246 West 17th Street

New York, NY

Final Report

April 7, 2009

Prepared by
Alissa Leigh Popovich
Structural Option

Prepared for
Dr. Ali Memari
Faculty Advisor

246 West 17th Street   New York, NY

Project Team

- Owner: Anthony Leichter
- Architects: Rawlings Architects
- Structural Engineers: Robert Silman Associates
- MEPF Engineers: Stanislav Slutsky
- General Contractor: Pav-Lak Construction

Building Information

- Occupation Type: Residential
- Size: 54,000 Square Feet
- Number of Stories: 10 Above Ground and 1 Below Ground
- Project Delivery Method: Design-Bid-Build

Structural Details

- Existing structure is steel moment frame, which has been encased in concrete for structural reinforcement and fireproofing
- Long-span beams and diagonal bracing have been added to support existing long span transfer beams beneath the new stories
- Original slab consists of concrete on composite deck
- New system consists of two-way flat plate concrete slab with both circular and rectangular concrete columns
- Additional lateral support provided by two sets of shear walls
- Foundation consists of spread footings and mat slab systems, both on a rat slab

Architectural Features

- Original structure was a three-story garage building circa 1925
- Project includes a seven-story addition and renovation of the existing building that will house a total of 34 condominium units
- Facade has two main components; the lower third features the original brick mass wall with new large punched windows, while the new stories above blend aluminum and IGU window walls, dark brick veneer, and metal paneling
- Roof consists of stilted pavers on built up polyethylene system

Mechanical & Electrical

- Individual AC systems for each condo unit
- Four boilers and four direct-fire hot water heaters in the cellar
- Electric radiant floor heat in each of the master bathrooms
- Three-phase, 4 wire, 208 Volt electrical feed
- Condo units feature wall-mounted and recessed ceiling lighting fixtures

Alissa Leigh Popovich   Structural

http://www.engr.psu.edu/ae/thesis/portfolios/2009/alp5013

Executive Summary

The current design of 246 West 17th Street consists of seven modern stories of flat-plate, two-way slab construction atop three stories of historic construction featuring steel framing and load-bearing masonry exterior walls. The weight of the concrete addition contributes an incredible amount of weight to the structure, requiring that the foundation be increased dramatically in size, that the historic steel columns be heavily reinforced, and that the long-span transfer beams be supported with very deep reinforcing beams. Furthermore, the lateral force resisting system makes no attempt to utilize the existing steel or the mass masonry exterior walls; instead, shear walls have been implemented and have been designed to take all lateral loads.

Structural Depth Study: System Optimization

This report explores an alternate design to the current concrete system (referred to herein as the "current design") in an attempt to lessen the degree of reinforcement required within the historic portion of the structure. The proposed design consists of steel framing with a lightweight concrete slab-on-deck system. To resist lateral forces, steel chevron braces have been implemented into the design in lieu of concrete shear walls.

Mechanical Breadth Study: HVAC Coordination

The conversion to a steel frame structure results in a significantly different floor system depth. To account for this change and allow for optimal coordination between the new structure and the mechanical HVAC system, the floor-to-floor heights have been increased on the newer stories and the system ducts have been resized to fit within the new interstitial space between ceiling and beam.

Construction Management Breadth: System Cost Study

The difference in cost of steel systems and concrete systems is apparent in areas such as the required materials and the associated labor. For this report, a system-oriented study has been carried out to evaluate the optimal design based on overall economy. As indicated, factors such as labor and material have both been taken into account to prove that the proposed steel system will in fact be most economical.

Acknowledgements

Robert Silman Associates

I would like to thank Eytan Solomon of RSA for recommending the 246 West 17th Street project to me, and also for all of his advice and assistance throughout the duration of my 5th year. I would like to thank RSA as a whole for allowing me to use this building as my senior thesis project, and for providing me with all the plans, tools, and information I could hope for.

The Pennsylvania State University

I would like to thank each of my professors in Architectural Engineering over the past 5 years. In particular, I would like to thank Dr. Ali Memari as my faculty advisor, Professor Kevin Parfitt, Professor Robert Holland, and Dr. Louis Geschwindner for their advice and insight regarding my senior thesis project.

Rawlings Architects

I would like to thank Kai Cheung providing me with all of the additional assistance and construction documents required for my thesis project.

Pandiscio Co

I would like to thank Sean Conway for providing me with some truly beautiful exterior and interior renderings of 246 West 17th Street.

I would also like to thank Tony Leichter, owner of 246 West 17th Street, for allowing me to use his building as the focus of my senior thesis project.

Lastly, I want to thank my family and friends for their unwavering support and camaraderie throughout thesis and beyond. These past few years are ones I am sure I will never forget.

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Introduction

246 West 17th Street is a high-end condominium building in New York, NY that originated as a three-story brick garage structure circa 1925. The current design includes an architectural renovation and structural retrofit of the historical portion along with the addition of seven stories atop the original structure. Originally 24,150 square feet, the building now contains nearly 54,000 square feet and 34 condominium units.

Figure 1: Building location map

Building Overview

Architecture

As with the original building, the cellar of 246 West 17th Street contains garage parking with added mechanical and storage spaces. The 1st floor has been altered to include three condominium units and two recreational spaces. The 2nd and 3rd floors of the original garage building each accommodate five condo units. The 4th floor marks the first story atop the historic structure. Here, the façade steps back from the brick structure below, providing residents in each of the three units on this floor with a personal terrace space. The 5th, 6th, and 7th floors have identical floor plans: each holds four units with balconies. The 8th floor again steps back, providing terrace areas for each of the two condo units. The 9th and 10th floors feature two condo units as well, each with personal balconies and private roof terraces above. The floor-to-floor heights range between 10’-7½” on a majority of the middle floors to 16’-6” on the first floor.

Figure 2: 246 West 17th Street entrance

Figures 1 and 2 show the site location in relation to Madison Square Park and the northern façade of 246 West 17th Street, respectively. More general figures can be found in Appendix A.

Building Envelope

The three-story historic mass masonry walls of the original 246 West 17th Street structure provide a solid base for the newly-added portion above. Much of the original façade and ornamentation remains intact in the current design, although the north and south elevations have been opened up with large bay windows to allow for more light on the interior.

The modern portion features a mix of glass and aluminum curtain walls, metal paneling, and dark brick veneers. These materials add a sense of modernity to the upper two-thirds of the structure above the historic base, which holds fast to the charm and historical context of the surrounding neighborhood. The structural backing of the paneling and brick veneer systems consists of cold-formed metal framing filled with batting insulation. Walls adjacent to the seismic joint are backed by a concrete wall, and the parapets are backed by 6" CMU to account for higher lateral loading on these areas. In addition to providing exterior aesthetics, the new stories succeed in bringing 246 West 17th Street up to the heights of the adjacent buildings, which previously towered over the 3-story garage.

Foundation

The soils under the historic slab of 246 West 17th Street are considered to be stable and have high bearing pressures when classified according to the New York City Building Code (NYCBC). The geotechnical investigation provided by Pillory Associates found there to be a layer of fill soil directly below the existing slab, followed by Glacial Alluvium and then Mica Schist Bedrock. The bearing pressure of the Glacial Alluvium is rather high at 3.5 tons/sf (7ksf), and Pillory states in their report that any new slab may hence be designed as slab-on-grade. The geotechnical engineers specifically recommend the use of either a spread footing foundation or a mat slab to replace the existing slab on grade. Ultimately, after the original slab was removed, both systems were utilized on site in the current design: Spread footings measuring 3'-10" thick were placed on a 2" rat slab on gravel on the southern half of the cellar, while a 3'-10" thick mat slab was placed on the same 2" rat slab on gravel on the northern half of the cellar. Since the cellar walls and perimeter foundations were able to be kept intact, no underpinning was required for the project.

Floor and Framing Systems

246 West 17th Street contains two distinct structural types. The first is represented by the historic portion, which features load-bearing masonry walls and steel framing. The second is represented by the modern portion, consisting of a concrete frame and slab system.

Historic System

The historic floor system of Floors 1, 2, and 3 consists of an 8" draped-mesh system, which is typical of 1920s New York construction (see Figure 3 on the next page). The concrete here is cinder-filled and is of varying quality, with an average strength of only 860psi. The steel framing in this portion has a regular bay size of 20'-8" by 35'-8" and consists of historic 30ksi W-shapes.

The girders span 35'-8" in the north-south direction, extending between the load-bearing masonry walls and the central column line. Slightly smaller historic beams spaced at 5'-6" o/c frame into these girders with a span of 20'-8". The tops of both the beams and girders are embedded in the concrete slab above, but the sizes were able to be determined through the use of the historic construction documents (see Appendix A) and structural probes. The girders were found to range between 24" and 28" in depth, while the beams range between 10" and 14" in depth.

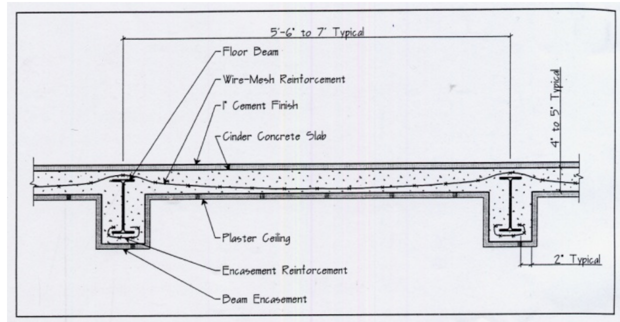

Figure 3: Draped-mesh slab system

The steel columns in the historic portion are also W-shapes (measuring 8" in total depth and flange width) and were originally encased in concrete for fireproofing purposes. These were stripped and re-encased by 26"x26" concrete columns for structural reinforcement due to the addition of the seven modern stories above.

Interface between the Historic and Modern Systems

Due to a setback on the 4th level, the original long-span roof girders now act as transfer beams supporting the seven full stories above. These beams have been structurally reinforced through the addition of steel long-span W-shapes, which act in pairs to support each original girder from either side (pictured in Figure 13). The floor system itself has also been reinforced to transfer lateral loads from the new structure above to the existing structure below: diagonal angle bracing has been added in a truss-like pattern beneath the slab level for this purpose.

Modern Framing System

The new levels feature 8-inch two-way flat-plate systems within concrete moment frames. Circular columns ranging from 14" to 18" in diameter are placed at interior locations at a relatively regular pattern. Rectangular columns flank the perimeter, and range in size between 10"x18" and 12"x18". The design strength of this concrete system is 5950psi.

Lateral System

The lateral force resisting system (LFRS) of 246 West 17th Street consists of four major shear walls. These span the entire height of the building, with two running east-west on either side of the vertical circulation core, and two running north-south along portions of the exterior walls, as illustrated in Figure 4 at right. Each shear wall is composed of 5950psi concrete and is 10" thick along the

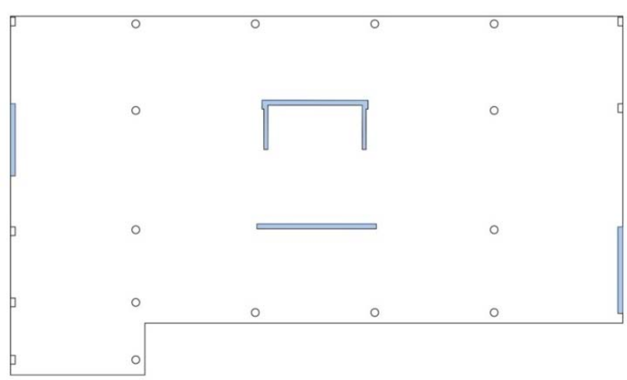

Figure 4: Current shear wall layout on typical floor

entire height. In the current design, neither the historic steel nor the mass masonry wall is depended upon for any lateral resistance.

Roof System

Multiple setbacks in 246 West 17th Street provide a variety of private terraces for the condominium owners. Façade setbacks occur at the 2nd, 4th, and 8th floors, in addition to a large decrease in the floor plan area at the roof level, as the building narrows around the stair and machine room bulkhead area. This decrease in area provides penthouse tenants with a private roof terrace. Each of these terraces is finished with concrete pavers and wrapped by either 3'-8" tall glass railings or a 5' tall parapet.

The typical roof system of 246 West 17th Street – which includes these terrace areas – features a single-ply EPDM roofing membrane topped with 4" of extruded polystyrene insulation, filter fabric, and 2'x2' pavers on adjustable pedestals to ensure that the interior finish level matches that of the outside terrace. This system rests on a low-slope topping slab, which is supported by the structural slab below.

Codes

The current design for 246 West 17th Street follows the guidelines upheld by the NYCBC. The standards here have evolved with New York City as it has grown over the years, so that many are custom to the city and many more are considered to be outdated. In July 2008, in an attempt to standardize and modernize the NYCBC, the code adopted the 2006 version of the *International Building Code* (IBC-06). This code in turn references the 2005 version of the *Minimum Design Loads for Buildings and Other Structures* by the American Society of Civil Engineers (ASCE7-05), a well-accepted national standard. The proposed design herein upholds to the provisions outlined in ASCE7-05 for the purposes of this report.

BAE/MAE Requirement: Computer Modeling

RAM Structural System (RAM SS) was chosen to model 246 West 17th Street due to the program's recognized abilities at handling relatively simple, orthogonal structures. For the proposed design, RAM SS was used to account for the self-weight of the materials for structural modeling accuracy. The program was also used to calculate and apply the forces due to wind load and seismic loading, using techniques and values obtained in the Computer Modeling of Frame Structures class.

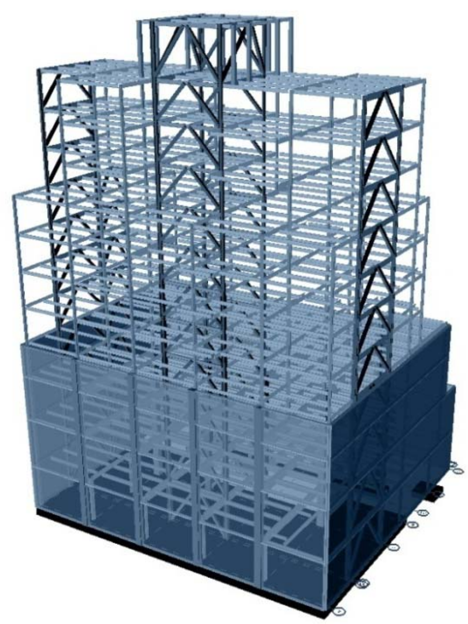

Figure 5: 3D RAM SS representation of the 246 West 17th Street proposed structure

Loading

Gravity Loads

The garage structure is believed to have been designed with a uniform live load of 75psf, as was typical for New York City building construction in the 1920s (as found in *Historical Building Construction* by Donald Friedman). A design of this magnitude is more than adequate to support the 40psf live load required today for residential construction, but additional tests were undertaken to ensure that the structure could support the current design live loads of the new lobby and terraces.

The table in Figure 6 below shows the current live loads and required live loads (per ASCE7-05) along with the loads to be used in the proposed thesis project design. Current, required, and proposed dead loads as calculated and per ASCE7-05 are also shown in the tables below, as seen in Figures 7 and 8. Because RAM SS was used to calculate the material self-weights, these values are exempt from the calculation of the superimposed dead loads.

Live Load Schedule	As Designed [psf]	As Required by ASCE7-05 [psf]	To be used in Proposed Design [psf]
Bulkheads	30	20	30
Main Roof	30	60	100
New Floor (Interior)	40	40	40
New Floor (Exterior)	60	100	100
Existing Floor (Interior)	40	40	40
Existing Floor (Exterior)	40	100	100
First Floor	100	100	100
Basement as Garage	100	40	40
Basement as Machine Room	100	40	40

Figure 6: Live load schedule

Dead Load Schedule	As Designed [psf]	As Calculated [psf]	To be used in Proposed Design [psf]
Bulkheads	130	140	140
Main Roof	70	70	70
Typ New Floor (Int)	20	20	20
Typ New Floor (Ext)	70	80	80
Typ Existing Floor	20	20	20
First Floor	35	35	35
Basement	105	160	160

Figure 7: Dead load schedule

Dead Load Schedule by Floor/Area Type	Pavers	Roofing	Hung Clg	Mech	Partitions	Finished Floor	1st Floor Fin. Floor	Total Dead Load
Bulkhead Roof	35	30	-	75	-	-	-	**140**
Roof Main	35	30	5	-	-	-	-	**70**
Roof Mech	-	-	5	75	10	-	-	**90**
Typ New Floor (Int)	-	-	5	-	10	5	-	**20**
Typ New Floor (Ext)	35	30	5	-	10	-	-	**80**
Typ Existing Floor	-	-	5	-	10	5	-	**20**
First Floor (Int)	-	-	5	-	10	-	20	**35**
First Floor (Ext)	35	-	5	-	10	-	-	**50**
Basement as Garage	-	-	-	150	10	-	-	**160**

Figure 8: Superimposed dead load tabulation

Lateral Loads

For accuracy in modeling, RAM SS was used to determine and apply the lateral loads for 246 West 17th Street, which were based on IBC-06 and ASCE7-05 design provisions. The resulting forces were checked against previously-calculated values, and were found to be slightly lower than those obtained by hand. As mentioned, the RAM SS model was decidedly more accurate because the hand calculations were performed under the assumption of a constant building width from base to bulkhead. In actuality, the building has multiple setbacks, resulting in a significant taper in the width in the north-south building direction. The difference between hand-calculated and model-calculated loads is much greater in this direction, which supports the belief that the RAM SS model-calculated loads are in fact correct. While the hand-calculated loads could have been used, they were found to be over conservative.

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.

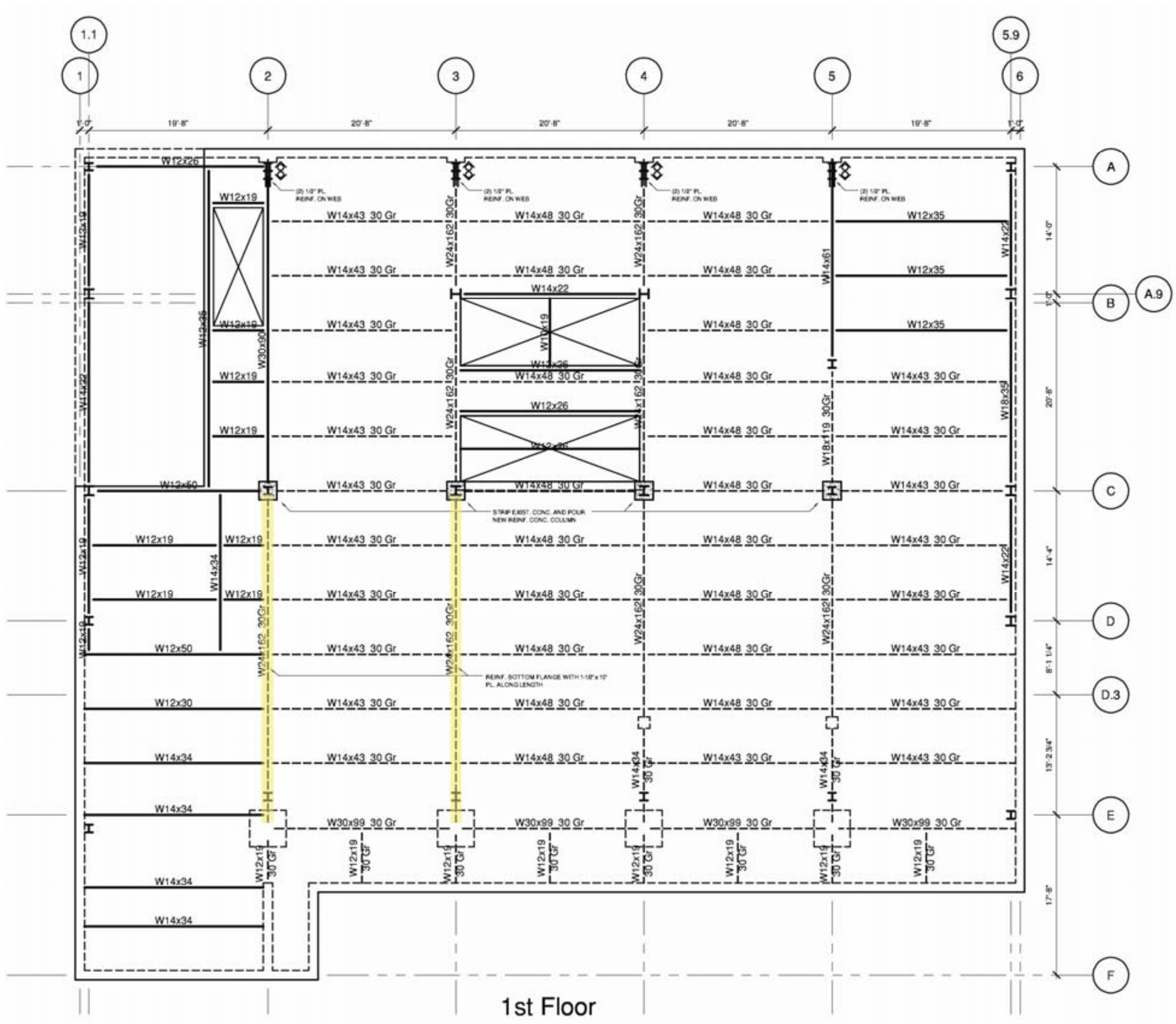

Figure 11: Noted girder failures on the 1st Floor

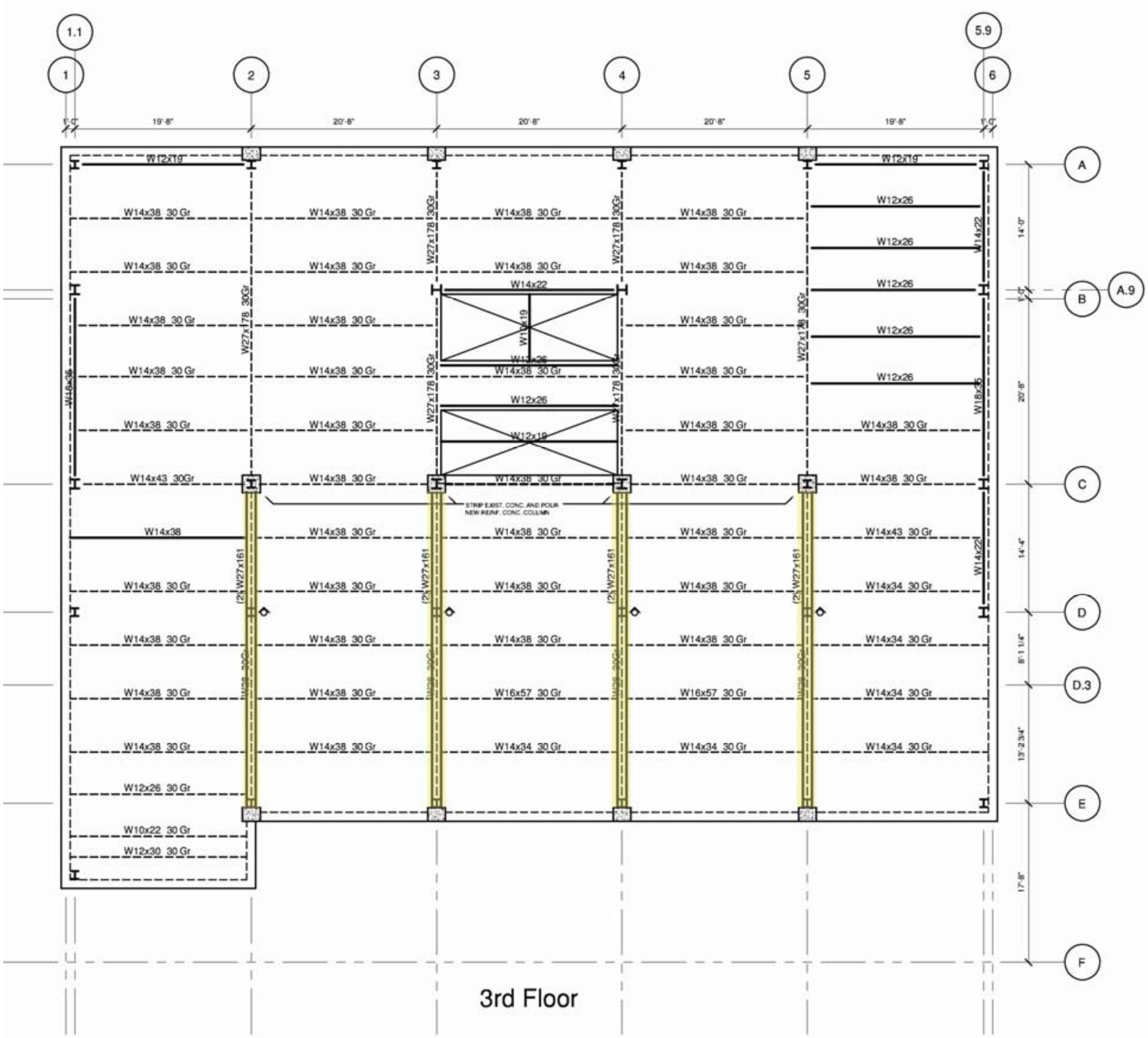

Figure 12: Noted girder failures on the 3rd Floor

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.)

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.

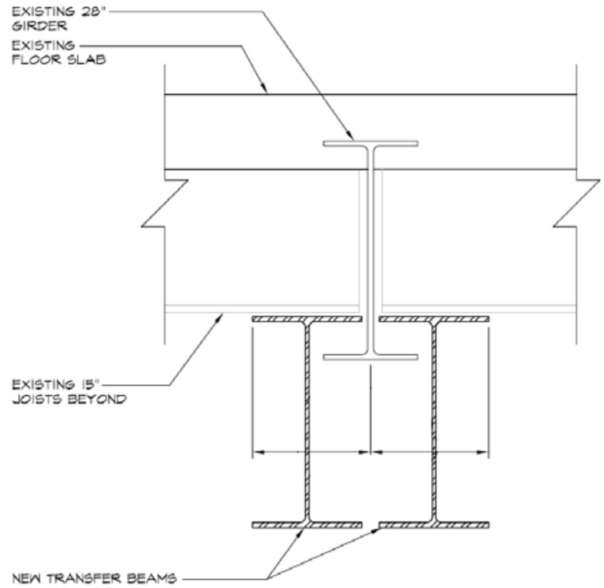

Figure 13: Section showing stacked girders as reinforcing members

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.

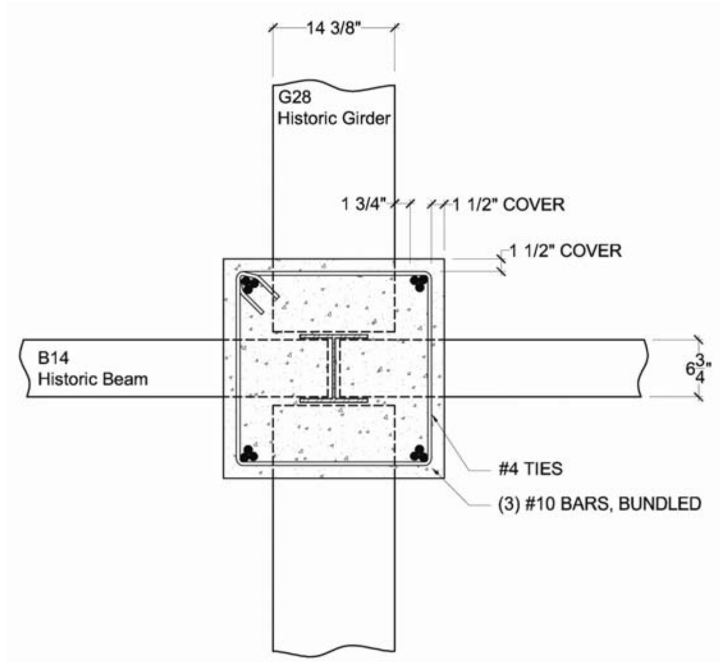

Figure 15: Plan view of historic beams framing into historic column; Subsequent concrete column design

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.

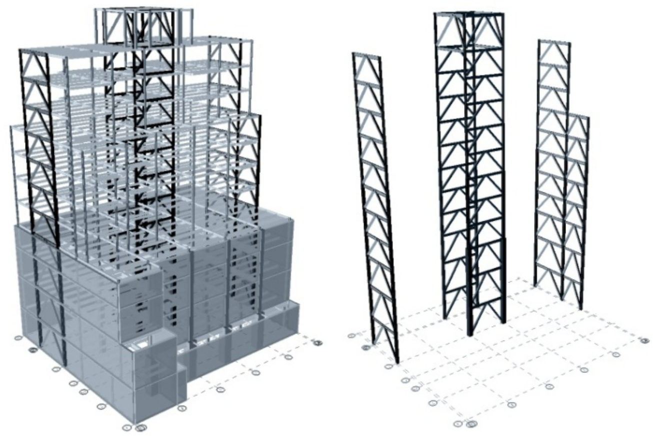

Figure 16: 246 West 17th Street lateral brace system

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.

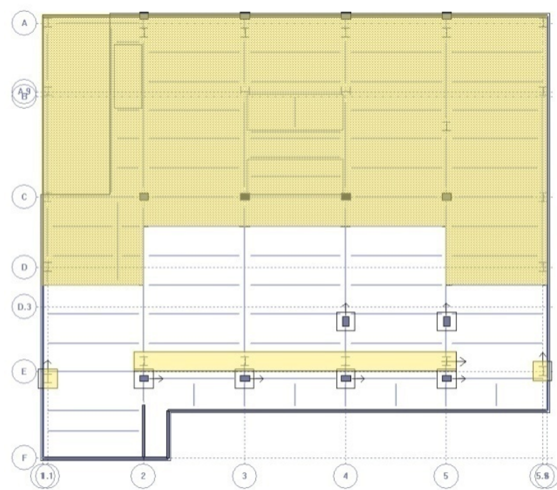

Figure 18: Final foundation layout

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.

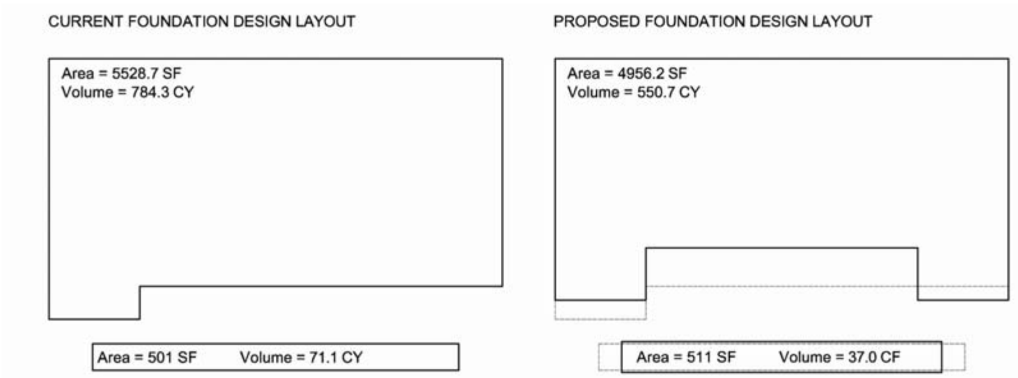

Figure 19: Graphical summary of effects on foundations

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

Mechanical Breadth – HVAC Coordination

The conversion of a concrete flat plate slab to a steel system has some serious implications on the overall floor system thickness. On the new stories where this change takes place, the floor system went from being a uniform thickness of 8" due to the flat plate slab to a maximum thickness of 20" due to the slab-on-deck, steel beams, and fireproofing (see Figure 21 below). In the current design, the HVAC system is able to maneuver freely in the interstitial space between the top of the ceiling and the underside of the slab; in the proposed design, the HVAC system is confined to 10" in depth for ductwork that must pass underneath the steel beams before reaching the plenum area.

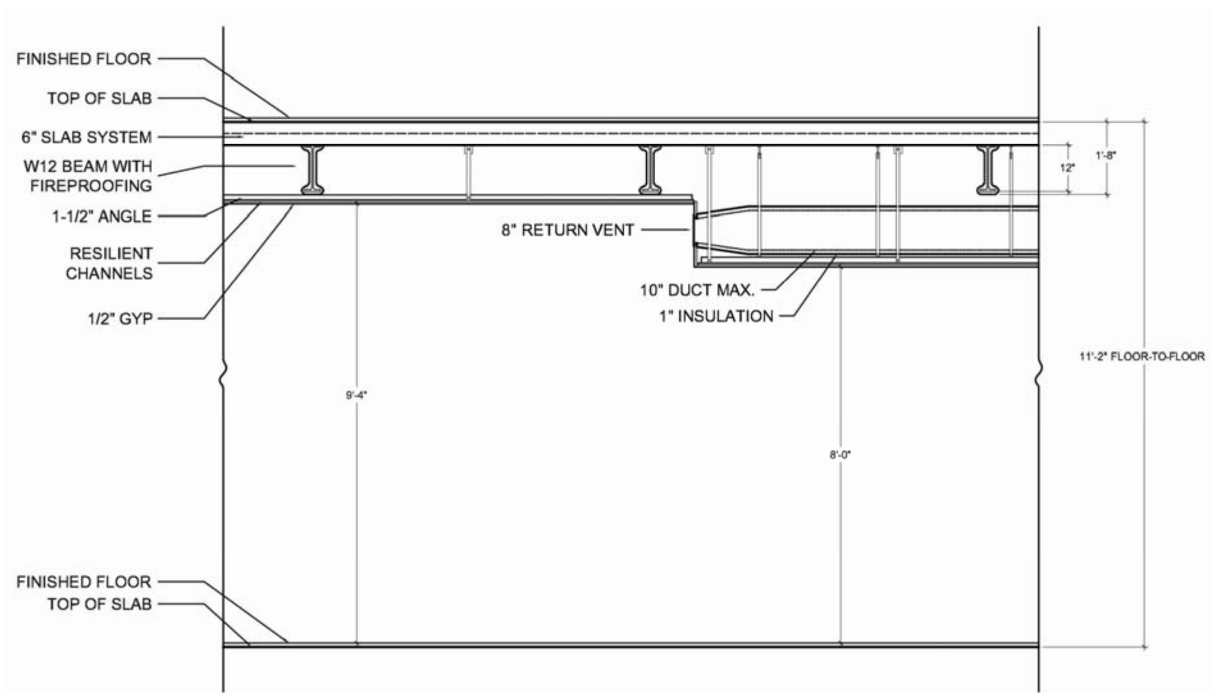

Figure 21: Typical floor section showing HVAC duct placement and ceiling heights

To compensate for this limited clearance, many of the existing ducts needed to be resized and redirected. New ducts were kept to a 4-to-1 width-to-depth ratio to limit the effects of frictional drag, while any new layouts took into consideration the location of the return and supply vents. Figure 22 at right shows the duct sizes that were determined to work with the typical new floor system and subsequent duct depth restriction.

Comparable Mechanical System Duct Sizes [inches]	
Current Size	Proposed Size
36x12	42x10
24x16	36x10
21x12	24x10
18x16	32x10
16x20	36x10
16x12	20x10
8x12	10x10 or 12x8

Figure 22: Proposed duct sizes

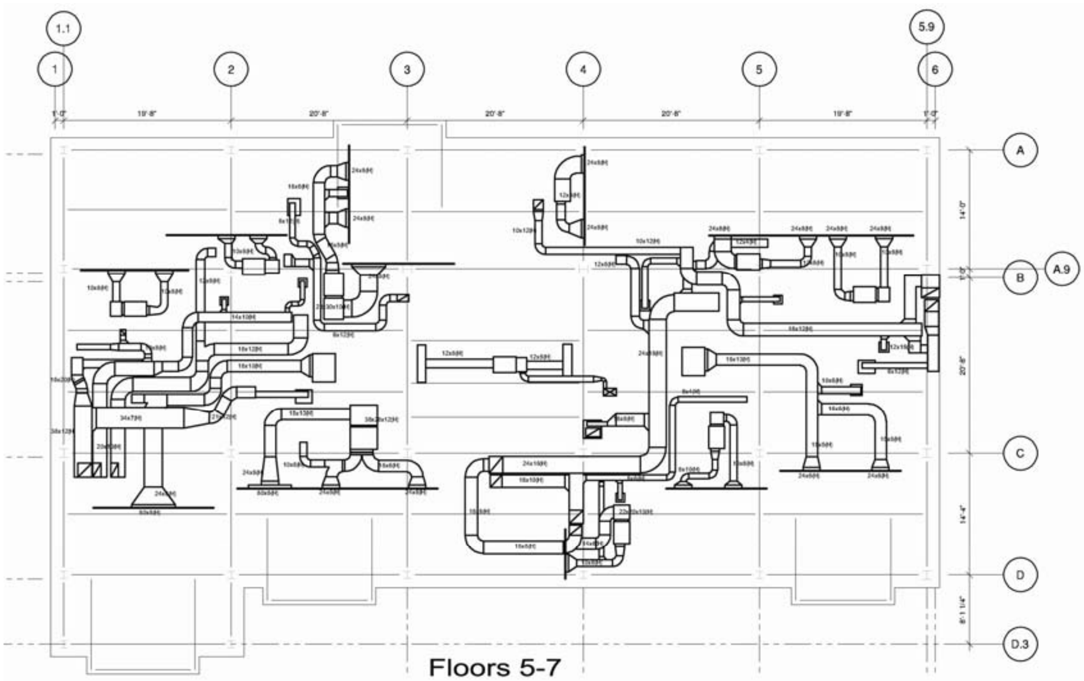

Figure 23: Current HVAC layout and sizes on typical floor

Figure 23 (above) shows the current HVAC layout for a typical floor. Figure 24 (below) shows the proposed HVAC layout. Ducts highlighted in yellow are those that were resized and/or redirected. For detailed views of these plans, see Appendix C.

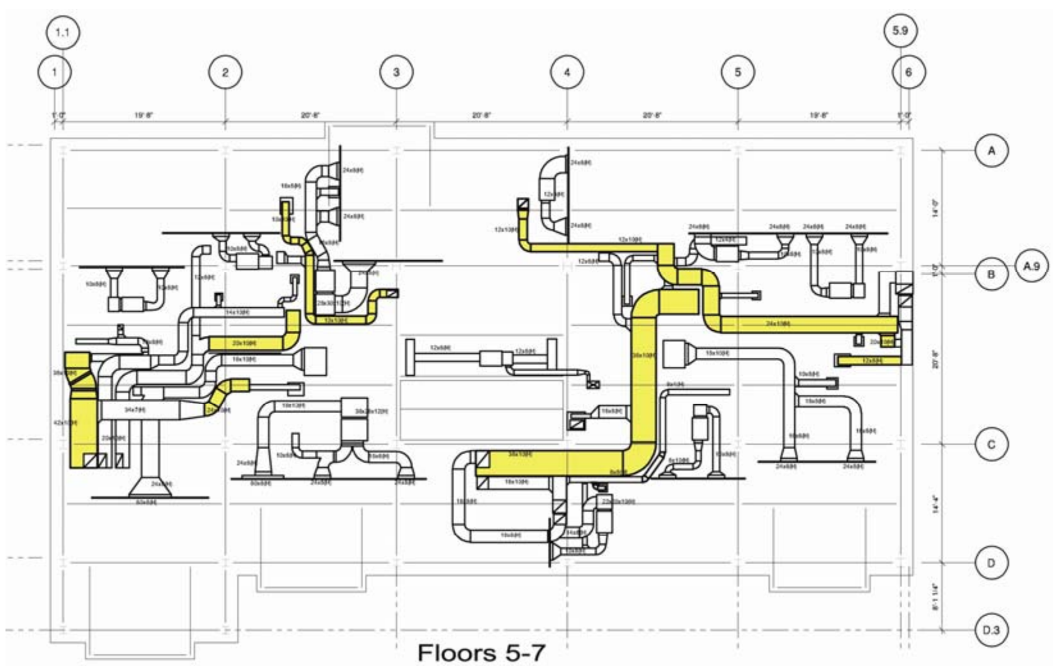

Figure 24: Proposed HVAC layout and sizes on typical floor

Construction Management Breadth – System Cost Study

Changing the structural system from the current design of concrete to the proposed design of steel resulted in a significant change in the overall construction cost: the total estimated cost of the proposed system was found to be 7.5% less than the original cost. Total costs are compared in Figure 25 below.

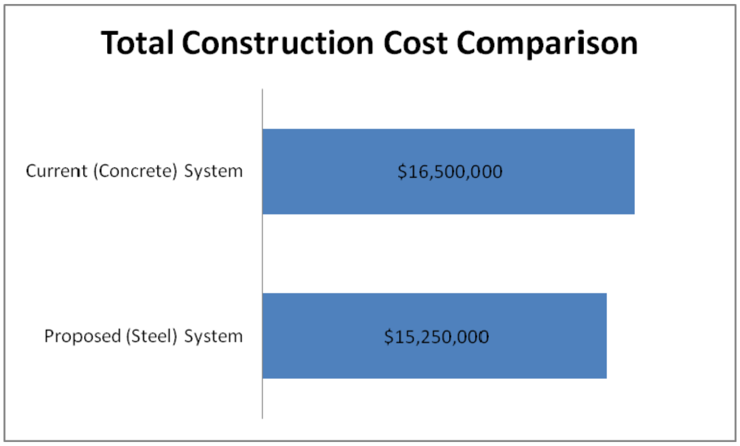

Figure 25: Comparison of total cost between current and proposed system

Current (Concrete) System	Proposed (Steel) System	Savings
Foundation Cost	Foundation Cost	In Foundations
$ 202,133	$ 140,687	$ 61,446
Superstructure Cost	Superstructure Cost	In Superstructure
$ 2,423,497	$ 1,235,295	$ 1,188,202
Total Cost	Total Cost	In Total Cost
$ 16,500,000	$ 15,250,352	$ 1,249,648

Figure 26: Estimated savings in cost due to structural system optimization

As seen above in Figure 26, savings were found in both the foundation and the superstructure systems. Foundations accounted for approximately 5% of the total savings, while the superstructure accounted for the remaining 95%. These are due largely in part to reductions in the required quantity of concrete and in the equipment rental period. Even though the required labor may be seen to increase due to the welding required for the proposed steel system, the anticipated, accelerated pace of the steel construction offsets these expenses. Detailed estimate sheets may be found in Appendix D. Notice that for these calculations, only the construction costs that varied between the two structural types were taken into account; all other costs were assumed to remain the same and therefore not contribute to an overall change in cost.

Conclusion

Simply put, the current concrete-based design does not achieve structural optimization due to the extensive weight of the system. The following design possibilities are attainable with the use of a lighter steel system, and have thus been focused on in this report: the utilization of historic members in the lateral system, the reduction in transfer beam reinforcement size, and the reduction in foundation size.

As the lateral system analysis proved, the historic columns and beams can be utilized in the lateral force resisting system while meeting and exceeding design expectations. Not only were the code requirements met, but the stringent limitations on deflection and drift set by the neighboring building were attained as well. As this was the controlling factor, strength requirements were also easily attained.

The structural analysis also showed that the foundation system can be significantly reduced in size due to the decreased building weight. This has material, cost, and labor savings overall.

The mechanical breadth study focused on system coordination. The new floor system depth was something that could not be ignored, and so this potential problem was maneuvered around. The mechanical system coordination was found to be more than feasible.

The construction management study focused on relative system cost. The change in structural systems resulted in multiple changes in material and labor types as well, but the end result was one of savings. 7.5% of the total construction cost was reduced due to the structural system change.

The proposed design, which focuses on material consistency and the integration of the new steel framing system with the historic framing system, proves to be efficient from both a structural standpoint and a cost standpoint. After a few minor modifications, the system has proven to be adequate for mechanical space requirements as well. In summary, all of the checkpoints were met, thereby achieving the ultimate goal of structural system optimization.