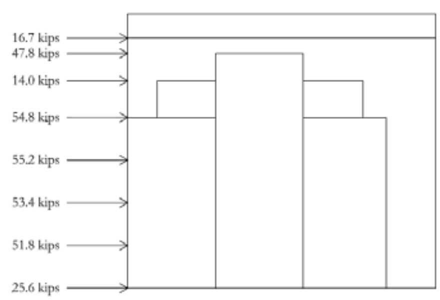


Figure 17: East-West Direction Story Shear

See Appendix A: pg.57-62 for calculations.

e. Seismic Loads

Equivalent Lateral Force

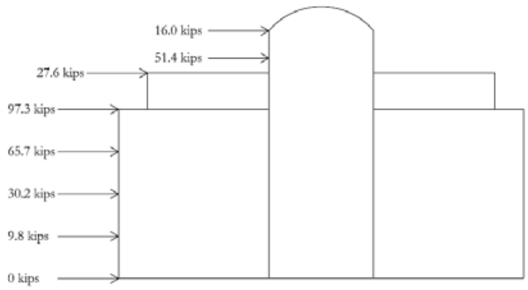


Figure 18: Story Shear

See Appendix A: pg.62-65 for calculations.

f. Analysis Codes and Reference Standards

National Building Code: International Code Council (ICC) 2006
    "International Building Code (IBC)"
Design Loads: American Society of Civil Engineers (ASCE) 7-05
    "Minimum Design Loads for Buildings and Other Structures"
Steel Reference Standard: American Institute of Steel Construction (AISC) 13th Edition
    "Specification for Structural Steel Buildings" (LRFD)
Concrete Reference Standard: American Concrete Institute (ACI) 318-02
    "Building Code Requirements for Structural Concrete"
Metal Deck Reference Standard: United Steel Deck (USD) 2006
    "Steel Decks for Floors and Roofs"
Steel Joist Reference Standard: Nucor-Vulcraft Group 2003
    "Steel Joists & Joist Girders"

g. Load Combinations

LRFD

1. 1.4D
2. 1.2D+1.6L+0.5S
3. 1.2D+1.6S+L
4. 1.2D+1.6S+0.8W
5. 1.2D+1.6S-0.8W
6. 1.2D+1.6W+L+0.5S
7. 1.2D-1.6W+L+0.5S
8. 1.237D+1.0E+L
9. 1.237D-1.0E+L
10. 0.9D+1.6W
11. 0.9D-1.6W
12. 0.863D+1.0E
13. 0.863D-1.0E

See Appendix A: pg.97 for Seismic Load Combination calculations.

iv. Proposed Concrete Structural System

a. Foundation System

For the redesign of the foundations, it was decided to change the augercast piles from the previously selected 16" dia. and 85 ton capacity to a different presented option of an 18" dia. and 105 ton capacity and of equal length, as per the geotechnical engineer, John D. Hynes & Associates, Inc. By changing the diameter of the augercast piles, the effect of the increased weight of the structure had less impact on the foundation configurations, which are mostly governed by geometrical constraints; see Figure 21: Proposed Concrete Structural System Foundation Plan & Figure 29: Proposed Concrete Structural System Pile Cap Configurations. Below is the column dowel reinforcement schedule corresponding to the appropriate columns and pile caps; see Figure 22: Proposed Concrete Structural System 2nd Floor Framing Plan.

Column Dowel Reinforcement Schedule		
Column	Size	Dowel Reinforcement
C1	20"x20"	4-#8
C2	20"x20"	4-#8
C3	20"x20"	4-#8
C4	24"x28"	4-#10
C5	24"x28"	4-#10

For calculations and other assumptions; see Appendix A: pg.66-68.

b. Framing System

As a result of changing the lateral system to shear walls, the framing system also had to be changed to concrete. It was determined based upon results from Technical Assignment #2 that a two-way flat plate system was comparative to the existing composite slab and metal deck floor system. The concrete strength was also changed from 4000 psi to 5000 psi in order as determined from the optimum analysis of the floor slabs.

Preliminary thicknesses of slabs were based upon the ACI code requirements for minimum slab thickness, however final designs incorporated a deflection analysis, enabling the thickness of the slabs to be reduced, due to the 33'-4" long span. Also, due to the punching shear existing at the column strips along this long span, drop panels with a 4" thickness also needed to be incorporated; see Figure 30: Proposed Concrete Structural System Drop Panel Details. The final slab thickness were 12" for the first through fourth floors and 14" for the fifth floor; see Figures 22 & 23: Proposed Concrete Structural System 2nd & 5th Floor Framing Plans, respectively.

A one-way slab with beams was implemented for the sixth floor as there is only one span that exists and it was also found to be 12" thick; see Figure 24: Proposed Concrete Structural System 6th Floor Framing Plan. Below is the slab reinforcement which was the result of the analysis of the critical strips for each slab in PCA Slab. The slabs should also be analyzed based upon the seismic loads in the diaphragms, however this was not feasible for the duration of this project.

Slab Reinforcement Schedule			
Story	Strip	Reinforcement	Spacing (in)
6th Floor	Column	#5	9
5th Floor	Column	#5	5
	Middle	#5	10
4th Floor	Column	#5	5
	Middle	#5	12
3rd Floor	Column	#5	5
	Middle	#5	12
2nd Floor	Column	#5	5
	Middle	#5	12
1st Floor	Column	#5	5
	Middle	#5	12

For calculations and other assumptions; see Appendix A: pg.69-81.

On the next page is the beam schedule for the one-way slab beams and was based upon the lateral analysis results from ETABS, as they acted in part of the concrete moment frame which frames the sixth floor; see Figure 24: Proposed Concrete Structural System 6th Floor Framing Plan.

Beam Schedule				
Beam	Size	Flexural Reinforcement	Shear Reinforcement	Spacing (in)
B1	24"x24"	3-#10	#3	5
B2	24"x24"	4-#10	#3	5

For calculations and other assumptions; see Appendix A: pg.82-83.

Preliminary column sizes were determined to be 16"x16" based upon the results from PCA Slab. All columns were designed using the CRSI Handbook, these results of these designs are presented in the column schedule below and on the subsequent pages. Design by CRSI Handbook was permitted as all the columns met the short column requirements as required; see Figures 22, 23 & 24: Proposed Concrete Structural System 2nd, 5th & 6th Floor Framing Plans. The final column sizes were determined for gravity loading with the exception of those on the sixth floor, which were based upon gravity and lateral analysis results from ETABS, as they acted as part of the concrete moment frame which frames the sixth floor.

Column Schedule				
C1	Floor	Bars	Bar Configuration	Ties
20"x20"	Basement	8-#10	3E	#3
	1st Floor	8-#10	3E	#3
	2nd Floor	8-#10	3E	#3
	3rd Floor	8-#10	3E	#3
	4th Floor	16-#10	5E	#3
C1	Floor	Tie Spacing (in)	Extended Bars	Splice Length (in)
20"x20"	Basement	18	8-#10	38
	1st Floor	18	8-#10	38
	2nd Floor	18	8-#10	38
	3rd Floor	18	8-#10	38
	4th Floor	18	NA	NA

Column Schedule				
C2	Floor	Bars	Bar Spacing	Ties
20"x20"	Basement	8-#10	3E	#3
	1st Floor	8-#10	3E	#3
	2nd Floor	8-#10	3E	#3
	3rd Floor	8-#10	3E	#3
	4th Floor	16-#10	5E	#3
C2	Floor	Tie Spacing (in)	Extended Bars	Splice Length (in)
20"x20"	Basement	18	8-#10	38
	1st Floor	18	8-#10	38
	2nd Floor	18	8-#10	38
	3rd Floor	18	8-#10	38
	4th Floor	18	NA	NA
C3	Floor	Bars	Bar Spacing	Ties
20"x20"	Basement	4- #10	2E	#3
	1st Floor	4- #10	2E	#3
	2nd Floor	4- #8	2E	#3
	3rd Floor	4- #8	2E	#3
	4th Floor	4- #8	2E	#3
C3	Floor	Tie Spacing (in)	Extended Bars	Splice Length (in)
20"x20"	Basement	18	4- #10	38
	1st Floor	18	4- #10	38
	2nd Floor	16	4- #8	30
	3rd Floor	16	4- #8	30
	4th Floor	16	NA	NA
C4	Floor	Bars	Bar Spacing	Ties
24"x28"	Basement	8-#8	3E	#3
	1st Floor	8-#8	3E	#3
	2nd Floor	8-#8	3E	#3
	3rd Floor	8-#8	3E	#3
	4th Floor	8-#10	3E	#3
	5th Floor	8-#10	3E	#3
	6th Floor	8-#10	3E	#3

Column Schedule				
C4	Floor	Tie Spacing (in)	Extended Bars	Splice Length (in)
24"x28"	Basement	16	8-#8	30
	1st Floor	16	8-#8	30
	2nd Floor	16	8-#8	30
	3rd Floor	16	8-#8	30
	4th Floor	18	8-#10	38
	5th Floor	18	8-#10	38
	6th Floor	18	NA	NA
C5	Floor	Bars	Bar Spacing	Ties
24"x28"	Basement	8-#8	3E	#3
	1st Floor	8-#8	3E	#3
	2nd Floor	8-#8	3E	#3
	3rd Floor	8-#8	3E	#3
	4th Floor	8-#10	3E	#3
	5th Floor	8-#10	3E	#3
	6th Floor	8-#10	3E	#3
C5	Floor	Tie Spacing (in)	Extended Bars	Splice Length (in)
24"x28"	Basement	16	8-#8	30
	1st Floor	16	8-#8	30
	2nd Floor	16	8-#8	30
	3rd Floor	16	8-#8	30
	4th Floor	18	8-#10	38
	5th Floor	18	8-#10	38
	6th Floor	18	NA	NA

For calculations and other assumptions; see Appendix A: pg.84-96.

c. Lateral Load Resisting System

Preliminary thickness of the shear walls was governed by IBC 2006 Fire Construction Rating requirements and to provide a 3 hour rating for the stair well and determined to be 8”. After analyzing the lateral system in ETABS, it was determined that this thickness of shear wall was adequate for drift, overturning and torsion; see Figure 19: ETABS Model. On the next page, the shear wall schedule show the results for the designs based upon ETABS.

Based on the configurations of the sixth floor, shear walls, which optimally replaced the North and South stair towers, could not laterally support this floor. Therefore, a concrete moment frame was utilized for the sixth floor, the designs for which were presented in the previous section, Framing System; see Figures 26, 27 & 28: Proposed Concrete Structural System Elevation Line A, Line A7 & Line 4.

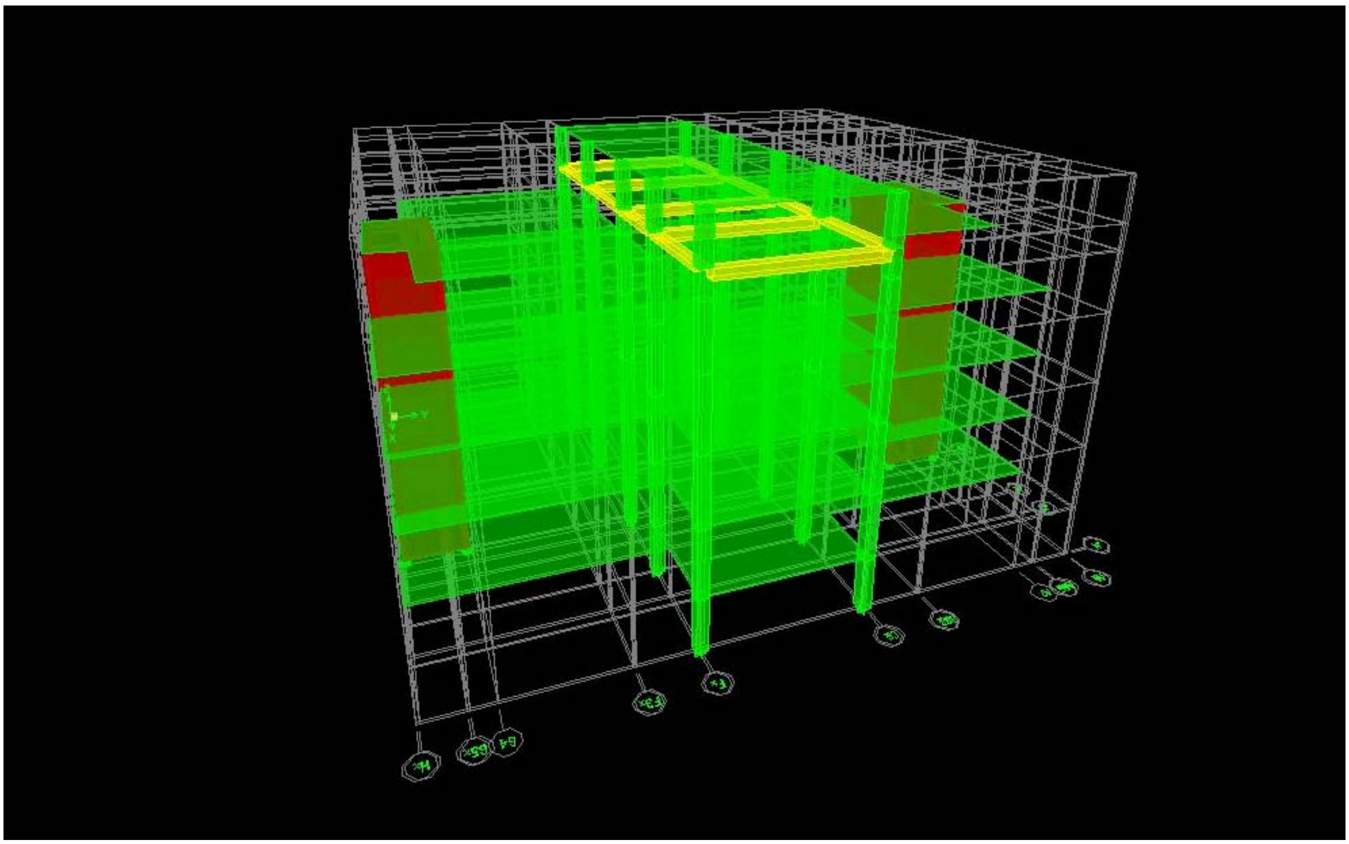


Figure 19: ETABS Model

Shear Wall Schedule					
Pier	Thickness (in)	Flexural Reinforcement	Spacing (in)	Shear Reinforcement	Spacing (in)
WA	8	#4	12	#4	10
WA7	8	#4	12	#4	10
WG4	8	#4	12	#4	10
WH	8	#4	12	#4	10
W43A	8	#4	12	#4	10
W43H	8	#4	12	#4	10
W5A	8	#4	12	#4	10
W5H	8	#4	12	#4	10
Spandrel	Thickness (in)	Flexural Reinforcement	Vertical Shear Reinforcement	Horizontal Shear Reinforcement	Spacing (in)
SA7	8	4- #4	4 legs- #4	#4	12
SG4	8	4- #4	4 legs- #4	#4	12

For calculations and other assumptions; see Appendix A: pg.97-113.

d. Roof Framing

As the roof needs to span over a large area in order to accommodate the column-free space as required by the fifth floor ballroom, a steel framed roof is required. The proposed roof framing system is very similar to the existing under the assumption that the existing roof system is flat as shown in Figure 1: The Duncan Center, and not gabled as indicated on Figure 7: Existing Steel Structural System Roof Framing Plan. The roof framing was designed in RAM Structural System; see Figure 20: RAM Structural System Model & Figure 25: Proposed Concrete Structural System Roof Framing Plan.

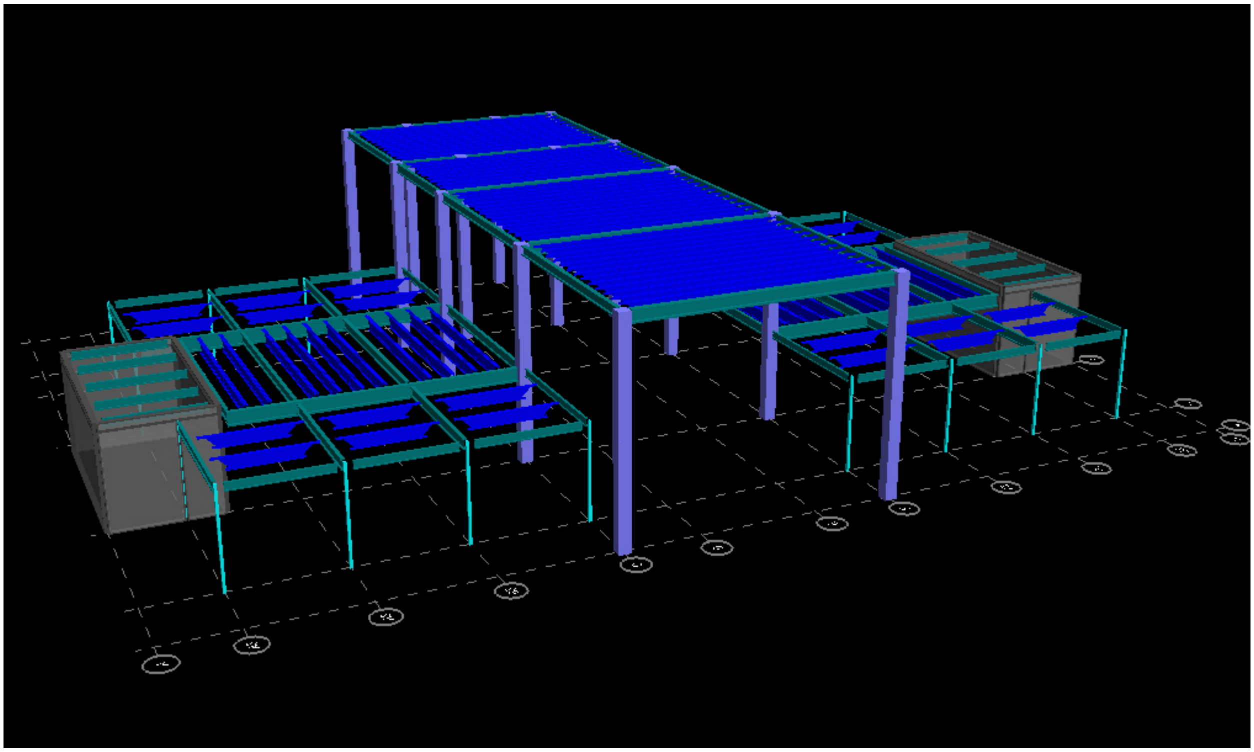


Figure 20: RAM Structural System Model

For calculations and other assumptions; see Appendix A: pg.114-121.

e. Foundation Plan

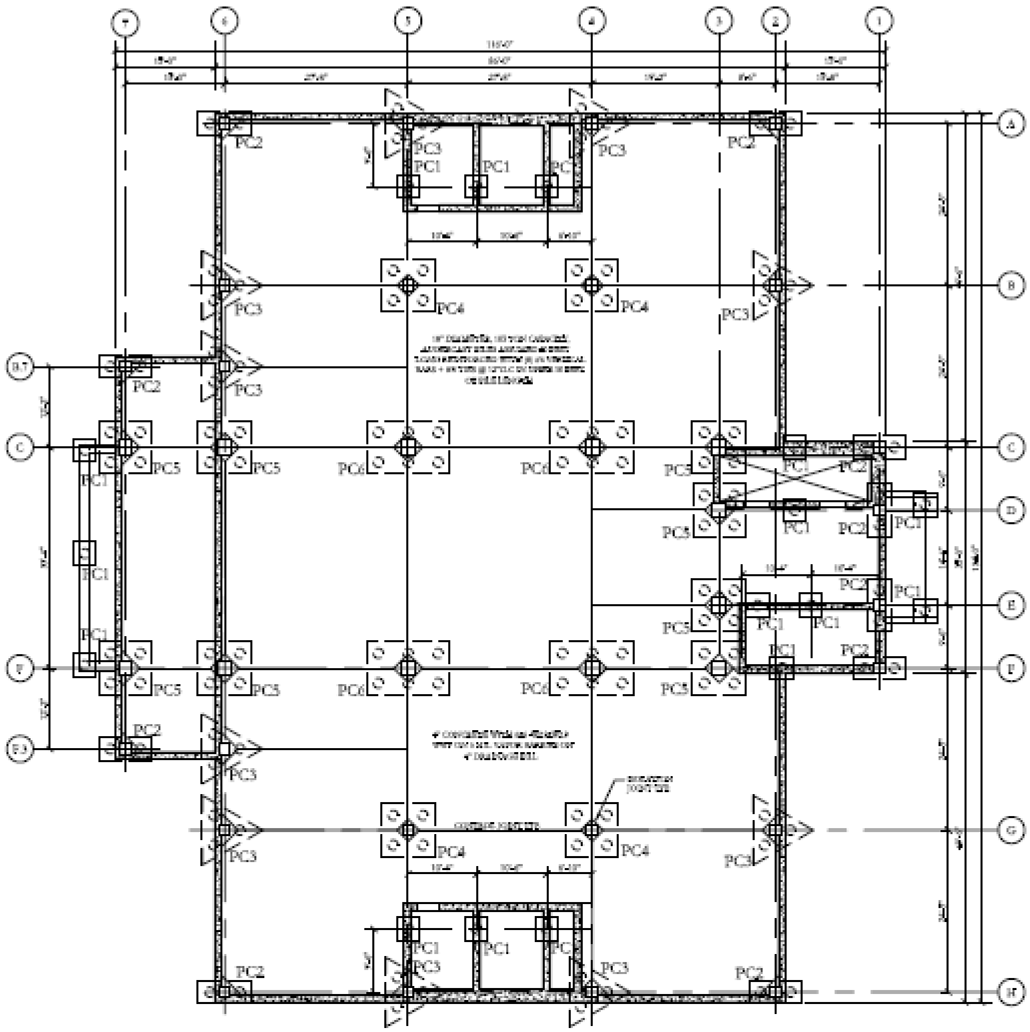


Figure 21: Proposed Concrete Structural System Foundation Plan

f. Framing Plans

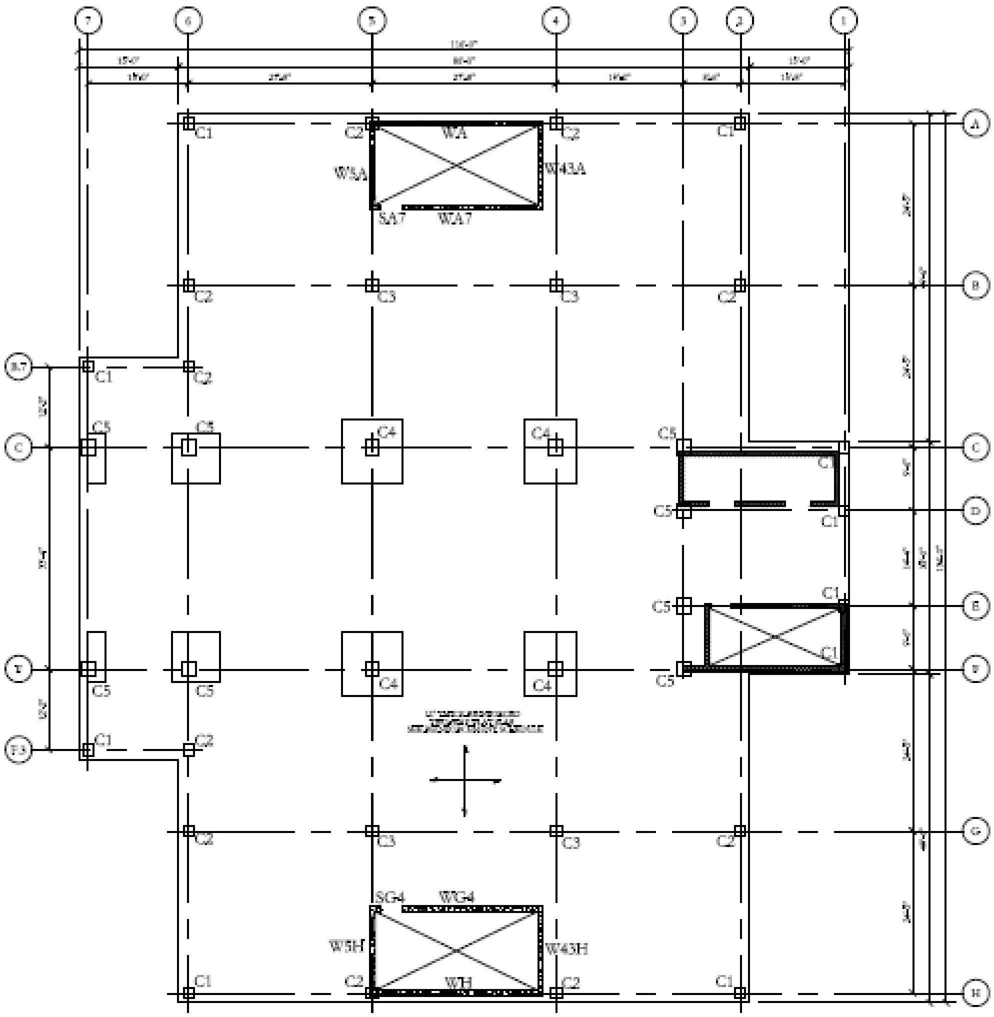


Figure 22: Proposed Concrete Structural System 2nd Floor Framing Plan

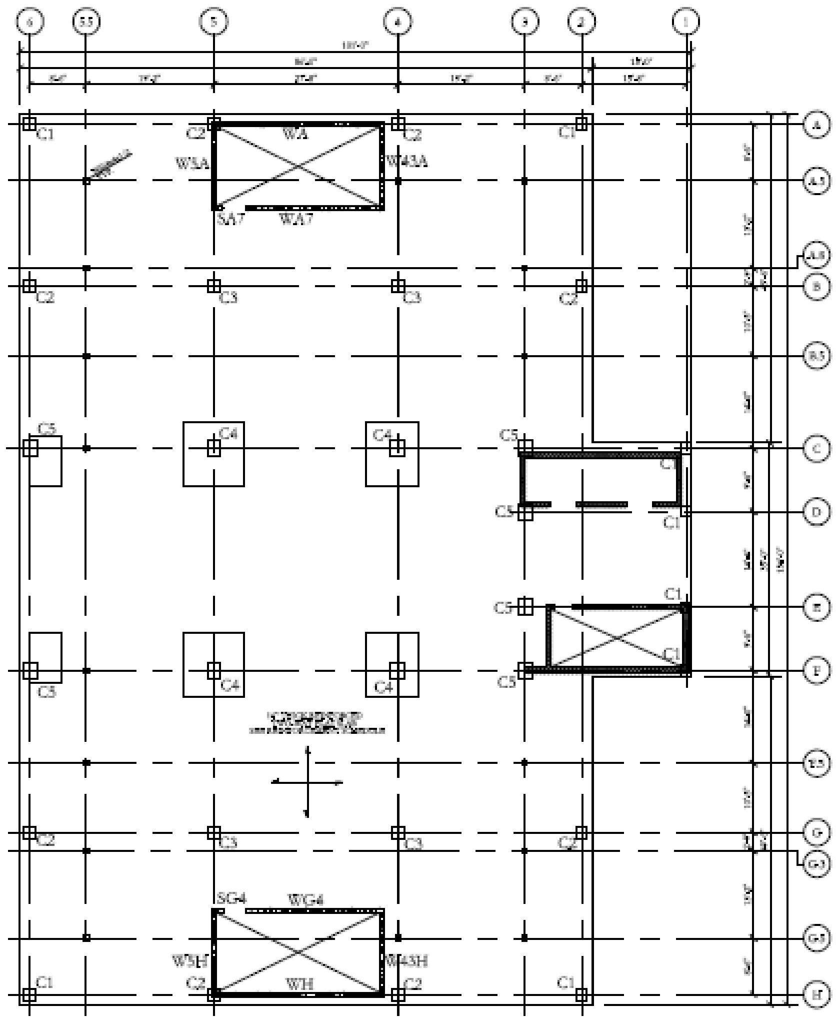


Figure 23: Proposed Concrete Structural System 5th Floor Framing Plan

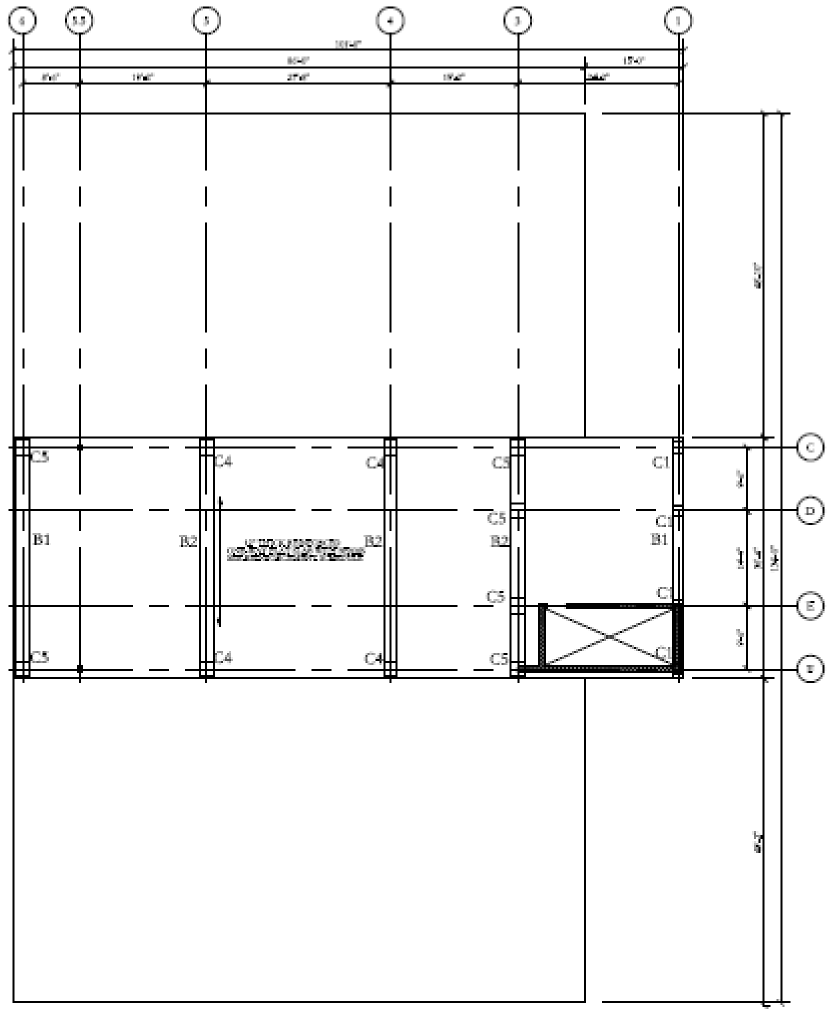


Figure 24: Proposed Concrete Structural System 6th Floor Framing Plan

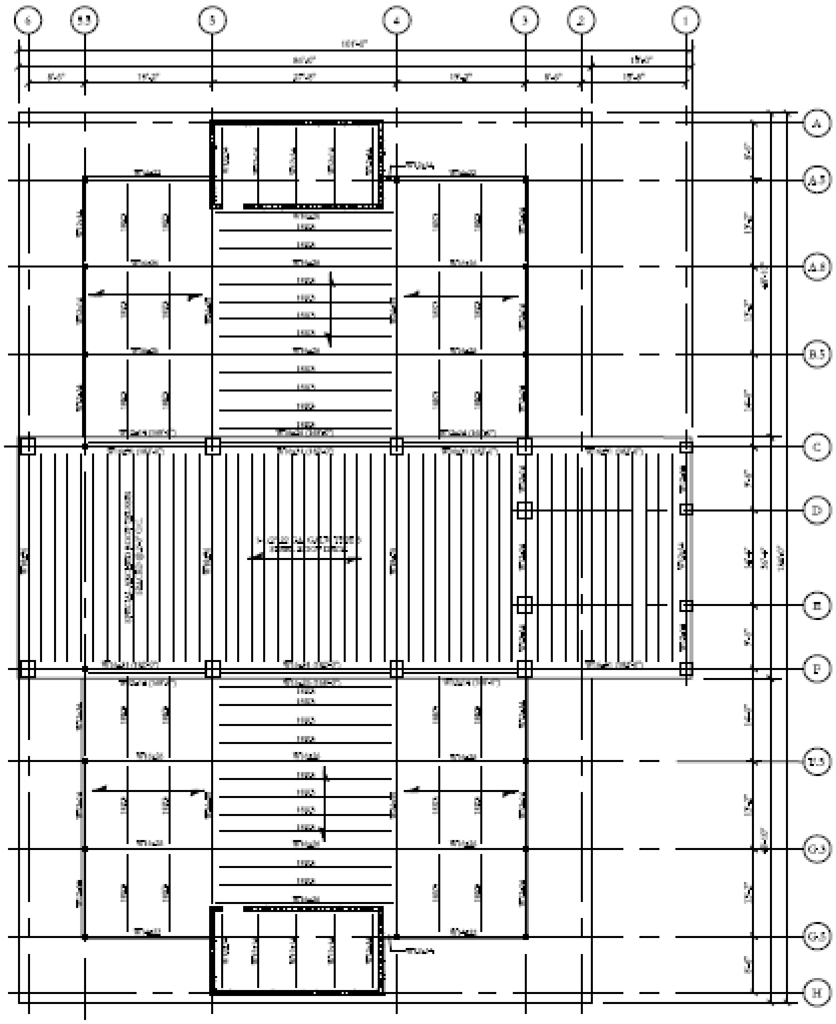


Figure 25: Proposed Concrete Structural System Roof Framing Plan

g. Elevations

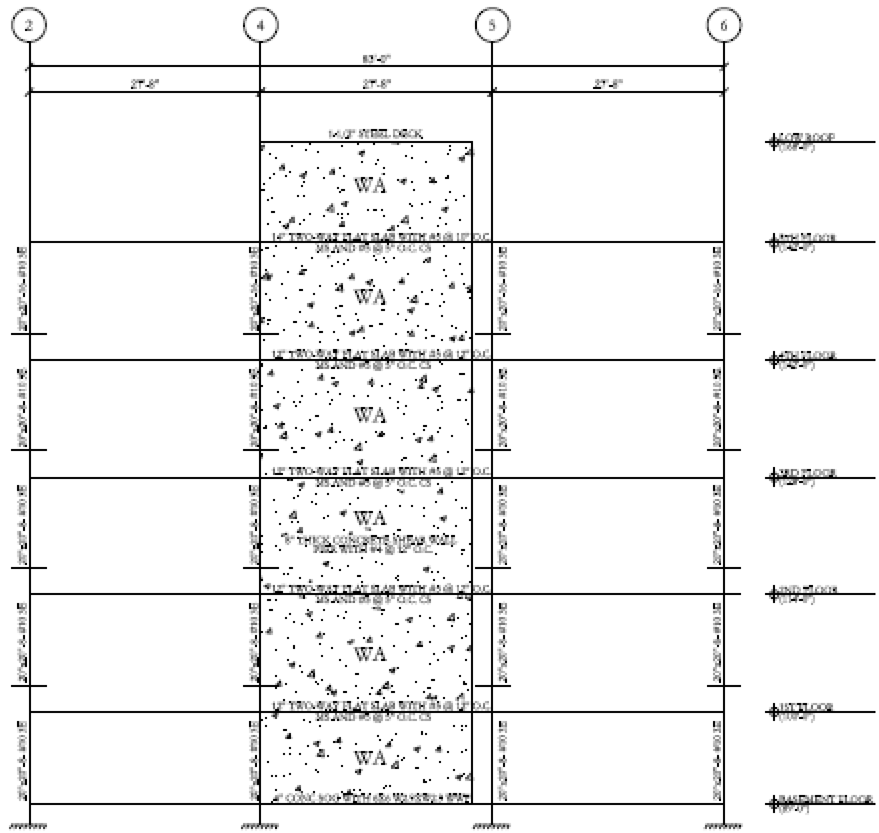


Figure 26: Proposed Concrete Structural System Elevation Line A

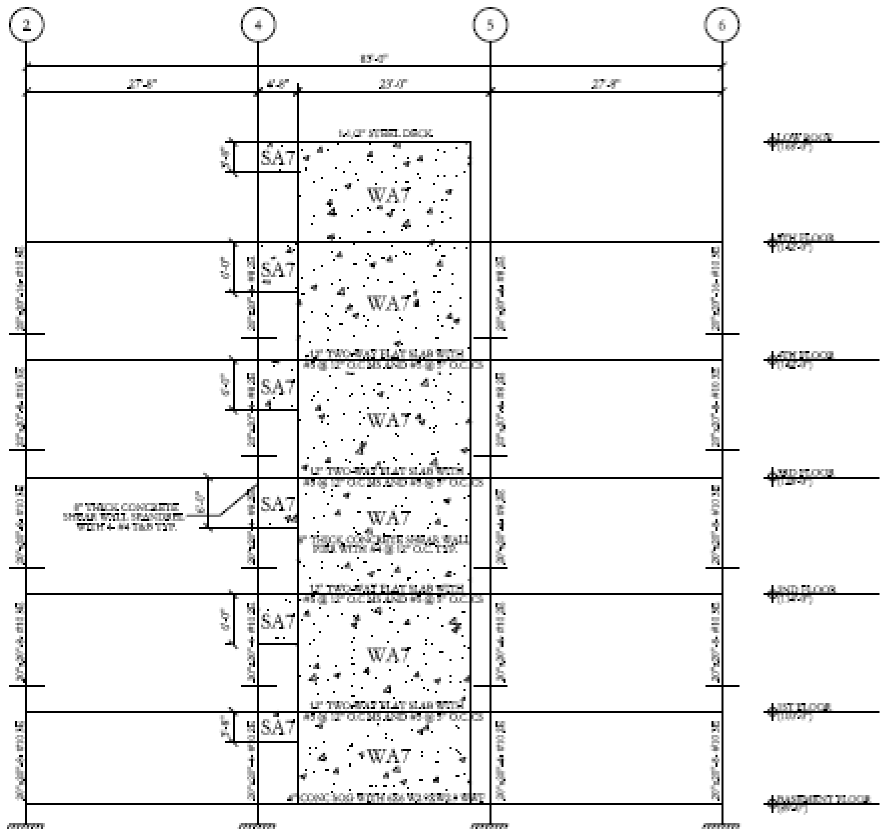


Figure 27: Proposed Concrete Structural System Elevation Line A7

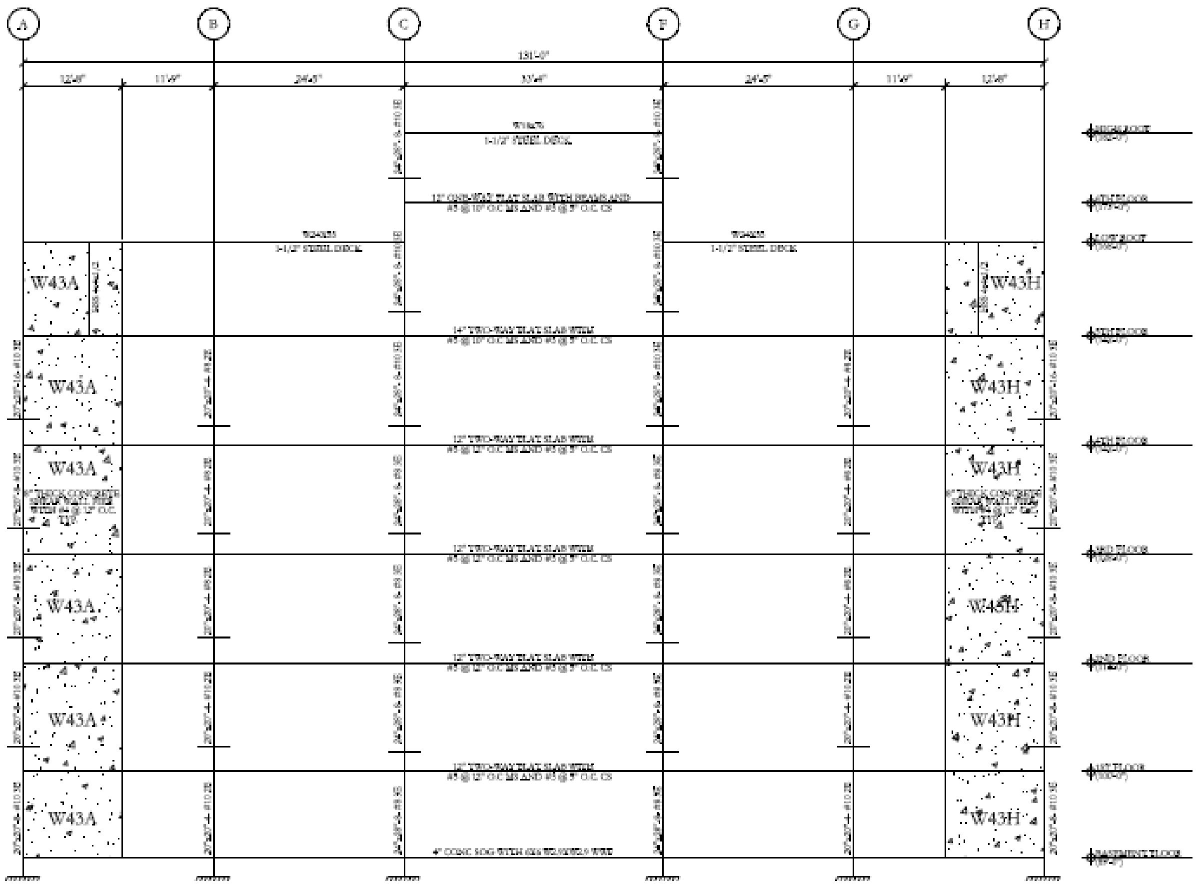


Figure 28: Proposed Concrete Structural System Elevation Line 4

h. Details

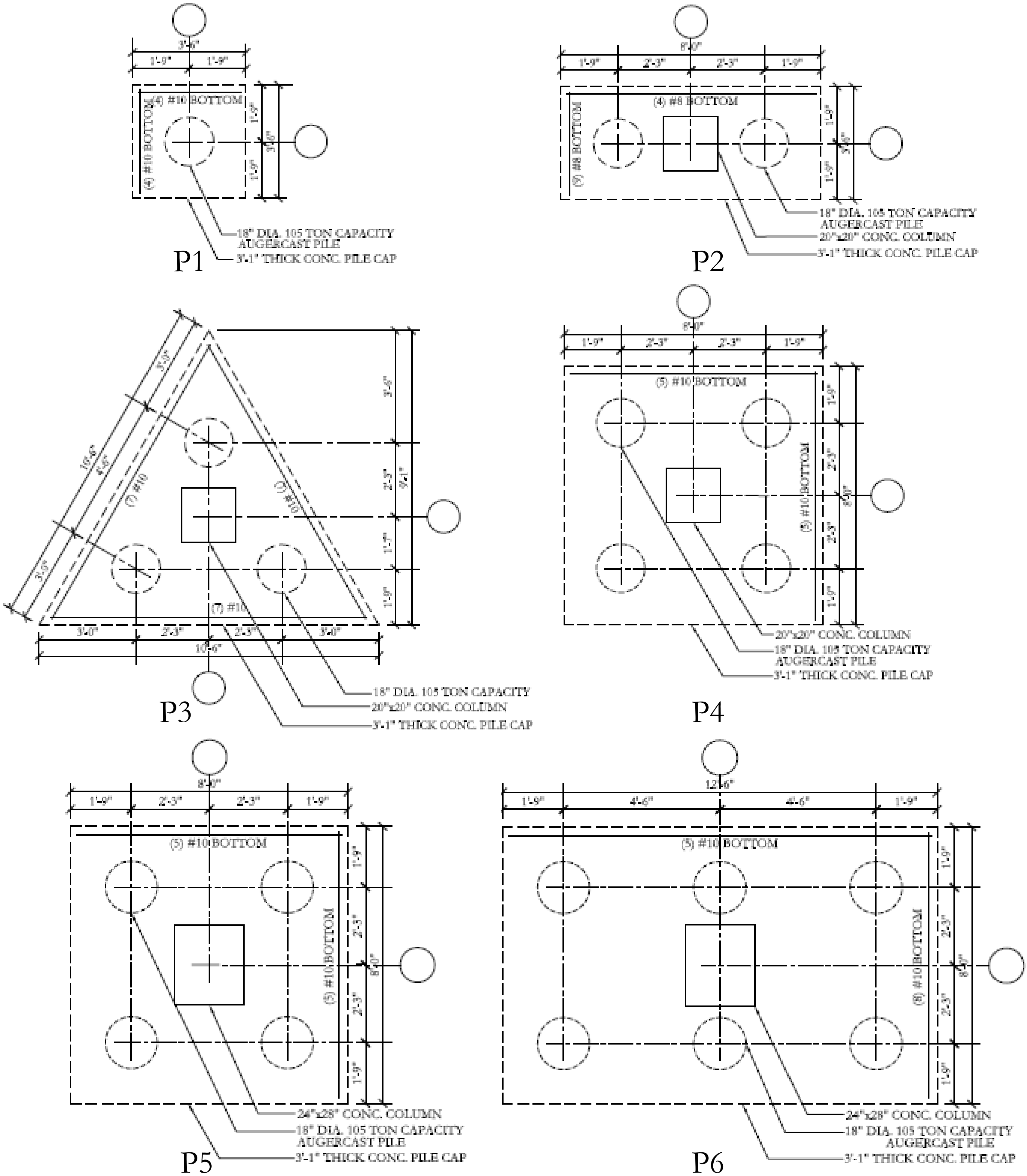
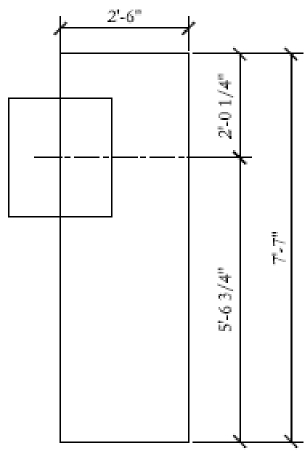
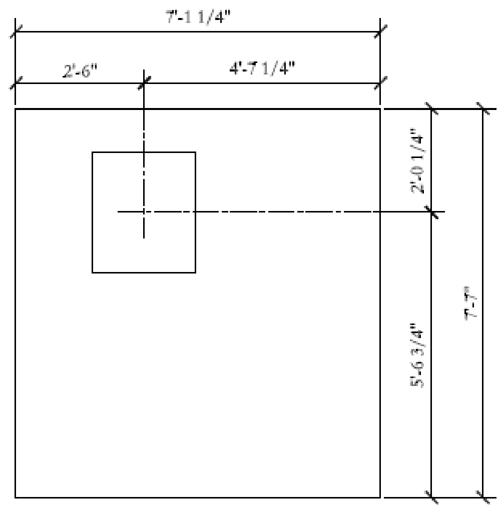


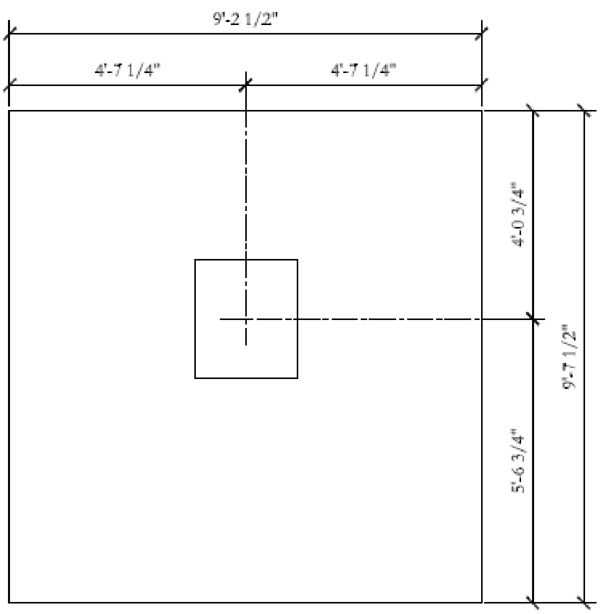
Figure 29: Proposed Concrete Structural System Pile Cap Configurations



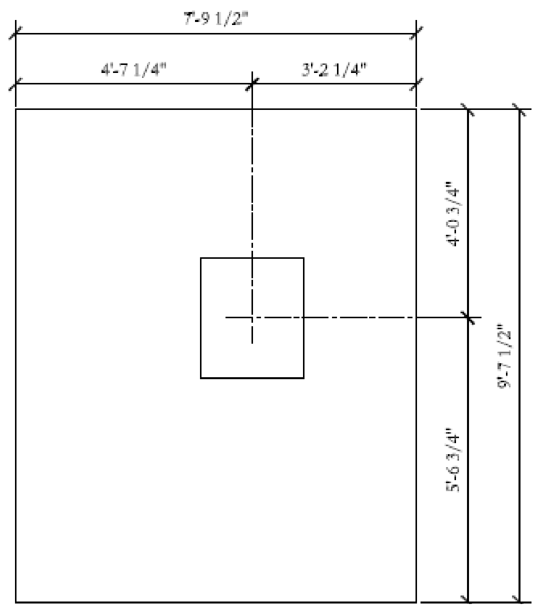
Line 7



Line 6



Line 5



Line 4

Figure 30: Proposed Concrete Structural System Drop Panel Details
Shown for Line C; Mirror for Line F

v. STRUCTURAL SYSTEM COMPARISON & DEPTH CONCLUSIONS

Based on the design of the proposed concrete structural system, it was found that the structural system did provide an increase in mechanical space, which can be seen from the table below; see Figure 31: Existing Steel & Proposed Concrete Structural System 2nd Floor Mechanical Plans. Also, the foundation system was not as dramatically impacted as had been expected with such an increased weight in the structure, which was made feasible by changing the pile diameter from 16" to 18" dia. and of equal length.

However, due to the need for a steel framed roof, in order to provide a column-free space in the ballroom with long spans, spray-on fireproofing is still required for at a least that small portion of the building. It is common for a concrete building to have a steel framed roof due to the long spans required and it is not anticipated that this will cause any difficulties.

Structurally, the two systems are comparative, despite the reduction of spray-on fireproofing and increase in mechanical ceiling to floor cavity space, and designed using the same criterion which were met. The final decision to recommend the proposed concrete structural system over the existing steel structural system will be based upon the acoustics and construction management analyses.

Mechanical Space Savings			
Floor	Mechanical Space		
	Existing Steel Structural System	Proposed Concrete Structural System	Increase
1st Floor	2'-3"	3'-0"	9"
2nd Floor	2'-3"	3'-0"	9"
3rd Floor	2'-3"	3'-0"	9"
4th Floor	2'-3"	3'-0"	9"
5th Floor	2'-3"	2'-10"	7"
6th Floor	2'-3"	2'-6"	3"

For calculations and other assumptions; see Appendix A: pg.122.

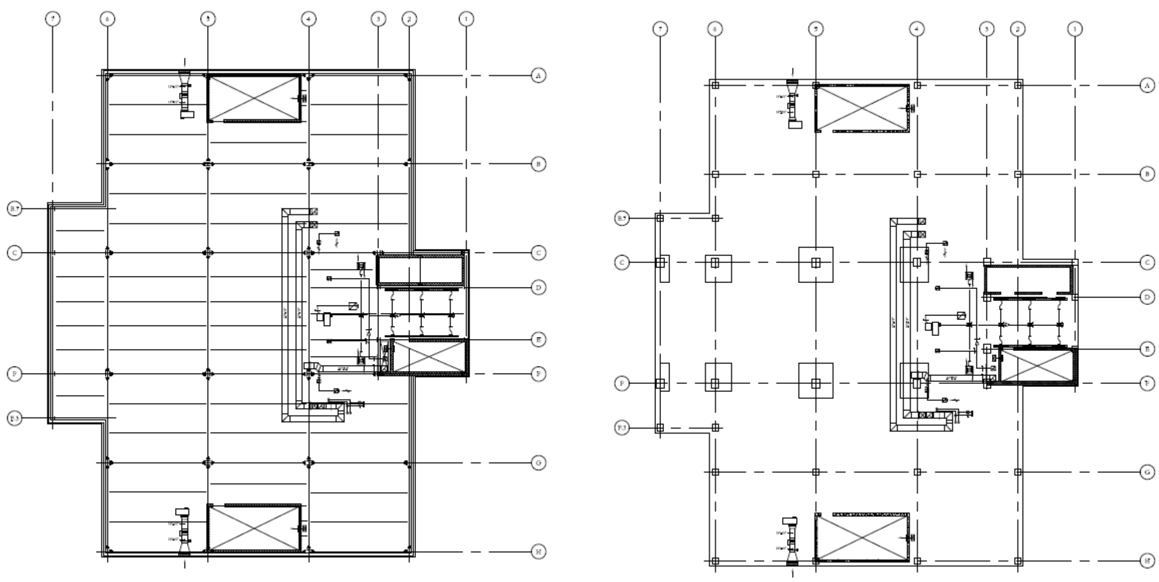


Figure 31: Existing Steel & Proposed Concrete Structural System 2nd Floor Mechanical Plans