Structural Depth Study – System Optimization

As previously described, the current structural design places a seven-story concrete system atop a three-story historic steel and load-bearing masonry system. The building weight is increased substantially, and both the foundation design and magnitude of reinforcement required on the historic members respond accordingly to compensate for this new load. In addition, none of the historic system is utilized in the LFRS; shear walls are implemented instead.

The intent of this study is to explore the implementation of an alternate design solution and the resulting implications on the aforementioned aspects of the structure. The proposed solution is to replace the modern concrete system with a steel framing system. The goals of this proposed design are as follows:

1) Reduce the overall weight of the building so that the size of the foundation might be decreased.
2) Decrease the size of the members reinforcing the long-span transfer beams on the 4th level, or change the type of reinforcing entirely to that of a smaller magnitude.
3) Change the column strengthening method from concrete encasement to steel plate reinforcement.
4) Utilize historic members in the LFRS.

In anticipation of the new structural system – in which the total floor system thickness was expected to increase by at least 12" due to new steel beams – the story heights of the new floors were increased by 6" each. This was done to ensure adequate space for the HVAC system in the proposed design, while having minimal effects on the floor-to-ceiling heights and the interior architectural aesthetics. These story height increases brought the overall building height to 119.986', which is just under the maximum allowed building height within the current zoning ordinance. (This ordinance states that the total building height, excluding parapets and bulkheads, shall be 120' for a building in the C6-2A / R8A contextually sensitive zone.)

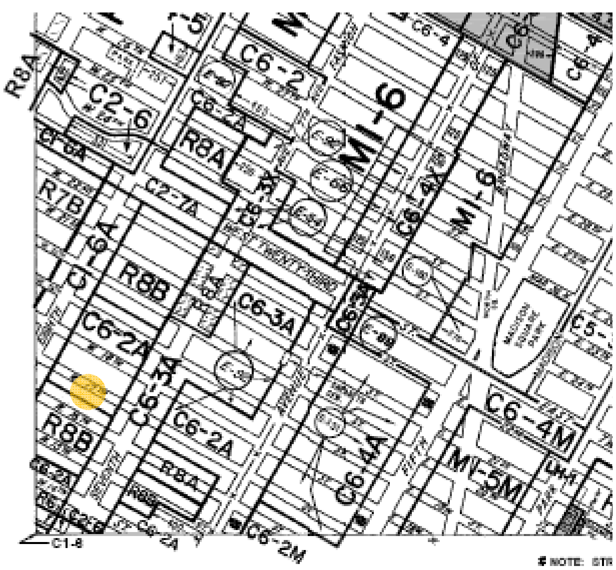

Figure 9: Zoning map depicting location of 246 West 17th Street

Let it be noted that these new story heights were factored into all structural analysis for the proposed design.

Gravity System Study and Design

Floor system design and fireproofing

The proposed slab-on-deck floor system was chosen based on the aforementioned gravity loads using the *United Steel Deck Catalogue* as a basis for design. Based on required span lengths and service load values, a 3-inch Lok-Floor composite system was found to be adequate when paired with 4ksi lightweight concrete. To ensure the accuracy of the design loads and particularly the self weight of the floor system, the proposed design was inserted into the RAM SS model before performing beam analysis and design.

For residential occupancies such as 246 West 17th Street, the NYCBC requires a 2-hour fire separation between floors and individual units per ASCE7-05. To meet this standard, the slab need only be 4½” in total thickness; however, a lightweight system of this thickness is prone to floor vibration. To reduce the effects of this vibration, a 6” slab shall be used instead. The underside of the deck does not need to be sprayed with fireproofing, but all exposed steel of the beams and columns shall need to be sprayed.

Preliminary research of existing historic steel shapes

The exact sizes of the existing structural steel could not be determined by visual inspection due to the nature of the draped mesh system, in which the tops of the beams and girders are encased in concrete. For this reason, a copy of the 1925 construction documents were obtained and deciphered to determine the historic beam sizes. The weight and moment of inertia about the strong bending axis of each historical section was determined using the AISC Historical Shapes Database Search Utility, along with the *AISC Manual of Steel Construction*, 13th edition. The said values were then used to find comparable modern sections to be input in the proposed design, which are listed in Figure 10 below.

Historic Section				**Modern Section**			**Section differences**	
Plan Description		*W [plf]*	*I_x [in^4]*	*Size*	*W [plf]*	*I_x [in^4]*	*W [plf]*	*I_x [in^4]*
5" (B5)	Beam	18.9	23.8	**W5x19**	19	26.3	0.1	2.5
10"-23.5# (B10)	Beam	23.5	122.9	**W10x22**	22	118	-1.5	-4.9
12"-28.5# (B12)	Beam	28.5	216.2	**W12x26**	26	204	-2.5	-12.2
12"-36# (B12A)	Beam	36	269.2	**W12x30**	30	238	-6	-31.2
14"-33# (B14)	Beam	33	334.3	**W14x34**	34	340	1	5.7
15"-33# (B15)	Beam	35	367.9	**W14x34**	34	340	-1	-27.9
15"-38# (B15)	Beam	38	442.6	**W14x38**	38	385	0	-57.6
15"-41# (B15)	Beam	41	456.7	**W14x43**	43	428	2	-28.7
15"-46# (B15)	Beam	46	484.8	**W14x48**	48	484	2	-0.8
15"-56# (B15)	Beam	56.5	742.3	**W16x57**	57	758	0.5	15.7
(2) 18"-48# (B64)	Beam	96	1474.2	**W16x100**	100	1490	4	15.8
24"G-140# (G24A)	Girder	140	4201.4	**W24x131**	131	4020	-9	-181.4
26"-90# (B26)	Girder	90	3043.1	**W24x94**	94	2700	4	-343.1
26"G-160# (G26)	Girder	160	5576.6	**W24x162**	162	5170	2	-406.6
28"G-180# (G28A)	Girder	180	7264.7	**W27x178**	178	7020	-2	-244.7
8"-32# (H8)	Column	32	105.7	**W8x31**	31	110	-1	4.3

Figure 10: Comparable modern sections for historic beams

Steel Beam Analysis and Reinforcement: Historic Members

The historic beams were modeled using the comparable sections found above while using a yield strength of 30ksi, and then loaded to test their performance under the new design loads. The majority of the historic members were found to be quite adequate, but a few that were subjected to higher live loads failed in bending toward the middle of the span-length. These members were noted to be long-span girders located on the 1st and 3rd floor, as shown in Figure 11 below and Figure 12 on the next page.

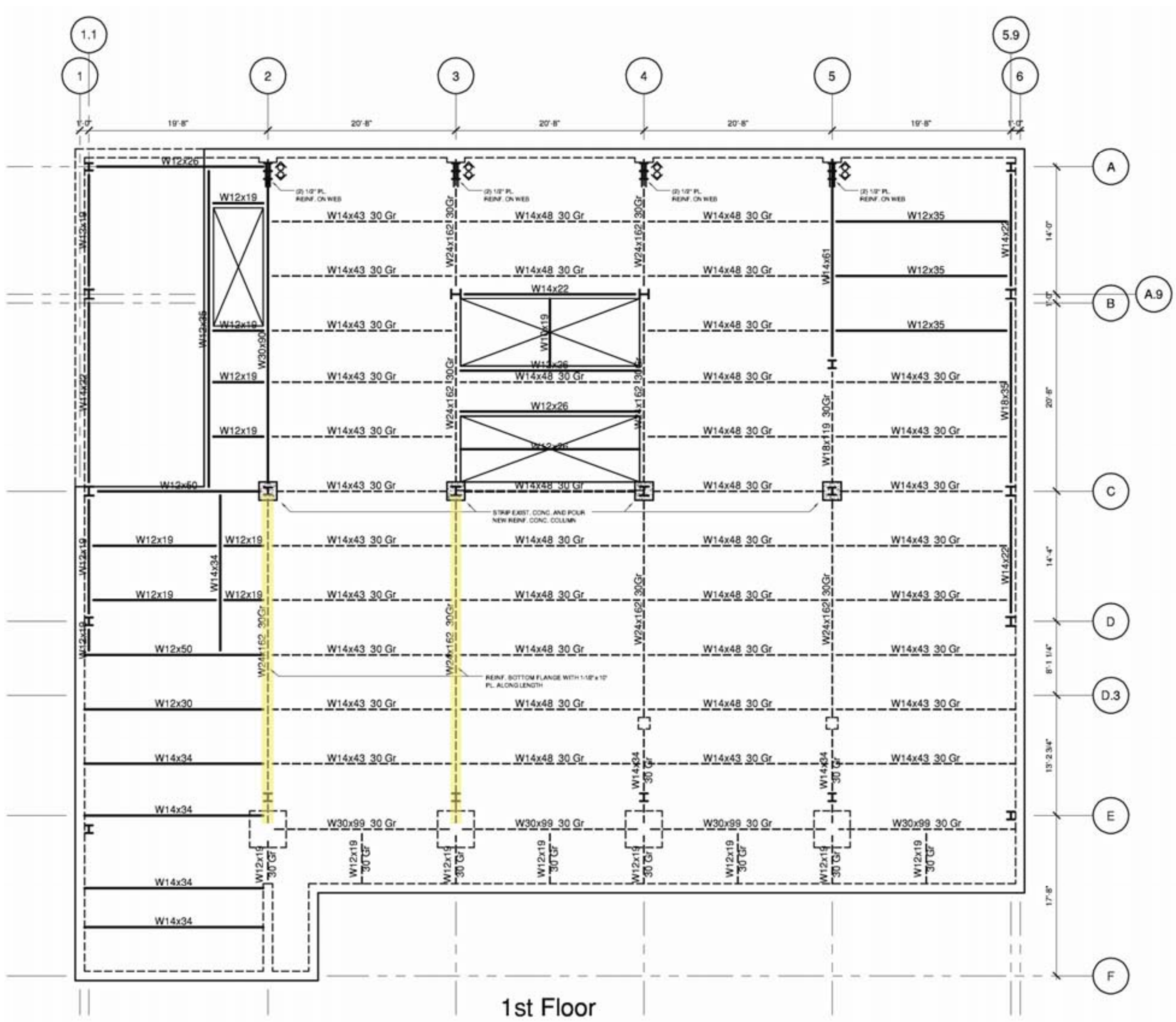

Figure 11: Noted girder failures on the 1st Floor

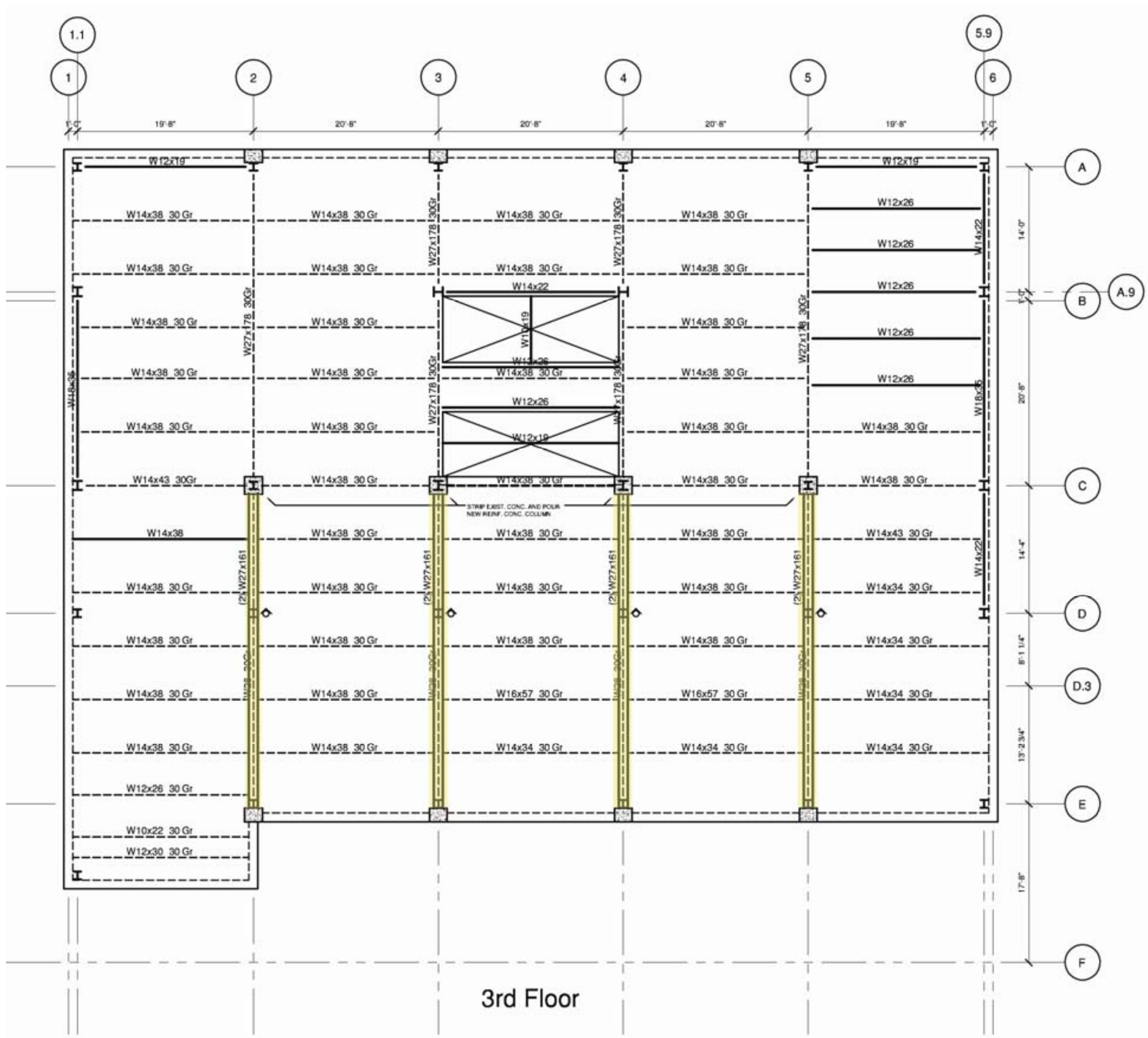

Figure 12: Noted girder failures on the 3rd Floor

To reinforce these long-span girders, various calculations were carried out that involved welding a strengthening member to the underside of the failing member. Strengthening members that were considered include WT-shapes, plates, and W-shapes. For the design of these strengtheners, simple hand calculations were executed based on the strengths and geometries of the two components. The plastic moment capacity ϕM_n was determined for the paired combination and compared to the maximum moment M_u acting on the beam. (Detailed calculations of this analysis can be found in the Appendix B.)

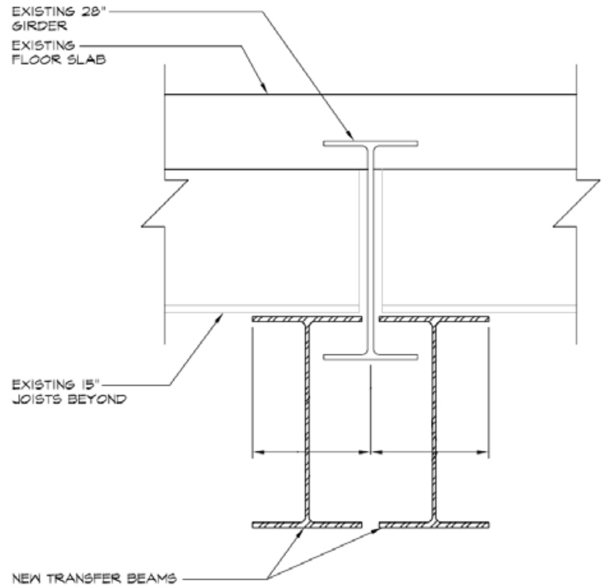

Figure 13: Section showing stacked girders as reinforcing members

As anticipated, the adequate reinforcing members for the 3rd Floor transfer beam was able to be decreased from that of the current design due to the decreased building weight: the required design went from (2)W27x194 beams to (2)W24x176 beams. See the figure at right for a detail of this reinforcing.
The adequate reinforcing for the failing 1st Floor long-span beam was found to be a 1.5”x10” (in cross-section) plate welded along the length of the beam.

Steel Beam Design: Modern Members

The new beam designs were limited to a depth of 14” (for interior beams) to minimize the effect on the architecture within. Non-composite design was first explored for all new stories; however, the 14” beam-depth restriction could not easily be met in many areas without also seeing a substantial increase in beam weight. Composite design was hence explored, within which a construction dead load equal to the weight of the wet concrete was added to the model.

To evaluate the economy of each optimized design, a comparison was made between the total weights of the composite and non-composite design options, seeing as the cost of steel is directly related to the total tonnage. To account for the shear studs along the length, each stud was assumed have an equivalent weight of 10 pounds of steel. The results proved to favor the composite design across the board. Below are sample calculations featuring standard beam designs that were evaluated for beam economy.

Composite vs. Non-Composite Beam Design					
Bm. Length [ft]	Design Options	Beam Size	# Studs	Total Wt. [lbs]	Final Design
20.67	Composite Design	W12x 14	8	**369.38**	W12x14 (8)
	Non-Composite Design	W14x 30	0	620.1	
21.67	Composite Design 1	W12x 14	27	**573.38**	W12x14 (27)
	Composite Design 2	W12x 26	14	703.42	
	Non-Composite Design	W14x 53	0	1148.51	
14.33	Composite Design 1	W14x 61	20	**1074.13**	W14x61 (20)
	Composite Design 2	W14x 53	36	1119.49	
	Non-Composite Design	W14x 90	0	1289.7	
14.33	Composite Design	W14x 74	28	**1340.42**	W14x74 (28)
	Non-Composite Design	W14x 109	0	1561.97	
Note: Shear stud equivalent weight taken as 10 lbs/stud					

Figure 14: Composite design economy and justification

Column design per gravity loading

Columns were designed using RAM SS to meet strength and serviceability provisions per ASCE7-05 and IBC-06. First, columns were analyzed and sized according to gravity loading, then those involved in the LFRS were checked under lateral loading (as will be discussed in upcoming sections of this report). Designs for the new steel members ranged between W12 and W14 members. These findings are detailed in the column schedule in Appendix B.

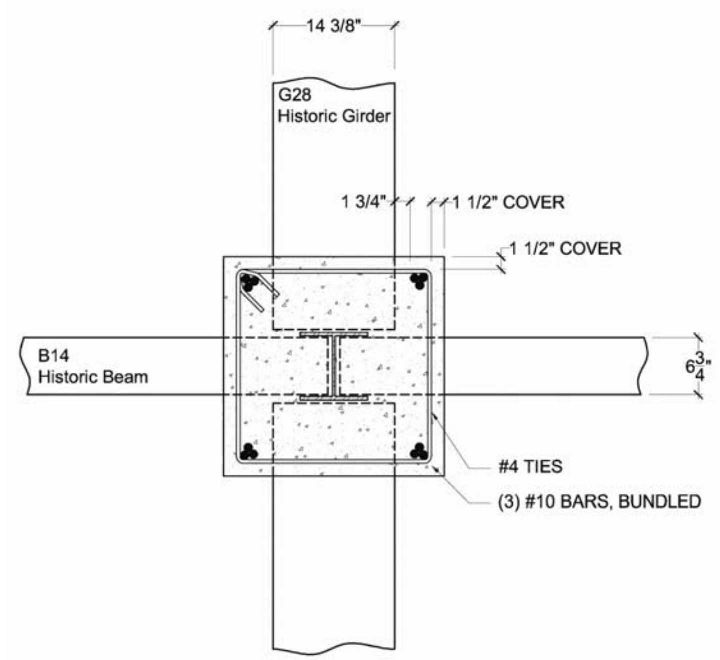

Figure 15: Plan view of historic beams framing into historic column; Subsequent concrete column design

As previously stated and illustrated in Figure 10, the historic members were inserted into the model using a comparable modern section to evaluate their condition under the proposed design loads. Due to the addition of the seven stories above, the historic members were found to fail under gravity loading.

To reinforce these members, the steel members were encased in a 4ksi concrete column. The historic steel was neglected, and instead minimum steel requirements were met using bar reinforcement. As shown in Figure 15 above, the geometry of the existing beam-to-column connection created a design challenge when considering how to run continuous reinforcement between stories: the beams prevented this from being done at all four sides, leaving only the corners open to do so. Hence, the solution was to bundled the rebar and confine it at the corners so that it could bi-pass the beams. The final column size was found to be 26”x26”; these were the minimum dimensions possible that would still allow for the rebar to be placed at the corners while meeting minimum concrete cover requirements. The design was investigated through the use of PCA column, with applied bending and axial loads that were obtained from the RAM SS model. The results of this PCA column investigation for the critical column case can be found in Appendix B of this report.

Lateral System Study and Design

As previously stated, the goals of the proposed design include the incorporation of historic members into the LFRS. To meet these goals, a steel system was chosen and placed in a location such that the adjacent columns and beams will be contributors to the lateral force resistance. Chevron braces consisting of rectangular hollow steel sections were selected as the primary method and placed similarly to the current shear wall locations to limit the effect on the interior architecture.

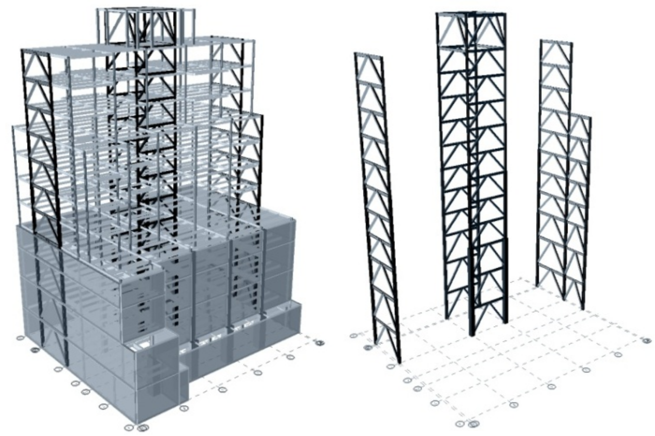

Figure 16: 246 West 17th Street lateral brace system

Design considerations

In the current design, the lateral load resisting properties of the historic masonry wall were neglected. For the proposed design, the walls running north-south were incorporated into the lateral system; these walls were repointed and left almost entirely intact, and they are therefore assumed to be able to take lateral load. The east-west running walls, however, were opened up substantially by the placement of new doors and windows, so the lateral resisting qualities of these walls were ignored in the proposed design.

All lateral loads were calculated and applied through use of the RAM SS program per ASCE7-05 and IBC-06 provisions. As previously noted, these values are accepted as being more accurate than the hand-calculated values.

Design challenges

1. Per ASCE7-05 design standards and recommendations, the story drift and overall deflection of the structure due to wind were limited to $h/600$ for the first 3 stories to limit the stress on the historic masonry wall. Above this level – where the exterior materials change from masonry to aluminum curtain wall – story drift and overall deflection due to wind was limited to $h/400$.

2. For seismic deflections, the story drift was limited to $0.020h$. The deflections obtained in the model results were elastic deflections, and therefore they had to be multiplied by the seismic amplification factor C_d to obtain the actual design deflections. These amplified values were required to meet the drift limit.

3. In addition to the said story drift limitations, a 2-1/4" seismic joint at the east end of the 6th Floor placed a more stringent limit on the overall story deflection at this level. Since building on the other side of the seismic joint is an 8 story masonry structure, it can be assumed that this

building shall deflect similarly to (if not less than) 246 West 17th Street under lateral loading. For this reason, the deflection of 246 West 17th Street was limited to half the width of the seismic joint (or 1-1/8") to account for sway from the other building, which would be coming from the opposite direction.

Results

After multiple iterations of unsuccessful trials, a virtual work analysis was run in the RAM SS program to view the members contributing most to the drift resistance. At last, the LFRS columns contributing most were realized, and so these were increased in size until drift criteria were met. The final deflections and story drifts as compared to the allowable values are shown below in Figure 16. The most efficient brace size was found to be that of HSS10x10x5/8 tubing.

DRIFT DATA			Wind Drifts [in]				Seismic Drifts [in]				
X-DIRECTION			Total Drift		Story Drift	Allowable Story Drift	Total Drift			Story Drift	Allowable Story Drift
Level	Total Ht.	Story Ht.	Load Case	Δ Wind	Δ Story	h/400	Load Case	Δ Elastic	Δ Amplified	Δ Story	$0.020h_{sx}$
BH	134.486	14.500	W1, W2	*1.869*	**0.230**	**0.44**	E2	0.684	*2.223*	**0.286**	**3.48**
Roof	119.986	11.167	W1, W2	*1.639*	**0.182**	**0.34**	E2	0.596	*1.937*	**0.224**	**2.68**
10	108.819	11.167	W1, W2	*1.457*	**0.184**	**0.34**	E2	0.527	*1.713*	**0.228**	**2.68**
9	97.652	11.167	W1, W2	*1.273*	**0.181**	**0.34**	E2	0.457	*1.485*	**0.224**	**2.68**
8	86.485	11.167	W1, W2	*1.092*	**0.177**	**0.34**	E2	0.388	*1.261*	**0.218**	**2.68**
7	75.318	11.167	W1, W2	*0.915*	**0.167**	**0.34**	E2	0.321	*1.043*	**0.205**	**2.68**
6	64.151	11.167	W1, W2	***0.748***	**0.161**	**0.34**	E2	0.258	***0.839***	**0.189**	**2.68**
5	52.984	11.167	W1, W2	*0.587*	**0.149**	**0.34**	E2	0.200	*0.650*	**0.172**	**2.68**
4	41.817	11.167	W1, W2	*0.438*	**0.138**	**0.22**	E2	0.147	*0.478*	**0.156**	**2.68**
3	30.65	14.400	W1, W2	*0.300*	**0.156**	**0.29**	E2	0.099	*0.322*	**0.172**	**3.46**
2	16.25	16.250	W1, W2	*0.144*	**0.144**	**0.33**	E2	0.046	*0.150*	**0.150**	**3.90**
1	0	0	N/A	*0*	**0**	**0**	N/A	0	*0*	**0**	**0**

W1 = Wind +X Direction
W2 = Wind –X Direction
W3 = Wind +Y Direction
W4 = Wind –Y Direction

E1 = Earthquake +X Direction
E2 = Earthquake –X Direction
E3 = Earthquake +Y Direction
E4 = Earthquake –Y Direction

$$\Delta_{amp} = \frac{C_d \Delta_e}{I}$$

$C_d = 3.25$
$I = 1.0$

$T_{calc} = 1.288$ seconds
$T_{model} = 1.292$ seconds

DRIFT DATA			Wind Drifts [in]				Seismic Drifts [in]				
Y-DIRECTION			Total Drift		Story Drift	Allowable Story Drift	Total Drift			Story Drift	Allowable Story Drift
Level	Total Ht.	Story Ht.	Load Case	Δ Wind	Δ Story	h/400, h/600	Load Case	Δ Elastic	Δ Amplified	Δ Story	$0.020h_{sx}$
BH	134.486	14.500	W3, W4	*1.629*	**0.014**	**0.44**	E4	1.979	*6.412*	**0.075**	**3.48**
Roof	119.986	11.167	W3, W4	*1.615*	**0.215**	**0.34**	E4	1.956	*6.337*	**0.862**	**2.68**
10	108.819	11.167	W3, W4	*1.400*	**0.223**	**0.34**	E4	1.690	*5.475*	**0.891**	**2.68**
9	97.652	11.167	W3, W4	*1.177*	**0.214**	**0.34**	E4	1.415	*4.585*	**0.836**	**2.68**
8	86.485	11.167	W3, W4	*0.963*	**0.206**	**0.34**	E4	1.157	*3.749*	**0.810**	**2.68**
7	75.318	11.167	W3, W4	*0.757*	**0.202**	**0.34**	E4	0.907	*2.939*	**0.797**	**2.68**
6	64.151	11.167	W3, W4	*0.555*	**0.193**	**0.34**	E4	0.661	*2.142*	**0.755**	**2.68**
5	52.984	11.167	W3, W4	*0.362*	**0.145**	**0.34**	E4	0.428	*1.387*	**0.687**	**2.68**
4	41.817	11.167	W3, W4	*0.217*	**0.177**	**0.22**	E4	0.216	*0.700*	**0.564**	**2.68**
3	30.65	14.400	W3, W4	*0.040*	**0.024**	**0.29**	E4	0.042	*0.136*	**0.075**	**3.46**
2	16.25	16.250	W3, W4	*0.016*	**0.016**	**0.33**	E4	0.019	*0.062*	**0.062**	**3.90**
1	0	0	N/A	*0*	**0**	**0**	N/A	0	*0*	**0**	**0**

Figure 17: Drift analysis results compared to allowable drift values

Design strength checks were carried out based on all lateral load combinations per ASCE7-05 as well. Drift was found to be the controlling factor, and all members passed as-designed for the drift limitation.

All diagonal braces shall be connected to columns using gusset plates at the base of the columns and at the mid-span of the beams. It should be noted that the brace frame connections at these locations shall be designed to carry the maximum probable brace force, "which may be approximated as 1.2 times the expected strength of the brace" (per the AISC Steel Design Guide Series 15: *Rehabilitation and Retrofit Guide*).

Effects on the Foundation System

The current foundation system was compared to the required foundation for the proposed design through the use of RAM SS. Considering the significant decrease in building weight, an attempt was made to resolve the mat slab in the northern portion of the into individual spread footings. Unfortunately, the required footing sizes were found to be too large and too close together for this to be economical. The design was hence converted back to a mat slab (with a slightly different geometry) and was found to have a required design thickness of 3'-0". It should be noted that this represents a 10" reduction in thickness when compared to the original foundation, which was 3'-10" thick.

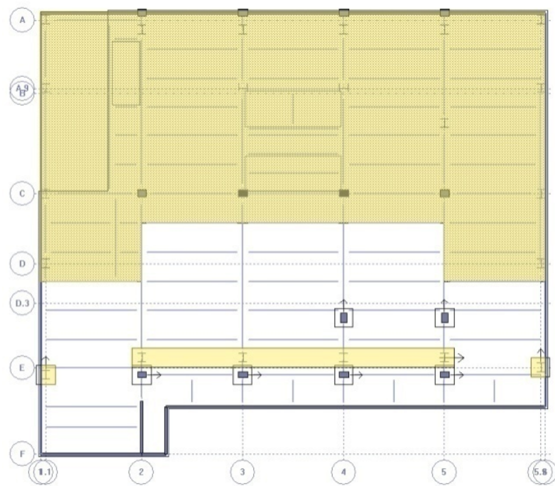

Figure 18: Final foundation layout

The current continuous footing was also analyzed in RAM SS in an attempt to optimize the design. As with the mat slab, individual spread footings were looked at but found to be too close together to be efficient. The design was reverted back to a continuous footing, in which the final design thickness was found to be 24". This is a significant decrease in thickness when compared to the original thickness of 46".

The final design layout is illustrated in Figure 18. A summary of the effects of the new design on the foundation may be found in Figures 19 and 20 on the next page. The cost savings associated with the optimization of the foundation system are shown in the upcoming pages of the Construction Management Breadth portion of this report.

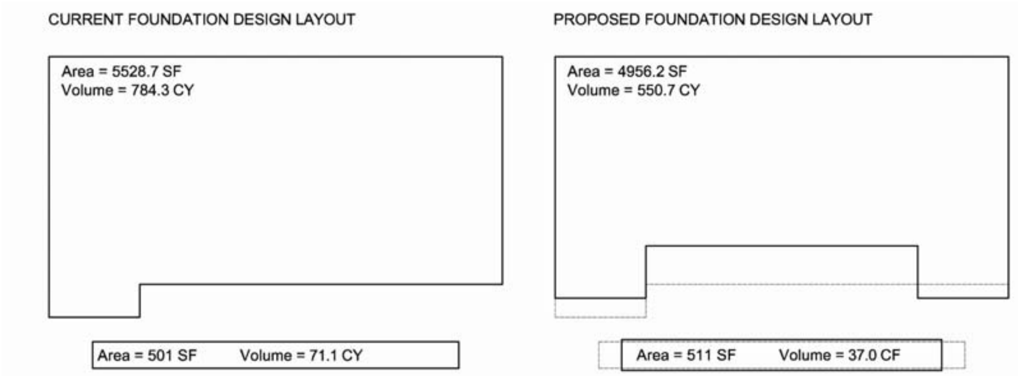

Figure 19: Graphical summary of effects on foundations

Foundation Design Comparison			
Mat Slab	Current Design	Proposed Design	Difference
Surface Area [SF]	5528.7	4956.2	-572.5
Thickness [inches]	46	36.0	-10.0
Concrete Volume [CY]	784.3	**550.7**	-233.6
Continuous Footing	Current Design	Proposed Design	Difference
Surface Area [SF]	501	511	10.0
Thickness [inches]	46	24.0	-22.0
Concrete Volume [CY]	71.1	**37.9**	-33.2
Spread Footing	Current Design	Proposed Design	Difference
Surface Area [SF]	-	32	32.0
Thickness [inches]	-	18.0	18.0
Concrete Volume [CY]	-	**1.8**	1.8
Total Conc. Volume [CY]	855.3	**588.5**	-266.8

Figure 20: Tabulated foundation design comparison