Depth – Structural Redesign

Proposal

Due to the location of the project and the needs of the client, the most efficient structural system was originally designed. This became apparent in the previous Technical Assignment #2 where the existing floor system was compared to four other proposed systems. The intention of this proposal will be to redesign and reevaluate the current cast-in-place reinforced concrete system to a Girder-Slab system using asymmetrical steel beams and precast concrete planks. The serviceability and strength of the new system will be checked using codes and loads from the following but not limited codes: IBC, ASCE7-05, and AISC

Problem Solution

Floor System

The proposed floor structure to be analyzed and implemented is the Girder-Slab system. It is a new and innovative way to build that combines the benefits of steel and concrete into one monolithic floor system. The system is comprised of an interior girder known as an open-web dissymmetric beam, or D-Beam, which supports precast, prestressed hollow-core slabs on its bottom flange. The D-Beams also have openings in some of the web to allow for grouting of the hollow core planks. Upon grouting, the system develops composite action and is able to resist lateral movement between the planks and beams.

After the initial calculations performed in Technical Assignment #2, DB8x42 beams were chosen with an 8" x 4'-0" hollow core plank. The construction and economic costs associated with this system will be analyzed and reviewed. The Girder-Slab may prove to have a shorter erection time but a longer lead time than cast-in-place concrete. The erection time will allow for a quicker construction time but may be outweighed by the initial costs of the system. These topics as well as others will be reviewed and compared in the following pages.

Lateral System

In order to allow for the fastest erection time, a lateral resisting system consisting of diagonal braced frames will be investigated. Braced frames will be located around the elevator core and central stairwell. The braced frames will have to be able to resist the seismic and wind loads set by previous Technical Assignments. This lateral system must be able to resist any torsion effects created by these loads.

Structural Gravity System

Girder-Slab Floor System

Girder-Slab is a steel and precast plank flooring system developed by Girder-Slab Technologies LLC. This is the first hybrid floor system that fully integrates steel and precast planks to create a monolithic slab assembly. Specifically targeting mid to high-rise residential construction, The system is comprised of an interior girder known as an open-web dissymmetric beam, or D-Beam, which supports precast, prestressed hollow-core slabs on its bottom flange. The D-Beams are cut from a parent wide flange section which produces two D-beams which can be seen in Figure 6. Typically, there are two basic D-Beam sections that will work with the use of 8" pre-cast slabs, DB-8 and DB-9. Beams are corrugated when cut in half which allows for grout to flow through the web and the hollow core plank openings. Upon grouting, the system develops composite action, allowing it to support residential live loads. The transformed Girder-Slab section is able to withhold over twice the moment capacity of a sole D-Beam. The Girder- Slab system will also reduce

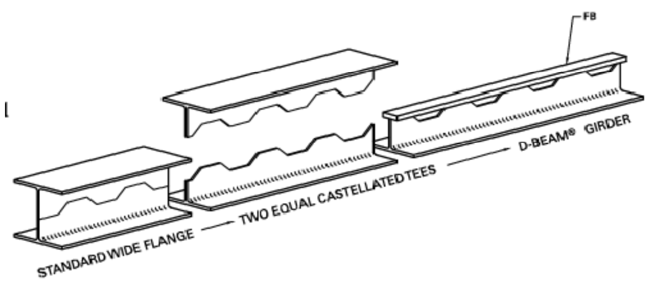


Figure 6: Two D-Beam Girders cut from a parent Wide-Flange

Figure 7: Composite transformed D-Beam Section

labor costs and improve construction operations. Girder-slab system and D-Beam girders are only distributed and assembled in New Jersey by Girder-Slab Technologies LLC.

In order to implement the Girder-Slab flooring system, the layout of the columns in the building needed to be changed. To allow for the floor planks to be aligned properly, an orthoganol grid was created. The Girder-Slab system was used for typical floors 2-16 during the structural redesign. In order to match the existing thickness of the flat plate floor slab (8”), DB-8’s were chosen with an eight inch hollow-core floor plank. In order to level the floor from differential deck cambers, a ¾” concrete topping will be used. The typical Girder-Slab layout can be seen in Figure 8.

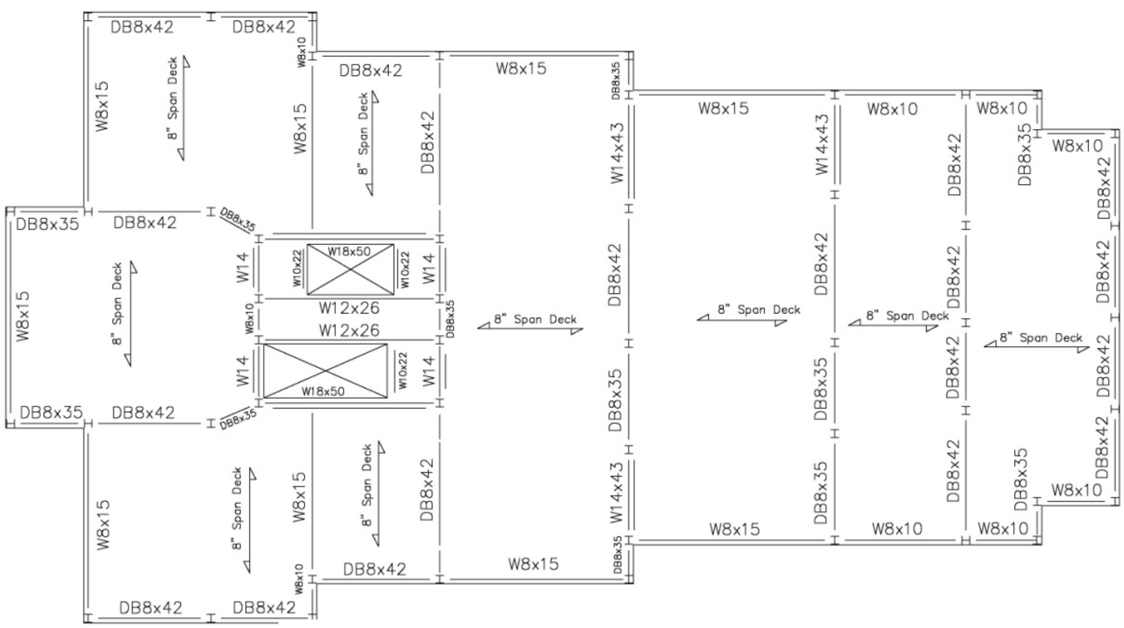


Figure 8: Typical floor plan utilizing Girder-Slab.

Typically, eight inch pre-cast planks will span the long direction of the bays, while DB8x42 will span the short directions. In some cases, DB8x35 will be used when acceptible. Along the plank edges, Wide-flange members will be used to span this long direction. J952 8” x 4’ Span Deck planks with 6-1/2” Ø strands will be used in the Girder-Slab system.

On the roof level, larger D-Beams were necessary to account for the increased live load. At this level, DB9's replace the DB8's in order to achieve the same spans while still maintaining the Girder-Slab system. Addtionally, a rubberized roof ver plank will be used to replace the hollow core concrete planks to withstand any severe weathering that may occur on the roof.

The Girder-Slab system was designed in accordance with the design specifications presented in the Girder-Slab Design Guide. This design guide uses Allowable Stress Design (ASD) calculations in accordance with with American Institute of Steel Construction (AISC).

When determining the types of D-Beams to use, the system must be checked twice, once for pre-composite action, and one for full composite. Pre-composite action occurs prior to grouting the system and before any curation takes place. The only load used in this design check is the weight of the pre-cast hollow core planks. Moment and deflection calculations are performed to make sure they meet the allowable criterion for the steel section. After the grout is injected and has cured, the transformed section is analyzed with the addition of the dead load of the plank, any superimposed dead load, and the live load per the occupancy (in accordance with ASCE7-05). At this point, the required section modulus is calculated and compared to the given transformed sections of the composite steel. Equation 1 exhibits how to calculate the required section modulus.

Equation 1: $$S_{reg} = \frac{M_{TL}}{O.6F_y}$$

Where: M_{TL} is the bending moment due to total loading

Fy is the yield strength of the steel

Deflection of the composite section is also checked against the allowable deflection criteria. In this case, the deflections were compared against the industry standard of L/360. Superimposed compressive stress on the concrete is checked against the allowable compressive stress. Next, the bottom flange tensile stress is checked for the total load and compared to the allowable yield stress of the steel section.

Equation 2: $$f_b = \frac{M_{DL}}{S_b} + \frac{M_{SUP}}{S_{b(Transformed)}} \leq 0.9F_y$$

Where: Sb is the section modulus of the D-beam before composite action

Sb(Transformed) is the section modulus of the transformed section

Finally, the last strength check is calculating the allowable shear stess of the D-Beam against the total loading.

Equation 3: $$f_v = \frac{R}{t_w b} \leq 0.4F_y$$

Where: R is the support reaction.

Figures 9 and 10 show some Girder-Slab details associated with the floor system.

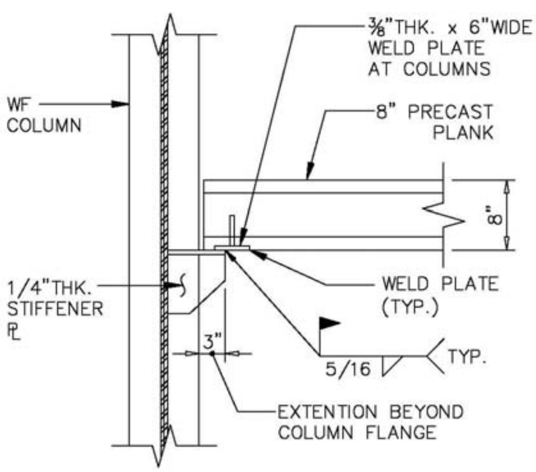


Figure 9: Typical precast plank details at columns

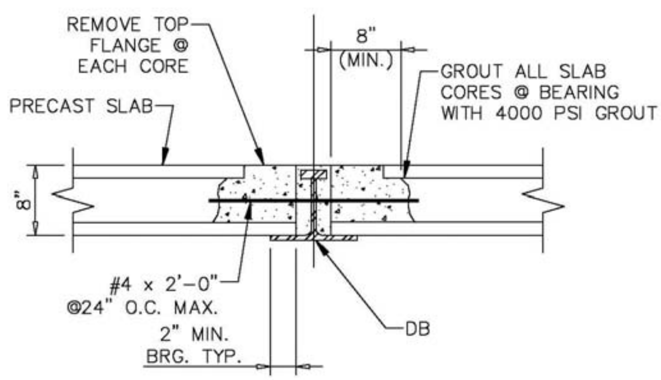


Figure 10: Typical detail of Girder-Slab system at D-Beam section

Girder-Slab Tree Columns and Connections

In order to use DB8 girders and hollow-core floor planks, wide-flange columns were designed for the gravity system of Southtown Building No. 5 as seen in Figure 4. Column sizes were determined using RAM Structural System and checked using hand calculations. To make the construction process more economical, the columns only change in size every 5 stories.

However, in order for some of the DB8x42 beams to span up to 19’, a “tree” column was utilized in some areas. A tree column is a WT section that is welded to a wide-flange column with a bevel weld and a fillet on both sides. This connection allows for a larger span of the D-beam without sacrificing any structural integrity. When selecting the WT shape to use, the same depth must be achieved as that of the D-Beam. In this particular case, a DB8 was used and therefore a WT8 section was selected. A typical connection was designed and a WT8x22.5 section was selected. Since the WT shape must be welded to the column, there will be a negative moment caused by this fixed connection of 44.5 ft-kips. Additionally, the WT will receive a shear force from the D-Beam of 19.8 K through a single plate connection with two bolts in each member. After sizing the WT shape, a 9” x 6” x ¾” plate with 7/16” A325N bolts will be able to resist the shear. Calculations for member sizing and connection may be found in the Appendix.

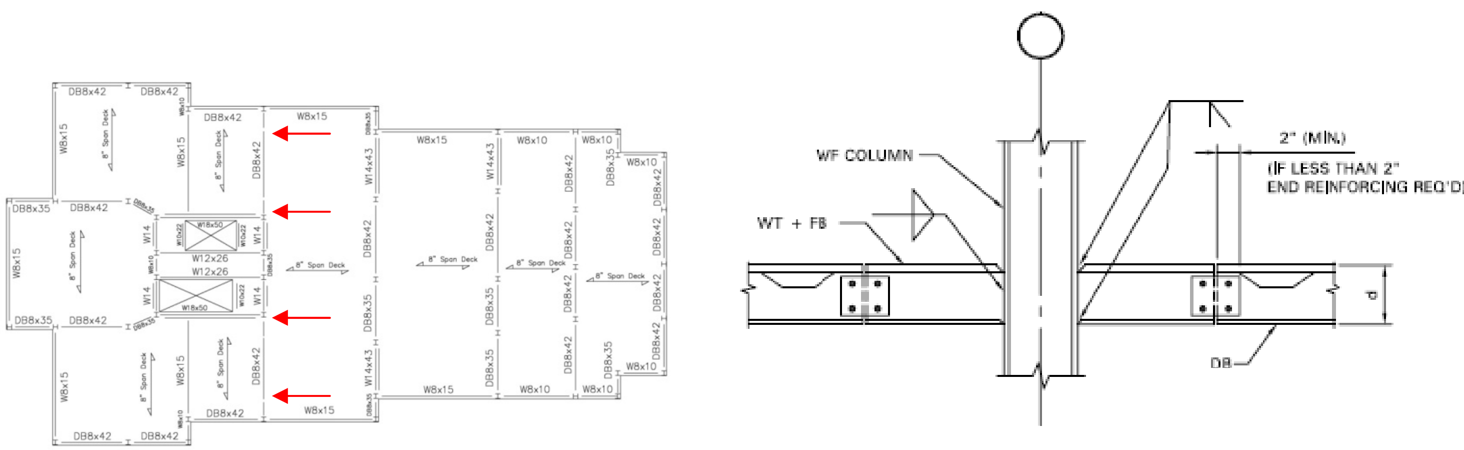


Figure 11: At left, red arrows indicate tree column locations located on a typical floor plan. At right, typical tree column connection utilizing WT shapes and single plate connection

Since tree column connections are very costly, they are only to be used in areas where the D-Beam spans over 16 feet. In such areas with shorter spans, an unstiffened seat connection is utilized. A typical connection for a DX8x42 spanning into the flange of a W12x96 was designed using Table 8-4 of AISC Steel Manual, 13th Edition. Once picking a steel angle size, limit states for web crippling, web yielding, seat angle flexure, angle shear yielding, weld rupture were checked. After the angle size met all requirements, a 3/8” fillet weld on both sides of the angle was determined adequate. A stabalizing angle was used at the top of the D-Beam in order to resist flexure and rotation of the D-Beam. A detail can be seen below in Figure 12.

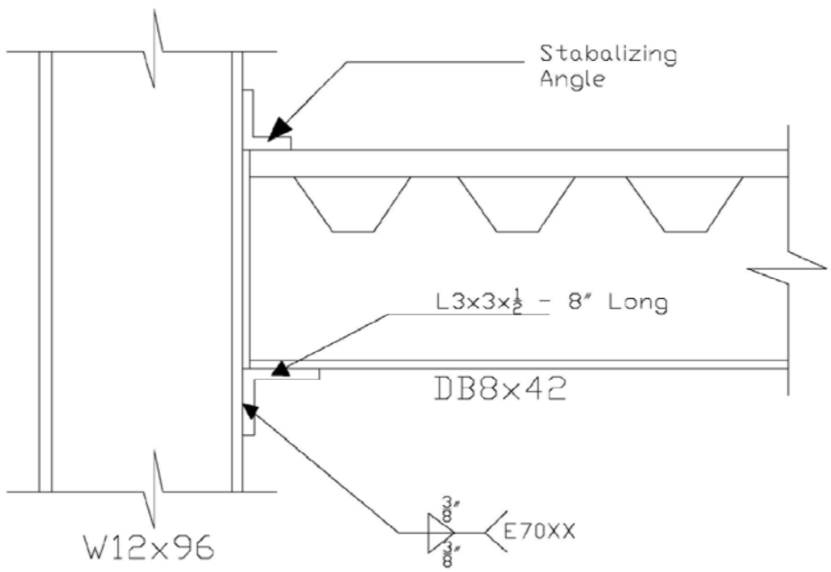


Figure 12: Connection detail using unstiffened seat connection at D-Beam – Column Flange location

Composite Beam Floor System

The first floor level of Southtown Building No. 5 was designed using a composite beam and concrete slab system. This system was chosen due to the increased live load per occupancy. Floor thickness was not critical in this floor because of the cellar bellow. Since the cellar is going to be an unoccupied space, the added thickness of the wide-flange girders will not add to the overall height of the building but detract from the story height of the cellar. With the first floor consisting of mostly public space, including the lobby, day care center, storage, fitness center, and conference room, the live load was increased from 40 psf of a typical residential floor to 100 psf. Using the column grid that was created by the Girder-Slab system above, the composite floor system has multiple bay sizes. The most common bay size, however, is a 27'-0" x 26'-0". Composite concrete and metal deck span perpendicular to beams spanning the 26'-0" distance and spaced 6'-9" o.c. Sixteen feet intermediate beams will then frame into the girders spanning 27'-0" which will, in turn, frame into the web of the wide-flange column.

The metal decking that was chosen was a 20 gauge USD 2" Lok-Floor deck with 4" concrete slab above for a total of six inches. The concrete compressive strength used was 3000 psi. The decking chosen was rated to span 7.85 feet without the use of shoring. The loading capacity of this deck is rated at 400 psf. Metal studs used were 4" x ¾" diameter, Grade 60.

The composite beams and girders were designed using American Institute of Steel Construction (AISC) Manual 13th Edition, Allowable Strength Design (ASD). The load combination used for this particular calculation was D+L. When designing a 27'-0" x 26'-0" bay by hand, a W12x19 with 14- ¾" diameter studs was found to be efficient as an intermediate beam. The maximum moment at midspan for this 26'-0" span was found to be 43.6 ft-kips.

The controlling factor for these intermediate beams was the deflection limitation. A moment of inertia that was required to limit the deflection to L/360 was figured to be 40.75 in^4. This moment of inertia was determined for a construction loading. The construction loading includes self-weight of the structure (i.e. deck, concrete, studs, and beams), as well as workers and equipment.

The deflections of the beams were also checked against live loads and total loads after the concrete cures and the system become fully composite. Given the industry standard of L/360 = 0.87". All beam sizes were well within this deflection criterion.

Once the intermediate beams were sized, a typical girder spanning 27'-0" was designed using a similar process. A maximum moment was found to be 121 ft-kips. By setting the deflection equation of a simple supported beam with 3 equal concentrated loads equal to the deflection limit of L/240. By manipulating this equation, a moment of inertia was found to be I_x= 448 in^4. This equation can be seen below.

Equation 4: $$f_b = \frac{M_{DL}}{S_b} + \frac{M_{SUP}}{S_{b(Transformed)}} \leq 0.9F_y$$

RAM results produced typical sizes for intermediate beams of W12x14 (18 studs) in this particular bay. This girder was checked against the necessary Ix value needed to limit the deflection. The girder in this bay spanning the 27’-0” length was sized W16x31 by RAM which was also checked against limiting deflection.

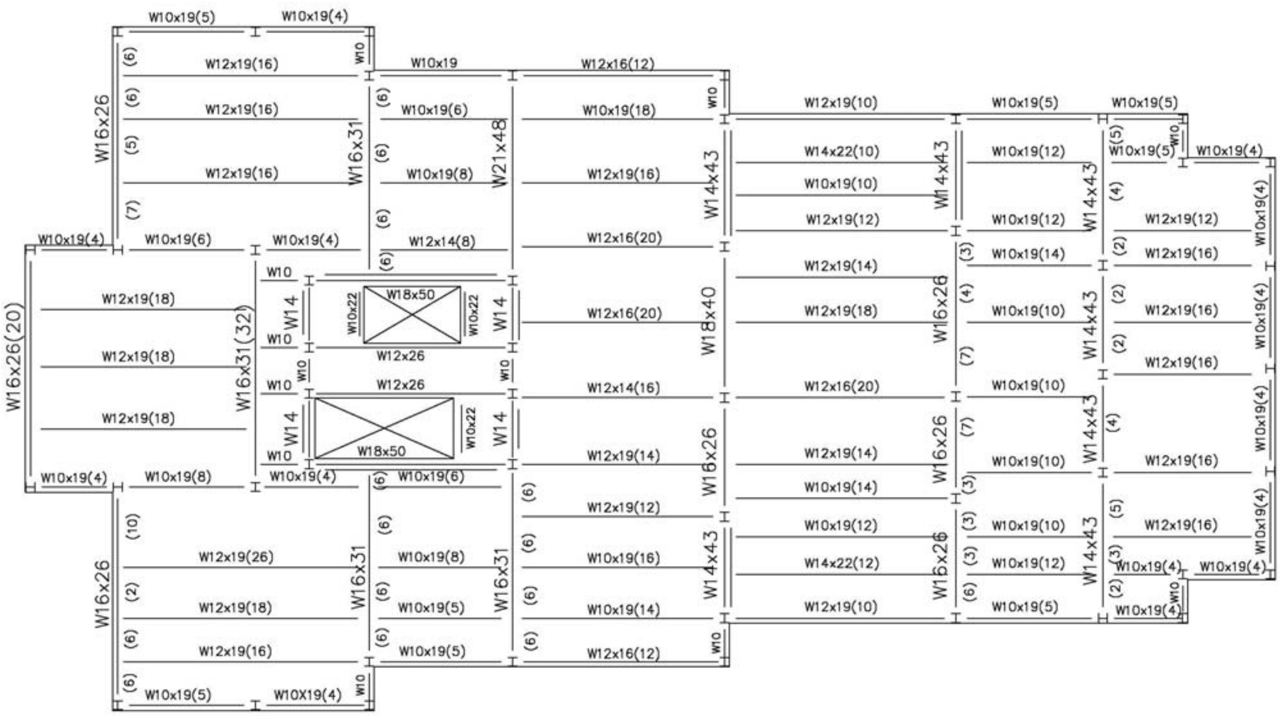


Figure 13: First floor structural plan utilizing steel beams and girders with composite steel deck

Composite Floor Connections

Three shear connections for the composite floor system were designed. All beams and girders that were modeled in RAM were assigned as pin-pin connections. In a typical bay, the three connections that were designed were 1) girder-web to intermediate beam-web, 2) girder-web to column-web and 3) beam-web to column-flange. The three connection types can be seen below in Figure 14.

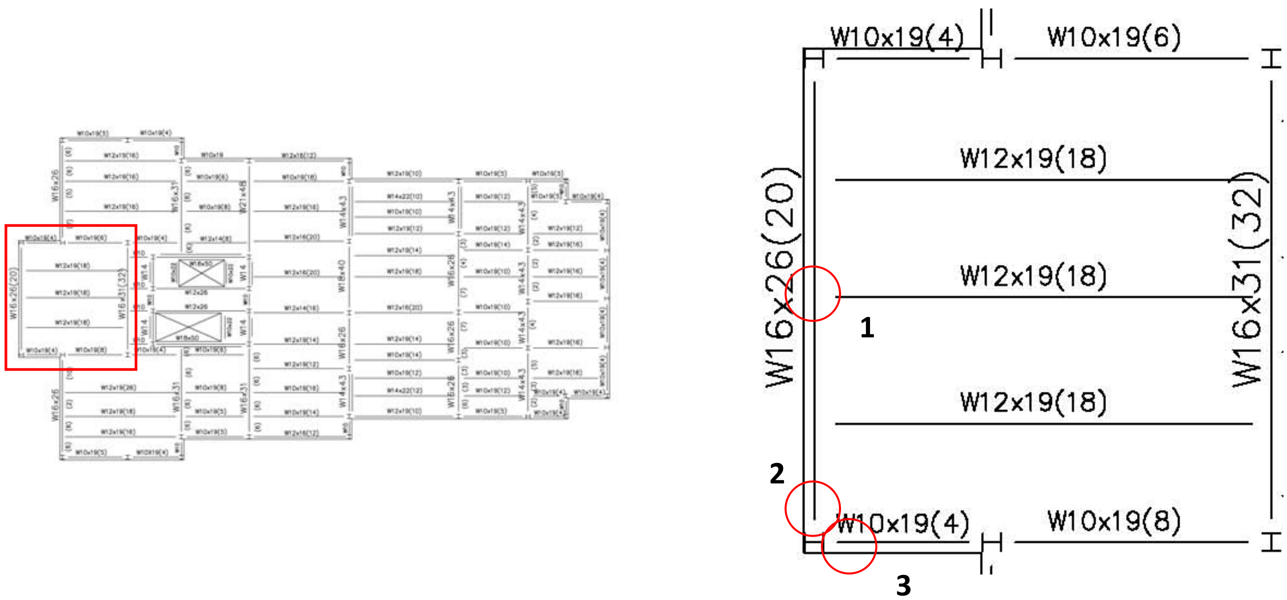


Figure 14: Typical composite steel bay with connection design locations. Typical bay located in red box on left.

Eccentric weld tables in Chapter 8 and design aids in chapter 10 of AISC Steel Manual, 13th Edition were used in the design of all connections. All calculations can be seen in the Appendix.

For connection 1, a 6"x5"x1/4" shear tab was used with 2-3/4" A325 Type-N Bolts. The beam will be coped at the top to allow for a level floor surface and ease of construction. A 3/16" E70XX fillet weld will be used to connect the shear tab to the web of the girder. This connection can be seen below in Figure 15.

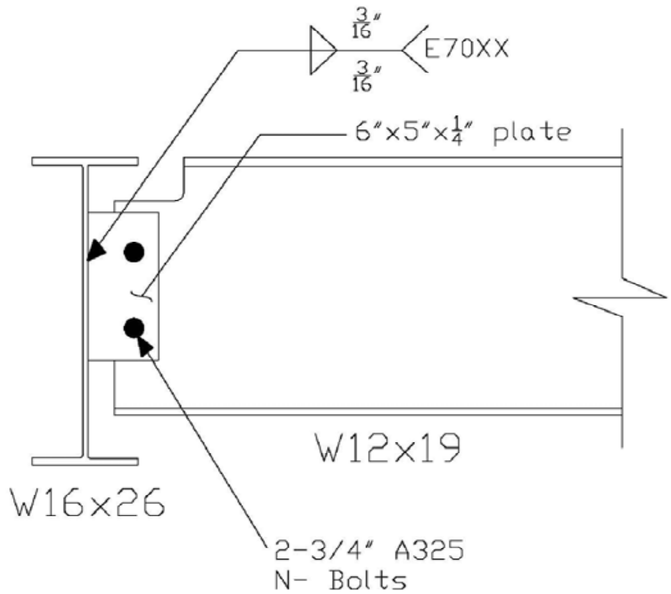


Figure 15: Shear tab connection at beam web – girder web location

For connection 2, a bolted/welded single angle connection was used. An 11-inch L4"x4"x3/8" single angle was utilized with 4-3/4" A325 Type-N Bolts. A 3/16" E70XX fillet weld will be used to connect the angle to the column web. A 3/8 inch weld return is employed at the top of the single angle. This connection can be seen below in Figure 16.

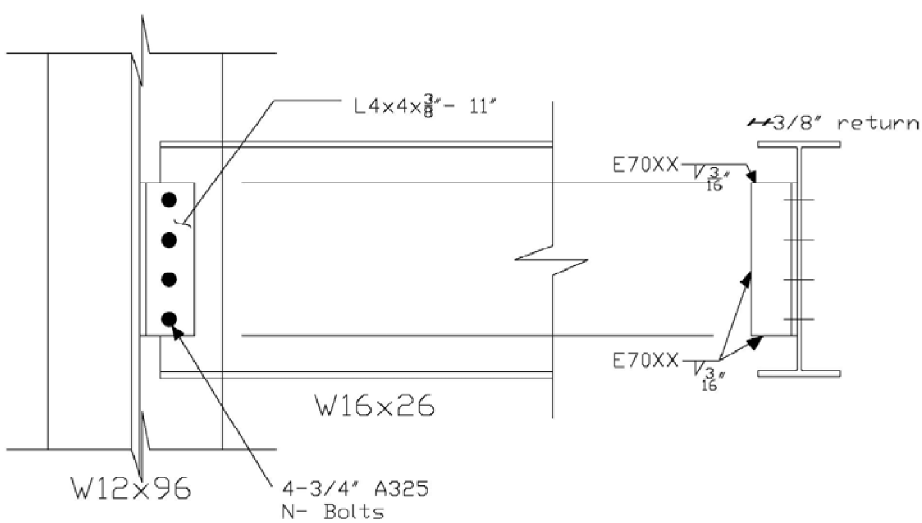


Figure 16: Bolted/welded single angle connection at girder web- column web location

Finally, for connection 3, bolted/welded double angle connection is used. A 6 inch L3”x3”x1/4” double angle was used with 2-3/4” A325 Type-N Bolts. The beam will be coped at the bottom for constructability. A 3/18” E70XX fillet weld will be used to connect the double angles to the flange of the column. This connection can be seen below in Figure 17.

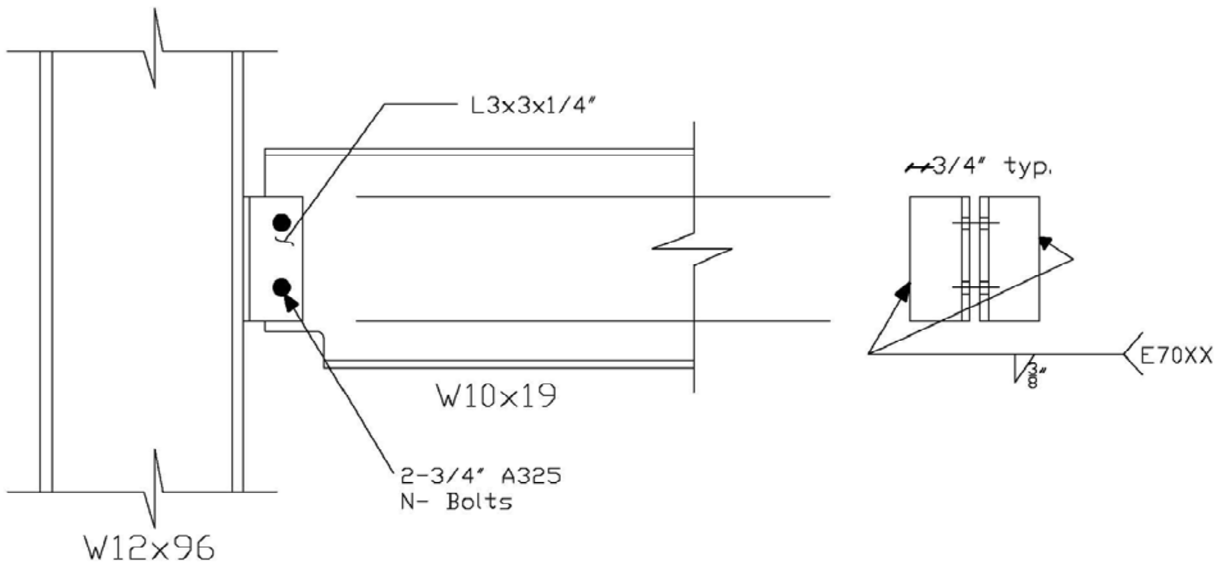


Figure 17: Bolted/welded double angle
connection at beam web-column flange location

Gravity Columns

Gravity columns in Southtown Building No. 5 were designed by RAM Structural System and cross checked with hand calculations. All calculations were done using Allowable Strength Design in accordance with the AISC Steel Manual, 13th Edition.

Columns were designed to be spliced at every 4 floors. This was purposely designed this way to allow for the fastest erection time possible. In a four-floor tier, the raising gang will erect the first two levels of framing and the decking crew will pour the topping material and level out the 2nd floor. After this, the raising gang will continue with the 3rd and 4th floors as the decking crew continues with the 1st floor. Following completion of the 1st floor, the decking crew continues with the 4th floor as the raising gang continues to the next tier. Finally, the decking crew finishes by decking the 3rd floor and the process repeats.

Columns were checked at three splice points, 4th, 8th and 12th floors. Most bays in the typical floors did not require tree columns for the Girder-Slab system to work properly but a few columns would require this connection type. For these particular columns, a combined loading of axial and bending occurs at the column. As previously mentioned, the designed tree column would be subjected to a 50 ft-kip moment. For these few columns, interaction equation H1-1a governed.

Equation 5: H1-1a: $\frac{\mathrm{Pr}}{Pc}+\frac{8}{9}\left(\frac{Mr}{Mc}\right)\leq 1$

Where P_r is the axial load on column

M_r is the bending moment on column

P_c is the axial strength of column

M_c is the bending strength of column

For all other columns that did not utilize a tree column connection, the design was based purely on axial loading. This total axial force was determined via column load take down which can be seen in the appendix. The axial force in a typical interior column was determined and then cross checked with the RAM results. Less than a 5% difference in error was found. For this same column, an elevation of the column line can be seen in Figure 18.

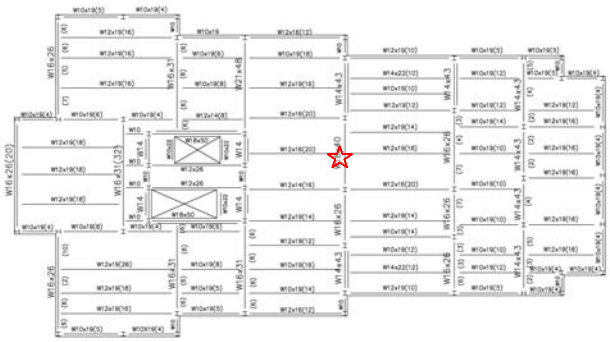
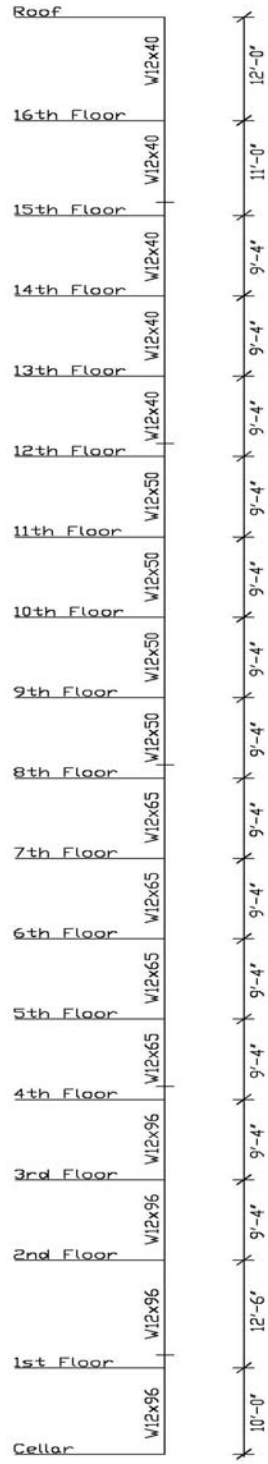


Figure 18: Gravity column line located at red star on floor plan above



Footing Redesign

With the proposal of switching from a concrete structural system to a steel system, comes the added benefit of reduced gravity loads. The original foundation wall is intended to stay in place to resist cladding and exterior column loads. Additionally, the mat foundation under the original shear walls can remain the same in depth and steel reinforcement but will be reduced in length since there is no long wall along the top in the North-South direction. This reduction in concrete mat will not add to the reduction in overall concrete since there will be newly placed braced frames in the North-South direction as described later in the Lateral Force Resisting System.

When redesigning typical interior footings, it was expected that the overall size of the footing would be reduced. However, when designing an interior footing, the size of the footings increased. This can be attributed to the larger spans and the reduced number of interior footings. With the use of steel beams instead of a flat plate floor system, the number of interior gravity columns was reduced from 13 on a typical concrete floor to 8 on a redesigned typical steel floor. Therefore, although individual footings will be increased in size, the overall amount of footings is reduced.

At the base, a ground column axial force of 617 kips must be transferred to the ground. As specified in the geotechnical report, an allowable bearing capacity of 12,000 psf can be used for foundations. At this interior footing location, a W12x96 was used. A 26" x 26" x 3"base plate was designed in according with AISC Steel Manual, 13th Edition. The base plate would be welded to the column and 4 anchor bolts would transfer the axial forces into the concrete pier. At this point, the concrete pier would then transfer the axial force to the footing.

The footing was designed by hand and cross-referenced with the Concrete Reinforcing Steel Institute (CRSI) Handbook, 2002. The design produced a 6'-0" square footing, 26 inches deep with (9) #6 bars in each direction compared to the existing footing, a 4'-6" square footing, 30 inches deep with (8) #8 bars in each direction. The new design requires 2.9 cubic yards per footing while the old design requires only 1.875 cubic yards. However, taking into account that there are 5 more interior footings in the original design, there is less overall concrete used in the proposed foundation design.

Structural Lateral System

Wind Design

Wind Loads were computed using Chapter 6 of ASCE7-05. Basic wind speeds for New York City were taken as 110 mph. The building exposure category was chosen to be C. The general parameters of the wind calculations can be seen in Table 1 below.

Table 1: General parameters	
Classification Category:	II
Basic Wind Speed, V:	110 mph
Importance factor, I:	1
Mean recurrence interval:	50 year
MRI factor:	1
Exposure Category:	C
a:	9.5
zg:	900
Topographic factor, Kzt:	1
Wind directionality factor, Kd:	0.85
Gust Factor, G (x-dir wind):	1.01
Gust Factor, G (y-dir wind):	0.969
Internal pressure coefficient, +GCpi:	0.18
Internal pressure coefficient, -GCpi:	-0.18
Windward pressure coefficient, Cp:	0.8
Side pressure coefficient, Cp:	-0.7

When sizing the lateral system for Southown Building No. 5, hand calculated wind forces were inputted into a RAM frame model. The applied story forces were then analyzed and found to be within 5% of the hand calculated story forces. Table 2 below shows a comparison between hand calculated story forces and RAM output story forces.

Table 2: Wind Applied Story Forces (kips)					
Level	Height (ft)	Manual	RAM Ouput	Manual	RAM Output
		N-S	N-S	E-W	E-W
Roof	157	39.9	43.11	20.3	21.92
16	145	75.7	79.26	38.4	40.2
15	134	68.4	72.24	34.6	36.54
14	124	61.3	65.32	31.0	33.03
13	115	57.5	61.61	29.0	31.08
12	106	60.1	64.24	30.3	32.39
11	96	59.3	62.6	29.9	31.57
10	87	55.5	59.26	27.9	29.79
9	78	57.8	61.28	29.0	30.75
8	68	56.9	59.9	28.4	29.9
7	59	53.0	55.19	26.4	27.49
6	50	54.8	55.53	27.2	27.56
5	40	53.4	54.87	26.4	27.12
4	31	49.2	49.73	24.2	24.44
3	22	50.0	49.84	24.4	24.29
2	12	55.6	50.01	27.0	24.16
	Total	908.2	943.99	454.4	472.23

Allowing RAM to calculate the different load cases given in Chapter 6 of ASCE7-05, the controlling load case was determined to be LC 1. Hand calculated wind forces, shears, and overturning moment is all shown below in Table 3.

Table 3: Wind Calculations						
Level	Force (k)		Shear (k)		Overturning Moment (ft-k)	
	N-S	E-W	N-S	E-W	N-S	E-W
Roof	39.9	20.3	39.9	20.3	6261	3179
16	75.7	38.4	115.5	58.6	16753	8499
15	68.4	34.6	183.9	93.2	24646	12494
14	61.3	31.0	245.2	124.2	30403	15402
13	57.5	29.0	302.7	153.2	34807	17622
12	60.1	30.3	362.7	183.5	38450	19453
11	59.3	29.9	422.1	213.4	40517	20483
10	55.5	27.9	477.6	241.3	41549	20989
9	57.8	29.0	535.4	270.2	41761	21079
8	56.9	28.4	592.3	298.7	40274	20309
7	53.0	26.4	645.2	325.1	38069	19179
6	54.8	27.2	700.1	352.3	35003	17616
5	53.4	26.4	753.5	378.8	30140	15150
4	49.2	24.2	802.7	403.0	24882	12492
3	50.0	24.4	852.6	427.4	18758	9403
2	55.6	27.0	908.2	454.4	10898	5453
Total					473173	238804

Seismic Design

Seismic loads applied to the building were computed in accordance with chapters 11, 12 and 19 of ASCE7-05. Roosevelt Island Southtown Building No. 5 has a site class of C and a seismic design category of B, thus allowing, by code, the use of the Equivalent Lateral Force Method. The Seismic Design Criteria can be seen below in Table 4.

Table 4: Seismic Design Criteria	
Ss	0.36
S1	0.07
Site Class	C
Fa	1.52
Fv	2.4
SMS	0.544
SM1	0.168
SDS	0.363
SD1	0.112
Ct	0.02
hn(ft)	187.25
x	0.75
Ta	1.02
TL	6
k	1.255
Occ. Category	II
Importance factor (I)	1
Seismic Design Cat.	B

As for wind, the seismic parameters were inputted into RAM and the Equivalent Lateral Forces were then calculated for the building stories. A comparison of these forces can be seen in Table 5.

Table 5: Equivalent Lateral Forces				
Level	Height (ft.)	Weight (k)	Manual Force (k)	RAM Output Force (k)
Main Roof	157	2006	81.8	84.2
16	145	1221	45.0	47.6
15	134	1221	40.8	43.5
14	124	1221	37.2	40.0
13	115	1221	33.8	36.6
12	106	1221	30.4	33.0
11	96	1221	27.0	28.9
10	87	1221	23.8	26.0
9	78	1221	20.6	22.4
8	68	1221	17.6	18.9
7	59	1221	14.6	15.3
6	50	1221	11.8	11.5
5	40	1221	9.0	9.0
4	31	1221	6.5	5.9
3	22	1221	4.1	2.8
2	12	1221	2.0	2.3
		Total	406.0	427.9

It can be seen that the Total base shear for Seismic Design is 470 kips. Although the overall weight of the building is reduced with the use of a structural steel and hollow core plank system, the design parameters change. Instead of using an R value of 4 for concrete shear walls, 3.25 is used for concentric braced frames. This lower R value, in addition to a lower period increases the seismic response coefficient, Cs. When the base shear is determined, the higher Cs value is multiplied by the overall weight of the building and, thus creating a higher base shear. These formulas can be seen below.

Equation 6: Minimum of $C_s = \dfrac{S_{DS}}{\left(R/I\right)}$, $C_s = \dfrac{S_{D1}}{T\left(R/I\right)}$ for T ≤ T_L

Where: S_{DS} = the seismic design spectral response acceleration parameter in the short period range as determined from Section 11.4.4 of ASCE7-05

R = response modification factor in Table 12.2-1 of ASCE7-05

I = the occupancy importance factor determined in accordance with Section 11.5.1 of ASCE7-05

S_{D1} = the design spectral response acceleration parameter at a period of 1.0 s, as determined from Section 11.4.4 of ASCE7-05

T = the fundamental period of the structure determined in Section 12.8.2 of ASCE7-05

Equation 7: $T_a = C_t h_n{}^x$

Where: h_n is the height in ft. above the base to the heighest level of the structure

C_t and $_x$ are determined from Table 12.8-2 of ASCE7-05

As shown in the above tables, the wind forces and base shear are much more dominant in the design of the building's lateral system. The base shear generated by the wind loads in the East-West direction amount to 454 kips and 908 kips in the North-South direction. Therefore, the building's braced frame lateral system was designed based on wind criteria and calculations.

Braced Frame Design

In order to keep consistent with the change of the gravity system from reinforced concrete to a Girder-Slab and composite steel system, the existing shear wall lateral system was replaced with a braced frame steel system. In the existing system, the shear walls are typically surrounding the elevator core in the building as seen in Figure 19.

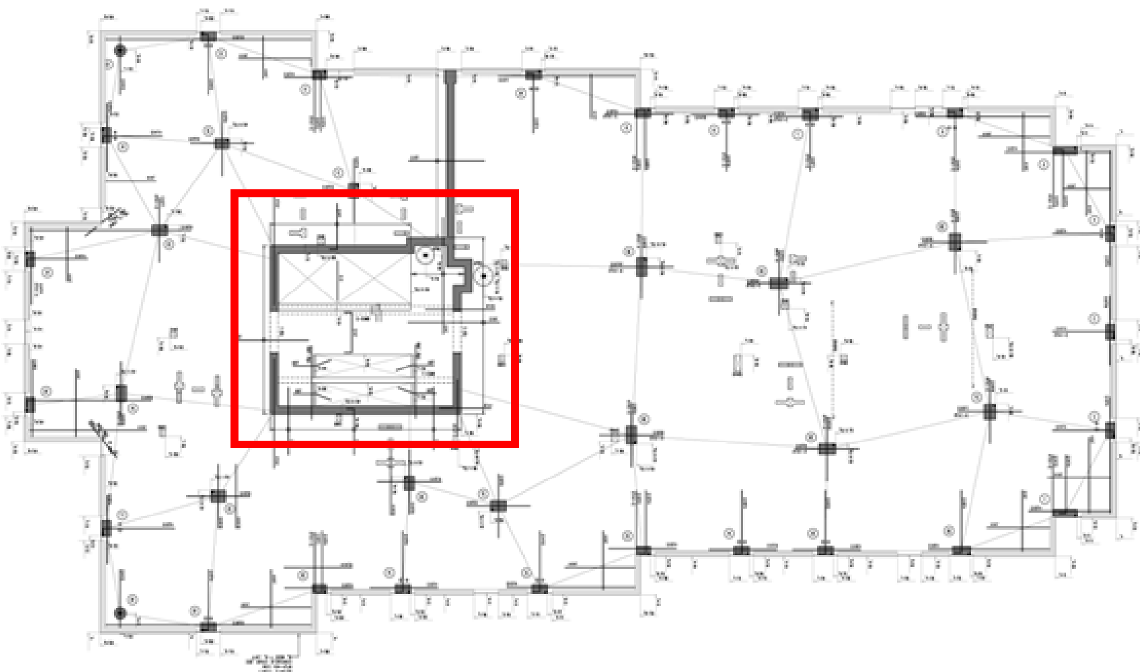


Figure 19: Typical existing structural floor plan with the elevator and stairwell core boxed in red

By replacing these walls with braced frames, the system needed to have more frames throughout the building in order to account for the long length of the North-South facing wall. This posed to be a problem with the existing architecture of the building. As shown in Figure 20, a layout of the braces can be seen.

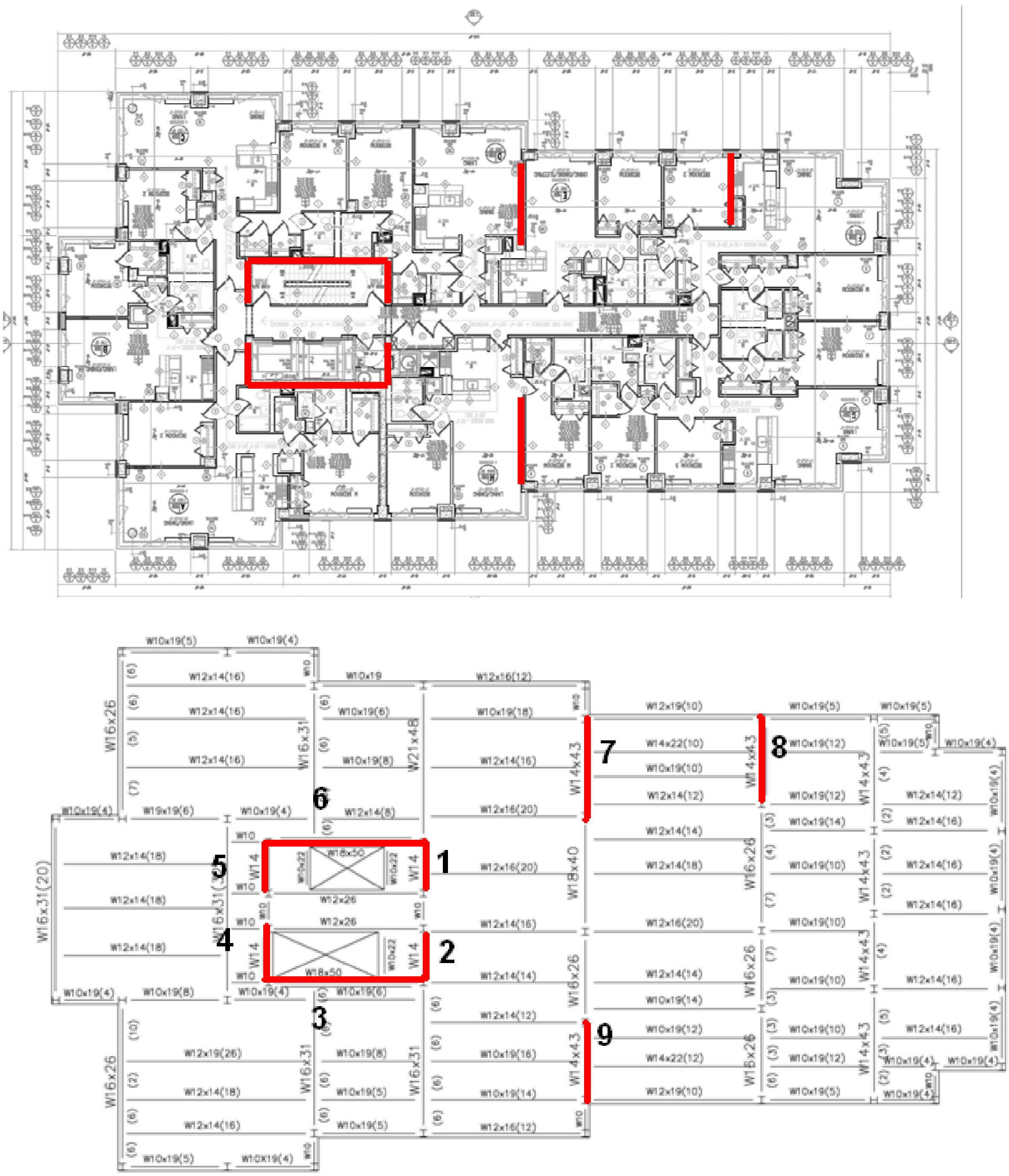
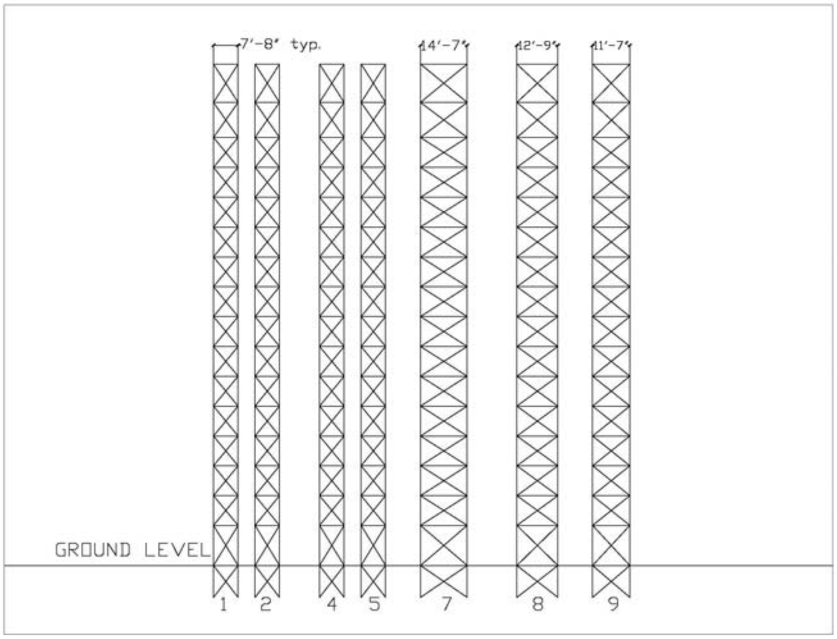


Figure 20: Above, the architectural floor plan can be seen with existing room layouts and partition walls. Below, a typical structural floor plan utilizing Girder-Slab floor system. Braced frame lateral systems are highlighted in red. Numbers correspond to frame number.

These braces utilize two different types of braced frames: the chevron brace and the cross brace. Frames 3 and 6 utilize the chevron brace while all of the other are fully crossed. Elevations for these braces can be seen in Figure 21.



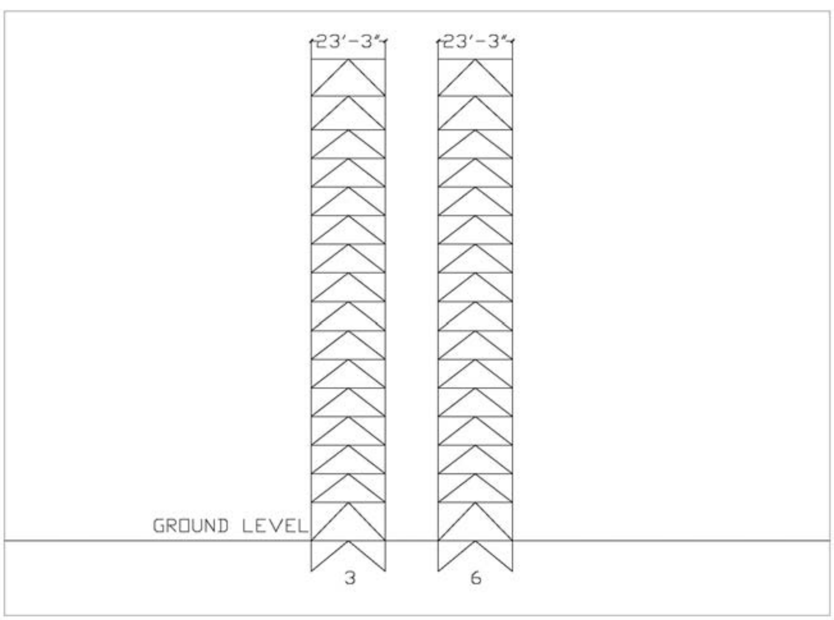


Figure 21: Braced frame lateral system. Frames 1, 2, 4, 5, 7, 8 and 9 utilize diagonal cross bracing while Frames 3 and 6 utilize Chevron braces.

The brace frames were designed using ASD load combination taken from ASCE7-05. The frames were assigned as gravity columns first and then assigned to lateral columns in RAM frame to determine initial member sizes. Once the story forces were applied to the building, the design of the frames became an iterative process. The overall displacement and torsion of the building was determined using RAM frame and it became apparent that the torsion was the controlling design factor. Using the industry standard of L/400 for the overall building displacement, 4.71 inches was the maximum displacement at the main roof. For Southtown Building No. 5, the maximum displacement was 3.53 inches due to wind in the N-S direction which occurred at the Main Roof level. Column sizes were increased to make sure that the displacement was within limits. A comparative analysis of story displacements and allowable displacements can be seen below in Table 6.

Table 6: Story Displacements			
			Wind
Level	Height (ft)	L/400 (in)	Max Displ. (in)
Roof	157	4.71	3.53
16	145	4.35	3.27
15	134	4.02	3.03
14	124	3.72	2.8
13	115	3.45	2.57
12	106	3.18	2.34
11	96	2.88	2.1
10	87	2.61	1.86
9	78	2.34	1.63
8	68	2.04	1.39
7	59	1.77	1.16
6	50	1.5	0.94
5	40	1.2	0.74
4	31	0.93	0.54
3	22	0.66	0.35
2	12	0.36	0.18
1	0	0	0

When the total building displacement was within limits, members were checked using RAM steel check and ASD load combinations from ASCE7-05. The controlling load combination for members varied throughout the frames. Members were sized accordingly in order to meet all necessary code requirements. Story displacement by seismic loading was also within acceptable code limitations. The maximum story displacement was found to be 3.1" at the main roof.

Maximum story drift for seismic loading was found to be 0.257". By multiplying this value by a Cd factor of 3.25, the code drift value was found to be 0.835". From Chapter 12 of ASCE7-05, the allowable story drift $\Delta = 0.020h_{sx}$ for occupancy category II and braced frame lateral system. Given that the building height is 157 feet, the maximum allowable story drift by code is 3.14" which is much larger than 0.835". Additionally, no torsional irregularity is considered for Southtown Building No. 5 since it is in the seismic design category B.

Overturning moment of the lateral system was checked for punching shear for the columns through the mat slab foundation. These calculations require the mat to be 36" thick. Since the existing mat foundation is 42" thick it will be able to resist the punching shear forces from the steel frame.

All member sizes and calculations can be found in the Appendix. Member sizes for braced frame 6 and 7 and 9 can be seen below in Figures 22 and 23, respectively.

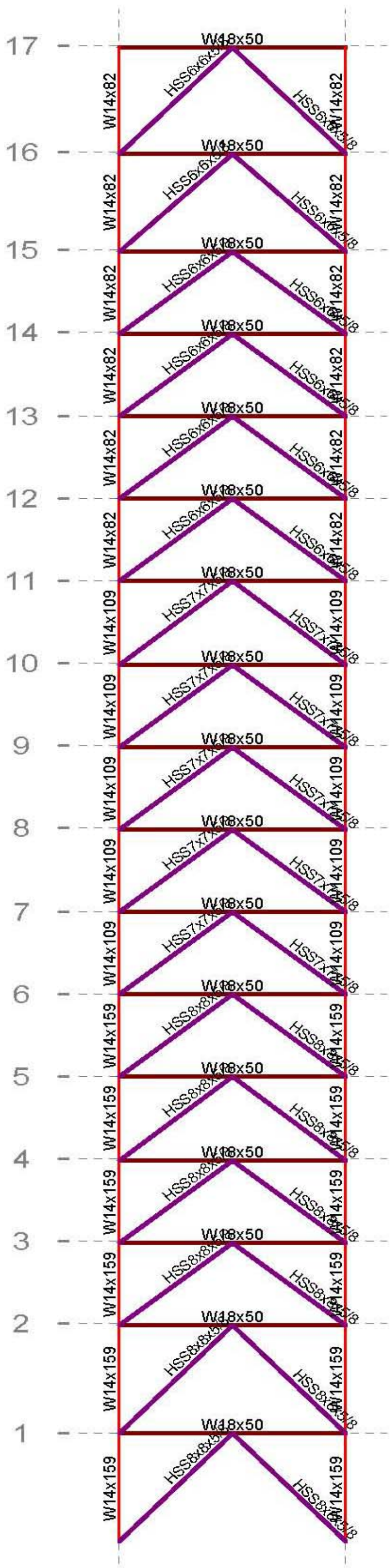


Figure 22: Frame 6 member sizes

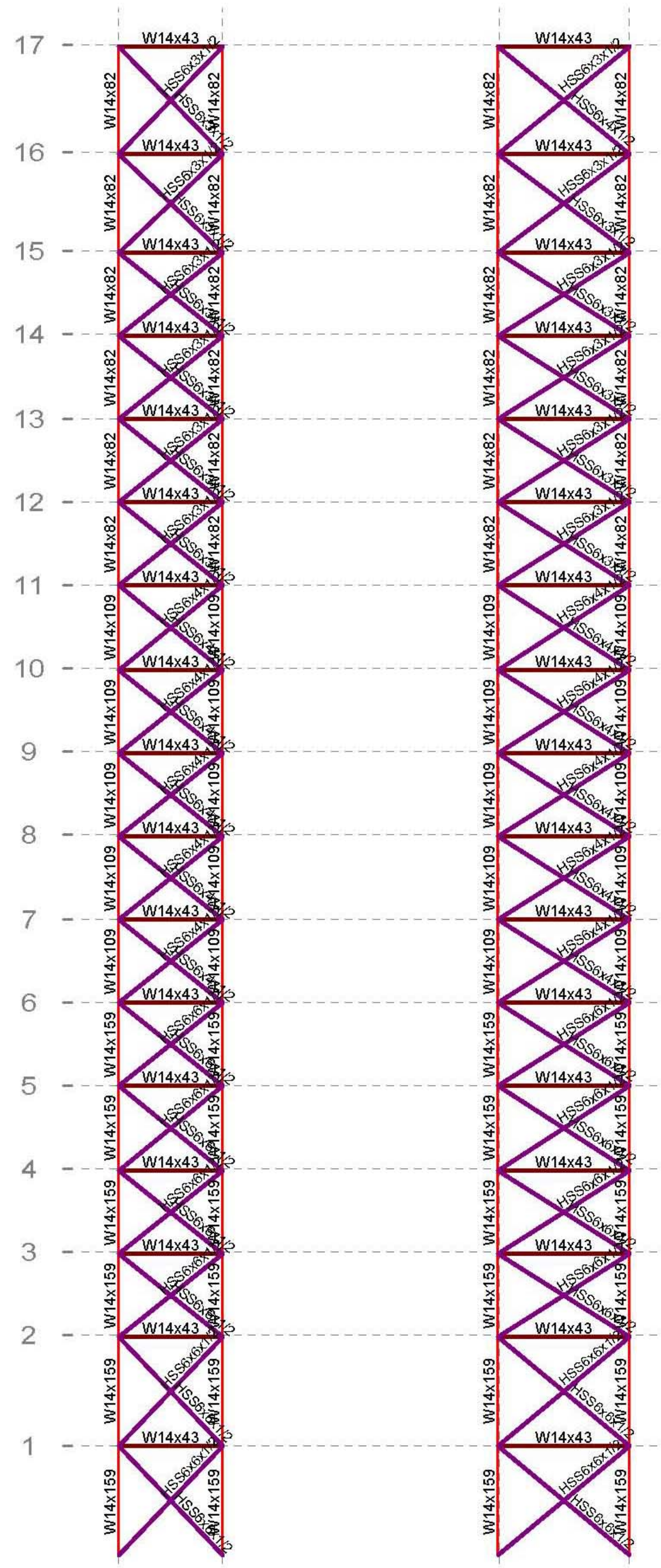


Figure 23: Frame 7 (left) and Frame 9 (right) member sizes

RAM Structural System Building Model

Southtown Building No. 5 was modeled using Bentley System RAM Structural System. To model the Girder-Slab system as accurately as possible, floors 2-Roof were modeled using a one-way deck. The deck was assigned the same weight as the 8" hollow-core floor planks and the ¾" topping material as specified before. Girder-Slab members were not designed using RAM but the model had to represent the typical floors as closely as possible to determine forces acting on adjacent columns and frames. The first floor was modeled using a composite steel deck and concrete system. Surface loads were applied to the floor diaphragms to accurately simulate applied forces on the floor slab. A 3D image of the RAM building model can be seen below in Figure 24.

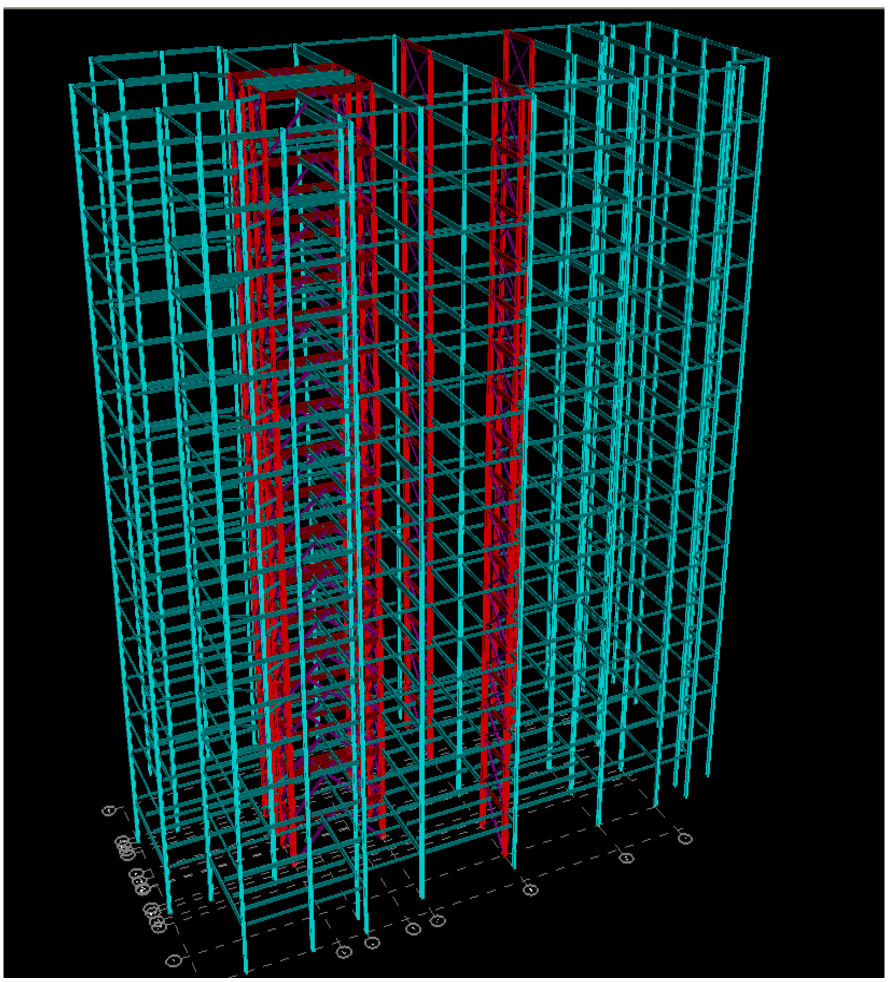


Figure 24: 3D RAM Building Model