The Residences of Sherman Plaza
Evanston, IL

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Structural Option
Spring 2005 Senior Thesis

The Pennsylvania State University
Department of Architectural Engineering

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Evanston, IL

PROJECT OVERVIEW
25 story Condominium
610,000 Total Square Feet
Includes 253 condominiums, lofts and penthouses.
Construction: December 2004-April 2006

PROJECT TEAM
Owner: Sherman Plaza Partners, LLC
Architects: Otis Koglin Wilson Architects/ Daniel P. Coffey & Associates, Ltd.
General Contractor: Focus Construction
Structural Engineer: Halvorson & Kaye Structural Engineers
Civil Engineer: V3 Consultants
MEP Engineer: Environmental Systems Design
Electrical Consultant: Huen Electric, Inc.
Fire Protection: Nova Fire Protection, Inc.

ARCHITECTURE
Second tallest building in Evanston, IL.
Includes 152,000 square feet of retail space, a 54,000 square foot health club, and an adjoining 1,585 car parking garage.
Includes a half acre intensive roof garden on the third, sixth, and seventh floors.

STRUCTURAL
Foundation: Belled Caissons
Superstructure: Reinforced cast-in-place concrete columns and beams
Floor: Two-way reinforced concrete slabs
Envelope: Face brick with limestone and cast stone detailing
Lateral System: Reinforced concrete shear walls and perimeter moment frames.

ELECTRICAL/LIGHTING
208/120V, 3 phase-4 wire primary voltage system
480/277V, 3 phase-4 wire secondary voltage
208/120V, 3-phase-4 wire emergency generator

MECHANICAL
Two 350 ton chillers
In-unit fan coil electrical heating units
Three air handling units @ 40,000 CFM in the 3rd floor mechanical room.
One air handling unit @ 2,500 CFM in the lobby ceiling.

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Executive Summary

The study presented in this report is the culmination of a year-long research and analysis of Sherman Plaza, a 25 story condominium building, located in Evanston, IL. The building is made up of a complex reinforced concrete structural system that has been designed with careful consideration. While the existing system is an adequate and efficient design, the building will be reanalyzed in order to gain a greater understanding of the complexities involved in designing a high-rise building's gravity and lateral systems. This study investigates a different structural system in an attempt to produce a new system that will improve constructability, shorten construction time, and lower costs without decreasing the building's quality.

To accomplish this goal, the existing reinforced concrete system was replaced by a new steel gravity and lateral system. The existing structural system has some drawbacks that can be improved upon with the new system. The reinforced concrete structure is somewhat difficult and time consuming to construct due to the need to place the formwork and shoring. The system also has a high weight, which results in the need for large foundations. The shear walls also need large foundations and grade beams for support.

RAM Structural System was used to design the gravity and lateral system of the new structure. Composite steel beams were chosen for the floor framing to limit the overall building height as much as possible. The beam sizes were also restricted to W16s on floors 2-7 and to W14s on levels 8-25. Despite these limitations, the building height was still increased from 260.5 feet to 283.25 feet, which is an overall increase of 22.75 feet. The lateral system was made up of a combination of moment and braced frames. This design resulted in a total building drift of 5.6254 inches, which is less than the allowable drift of H/600, or 5.665 inches.

With this new design, the building weight was greatly reduced. The foundations sizes were able to be decreased. The original foundation sizes varied between a 15'-6" bell diameter and 6'-0" shaft diameter caisson to a 6'-0" bell and 2'-6" shaft. The new design results in caisson sizes that range between a 3'-0" bell and a 7'-0" bell.

By changing the primary structural system of Sherman Plaza from reinforced concrete to structural steel, other building systems were also impacted. Two breadth studies were performed to determine the effect the structural material change had on the construction management and on building acoustics.

An estimate was performed of the costs of the exterior cladding and structural materials for each system from R.S. Means. The steel system resulted in a total cost of $17.45 million, and the reinforced concrete system had a total cost of $25.63 million. The steel system, therefore, was $8.18 million less expensive than the concrete system. R.S. Means was also used to perform a schedule estimate. The steel system took a total of 1146 days to complete, while the concrete system took 2660

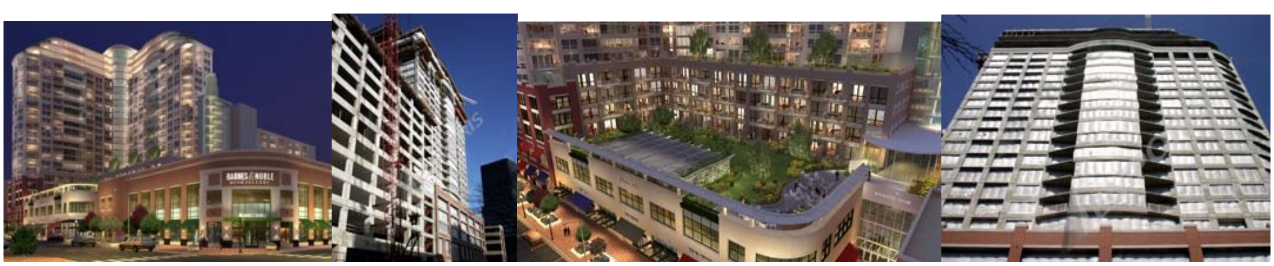
days. Therefore, the steel system could be erected 1514 days faster than the concrete system.

The first acoustical analysis was a study of the transmission loss of the floor system. The original floor system, an 8 inch thick reinforced concrete slab, was found to have an adequate transmission loss. The new system, however, was made up of 3 inches of concrete on top of composite metal deck and was not acceptable according to the NC-25 noise criteria curve. Even with additional sound absorbing floor and ceiling materials, the new system did not meet NC-25 criteria, but did fall below NC-30 standards. The second analysis investigated the transmission loss of one of the concrete shear walls from the original structural system. The shear wall was found to have an acceptable transmission loss to reduce the mechanical room noise. Based on the acceptable sound pressure levels, a new wall system was chosen to replace this wall. Three alternatives were analyzed. The chosen wall system was made up of 3 5/8” steel channel studs with two layers 5/8” gypsum board on both sides and 3” mineral-fiber insulation in the cavity and met the NC-25 criteria.

In all, the steel structural system was an effective design for this building. The composite steel produced an efficient gravity system that worked well with the given column layout. The drawback to this system, however, was that the structural system’s ceiling to floor section depth was greater than that of the existing concrete system. The newly designed lateral system also produced acceptable results. The design, however, uses a large number of braced frames and moment connections that will increase construction time and costs. The architectural constraints also made the placement of the frames within the building difficult. Since the building was designed to have shear walls provide the lateral resistance, the architecture of the building did not provide many options for braced frame locations. The braced frame system could possibly have been improved if other locations for the frames could have been tested.

Despite any drawbacks of the new steel structural system, the cost estimate and comparison of the two systems showed that the steel system is less expensive by $8.18 million. This savings in cost could compensate for the increase in building height and the large number of lateral braced frames and moment connections. In addition, according to the schedule estimate, the steel system could be erected 1514 days faster than the concrete system. Therefore, the new steel redesigned system is a viable alternative to the existing reinforced concrete structural system.

BUILDING DESCRIPTION



Building Description

Location:

The Residences of Sherman Plaza are located in the diverse community of downtown Evanston, IL, just north of the city of Chicago on Lake Michigan. The 25 story condominium is the second tallest building in Evanston and offers residents luxuries that were once only available in downtown Chicago, such as a 54,000 square foot health club, garage parking, ½ acre rooftop garden, and easy access to the El and Metra trains. The building is zoned in the Downtown Retail/Development Core in Evanston, IL. The condominium is located on a site that was originally a parking garage with 566 spaces. The new construction will house 253 condominiums, lofts and penthouse residences, 152,000 square feet of retail space, and a new adjoining 1,585 car parking garage.

Architecture:

The Sherman Plaza condominium building has a rectangular base containing the retail spaces and health club and is topped by a twenty-three story L-shaped condominium tower. The retail spaces are located on the first two floors and are occupied by retail tenants, such as Barnes & Noble Booksellers, Pier 1 Imports, and Washington Mutual. Residents can choose from a one bedroom condominium, a two bedroom loft, or a 2-4 bedroom penthouse suite with private terrace. The walls of the bottom two retail floors are covered in hard fired natural clay face brick with an encircling three foot base made of precast panels and granite. The face brick continues up to the seventh residential floor and the rest of the walls are primarily smooth formed concrete panels.

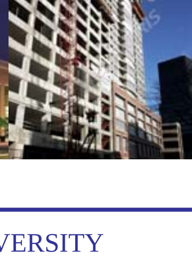
The building steps back on the third, sixth and seventh floors and the roofs of these floors are covered by an intensive garden. The top three penthouse levels are also stepped back and have large cast-in-place concrete “eyebrows” covering the balconies. The intensive green roof is comprised of layers of intensive soil, filter fabric, drainage/water retention channel elements, and prefabricated drainage courses. The concrete eyebrow roofs on the top three floors are covered with a single-ply elastomeric EPDM fully adhered roofing system above 2 inch roof insulation.

A parking garage is connected to the condominium tower on its west side, and it extends up fourteen stories and holds 1,585 cars. The garage, however, is structurally separate from the tower, and therefore, will not be included in this study.

Primary Project Team:

- **Owner**: Sherman Plaza Partners, LLC.
- **General Contractor**: Focus Construction
- **Architects**:
    - Design Architect: Daniel P. Coffey & Associates, Ltd.
        - www.dpcaltd.com
    - Associate Architect: Otis Koglin Wilson Architects
        - www.okwarchitects.com
- **Structural Engineer**: Halvorson & Kaye Structural Engineers
    - www.halvorsonkaye.com
- **Civil Engineer**: V3 Consultants
    - www.v3consultants.com
- **MEP Engineering**: Environmental Systems Design
    - www.esdesign.com
- **Electrical Consultant**: Huen Electric, Inc.
    - www.huenelectric.com
- **Plumbing Engineering**: Great Lakes Plumbing and Heating
    - www.glph.com
- **Fire Protection**: Nova Fire Protection, Inc.
    - www.novafire.com

BUILDING SYSTEMS



Building Systems

Electrical:

Sherman Plaza's electrical system is powered by a 480/277 Volt and a 208/120 Volt system. The 480V transformer provides power primarily to the cooling tower and mechanical system. The 208/120V, 3 phase-4 wire voltage system is used to power the rest of the building.

Lighting:

The lighting in Sherman Plaza is primarily standard fluorescent lighting. Accent wall sconces are used in the units' entryways and bathrooms. The roof gardens have extra lighting features with plant up-lighting, landscape, patio and bollard lighting. The bottom retail floor has surface mounted exterior light fixtures in between storefront windows.

Mechanical:

The primary mechanical equipment for Sherman Plaza is located in the 3rd floor and 26th floor mechanical equipment rooms. A cooling tower plant and two chillers are located on the second story roof of the retail building and are sized to service all 25 stories, including the retail area. Air handling units at 40,000 CFM are located in the 3rd floor mechanical room with another smaller air handling unit servicing the lobby at 2,500 CFM. Each of the lofts, condominiums, and penthouses has an individually controlled in-unit electric heat and air conditioning unit. Each of the residences has a fan coil unit with electrical heating with standard ducts. Exhaust fans in the 26th floor mechanical room provide exhausts for the kitchens, toilets, and in-unit dryers.

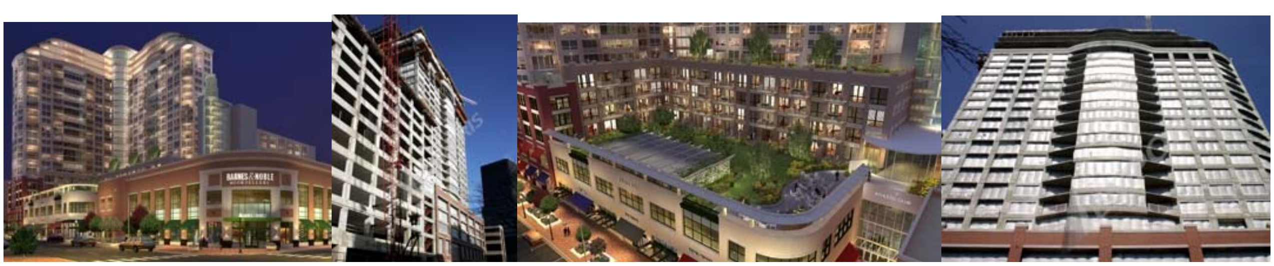
Fire Protection:

Sherman Plaza follows the 1996 BOCA National Fire Protection Code. Exterior bearing walls and interior columns have a 3 hour fire resistive rating. Stair and shaft enclosures and floor construction has a 2 hour rating, and dwelling unit separations and exit corridors have a 1 hour fire rating. The building contains a state-of-the-art fire alarm and sprinkler system, as well as basic fire suppression materials.

Transportation:

The building is served by three elevators: two passenger and one freight elevator. The passenger elevators are hydraulic type with a 2500 lb. capacity and a speed of 125 feet per minute. The freight elevator is an electric traction type with a 5000 lb. capacity and a speed of 500 feet per minute. A set of escalators serve the first two floors in the retail area. The escalator is a clat-step reversible type for ascending and descending passenger service with a speed of 100 feet per minute.

EXISTING STRUCTURAL SYSTEM



Existing Structural System

Gravity System:

The primary structural system of Sherman Plaza is reinforced cast-in-place concrete two-way slabs, beams, and columns. The primary floor system is two-way slabs, but there are some one-way slabs in irregular areas. The slab thickness of every floor is 8" with the exception of the first retail floor, which has a slab thickness of 9". The building is surrounded by perimeter edge beams, and there are interior edge beams surrounding slab openings for stairs and elevator shafts. The third, sixth and seventh floor framing has additional beams to account for the large loads due to the green roofs on those levels as the building steps back. The twenty-third floor framing also has large transfer girders to account for the change in the column grid for the penthouse levels.

The slab reinforcement remains fairly constant from floor to floor on the stories above the two retail floors. The bay sizes, however, differ throughout the plan, which causes the reinforcement size to change throughout a floor. The slab is required to have a minimum of #6@12" top reinforcement at column strip intersections, #5@12" bottom reinforcement at middle strip intersections, and #5@12" top and bottom reinforcement at intersections of the column strip and middle strip. The typical floor of the building begins on level 8, and this floor plan is continued up to floor 22. The last three floors differ, because they are penthouse levels.

In general, the columns are lined up along a grid, but the spacing of the columns varies. Most bays are either 14'x14' or 21'x21' square bays. Column sizes on the ground floor vary from 18"x54" on the building perimeter to 36"x36" as a typical interior column size. Column sizes differ on the upper floors and vary between a 20" diameter circular column, a 24"x24" square interior column and a 13"x36" on the perimeter. Figure 1 shows the typical floor plan, which extends from level 8 to 22.

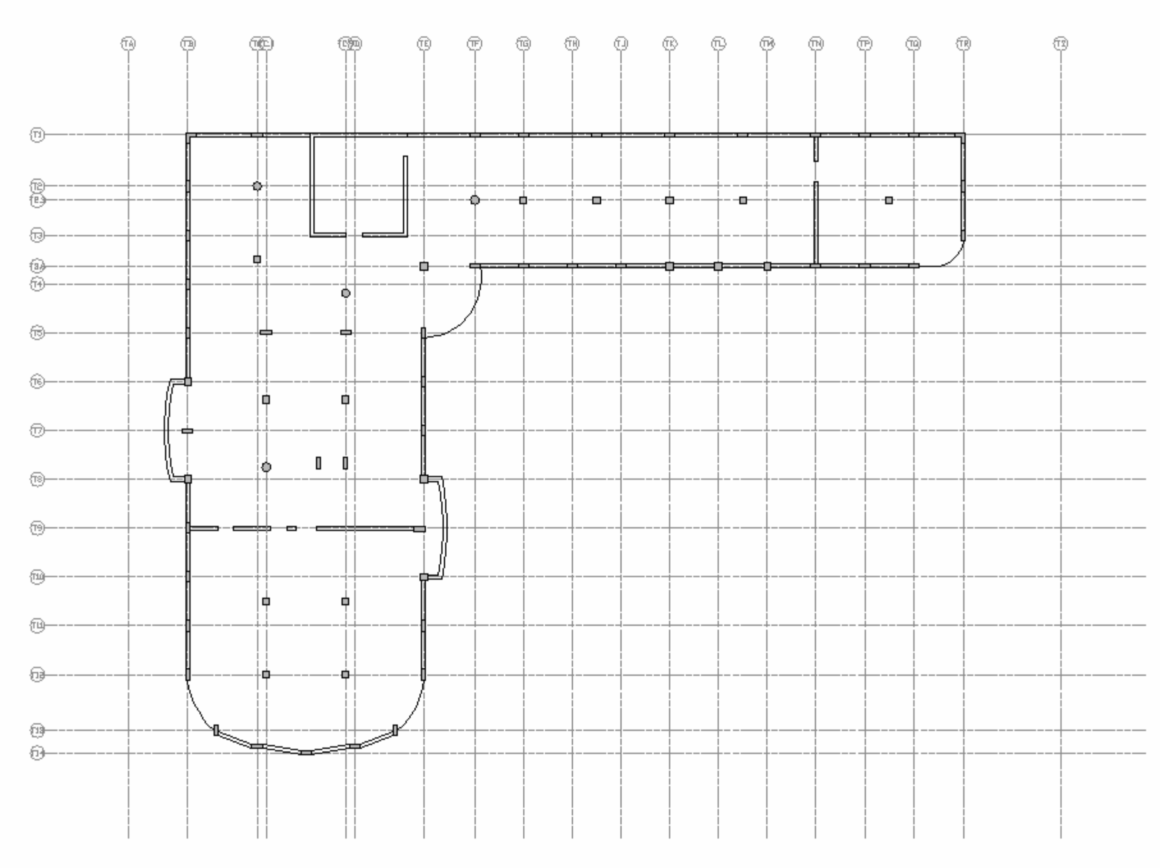


Figure 1: Typical Floor Plan: Levels 8-22

Lateral Resisting System:

The lateral support for the building is made up of a combination of reinforced concrete shear walls and perimeter moment frames for the first twenty-two stories. There are shear walls located around the elevator core, near the intersection of the L-shape of the building. There is also a shear wall in each arm of the L-shape. The elevator core shear walls are 18” thick for the first six floors, 16” thick for floors 7 to 22, and 12” thick for the last three floors. The shear walls located in the L-shape’s arms are 18” thick for the first six floors, 15” for floors 7 to 12, and 12” thick for the remaining floors. The reinforcement for the shear walls is #5@12”, in general. The moment frames are made up of deep edge beams around the building’s perimeter. A typical perimeter beam is a 13”x34” beam with 4 #7 reinforcement bars on top and bottom. The typical perimeter columns are 13”x36” with 8#7 bars.

The top three floors of Sherman Plaza are penthouse levels and have a different column grid than the rest of the building. Therefore, the moment frames do not continue up to these floors. Instead, it is assumed that the shear walls on this level will take all the lateral load. From the 6^{th} floor down, the shape of the shear walls changes, and there is an additional moment frame due to the area where the building steps back. Figures 2-4 show the shear wall and moment frame layouts of each floor.

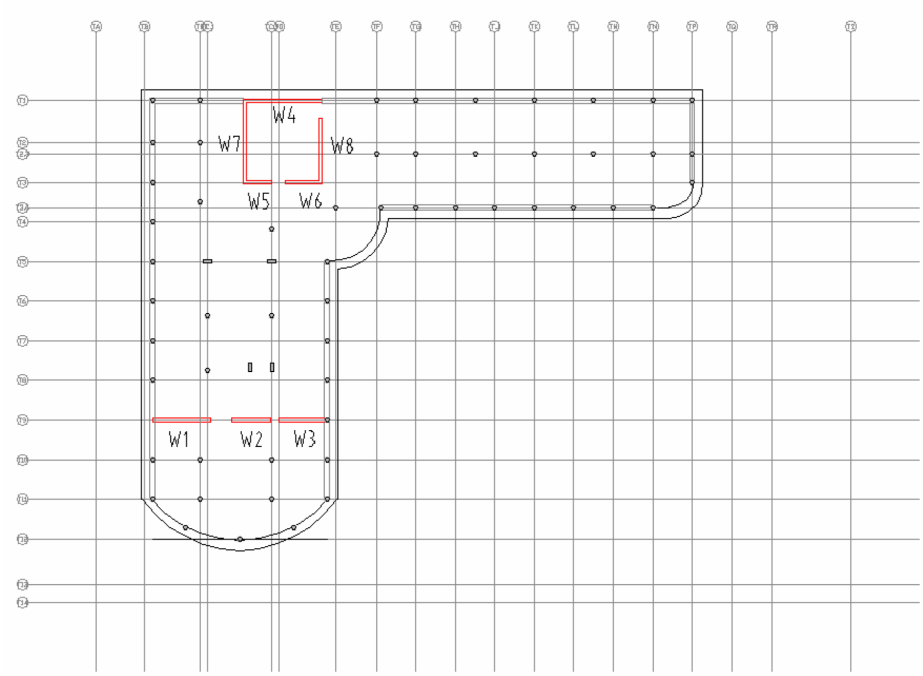


Figure 2: Level 25 (Shear walls are in red.)

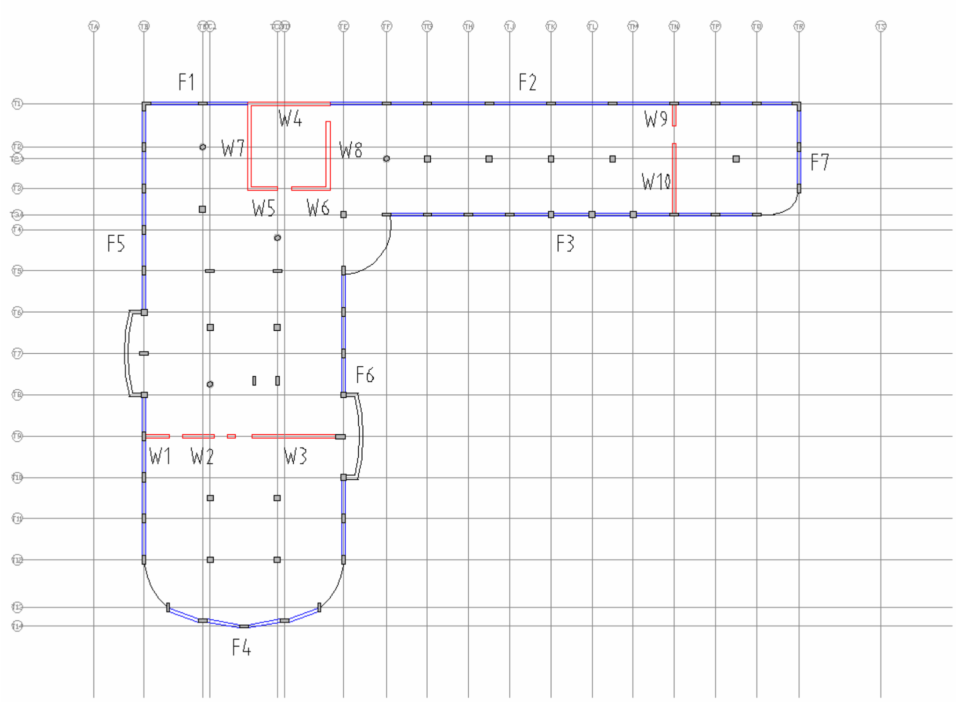


Figure 3: Typical Floor Plan: Levels 8-22
(Shear walls are in red and moment frames are in blue.)

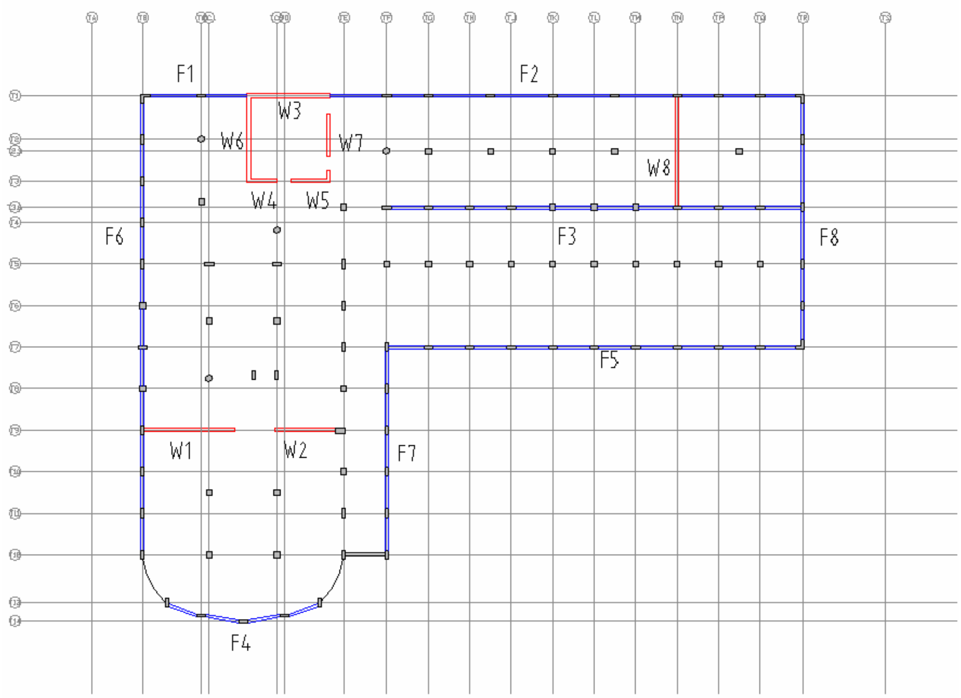
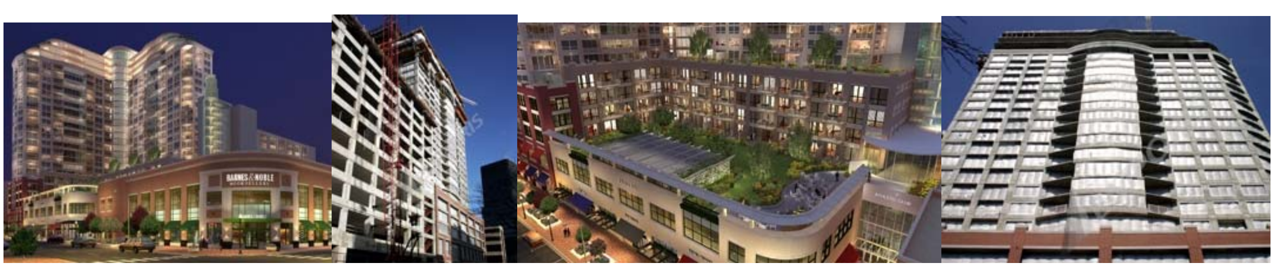


Figure 4: Level 6
(Shear walls in red and moment frames in blue.)

Foundation:

The foundation of Sherman Plaza consists of reinforced concrete belled-caissons, extending to hardpan at approximately 70 feet below grade. All the caissons will bear on hardpan soil strata with a minimum allowable bearing capacity of 30 ksf, except where the drawings indicate a minimum of 50 ksf. The largest caissons have a 15'-6" bell diameter and a 6'-0" shaft diameter in size and are spaced at 28'-0", in general. The sizes vary down to a 6'-0" bell diameter and 2'-6" shaft diameter, spaced at either 14'-0" or 21'-0", in general. Above the caissons is a 5" slab-on-grade with one layer of 6x6-W2.1xW2.1 W.W.F. Grade beams are located underneath the building's shear walls.

PROBLEM STATEMENT AND SOLUTION OVERVIEW



Problem Statement

Sherman Plaza is a complex building, and its structural system has been designed with careful consideration. While the existing system is an adequate and efficient design, the building will be reanalyzed in order to gain a greater understanding of the complexities involved in designing a high-rise building's gravity and lateral systems. This study investigates a different structural system in an attempt to produce a new system that will improve constructability, shorten construction time, and lower costs without decreasing the building's quality.

Several floor framing systems were analyzed in Structural Technical Report 2 to determine which could provide a suitable alternative to the existing system. It was found that the existing system had some drawbacks:

- The current building design has a reinforced cast-in-place concrete structure. This system is somewhat difficult and time-consuming to construct due to the need to place the formwork and shoring.
- The existing structural system is somewhat inefficient in terms of material usage. Due to the limited strength of the structural material, the bay sizes are restricted, resulting in a dense column grid. The lateral resisting system also uses a large amount of concrete for the shear walls and the large columns and edge beams that make up the moment frames.
- The reinforced concrete system has a high weight, which results in the need for large foundations. The shear walls also need a large grade beams for support.

To achieve the goals of shortening construction time and reducing weight and building costs, the structural depth study of this report examines the effects of changing the structural material from concrete to steel. In addition to the structural analysis, two other building disciplines were investigated in order to determine the effect that the structural material change would have on them. A study was performed in the construction management breadth area in order to compare the time and costs of the two structural systems. The acoustics of the new wall and floor systems were also analyzed to determine if they provide adequate sound transmission loss.

Problem Solution Overview

The building analysis in this report attempts to produce a structural system that is efficient in material usage, constructability, time and has a lower weight than the existing system. To accomplish this goal, the existing reinforced concrete system was replaced by a new structural steel system. A study was performed to investigate the effectiveness of this system and the impact it will have on other aspects of the building.

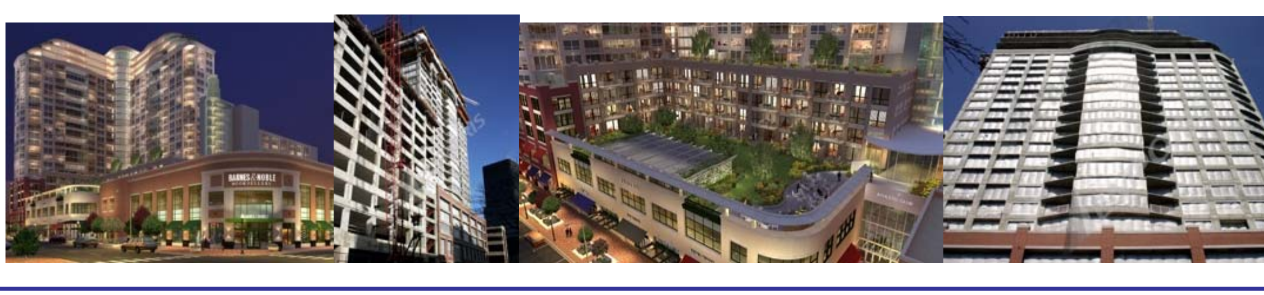
In Structural Technical Report 2, two steel floor framing systems were analyzed to compare the pros and cons with the existing system. It was found that both a composite and non-composite system would work for the building, but each of the systems has its strengths and weaknesses. A drawback to the steel systems is that they have a large ceiling to floor section depth. This larger depth increases the total building height, which causes an increase in costs in items, such as exterior cladding, mechanical equipment, etc. Therefore, the non-composite system was not considered, because it has an even larger section depth than the composite system. Although the composite system depth is larger than the existing concrete depth, the cost and schedule analysis determines if the other advantages to the steel system outweigh this drawback.

The steel system allows savings in time and cost in other areas and has the following advantages over the existing concrete system:

- The building's weight was reduced by switching to steel, which in turn allowed the size of the foundations to be reduced. Since the foundations are belled caissons extending 70 feet with a maximum diameter of 15 feet, a smaller caisson size resulted in significant savings in concrete.
- The steel system is easier to construct, because it does not require the use of the formwork and shoring necessary for the concrete flat plate floor system. Therefore, the construction time was reduced.
- The use of steel for both the gravity and lateral systems of the building eliminates the need to schedule both concrete and steel workers on the construction site which shortens the construction time.

There are also several considerations other than the floor framing to take into account about the structural system when changing the structural material. As already stated, the foundations were resized because of the decrease in building weight. In addition, the lateral resisting system was redesigned. The current system incorporates both concrete moment frames and shear walls. The concrete moment frames were replaced by steel frames. It was determined that steel braced frames would also be used instead of shear walls, in order to eliminate the need to tie the steel beams into the concrete walls. A steel lateral system required much less material, because the shear walls and large grade beams underneath were eliminated.

STRUCTURAL SYSTEM ANALYSIS AND REDESIGN



Structural System Analysis and Redesign

Gravity System:

The existing reinforced cast-in-place concrete system was replaced with a structural steel system in an effort to better understand the complexity of designing a high-rise building's gravity and lateral systems. Due to architectural constraints, it was determined that the column grid would remain the same as the existing system. The floor plans were then entered into RAM Structural System to design the new floor framing.

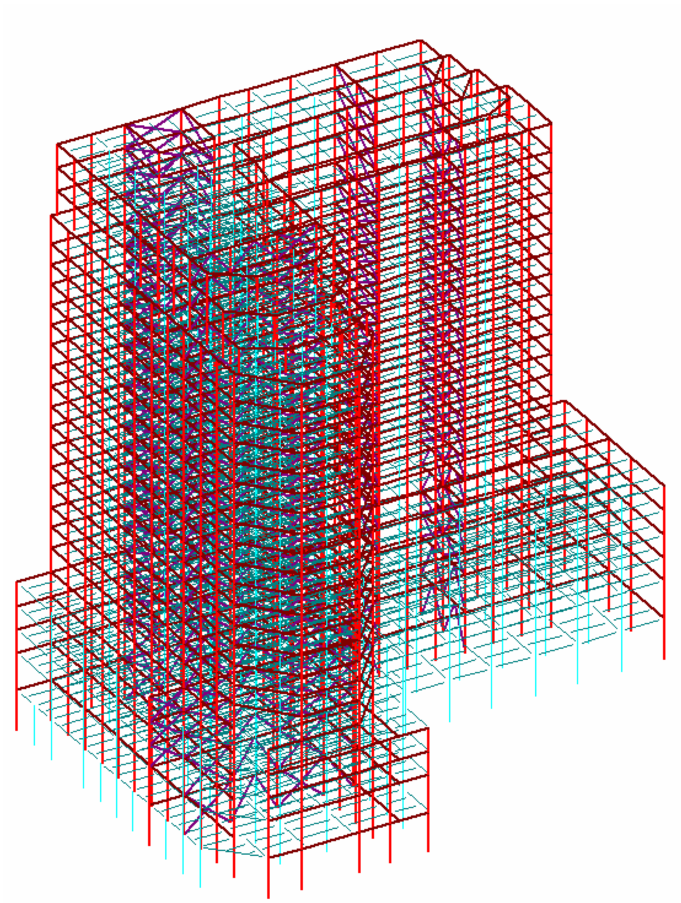


Figure 5: 3D Model of Building in RAM Steel

A 1.5" 18 gage composite Lok-Floor deck was chosen from the United Steel Deck catalogue. The deck can span 10.10 feet and will hold a maximum uniform live service load of 400 psf. The deck is topped by 2" lightweight concrete, and the shear studs are 3" long by ¾" diameter. Surface loads were applied to each floor, and a line load was applied to the perimeter of each floor to account for the weight of the cladding material. Table 1 contains the surface loads that were used in the RAM model.

Table 1: Surface Loads		
	Superimposed	
	Dead Load	Live Load
Retail	25 psf	100 psf
Residential	15 psf	80 psf
Storage	25 psf	100 psf
Roof Garden	15 psf	80 psf

The entire second level was applied with the retail area loads. The third, fourth and fifth floors contain both residential areas and storage areas. The sixth and seventh floors are residential floors but are also applied with a higher load where the building steps back to account for the extra load due to the intensive roof garden. The remaining floors are all residential levels and are applied with the typical residential load. On the top three penthouse levels, the slab projects several feet beyond the beam edge to account for the additional weight of the concrete eyebrow overhangs.

The gravity beams were designed by RAM Structural System. In an effort to limit the overall building height, the beams on floors 8-25 were restricted to W14s and beams on floors 2-7 were restricted to W16s. For level 8, the typical floor, the beam sizes range between W8x10 and W14x22. The normal size for an in-fill beam spanning 21' is W10x12 with 16 shear studs. A typical girder size is W14x22 with 24 studs or a W12x19 with 24 shear studs, spanning 21'. See Figure 6 for the beam sizes on the typical floor.

For the lower levels, the beam sizes range between a W8x10 and a W16x31. The normal size for an in-fill beam spanning 21' is W12x14 with 8 shear studs, and a typical girder size is W14x22 with 20 studs or a W16x26 with 16 studs. For the top three penthouse levels, the beam sizes range between W8x10 and W12x22. The typical in-fill beam size is W10x12 with 10 studs, and a normal girder size is W12x19 with 30 shear studs. Only the 22nd floor differs from these typical sizes, because it contains large transfer girders which hold extra weight due to the different column grid of the penthouse levels. The beam sizes of the 22nd floor range between W8x10 and W24x117. See Figures 6-9 for the beam sizes of the typical lower floor, penthouse level and the 22nd floor.

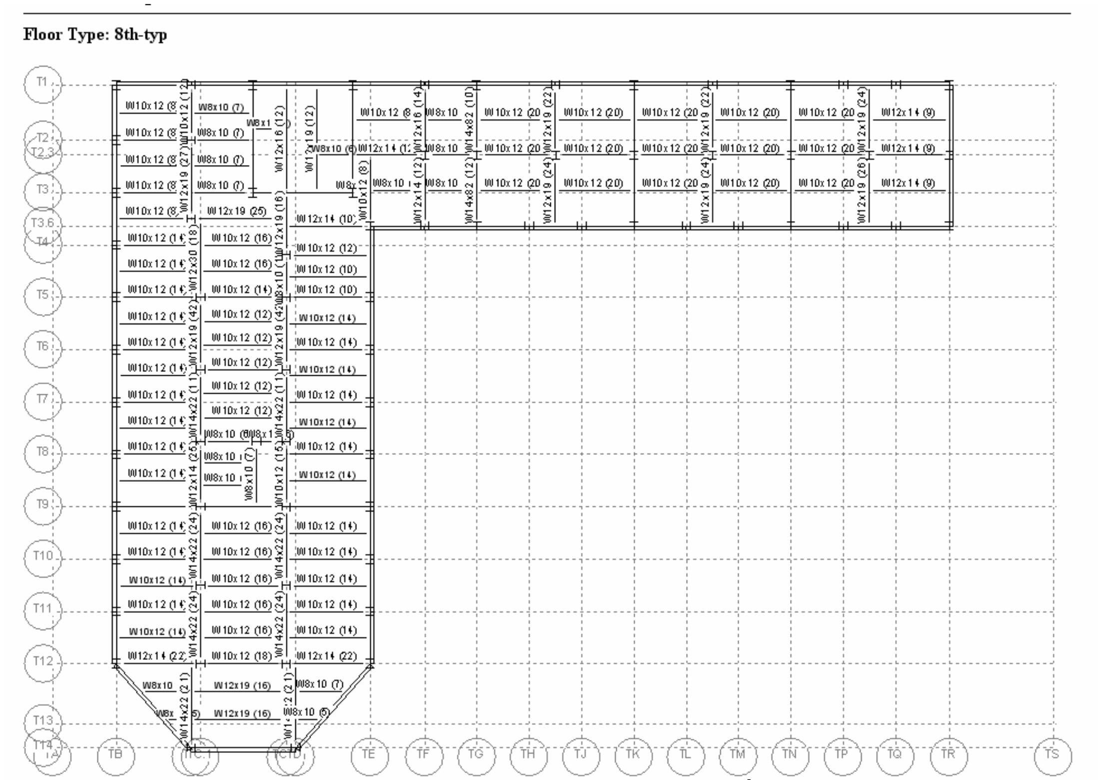


Figure 6: Beam Sizes for Typical Floor (8th Floor Plan)

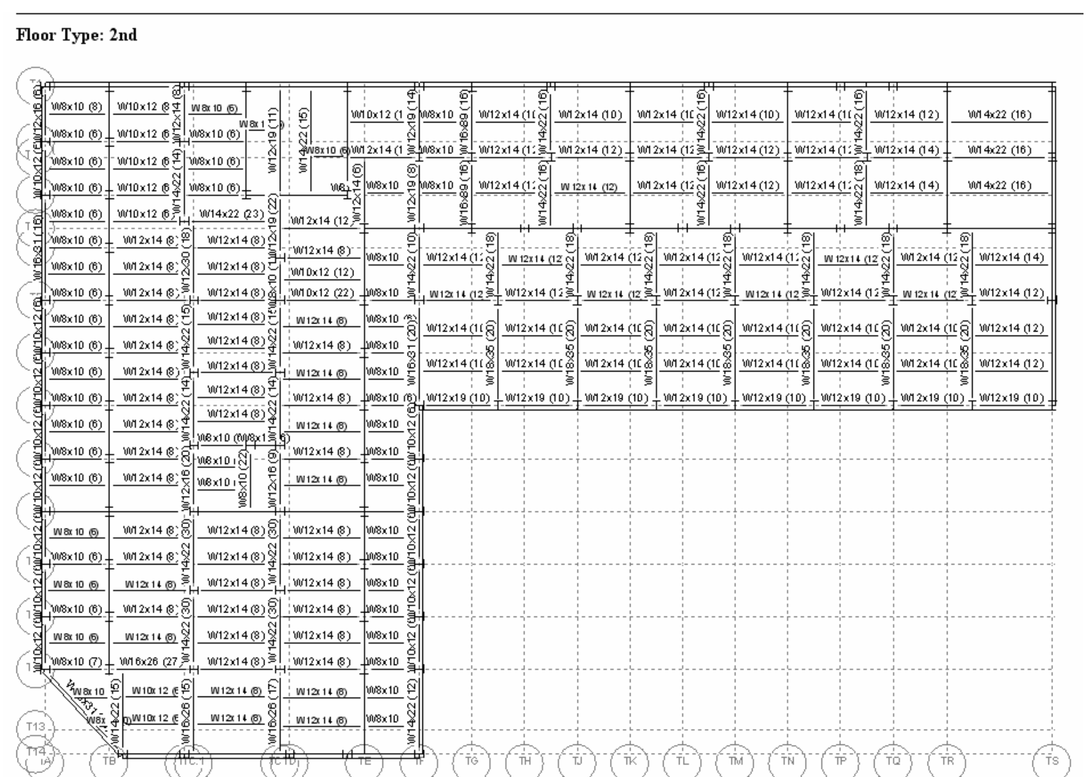


Figure 7: Beam Sizes for Typical Lower Floor (2nd Floor Plan)

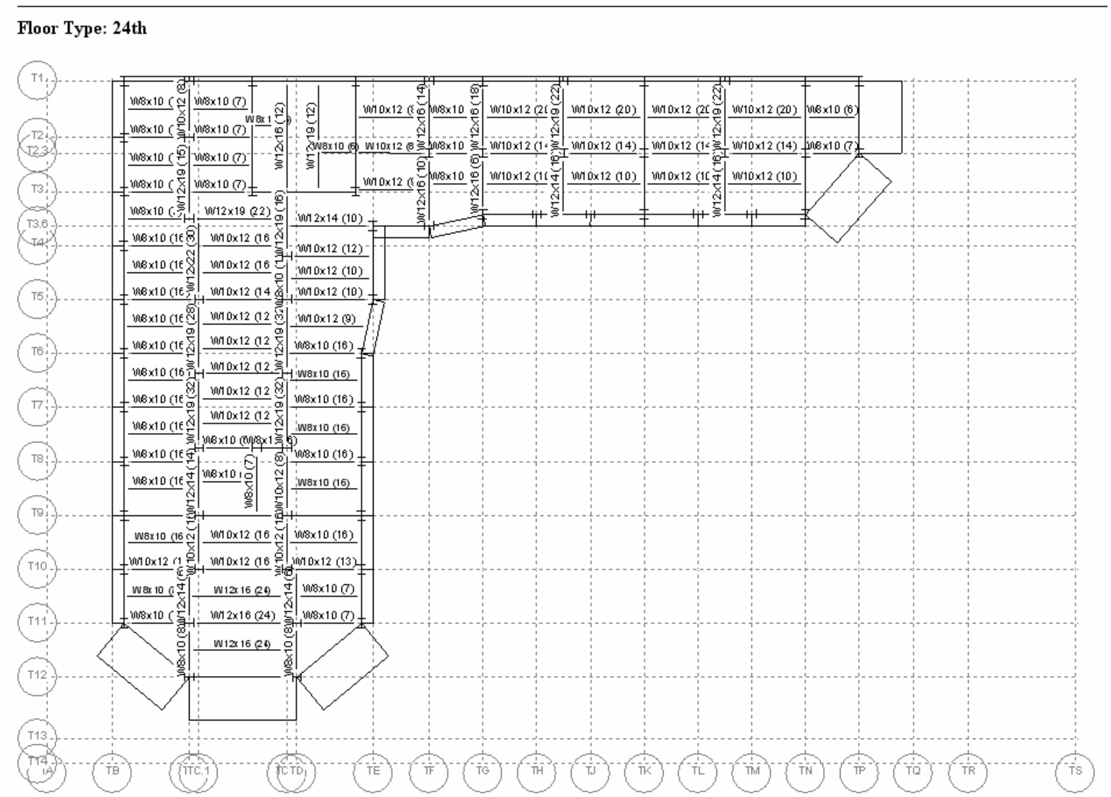


Figure 8: Beam Sizes for Typical Penthouse Level (24th Floor Plan)

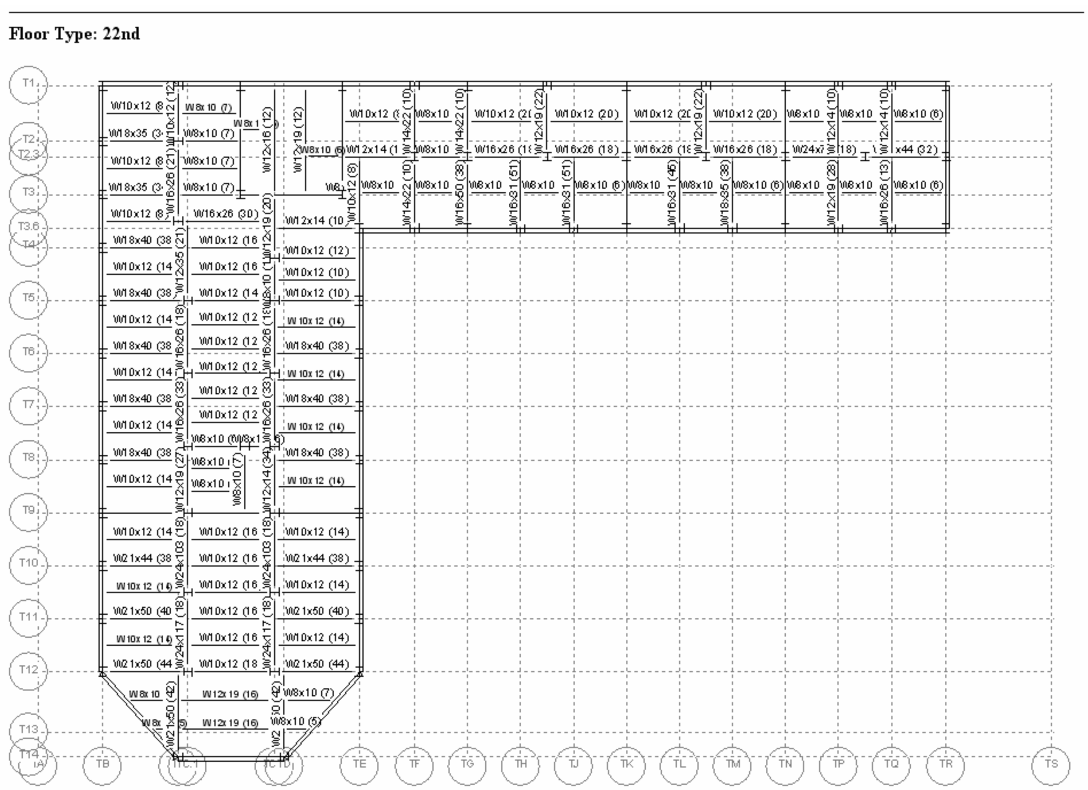


Figure 9: Beam Sizes for 22nd Floor

After designing the beams, the new building height was calculated in order to input the new floor to floor heights with which to design the columns. Each level retains the same floor to ceiling height as the existing system, but the structural materials of the steel system created a larger ceiling to floor section depth than the flat plate concrete system. The original building was 260.5 feet high and the redesigned building is 283.25 feet. Table 2 shows the new building height calculations.

Table 2: Overall Building Height Calculations

Floor #:	Beam Size	Depth (in.)	Deck (in.)	Total Depth (in.)	Original Story Height (ft.)	New Story Height (ft.)	Actual Story Height (ft.)
2	W16x31	15.9	5	20.9	18.6667	19.7417	19.75
3	W16x31	15.9	3.5	19.4	17.5	18.4500	18.5
4	W16x31	15.9	3.5	19.4	10.3333	11.2833	11.3333
5	W16x31	15.9	3.5	19.4	10.3333	11.2833	11.3333
6	W16x31	15.9	3.5	19.4	11	11.9500	12
7	W16x31	15.9	3.5	19.4	11	11.9500	12
8	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
9	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
10	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
11	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
12	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
13	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
14	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
15	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
16	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
17	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
18	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
19	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
20	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
21	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
22	W14x30	13.8	3.5	17.3	9.1667	9.9417	10
23	W24x68	23.7	3.5	27.2	12	13.6000	13.6667
24	W14x43	13.7	3.5	17.2	10.6667	11.4334	11.5
25	W14x38	14.1	3.5	17.6	10.6667	11.4667	11.5
ROOF	W14x38	14.1	3.5	17.6	10.8333	11.6333	11.6667
				TotalHeight =	260.5005	281.9172	283.25

After inputting the new floor to floor height, the columns were designed in RAM Steel. The column sizes range from W10x33 and W14x193. The typical column size on the ground floor is W14x145 to W14x132. On the column lines that extend from the ground floor to the roof, the typical column size at the 25th floor is W14x43. On the three penthouse levels, the typical size is a W10x33 due to the fact that they extend up only three floors. See Figures 10-11 for the column layout and column schedule.

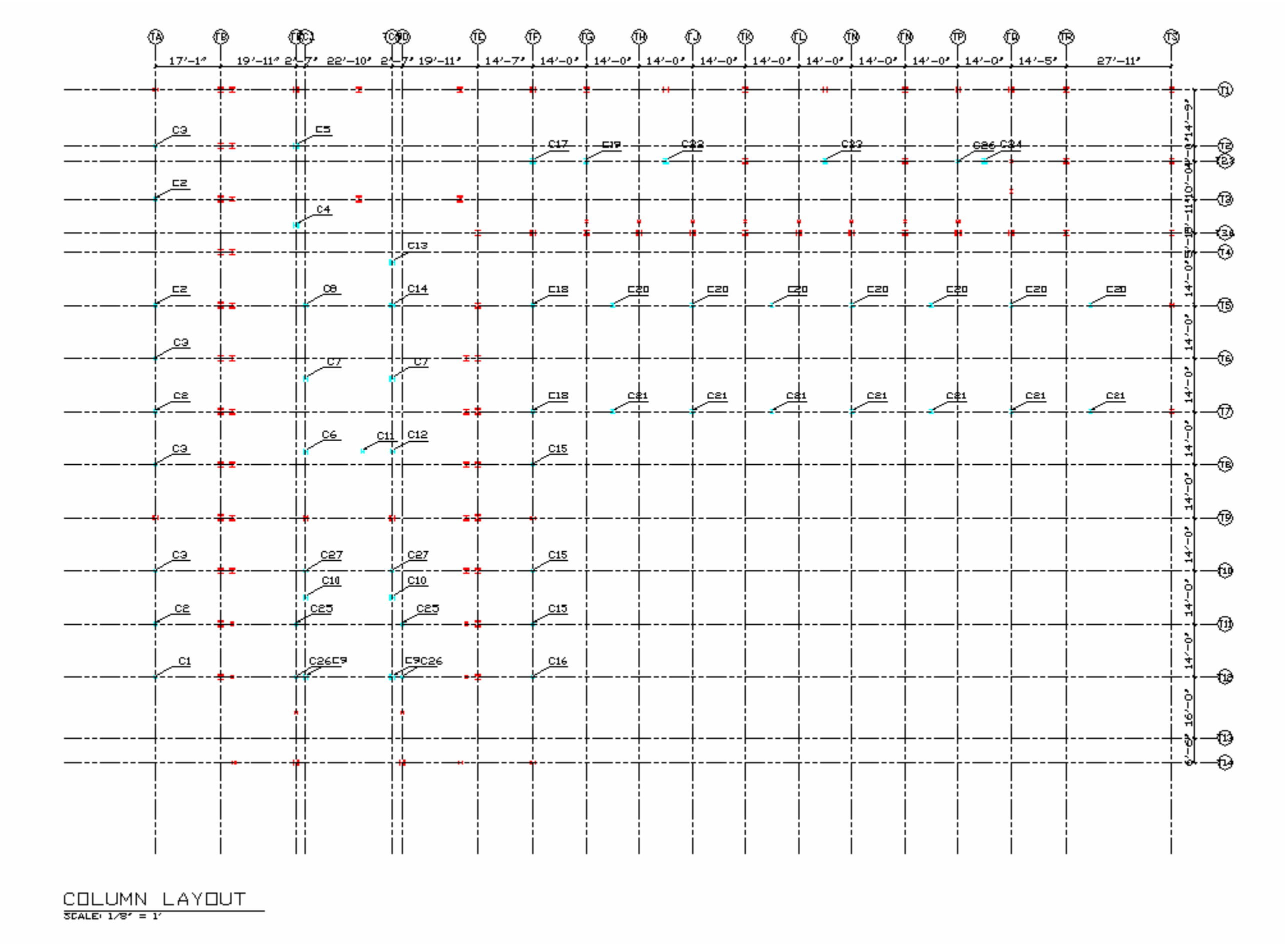


Figure 10: Column Layout from RAM Steel

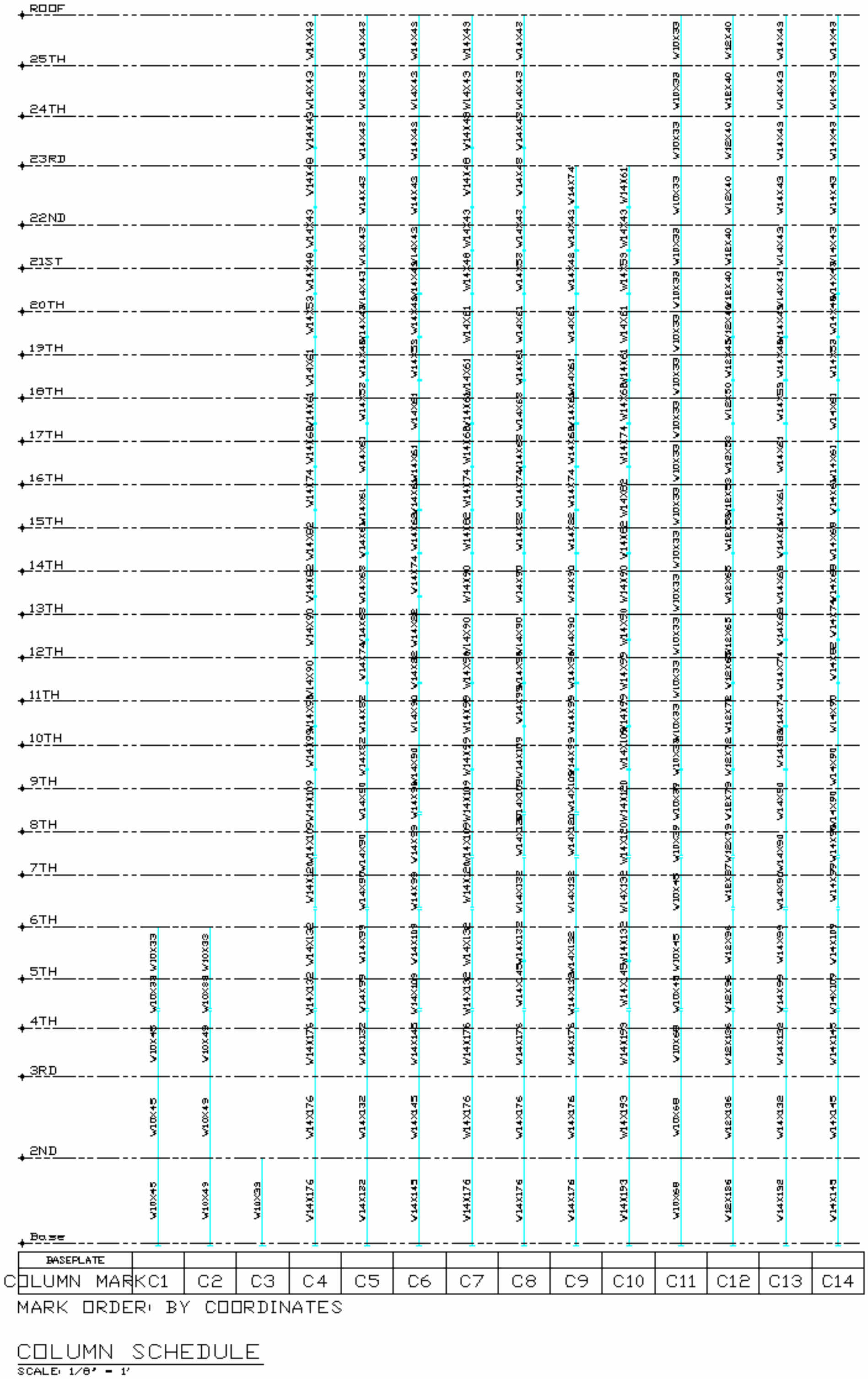


Figure 11a: Column Schedule

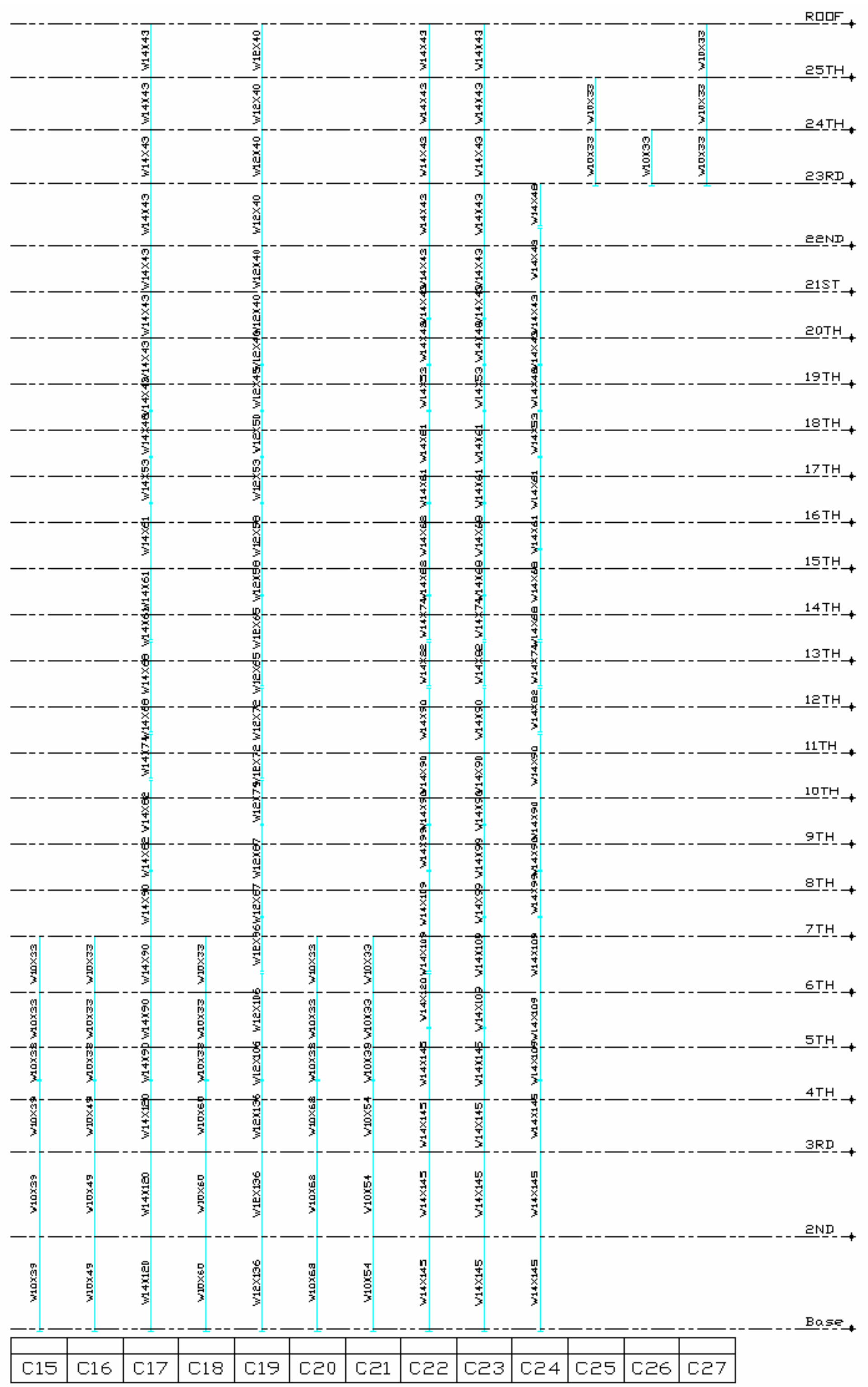


Figure 11b: Column Schedule

Lateral System:
After designing the building's structural steel gravity system, the existing lateral system, which is made up of both concrete shear walls and moment frames, was redesigned using a combination of steel moment and braced frames. While the concrete shear walls could have been used in the new design of the building, it was decided that they would be eliminated in an attempt to reduce the weight of the building and to shorten the construction time.

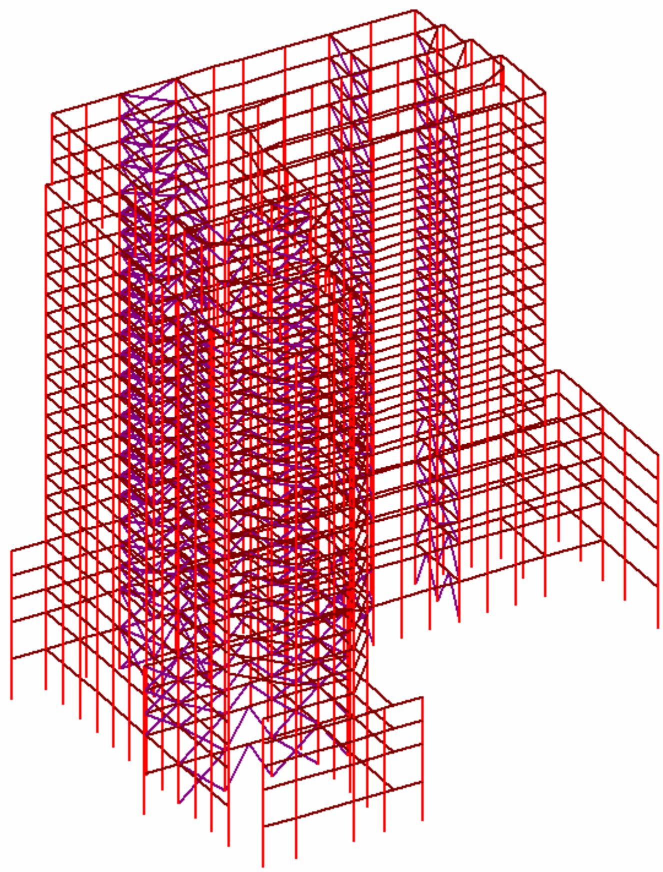


Figure 12: 3D Model of Lateral System

After a few lateral system designs were tested in the RAM Steel model, it was determined that the drift limit would control the design. The drift limit of Sherman Plaza was set at H/600 for other trades to use, such as windows and exterior cladding material. Since these materials will not be changed in the building redesign, the allowable drift for the structural steel building will be H/600. For a building of 283.25 feet, this drift value is 5.665 inches.

When choosing the locations of the lateral elements, it was important to lessen the impact on the architectural design as much as possible. The braced frames, therefore, were placed in the locations of the original shear walls. The moment frames, also, replaced the existing concrete moment frames around the building's perimeter. With this design, the building drift was too high in the Y direction, so another braced frame was added where there is a wall between two residential units. See Figure 13 for the location of the braced frames and shear walls. Frames A-G are braced frames, and Frames G-Q are moment frames.

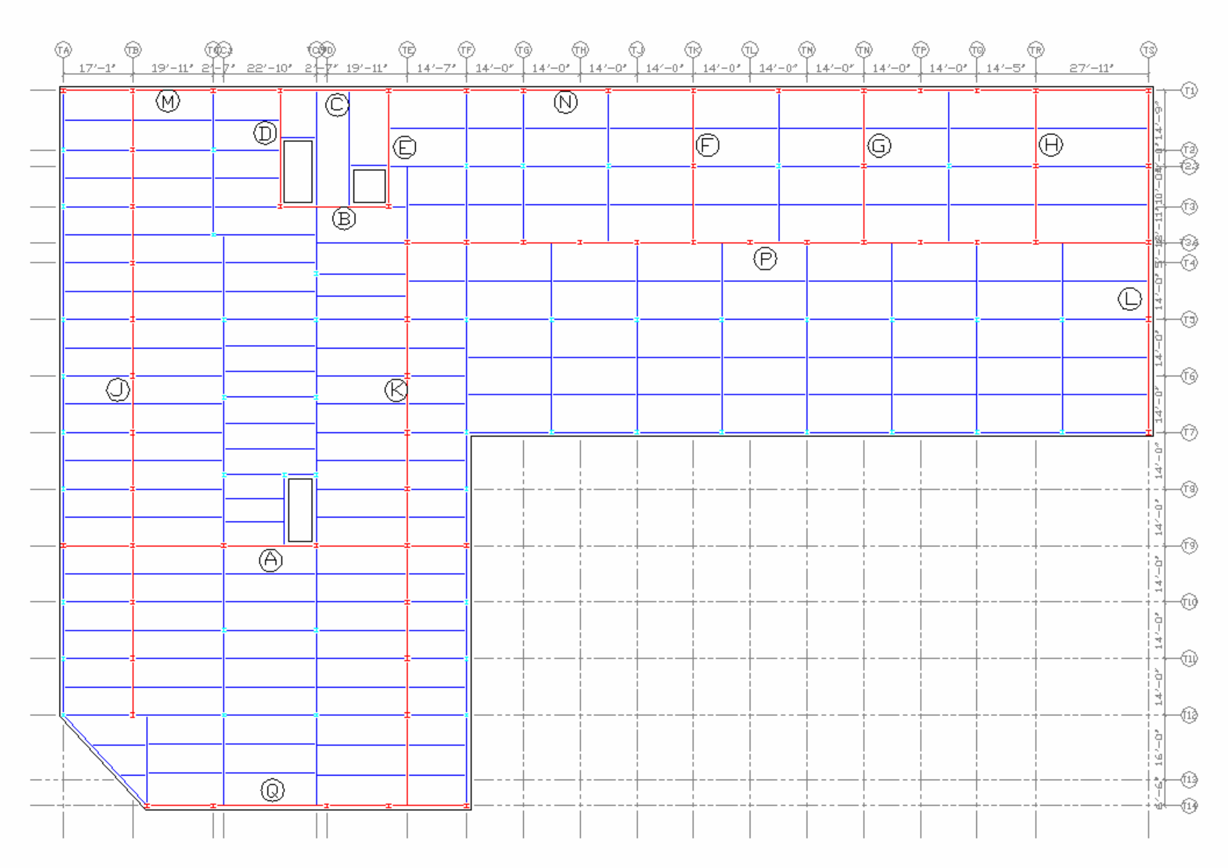


Figure 13: Moment and Braced Frame Locations

After determining the locations of the lateral elements, the architecture of each floor was analyzed to determine the shape of the bracing elements. Cross braces were used in bays with no openings. Several of the bays contained doorways, so these bays contain either chevron or diagonal braces. See Figures 14-20 for the braced frame designs.

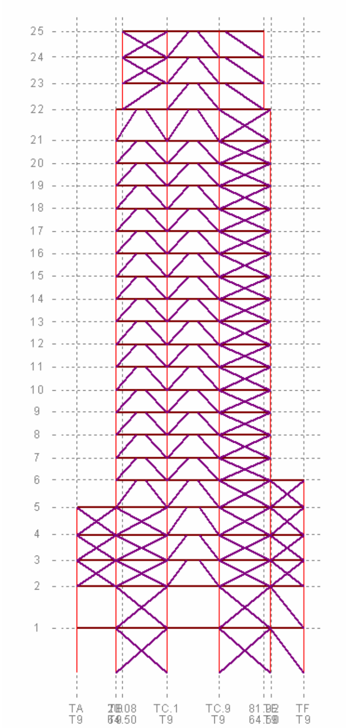


Figure 14: Braced Frame A

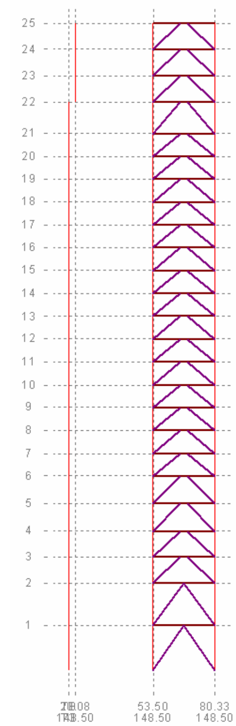


Figure 15: Braced Frame B

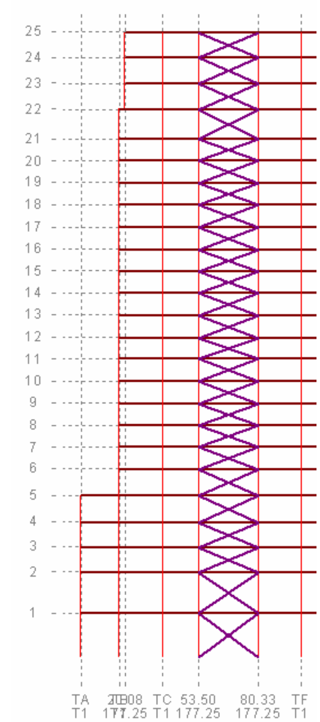


Figure 16: Braced Frame C

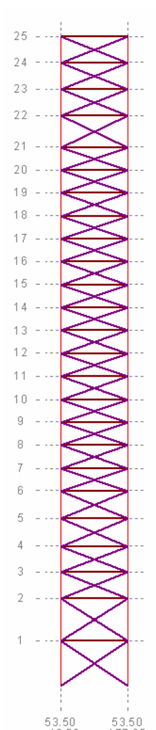


Figure 17: Braced Frame D

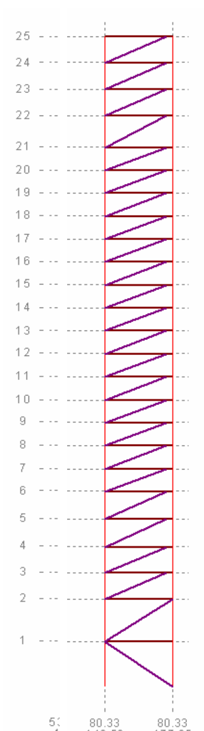


Figure 18: Braced Frame E

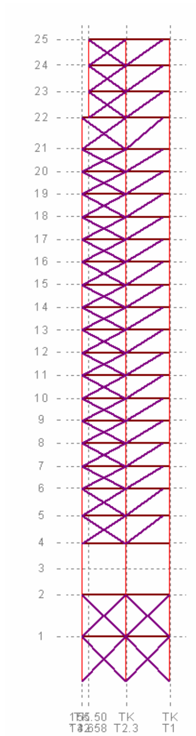


Figure 19: Braced Frame F

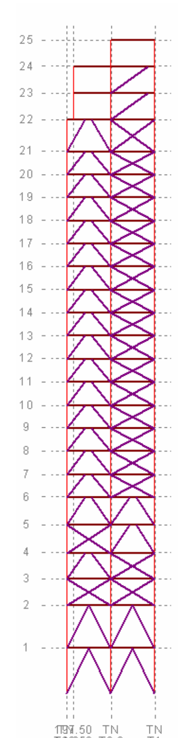


Figure 20: Braced Frame G

Next, the seismic calculations were updated with the new building height and weight. The full hand calculations can be found in Tables A.5-A.6 in the Appendix. The analysis of the building, however, was performed with loads generated by RAM Steel using the code provisions of ASCE 7-02. The computer generated loads have been compared to the hand calculations and both result in very similar values. The error in the hand calculations can be attributed to the estimate of the building weight on each floor. A summary of the computer loads which were used to design the lateral system are presented in Tables 3-4. The wind load cases are taken from the four load cases in ASCE 7-02, Figure 6-9.

Table 3: Wind Load Cases according to ASCE 7-02, Figure 6-9

	Wind1 X	Wind1 Y	Wind2 X+E	Wind2 X-E	Wind2 Y+E	Wind2 Y-E	Wind3 X+Y		Wind3 X-Y		Wind4 CW		Wind4 CCW	
Level	Fx (kips)	Fy(kips)	Fx (kips)	Fx (kips)	Fy(kips)	Fy (kips)	Fx (kips)	Fy (kips)	Fx (kips)	Fy (kips)	Fx (kips)	Fy(kips)	Fx (kips)	Fy (kips)
ROOF	21.25	28.34	18.6	18.6	24.8	24.8	15.94	21.25	15.94	-21.25	13.95	18.6	13.95	18.6
25	43.92	61.55	38.43	38.43	53.85	53.85	32.94	46.16	32.94	-46.16	28.82	40.39	28.82	40.39
24	46.33	62.98	40.54	40.54	55.11	55.11	34.75	47.23	34.75	-47.23	30.41	41.33	30.41	41.33
23	52.08	67.86	45.57	45.57	59.37	59.37	39.06	50.89	39.06	-50.89	34.18	44.53	34.18	44.53
22	48.95	63.85	42.83	42.83	55.87	55.87	36.72	47.89	36.72	-47.89	32.13	41.9	32.13	41.9
21	40.99	53.48	35.86	35.86	46.8	46.8	30.74	40.11	30.74	-40.11	26.9	35.1	26.9	35.1
20	40.66	53.07	35.58	35.58	46.43	46.43	30.49	39.8	30.49	-39.8	26.68	34.83	26.68	34.83
19	40.32	52.64	35.28	35.28	46.06	46.06	30.24	39.48	30.24	-39.48	26.46	34.55	26.46	34.55
18	39.96	52.2	34.97	34.97	45.67	45.67	29.97	39.15	29.97	-39.15	26.23	34.25	26.23	34.25
17	39.6	51.74	34.65	34.65	45.27	45.27	29.7	38.8	29.7	-38.8	25.98	33.95	25.98	33.95
16	39.21	51.26	34.31	34.31	44.85	44.85	29.41	38.45	29.41	-38.45	25.73	33.64	25.73	33.64
15	38.82	50.76	33.96	33.96	44.42	44.42	29.11	38.07	29.11	-38.07	25.47	33.31	25.47	33.31
14	38.4	50.24	33.6	33.6	43.96	43.96	28.8	37.68	28.8	-37.68	25.2	32.97	25.2	32.97
13	37.97	49.7	33.22	33.22	43.49	43.49	28.47	37.27	28.47	-37.27	24.92	32.61	24.92	32.61
12	37.51	49.13	32.82	32.82	42.99	42.99	28.13	36.85	28.13	-36.85	24.62	32.24	24.62	32.24
11	37.03	48.52	32.4	32.4	42.46	42.46	27.77	36.39	27.77	-36.39	24.3	31.84	24.3	31.84
10	36.52	47.89	31.95	31.95	41.9	41.9	27.39	35.91	27.39	-35.91	23.96	31.42	23.96	31.42
9	35.97	47.21	31.48	31.48	41.31	41.31	26.98	35.4	26.98	-35.4	23.61	30.98	23.61	30.98
8	35.22	46.45	30.81	30.81	40.64	40.64	26.41	34.84	26.41	-34.84	23.11	30.48	23.11	30.48
7	37.1	53.44	32.46	32.46	46.76	46.76	27.82	40.08	27.82	-40.08	24.35	35.07	24.35	35.07
6	38.85	68.25	34	34	59.72	59.72	29.14	51.19	29.14	-51.19	25.5	44.79	25.5	44.79
5	36.57	60.66	32	32	53.08	53.08	27.43	45.5	27.43	-45.5	24	39.81	24	39.81
4	34.35	57.17	30.06	30.06	50.02	50.02	25.76	42.88	25.76	-42.88	22.54	37.52	22.54	37.52
3	42.93	71.85	37.57	37.57	62.87	62.87	32.2	53.89	32.2	-53.89	28.17	47.15	28.17	47.15
2	51.31	86.56	44.9	44.9	75.74	75.74	38.48	64.92	38.48	-64.92	33.67	56.81	33.67	56.81

Table 4: Seismic Load Cases according to ASCE 7-02				
	EQ X+E	EQ X-E	EQ Y+E	EQ Y-E
Level	Fx (kips)	Fx (kips)	Fy (kips)	Fy (kips)
ROOF	17.65	17.65	13.89	13.89
25	18.44	18.44	14.4	14.4
24	18.69	18.69	14.47	14.47
23	18.24	18.24	14	14
22	17	17	12.91	12.91
21	16.11	16.11	12.13	12.13
20	15.23	15.23	11.37	11.37
19	14.36	14.36	10.62	10.62
18	13.5	13.5	9.89	9.89
17	12.64	12.64	9.17	9.17
16	11.8	11.8	8.47	8.47
15	10.98	10.98	7.79	7.79
14	10.16	10.16	7.13	7.13
13	9.35	9.35	6.48	6.48
12	8.56	8.56	5.85	5.85
11	7.78	7.78	5.24	5.24
10	7.02	7.02	4.65	4.65
9	6.27	6.27	4.09	4.09
8	5.54	5.54	3.54	3.54
7	7.72	7.72	4.83	4.83
6	7.08	7.08	4.3	4.3
5	5.77	5.77	3.39	3.39
4	4.07	4.07	2.3	2.3
3	3.25	3.25	1.74	1.74
2	1.79	1.79	0.85	0.85

After a trial and error of several shapes and sizes, including W shapes, double angles, and tube shapes, the double angles resulted in the least drift. The bracing members were sized at 2L8x8x¾. The columns and beams that were a part of the moment and braced frames also needed to be resized. The gravity columns and beams were relatively small sizes and therefore did not provide much lateral resistance. The lateral beam sizes range between W16x89 on the lower floors to W14x82 on the upper floors. The columns in the frames along the Y axis were sized as W14x370 to W14x257, in general, and the columns in frames along the X axis were sized from W14x132 to W14x370. This design produced an acceptable building drift. The drift values for each load case and for critical load combinations are listed in Table 5.

Table 5: Drift for Load Cases and Combinations		
Load Cases	Drift X (in.)	Drift Y (in.)
D	-0.0863	-0.0825
Lp	-0.928	-0.6116
W1	3.1386	0.0354
W2	-0.4549	4.0159
W3	2.8335	0.0794
W4	2.6591	-0.0175
W5	-0.5449	3.4314
W6	-0.2512	3.5963
W7	2.0128	3.0384
W8	2.6952	-2.9854
W9	1.9227	2.7485
W10	1.5997	2.5687
E1	0.8287	0.07
E2	0.7681	0.0379
E3	0.011	0.8237
E4	0.0986	0.8703
Load Combinations	Drift X (in.)	Drift Y (in.)
1.2D + 0.5Lp + 1.3W2	-0.7914	4.8158
1.2D + 0.5Lp + 1.3W5	-0.9083	4.0561
1.2D + 0.5Lp + 1.3W6	-0.5265	4.2704
1.2D + 0.5Lp + 1.3W8	3.3038	-4.2858
1.2D + 0.5Lp - 1.3W1	-4.2802	-0.4508
1.2D + 0.5Lp - 1.3W2	0.3915	-5.6254
1.2D + 0.5Lp - 1.3W5	0.5085	-4.8657
1.2D + 0.5Lp - 1.3W6	0.1267	-5.0799
1.2D + 0.5Lp - 1.3W7	-2.8165	-4.3547
1.2D + 1.3W2	-0.695	5.1216
1.2D + 1.3W5	-0.8119	4.3619
1.2D + 1.3W6	-0.4301	4.5762
1.2D - 1.3W1	-4.1838	-0.145
1.2D - 1.3W2	0.4879	-5.3196
1.2D - 1.3W5	0.6049	-4.5599
1.2D - 1.3W6	0.2231	-4.7742
1.2D - 1.3W7	-2.721	-4.0489
0.9D + 1.3W1	4.0026	-0.0283
0.9D + 1.3W2	-0.6691	5.1464
0.9D + 1.3W6	-0.4042	4.6009
0.9D - 1.3W1	-4.1579	-0.1202
0.9D - 1.3W2	0.5138	-5.2949
0.9D - 1.3W5	0.6308	-4.5351
0.9D - 1.3W6	0.2489	-4.7494
1.2D +0.5Lp -1.0E1	-1.0286	-0.4748
1.2D +0.5Lp -1.0E3	-0.2109	-1.2285
1.2D +0.5Lp -1.0E4	-0.2985	-1.2751

As can be seen in Table 5, the lateral system design was controlled by the load combination, 1.2D+0.5Lp-1.3W2. This combination produced a drift of 5.6254 inches, which is less than the allowable drift of H/600 = 5.665 inches.

Foundation:

The original foundation design called for large belled caissons due to the large building weight. The new foundation sizes were estimated according to the new column loads. The allowable soil bearing capacity for the building site's soil is 30 ksf. To get the area of the new caisson bell, the column load in units of kips was divided by the allowable soil bearing capacity. The area of the bell was then used to find the new bell diameter. See Table A.7 in the Appendix for the full calculations of each foundation. The layout of the new foundations is shown in Figure 21.

The original foundation sizes varied between a 15'-6" bell diameter and 6'-0" shaft diameter caisson to a 6'-0" bell and 2'-6" shaft. The new design results in caisson sizes that range between a 3'-0" bell and a 7'-0" bell.

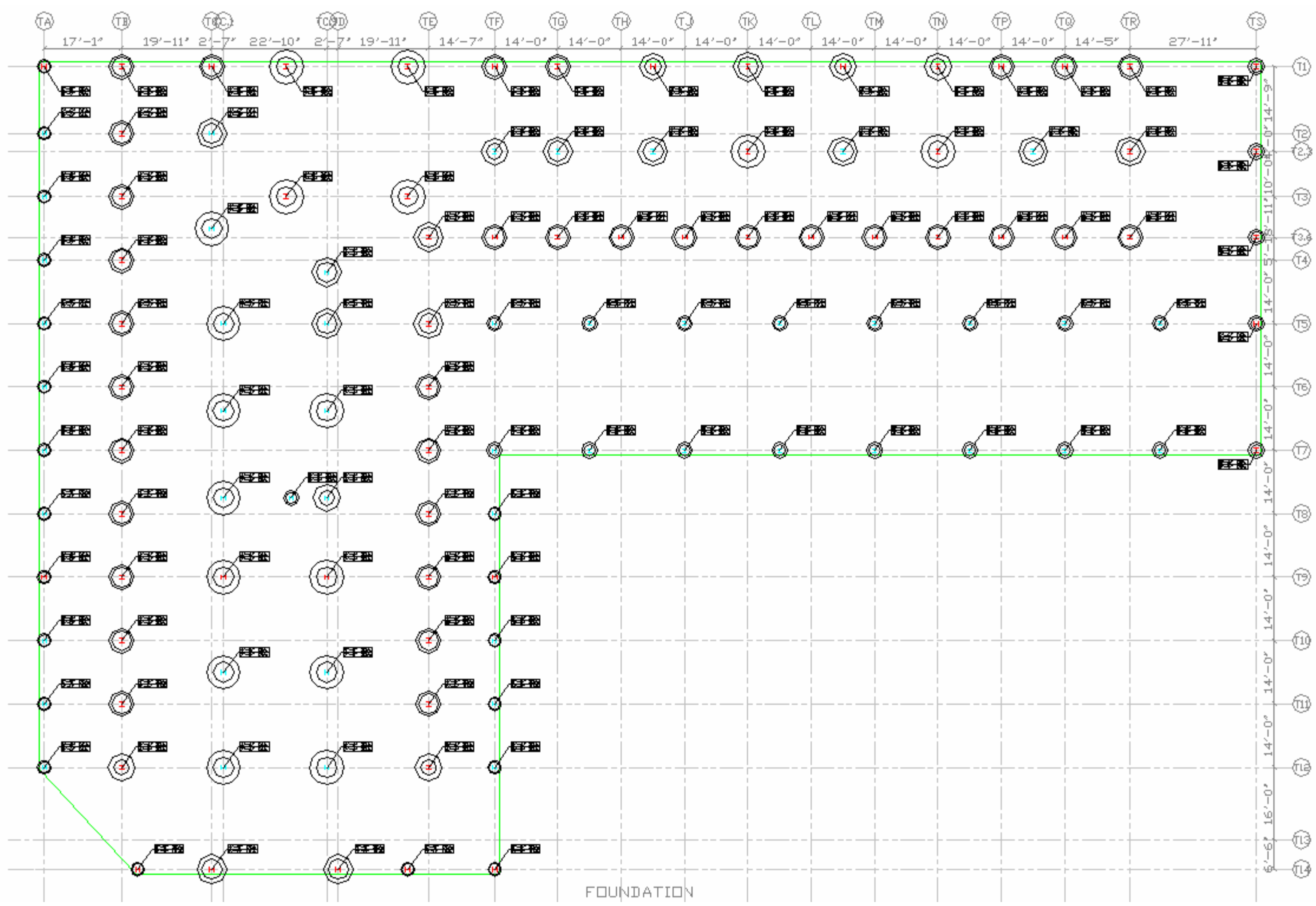


Figure 21: Foundation Plan

Summary:

The program, RAM Structural System, was used to design both the gravity and lateral system of Sherman Plaza. The column grid was not altered due to architectural restraints, and a layout of beams and girders was designed. The plans were entered into RAM, and a composite metal floor deck was chosen. The deck is a 1.5” 18 gage composite Lok-Floor and is topped by 2” lightweight concrete. The shear studs are 3” by ¾” diameter. The gravity system was designed and resulted in beam sizes that ranged between W8x10 and W16x31 on the lower floors and betweenW8x10 and W14x22 on the upper floors. From these beam sizes, the new building height was calculated to be 283.25 feet. The gravity columns were then designed, and it was found that the sizes varied between W14x193 and W10x33.

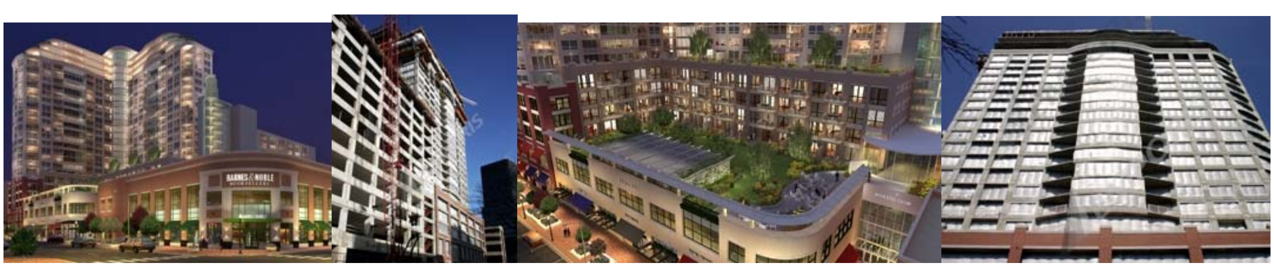
After designing the gravity system, the lateral system was designed using RAM Steel. The drift limit of the building was set to H/600, which results in a maximum allowable drift of 5.665 inches at the top story. To achieve an acceptable drift, it was determined that the building required braced frames in the locations of the original shear walls. Moment frames were also placed around the building’s perimeter. The lateral beam sizes were increased to W16x89 and W14x82, and the column sizes ranged between W14x257 and W14x370. The lateral bracing is made up of 2 L8x8x3/4. This design resulted in a drift of 5.6254 inches, which is below the allowable drift.

Using the new building design, new foundation sizes were estimated. The column load at the ground floor was found from RAM Steel. This value was used to find the caisson surface area, by dividing the column load by the allowable soil bearing pressure of 30 ksf. The surface area was then used to find the bell diameter for each of the caissons.

In all, the steel structural system was an effective design for this building. The composite steel produced an efficient gravity system that worked well with the given column layout. The drawback to this system, however, was that the structural system’s ceiling to floor section depth was greater than that of the existing concrete system. This increase in depth at each floor resulted in an increase of building height from 260.5 feet to 283.25 feet, which is an increase of 22.75 feet. The newly designed lateral system also produced acceptable results. The design, however, uses a large number of braced frames and moment connections that will increase construction time and costs.

The building weight was dramatically reduced, which allowed the foundation sizes to be decreased. Due to the fact that the caissons extend down 70 feet, this size reduction will result in large savings in concrete and in construction time. This savings will be investigated further in the construction management breadth study.

CONSTRUCTION MANAGEMENT BREADTH STUDY



Construction Management Breadth Study

Cost Analysis:

By changing the structural material of Sherman Plaza, the building cost will also be affected. This change will impact the costs of materials, equipment and labor. R.S. Means was used to estimate the costs of both the reinforced concrete system and the structural steel system for a comparison between the two. In this estimate, only the structural materials and the exterior cladding material were considered. Therefore, in the steel system, the materials that were considered are the beams, columns, lateral bracing, shear studs, metal deck, foundations, and concrete slab. The amount of structural materials was either estimated or taken from the takeoff from RAM Steel. For the concrete system, the beams, columns, slabs, foundations, and shear walls were considered. The exterior cladding cost was estimated for each system, because the steel structural system results in a building with a greater height and therefore greater cladding cost. Table 6 shows the cost estimate for the steel system. Table 7 shows the cost estimate for the reinforced concrete system. The full calculations and takeoff can be found in the appendix.

Table 6: Steel System Cost Summary		
	Total Length (ft.)	Total Cost
Steel Beams	99845.69	3113651.455
	Total Length (ft.)	Total Cost
Steel Columns	23015.2	3750311.381
	Total Length (ft.)	Total Cost
Lateral Bracing	25884.8	694489.184
	Total No. Studs	Total Cost
Shear Studs	67681	92722.97
	Total Sq. Feet	Total Cost
Metal Deck	593800	1146034
	Total Sq. Feet	Total Cost
Concrete Slab	5500	569525
	Total Cu. Yards	Total Cost
Foundations	3837.03	3473184.2
	Total Sq. Feet	Total Cost
Ext. Cladding	220668.07	4614169.3
Total Cost Steel System:		17454087.5

Table 7: Concrete System Cost Summary		
	Total Cu. Yards	Total Cost
Concrete Columns	3316.452	3028635.148
	Total Cu. Yards	Total Cost
Concrete Slab	14662	7169718
	Total Cu. Yards	Total Cost
Shear Walls	2265.222	377100.3204
	Total Cu. Yards	Total Cost
Foundations	3509.74	10790575.1
	Total Sq. Feet	Total Cost
Ext. Cladding	203964.58	4264899.4
Total Cost Concrete System:		25630928.0

Schedule Estimate:

The schedule estimate was also performed using data from R.S. Means. For each of the structural materials considered in the cost analysis above, the daily output was found and used to find the total number of days to perform each task. The days for each task were then added to provide an estimate of total days to erect each structural system. Table 8 and Table 9 provide the schedule estimates for the steel and concrete systems, respectively. The full calculations and takeoffs can be found in the appendix.

Table 8: Steel System Schedule Summary		
	Total Length (ft.)	Total Days
Steel Beams	99845.69	143.8260964
	Total Length (ft.)	Total Days
Steel Columns	23015.2	24.06033438
	Total Length (ft.)	Total Days
Lateral Bracing	25884.8	115.0435556
	Total No. Studs	Total Days
Shear Studs	67681	70.50104167
	Total Sq. Feet	Total Days
Metal Deck	593800	138.0930233
	Total Sq. Feet	Total Days
Concrete Slab	5500	34.375
	Total Cu. Yards	Total Days
Foundations	3837.04	619.3
Total Days Steel System:		1145.2

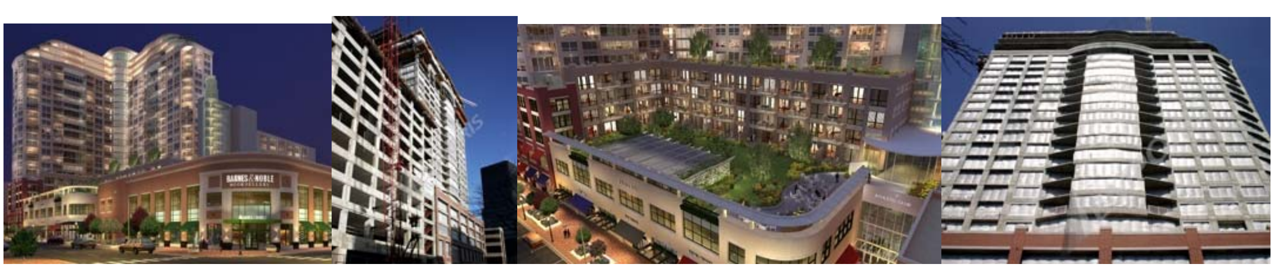
Table 9: Concrete System Schedule Summary		
	Total Cu. Yards	Total Days
Concrete Columns	3316.452	222.60
	Total Cu. Yards	Total Days
Concrete Slab	14662	484.85
	Total Cu. Yards	Total Days
Shear Walls	2265.222	29.810
	Total Cu. Yards	Total Days
Foundations	3509.74	1922.3
Total Days Concrete System:		2659.6

Summary:

An estimate was performed of the costs of the exterior cladding and structural materials for each system from R.S. Means. The steel system resulted in a total cost of $17.45 million, and the reinforced concrete system had a total cost of $25.63 million. The steel system, therefore, was $8.18 million less expensive than the concrete system.

R.S. Means was also used to perform a schedule estimate. The steel system took a total of 1146 days to complete, while the concrete system took 2660 days. Therefore, the steel system could be erected 1514 days faster than the concrete system. These values, however, are based on the crew type that is used to perform each task. If the same crew does not perform the tasks for each of the buildings, then these results may not be completely accurate.

ACOUSTICS BREADTH STUDY



Acoustics Breadth Study

Introduction:

By changing the primary structural system of Sherman Plaza from reinforced concrete to structural steel, other building systems were also impacted. The building's acoustics were affected, because the new structural materials have a different Sound Transmission Class than the existing materials. The new system should provide the same, if not better, sound isolation as the existing system. In cases when the sound transmission value was too high, other alternatives were considered to bring the sound pressure levels to acceptable values.

The sound transmission was considered in locations that were directly affected by the change of structural materials. The first case to be considered was the sound transmission through the floor, because the new floor system is considerably thinner than the original floor. The second case considered an area that contained a concrete shear wall in the original structural design. The new design, therefore, would need to provide a new wall design that would provide comparable transmission loss.

Floor System Transmission Loss:

The transmission loss of the floor system was analyzed between the residential and retail portions of the building. The transition between the retail and residential was a critical area, because the retail area, which contains a health club, had a higher sound level and required a greater sound barrier to isolate sound from the residential dwellings.

First, the transmission loss (TL) of the floor systems was determined from tables of TL data for common building elements. The source room sound pressure level (L1) was also determined from a table of noise level data for common building activity noise sources. The Noise Reduction value of the floor system was found using the following equation:

$$NR = TL + 10 \log(a/S)$$

The value of the variable "a" was found by multiplying the surface area of the receiving room's materials by their sound absorption coefficient. The noise reduction is then used to find the receiving room's sound pressure level (L2) using the following equation:

$$L2 = L1 - NR$$

The acceptable range of noise criteria for a residential space is from NC-25 to NC-35. To be conservative, the sound pressure levels will be compared with NC-25. These decibel values can be found from Figure 22.

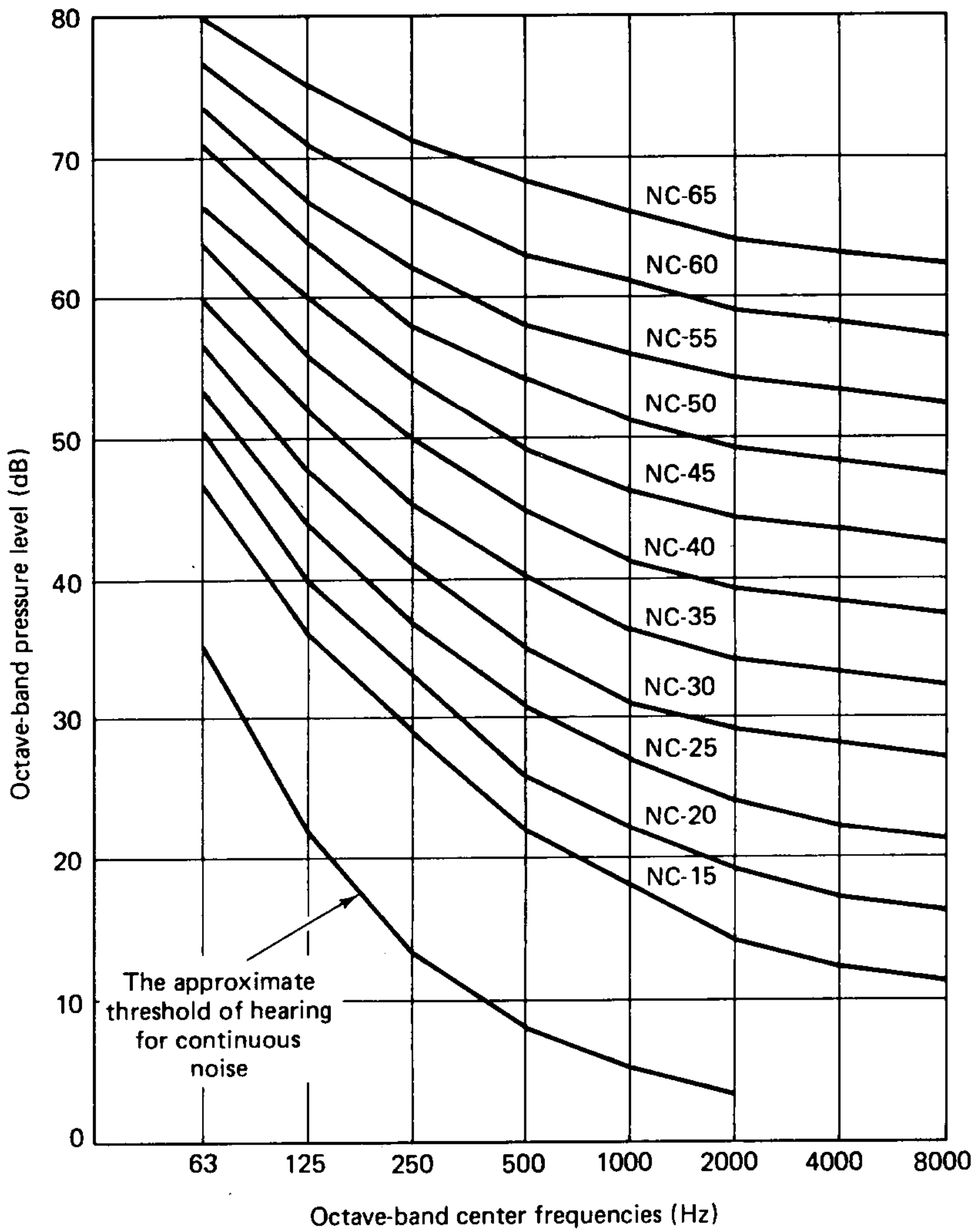


Figure 22: Noise Criteria Curve

The original floor system is composed of an eight inch thick concrete slab. The original floor system was found to have acceptable transmission loss, but the new floor system did not. The new floor system is made up of a three inch concrete slab on composite metal deck. The calculations can be found in Tables 10-11. Figures 23-24 show the residential sound pressure level versus the NC-25 noise criteria curve.

Original Floor System: 8" Reinforced Concrete Floor							
Source Room: Health Club							
Receiving Room: Residential Area							
	Surface Area:		Sound Absorption Coefficients				
Concrete Walls	490	0.1	0.05	0.06	0.07	0.09	0.08
Partition Walls	2170	0.55	0.14	0.08	0.04	0.12	0.11
Floor	1344	0.04	0.04	0.07	0.06	0.06	0.07
Ceiling	1344	0.01	0.01	0.02	0.02	0.02	0.02
Windows	0	0.35	0.25	0.18	0.12	0.07	0.04
a = S*alpha		1309.7	395.5	323.96	228.62	412.02	398.86
TL of Floor		38	48	56	60	67	72
10 log(a/S)		6	1	0	0	1	1
NR = TL + 10 log(a/S)		44	49	56	60	68	73
Sound Pressure Level		78	84	89	86	80	72
L2 = L1 - NR		34	35	33	26	12	0
NC-25		44	37	33	27	25	23

Table 10: Noise Reduction Original Floor System

New Floor System: 3" Concrete Slab on Composite Metal Deck							
Source Room: Health Club							
Receiving Room: Residential Area							
	Surface Area:		Sound Absorption Coefficients				
Concrete Walls	490	0.1	0.05	0.06	0.07	0.09	0.08
Partition Walls	2170	0.55	0.14	0.08	0.04	0.12	0.11
Floor	1344	0.04	0.04	0.07	0.06	0.06	0.07
Ceiling	1344	0.01	0.01	0.02	0.02	0.02	0.02
Windows	0	0.35	0.25	0.18	0.12	0.07	0.04
a = S*alpha		1309.7	395.5	323.96	228.62	412.02	398.86
TL of Floor		48	42	45	56	57	66
10 log(a/S)		6	1	0	0	1	1
NR = TL + 10 log(a/S)		54	43	45	56	58	67
Sound Pressure Level		78	84	89	86	80	72
L2 = L1 - NR		24	41	44	30	22	5
NC-25		44	37	33	27	25	23

Table 11: Noise Reduction New Floor System

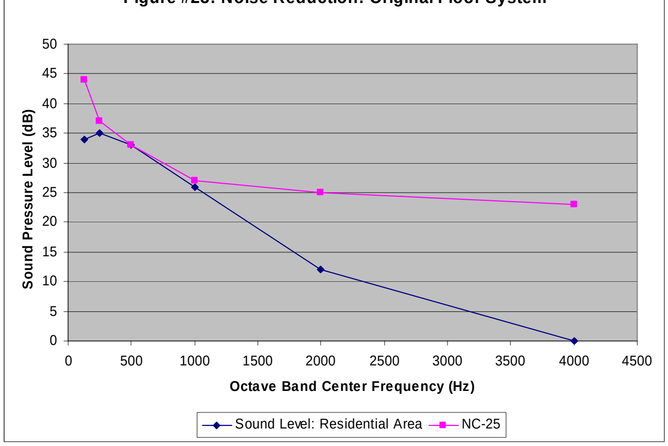


Figure #23: Noise Reduction: Original Floor System

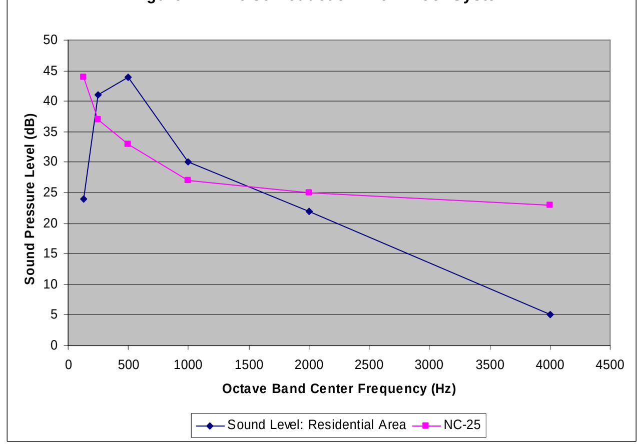


Figure #24: Noise Reduction: New Floor System

In order to improve the transmission loss of the floor system, several alternative systems were analyzed. First, suspended acoustical ceiling tiles were added to the floor system. The tiles improved the sound absorption of the room, but still did not result in acceptable sound pressure levels. Second, carpeting with a foam underlayment was added, but this system also did not have acceptable values. Next, the entire system was considered with both acoustical ceiling tiles and a carpeted floor. This system had values that were almost acceptable for NC-25 and were acceptable for NC-30, which is still in the preferred range for residential spaces. Tables 12-14 show the calculations for these floor systems, and Graphs 25-27 show the residential area sound pressure level versus the noise criteria curves.

New Floor System: Including Acoustical Ceiling Tiles

Source Room: Health Club
Receiving Room: Residential Area

	Surface Area:			Sound Absorption Coefficients			
Concrete Walls	490	0.1	0.05	0.06	0.07	0.09	0.08
Partition Walls	2170	0.55	0.14	0.08	0.04	0.12	0.11
Floor	1344	0.04	0.04	0.07	0.06	0.06	0.07
Ceiling	1344	0.76	0.93	0.83	0.99	0.99	0.94
Windows	0	0.35	0.25	0.18	0.12	0.07	0.04
a = S*alpha		2317.7	1631.98	1412.6	1532.3	1715.7	1635.34
TL of Floor		48	42	45	56	57	66
10 log(a/S)		8	7	6	6	7	7
NR = TL + 10 log(a/S)		56	49	51	62	64	73
Sound Pressure Level		78	84	89	86	80	72
L2 = L1 - NR		22	35	38	24	16	0
NC-25		44	37	33	27	25	23

Table 12: Noise Reduction Floor System with Ceiling Tiles

New Floor System: Including Sound Absorbing Floor Material

Source Room: Health Club
Receiving Room: Residential Area

	Surface Area:			Sound Absorption Coefficients			
Concrete Walls	490	0.1	0.05	0.06	0.07	0.09	0.08
Partition Walls	2170	0.55	0.14	0.08	0.04	0.12	0.11
Floor	1344	0.08	0.24	0.57	0.69	0.71	0.73
Ceiling	1344	0.01	0.01	0.02	0.02	0.02	0.02
Windows	0	0.35	0.25	0.18	0.12	0.07	0.04
a = S*alpha		1363.46	664.3	995.96	1075.34	1285.62	1285.9
TL of Floor		48	42	45	56	57	66
10 log(a/S)		6	3	5	5	6	6
NR = TL + 10 log(a/S)		54	45	50	61	63	72
Sound Pressure Level		78	84	89	86	80	72
L2 = L1 - NR		24	39	39	25	17	0
NC-25		44	37	33	27	25	23

Table 13: Noise Reduction Floor System with Sound Absorbing Floor

New Floor System: Including Sound Absorbing Floor and Ceiling Materials

Source Room: Health Club
Receiving Room: Residential Area

	Surface Area:	Sound Absorption Coefficients					
Concrete Walls	490	0.1	0.05	0.06	0.07	0.09	0.08
Partition Walls	2170	0.55	0.14	0.08	0.04	0.12	0.11
Floor	1344	0.08	0.24	0.57	0.69	0.71	0.73
Ceiling	1344	0.76	0.93	0.83	0.99	0.99	0.94
Windows	0	0.35	0.25	0.18	0.12	0.07	0.04
a = S*alpha		2371.46	1900.78	2084.6	2379.02	2589.3	2522.38
TL of Floor		48	42	45	56	57	66
10 log(a/S)		8	7	8	8	9	9
NR = TL + 10 log(a/S)		56	49	53	64	66	75
Sound Pressure Level		78	84	89	86	80	72
L2 = L1 - NR		22	35	36	22	14	0
NC-25		44	37	33	27	25	23

Table 14: Noise Reduction Floor with Sound Absorbing Floor and Ceiling

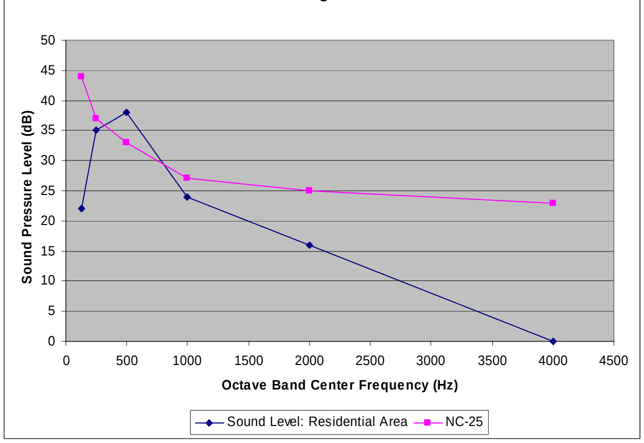


Figure 25: Noise Reduction: New Floor System with Acoustical Ceiling Tiles

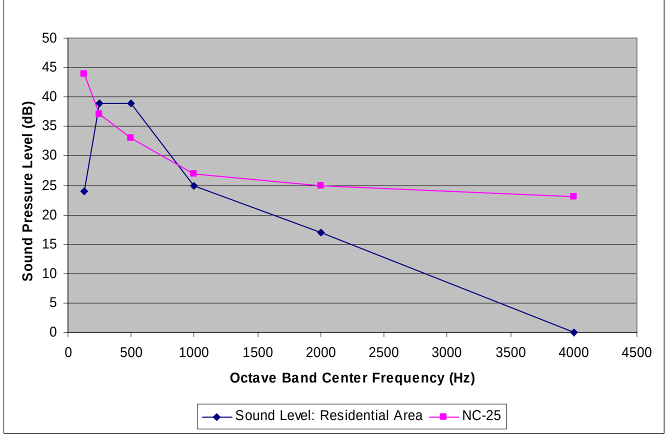


Figure 26: Noise Reduction: New Floor System with Sound Absorbing Floor Material

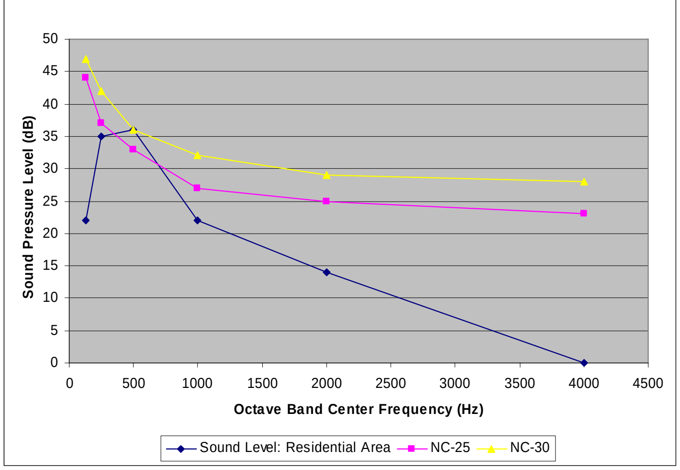


Figure 27: Noise Criteria: New Floor System with Sound Absorbing Floor and Ceiling Materials

Shear Wall Transmission Loss:

The transmission loss of the wall systems was analyzed in a location where there was a concrete shear wall in the original structural design. The area providing the source sound pressure level is a mechanical room, and the receiving room is a residential unit. The existing concrete wall was analyzed to determine if it provided adequate transmission loss. Next several alternative walls were analyzed, and the one with the best sound transmission loss was chosen to replace the existing wall.

This analysis uses the same procedure as the floor transmission loss calculations. The original shear wall is a 12 inch reinforced concrete wall and has an adequate transmission loss to produce sound pressure levels that are below the noise criteria curve, NC-25.

Original Wall System: 12" Reinforced Concrete Shear Wall

Source Room: Mechanical Room
Receiving Room: Residential Area

	Surface Area:		Sound Absorption Coefficients				
Concrete Walls	972	0.1	0.05	0.06	0.07	0.09	0.08
Floor	702	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling	702	0.01	0.01	0.02	0.02	0.02	0.02
a = S*alpha		118.26	76.68	93.42	103.14	122.58	105.84
TL of Wall		44	48	56	58	64	67
10 log(a/S)		0	0	0	0	0	0
NR = TL + 10 log(a/S)		44	48	56	58	64	67
Sound Pressure Level		86	85	84	83	82	80
L2 = L1 - NR		42	37	28	25	18	13
NC-25		44	37	33	27	25	23

Table 15: Noise Reduction Original Wall System

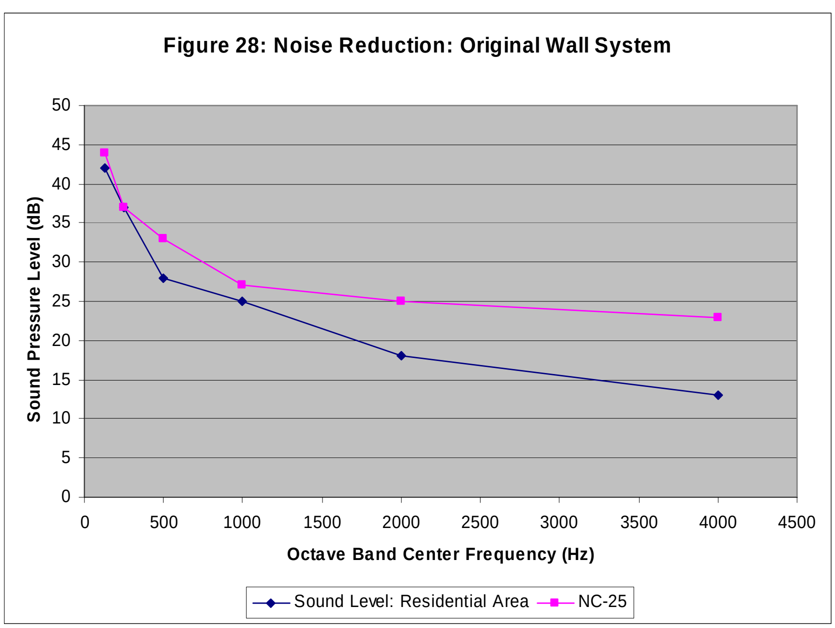


Figure 28: Noise Reduction: Original Wall System

Next, these calculations were performed on several wall assemblies until one was found that had acceptable transmission loss values. The walls that were considered were:

1. 2 ½” steel channel studs 24 in. o.c. with 5/8” gypsum board both sides, with 2” glass-fiber insulation in cavity
2. 2 ½” steel channel studs 24 in. o.c. with two layers 5/8” gypsum board one side, one layer other side, with 2” glass-fiber insulation in cavity
3. 3 5/8” steel channel studs 24 in. o.c. with two layers 5/8” gypsum board both sides, with 3” mineral-fiber insulation in cavity

The calculations can be found in Tables 16-18, and Figures 29-31 show the sound pressure levels versus NC-25. The calculations show that the first two alternative walls were inadequate, but the third wall produced acceptable values.

New Wall System: Typical Residential Wall Partition

Source Room: Mechanical Room
Receiving Room: Residential Area

	Surface Area:		Sound Absorption Coefficients				
Partition Walls	972	0.55	0.14	0.08	0.04	0.12	0.11
Floor	702	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling	702	0.76	0.93	0.83	0.99	0.99	0.94
a = S*alpha		1082.16	810	681.48	754.92	832.68	780.84
TL of Wall		26	41	52	54	45	51
10 log(a/S)		7	5	5	5	5	5
NR = TL + 10 log(a/S)		33	46	57	59	50	56
Sound Pressure Level		86	85	84	83	82	80
L2 = L1 - NR		53	39	27	24	32	24
NC-25		44	37	33	27	25	23

Table 16: Noise Reduction Wall Alternative 1

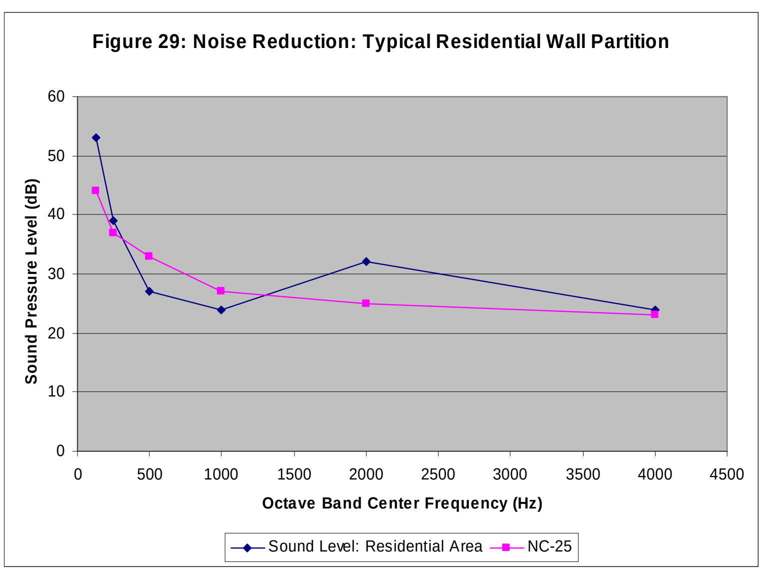


Figure 29: Noise Reduction: Typical Residential Wall Partition

New Wall System: Residential Partition Wall with Extra Layer Gypsum Board							
Source Room: Mechanical Room							
Receiving Room: Residential Area							
	Surface Area:		Sound Absorption Coefficients				
Partition Walls	972	0.55	0.14	0.08	0.04	0.12	0.11
Floor	702	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling	702	0.76	0.93	0.83	0.99	0.99	0.94
a = S*alpha		1082.16	810	681.48	754.92	832.68	780.84
TL of Wall		31	43	55	58	61	51
10 log(a/S)		7	5	5	5	5	5
NR = TL + 10 log(a/S)		38	48	60	63	66	56
Sound Pressure Level		86	85	84	83	82	80
L2 = L1 - NR		48	37	24	20	16	24
NC-25		44	37	33	27	25	23

Table 17: Noise Reduction Wall Alternative 2

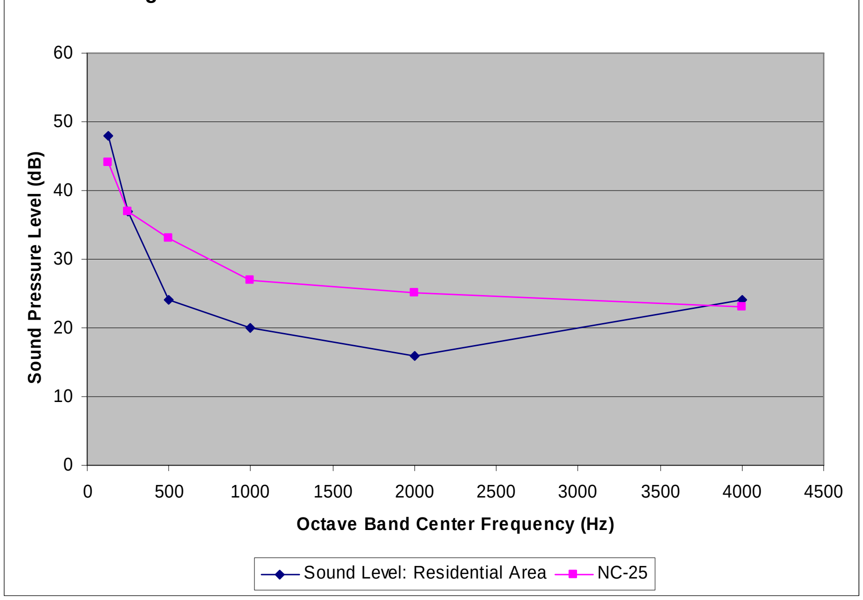


Figure 30: Noise Reduction: Residential Wall Alternative 2

New Wall System: Residential Partition Wall with Two Extra Layers Gypsum Board

Source Room: Mechanical Room
Receiving Room: Residential Area

	Surface Area:			Sound Absorption Coefficients			
Partition Walls	972	0.55	0.14	0.08	0.04	0.12	0.11
Floor	702	0.02	0.03	0.03	0.03	0.03	0.02
Ceiling	702	0.76	0.93	0.83	0.99	0.99	0.94
a = S*alpha		1082.16	810	681.48	754.92	832.68	780.84
TL of Wall		38	52	59	60	56	62
10 log(a/S)		7	5	5	5	5	5
NR = TL + 10 log(a/S)		45	57	64	65	61	67
Sound Pressure Level		86	85	84	83	82	80
L2 = L1 - NR		41	28	20	18	21	13
NC-25		44	37	33	27	25	23

Table 18: Noise Reduction Wall Alternative 3

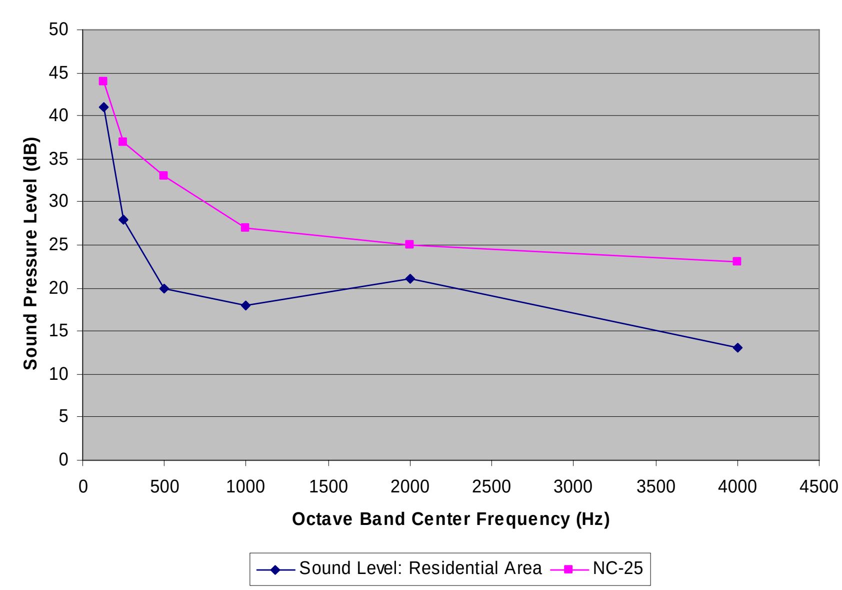


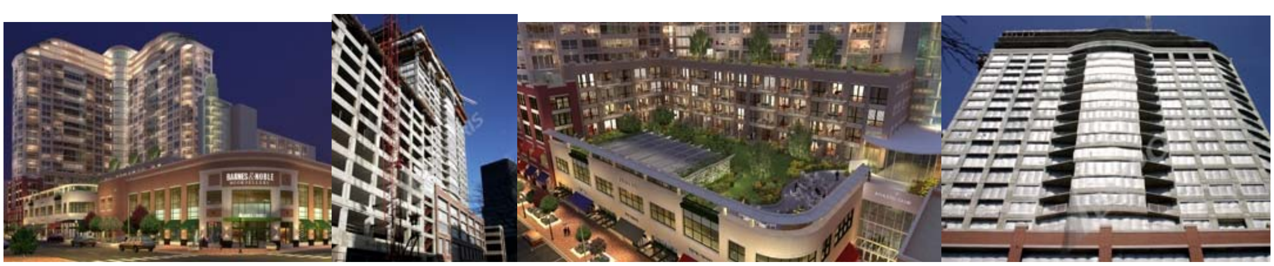
Figure 31: Noise Reduction: Residential Wall Alternative 3

Conclusion:

The first acoustical analysis was between the existing 8 inch concrete floor system and the new floor system of a 3 inch slab on metal deck. The existing floor system was found to have an adequate transmission loss. The new system, however, was not acceptable according to the NC-25 noise criteria curve. The first two alternatives were also not acceptable. The final alternative combined both acoustical ceiling tiles and a sound absorbing floor material. This system was close to being adequate for NC-25, but was below the values of NC-30, which is in the preferred range for a residential area.

The second analysis investigated the transmission loss of one of the concrete shear walls from the original structural system. The shear wall was found to have an acceptable transmission loss to reduce the mechanical room noise. Based on the acceptable sound pressure levels, a new wall system was chosen to replace this wall. Three alternatives were analyzed. The chosen wall system was made up of 3 5/8" steel channel studs with two layers 5/8" gypsum board on both sides and 3" mineral-fiber insulation in the cavity.

CONCLUSIONS



This analysis presented in this report aims to produce a new building design that is efficient in material usage, constructability, and time, while maintaining the quality of the original design. To accomplish this goal, the existing reinforced concrete system was replaced by a new steel gravity and lateral system.

Composite steel beams were chosen for the floor framing to limit the overall building height as much as possible. The beam sizes were also restricted to W16s on floors 2-7 and ranged between W8x10 and W16x31. On levels 8-25 the beams were limited to W14s and ranged in size from W8x10 to W14x22. Despite these limitations, the building height was still increased from 260.5 feet to 283.25 feet, which is an overall increase of 22.75 feet. The gravity column sizes varied between W10x33 and W14x193. The lateral system, however, required larger member sizes to provide adequate stiffness and lateral resistance. The beam sizes ranged between W16x89 and W14x82, and the columns were sized W14x257 to W14x370. The cross bracing was made up of 2 L8x8x3/4. This design resulted in a total building drift of 5.6254 inches, which is less than the allowable drift of H/600, or 5.665 inches.

With this new design, the building weight was greatly reduced. The foundations sizes were able to be decreased. The original foundation sizes varied between a 15’-6” bell diameter and 6’-0” shaft diameter caisson to a 6’-0” bell and 2’-6” shaft. The new design results in caisson sizes that range between a 3’-0” bell and a 7’-0” bell.

By changing the primary structural system of Sherman Plaza from reinforced concrete to structural steel, other building systems were also impacted. Two breadth studies were performed to determine the effect the structural material change had on the construction management and on building acoustics.

An estimate was performed of the costs of the exterior cladding and structural materials for each system from R.S. Means. The steel system resulted in a total cost of $17.45 million, and the reinforced concrete system had a total cost of $25.63 million. The steel system, therefore, was $8.18 million less expensive than the concrete system. R.S. Means was also used to perform a schedule estimate. The steel system took a total of 1146 days to complete, while the concrete system took 2660 days. Therefore, the steel system could be erected 1514 days faster than the concrete system.

The first acoustical analysis was between the existing 8 inch concrete floor system and the new floor system of a 3 inch slab on metal deck. The existing floor system was found to have an adequate transmission loss. The new system, however, was not acceptable according to the NC-25 noise criteria curve. Even with additional sound absorbing floor and ceiling materials, the new system did not meet NC-25 criteria, but did fall below NC-30 standards.

The second analysis investigated the transmission loss of one of the concrete shear walls from the original structural system. The shear wall was found to have an acceptable transmission loss to reduce the mechanical room noise. Based on the acceptable sound pressure levels, a new wall system was chosen to replace this wall. Three alternatives were analyzed. The chosen wall system was made up of 3 5/8" steel channel studs with two layers 5/8" gypsum board on both sides and 3" mineral-fiber insulation in the cavity.

In all, the steel structural system was an effective design for this building. The composite steel produced an efficient gravity system that worked well with the given column layout. The drawback to this system, however, was that the structural system's ceiling to floor section depth was greater than that of the existing concrete system. This increase in depth at each floor resulted in an increase of building height from 260.5 feet to 283.25 feet, which is an increase of 22.75 feet.

The newly designed lateral system also produced acceptable results. The design, however, uses a large number of braced frames and moment connections that will increase construction time and costs. The architectural constraints also made the placement of the frames within the building difficult. Since the building was designed to have shear walls provide the lateral resistance, the architecture of the building did not provide many options for braced frame locations. The braced frame system could possibly have been improved if other locations for the frames could have been tested.

The use of an all steel system, however, caused the building weight to be dramatically reduced. The foundation sizes in turn were able to be decreased. Due to the fact that the caissons extend down 70 feet, this size reduction will result in large savings in concrete and in construction time.

Despite any drawbacks of the new steel structural system, the cost estimate and comparison of the two systems showed that the steel system is less expensive by $8.18 million. This savings in cost could compensate for the increase in building height and the large number of lateral braced frames and moment connections. In addition, according to the schedule estimate, the steel system could be erected 1514 days faster than the concrete system. These facts make the new steel redesigned system a viable alternative to the existing reinforced concrete structural system.

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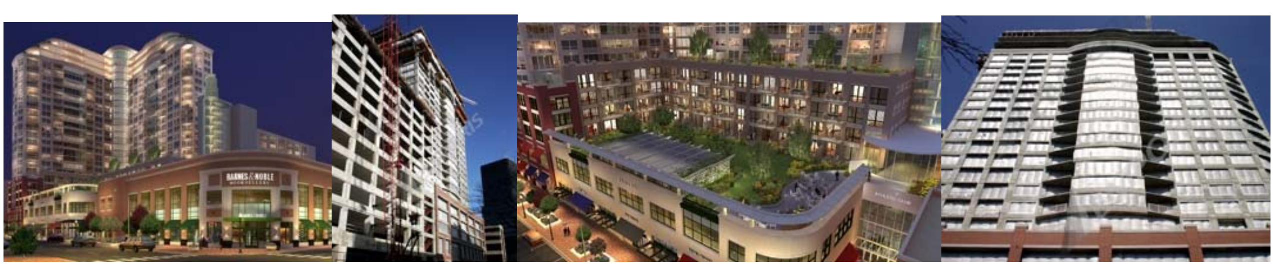

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