House of Sweden

Structural System and Existing Conditions Report

2900 K St. NW
Washington, DC 20007

The Pennsylvania State University
Department of Architectural Engineering
Senior Thesis 2008-2009

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HOUSE OF SWEDEN

Georgetown, Washington, D.C.

Kimberlee McKitish

Structures Option

Building Statistics

- Construted from August 4, 2004 to May 12, 2006
- Delivered in a Design - Bid - Build method
- **North Building:**
  $22.1 Million Overall Building Cost
  7 Building Levels Above Grade
  170,000 SF of Office and Residential Space
- **South Building:**
  $19.7 Million Overall Building Cost
  6 Building Levels Above Grade

Architecture

- Built on a single foundation with two separate towers rising out of the site
- The glass façade of the south building is backlit to create the illusion of a floating jewel rising above the Potomac River on a light colored stone podium
- The north building is clad in glass and metal paneling with a light stone base
- The roofing is rigid insulation topped with ballast over monolithic EDPM waterproofing membrane

Mechanical and Electrical Systems

- A central plant located in the penthouse of the north building runs the mechanical system for both buildings except in the embassy, which has its own ventilation system
- The electrical system is a 277/480 V, 3 phase, 4 wire system for public space lighting and steps down to 120/208 V for receptacles and incandescent lighting

Project Team

- **Owner:** LANO Armada Harborside, LLC
- **General Contractor:** Armada Hoffler
- **Tenant-South Building:** SFV National Property Board
- **Architect of Record:** VOA Associates, Inc.
- **Architect-South Building:** Wingardh Arkitektkontor AB
- **Structural Engineer:** Tadjer - Cohen - Edelson
- **MEP Engineer:** Tolk, Inc.
- **Civil Engineer:** Wiles Mensch Corp

Structural System

- Post-tensioned, two-way concrete slab system with drop panels and piles supporting a mat foundation
- North building typical bay sizing is 30' x 30', slab thickness is 7"-8", and concrete strength is 6 or 8 ksi
- South building typical bay sizing is 32' x 22', slab thickness is 10"-12", and concrete strength is 6 or 8 ksi
- North building lateral system is shear walls to the fourth floor then concrete moment frame, north building is all concrete moment frame

Special Systems

- Due to the sensitive nature of the building, intrusion detection was a necessary part of the design
- Interior protected areas were outfitted with redundant state-of-the-art intruder detection systems
- Also inclulded is surge protection and tamper protection on system components

http://www.engr.psu.edu/ae/thesis/portfolios/2009/kam5001

TABLE OF CONTENTS

EXECUTIVE SUMMARY

The House of Sweden is a mixed-use building house commercial, residential, and assembly space as well as the second embassy for Sweden. It is a signature building with distinctive architecture, cladding, and lighting located in Georgetown, Washington, D.C. The focus of this report was the north building, a seven story building with a post-tensioned flat slab concrete moment frame with a below-grade parking level.

The primary goal of this report is to design a steel structural solution for the building while decreasing the cost and schedule. This solution needs to take into account the height restriction along the Potomac River in Georgetown as well as the distinctive architecture of this signature building.

In a previous report, it was determined that composite steel beams might be a possible solution to the unique architecture of this building. Through extensive research and preliminary designs, it was decided that castellated beams would help minimize the floor depth to help keep an acceptable floor-to-ceiling height for this building. Also, four different structural combinations were initially considered. Since seismic loads were fairly high for this building, light-weight concrete was compared to normal weight concrete to see if this would have a significant impact on the loads while not increasing the cost significantly. Also, moment frames were compared to braced frames, since there were limited locations that the frames could be placed so as not to impact the architecture. After evaluation, it was decided that the normal weight concrete braced frames would be an acceptable solution for this building.

A breadth study was conducted into the feasibility of moving the mechanical equipment to the parking level to free up the penthouse space for apartments. Since this space is at the top of the building and has a view of the river, it is the most valuable real estate in that building. A parking study was conducted and determined that the chillers and cooling towers could be moved to provide room for three new apartments. The waterproofing of this space was looked at, along with the acoustical impact of the AHU left in the penthouse on the apartments that share the wall.

A second breadth study was conducted to look at the feasibility of these redesigns on the cost and schedule of the project. This evaluation took into account vendor, general contractor, subcontractor and RSMeans input. It was determined that these redesigns were feasible, would not impact the schedule in too negative a way, and would save the owner approximately 11% of the original budget.

Acknowledgements

The author of this report wishes to recognize and sincerely thank the following individuals for their patience, understanding, and guidance to aid in the completion of this thesis study. Without their help, the completion of this report would not have been possible.

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INTRODUCTION

This Final Report contains a detailed summary of the structural redesign of the House of Sweden. It includes background information on the building along with a detailed description of the structural system. The problem is stated and a synopsis of the solution steps is outlined. This report also discusses the applicable design codes and the design practices used for analysis of the House of Sweden's structural serviceability and strength. It then addresses some of the impacts of the structural redesign.

BACKGROUND

House of Sweden (Cover Figure) is located in Georgetown, Washington D.C. at the intersection of Rock Creek and the Potomac River. This development is built on a single mat foundation with a parking garage level and has two separate towers that rise out of the site above the garage. The south building consists of five stories and a mechanical penthouse; the north building is six stories and a mechanical penthouse. Construction of the two buildings began on August 4, 2004 and finished on May 12, 2006. It was delivered in a design-bid-build method and the design of the south building was commissioned as a competition in Sweden.

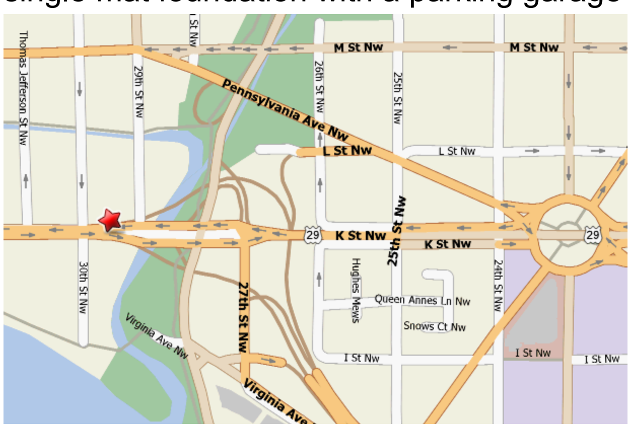


Figure 1: Site Location of the House of Sweden

BUILDING OVERVIEW

Architecture

House of Sweden inhabits one of the most perfect sites in Washington, D.C. Located at the intersection of Rock Creek and the Potomac River in scenic Georgetown, both buildings possess breathtaking views of the river, the Kennedy Center, and Watergate. Built on a single foundation, two separate towers rise out of the site, while sharing a below-grade parking garage.

The south building was designed by Wingardh Arkitektkontor AB and houses the Swedish Embassy along with an exhibit hall, convention center, rooftop terrace, and apartments. The architects designed this building to be “a shimmering jewel in the surrounding parkland.” To accomplish this goal, the base of the building was clad in light stone, while the upper floors were clad in glass laminated with a traditional Nordic blond wood pattern. This glass façade is backlit at night to create the illusion of the structure floating above the river. The south building has received Sweden's most prestigious architecture award; the Kasper Salin Prize for best building.

The north building houses offices and apartments, and incorporates expansive balconies and long stretches of ribbon windows to maximize exterior views. The façade employs the same type of light stone on the podium, but the upper floors are clad in metal panels. This allows the north building relate to the south building, yet keep its own identity. Photographs have been provided in Appendix A.

Building Envelope

Both building envelopes are steel stud construction with faced blanket insulation and gypsum wallboard attached. The north building uses a standoff system to attach light stone panels to the podium of the building and metal paneling to the upper floors. The south building uses the same standoff system and light stone paneling on the lower level. The upper levels employ a different standoff system of laminated glass panels as cladding. None of these cladding systems are used as a barrier system, which is why the insulation is faced to prevent moisture penetration. The roofing on the north building is rigid insulation topped with ballast over monolithic EDPM waterproofing membrane. The south building uses the rigid insulation and ballast over monolithic EDPM waterproofing membrane around the perimeter and a concrete topping slab over the same monolithic EDPM waterproofing membrane for the roof terrace.

Mechanical System

The House of Sweden's mechanical system has a central plant on the penthouse level of the north building that contains water chillers and boilers. These units provide conditioning for all the air handling units in both buildings. The north building has two 100% outdoor AHUs and three AHUs. These are connected to variable air boxes so that each residential unit and the various commercial spaces can condition their spaces as they see fit. The south building has two 100% outdoor AHUs that connect to variable air boxes and provide air to the residential units and corridors. The embassy has its own AHU and mechanical room. The parking garage has three fan coils units to exhaust gases from the underground parking level.

Electrical System

The electrical power for the House of Sweden is supplied by PEPCO. The power supply enters on the 30th street side of the north building in two places through a transformer vault. The lines run through 2500A buses before being distributed to main panelboards. The main switchboard room is located at the level below the main lobby. It contains panelboards for both 120/208V and 277/480V feeds from the transformers. There are electrical rooms located on every floor of both buildings. Backup power is supplied by a standby generator and plans for a future generator exist.

Lighting System

To respond to the architect's desire to have the buildings look like sparkling jewels floating above the landscape, the most unique lighting feature of the buildings is the backlit curtain wall on the south building. It is lit with what is considered recessed step lights; wall washers that present a soft indirect lighting effect to viewers. The corridors utilize cost effective 2'x2' recessed fluorescent light fixtures. The north building lobby uses ceiling mounted 6" recessed downlights and the south building uses the same 2'x2' recessed fluorescent light fixtures in the corridors, except that they are covered by hole-punched panels. All the lights in these public spaces are run on 277V so as to be energy efficient. The apartment and office areas have been outfitted to suit the tenants, and therefore, are not covered in this overview of the system.

Telecommunications System

This building is a high tech office and apartment space. Not only is the building provided with phone service, it has excellent in-house cellular coverage throughout the entire two buildings. The apartment spaces can also choose from a wide range of technology services including cable TV and high-speed internet access via a broadband cable network. Wi-Fi is also available throughout the apartment units and the commercial reception and conferencing spaces. Since the developer wanted to cater to business professionals, they also decided to offer a VoIP phone service. This service allows tenants to not only place a call with a land phone, they can also use a computer headset and microphone and all calls are communicated over a high-speed internet network. This improves clarity of a call and offers many services such as conference calling and voicemail that a professional will use every day.

Special Systems

Due to the sensitive nature of this building, intrusion detection was a necessary part of the design. This protection includes, but is not limited to, intruder detection in interior protected areas through various means and intruder detection through the building envelope. It also covers surge protection to equipment, card key access to secure areas, and tamper protection on switches, controllers, annunciators, pull boxes, and other system components.

STRUCTURAL SYSTEM OVERVIEW

Foundation

Cast-in-place piles support a mat foundation. These piles are 16” in diameter with a concrete compressive strength of f'_c = 6,000 psi and exist under the north perimeter of the parking garage. The mat foundation exists over the entire parking garage. It is a minimum of 38” thick, and 42” at the columns with a concrete compressive strength of f'_c = 4,000 psi and rests on a 2” thick mud slab. It is reinforced with rebar varying from #18 bars to #6 bars and at a variety of spacings. This foundation is either set on the piles at the north perimeter, or held with tie-downs. Columns from both the north and south buildings are supported on the mat foundation.

Framing System

House of Sweden is located in Georgetown, Washington, DC; therefore, the use of a post-tensioned concrete structural system was an obvious choice to help minimize the slab thickness and maximize the number of floors. Most of the floors above grade are two-way post-tensioned concrete flat slabs.

The north building has seven levels above grade. The first floor slab is 9”-10.5” thick reinforced with #4 and #5 bars and the drop panels are 5”, 8”, or 10” thick and reinforced with #7 and #8 bars. The second through seventh floors are 7”-8” thick with drop panels reinforced with #5 and #6 bars. Typical concrete strength on these floors is 6 ksi or 8 ksi. Concrete strength and slab thickness vary on each floor, which means that the slabs were not placed as single, monolithic pours and they had to be completed in sections. Because of the irregular building shape, there is no typical bay spacing, although many bays were kept at 30’ x 30’, possibly accounting for the change in slab strength and thickness.

The south building has five levels above grade. The first floor slab is a 9”-12” thick reinforced with #4-#6 bars and the drop panels are 8”, 10”, or 12” thick and reinforced with #6- #9 bars. The second through fifth floors are 10”-12” thick with drop panels reinforced with #5 and #6 bars. Typical concrete strength is 6 ksi or 8 ksi. Concrete strength and slab thickness vary on each floor, which means that the slabs were not placed as single, monolithic pours, and they had to be completed in sections. Because of the irregular building shape, there is no typical bay spacing, although many bays were kept at 32’ x 22’, possibly accounting for the change in slab strength and thickness.

The penthouse roof of the north building is similar to the floor slabs. It is a two-way, post-tensioned slab, 7" thick with a concrete strength of 6 ksi. It has drop panels reinforced with #4 and #5 bars. This roof was designed to hold a 30 psf snow load, plus snow drift load around the mechanical equipment.

The main roof of the south building is similar to the floor slabs. It is a two-way, post-tensioned slab, 10" or 12" thick with a concrete strength varying from 6 ksi to 8 ksi. The drop panels are reinforced with #5 and #6 bars. This roof was designed to hold a 30 psf snow load plus snow drift load around the mechanical equipment and the penthouse to the north. Since the south half of the roof includes a convention space, it was designed to hold a 100 psf terrace load plus a 25 psf paver load.

Lateral System

Slab-column concrete moment frames make up the lateral system of the north building. This system resists lateral loads in the north-south and east-west direction depending upon the orientation of the frame. Shear walls exist in the north building extending from the first floor to below the fifth floor slab. These walls were added to help combat the extra lateral forces induced in the slabs due to the presence of numerous sloped columns in this building. These walls vary in width and are 8 " or 12" thick with concrete strength of 6 ksi reinforced with #4 bars at 12" spacing in two curtains. They were not added to be part of the lateral system to resist wind or earthquake loading, however, by their very nature, they have become part of this system.

The north building has a slab-column concrete moment frame to resist lateral loads in both the north-south and east-west directions. No shear walls were necessary in this building because of the lack of sloped columns and the fact that this is a low-rise building and shear walls are not normally present in this type of building in the Washington, DC area.

Lateral loads imposed on the buildings are distributed through the following load path and the loads are distributed by relative stiffness which will be discussed later:

1. Exterior glass curtain wall
2. Perimeter slab
3. Concrete moment frames (and shear walls in the south building)
4. Mat slab foundation

Refer to Figure 2. for a layout of the columns and shear walls that contribute to the lateral load resisting system in the north building. Refer to Figure 3. on the next page for a layout of the columns that contribute to the lateral load resisting system in the south building.

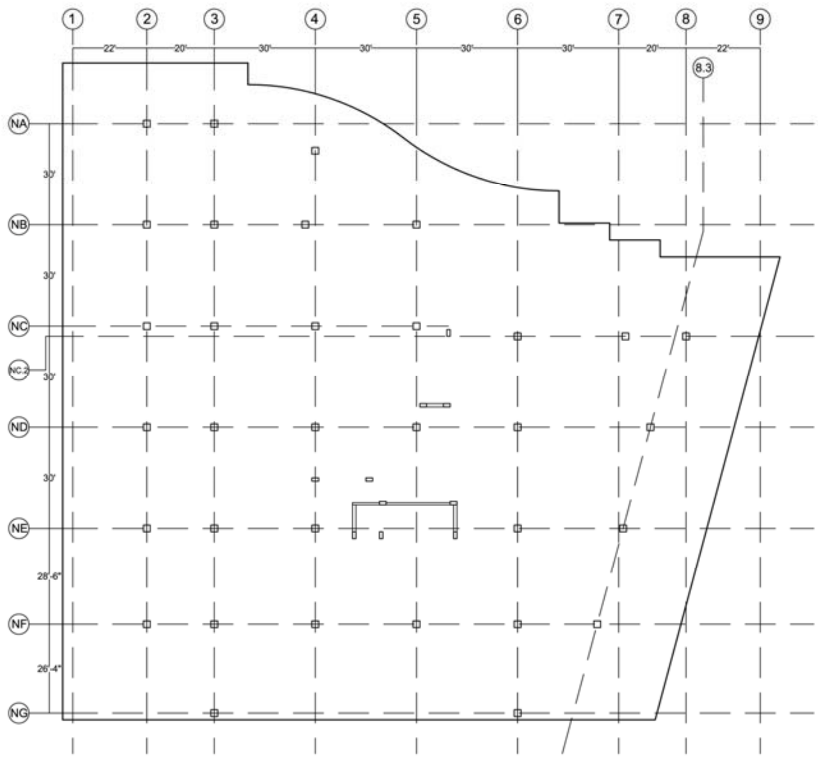


Figure 2: Typical North Building Column and Shear Wall Layout

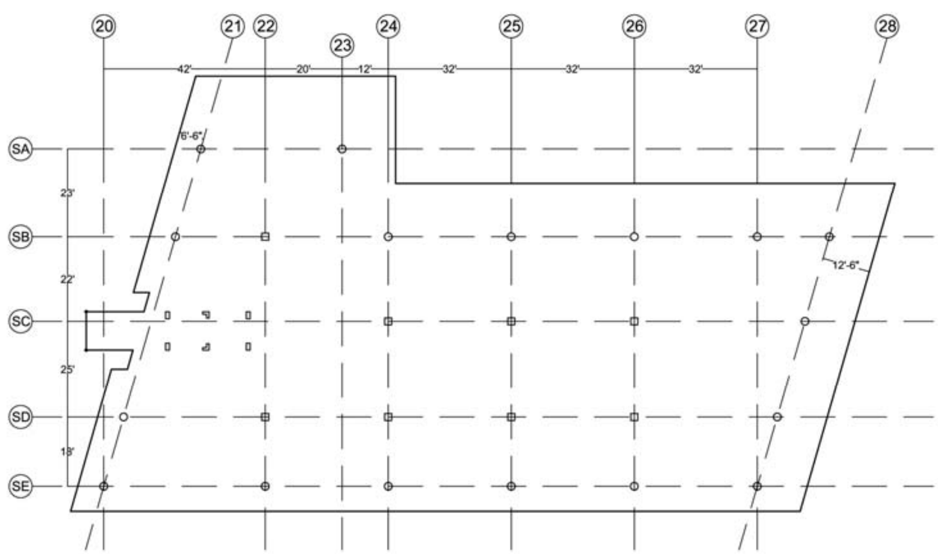


Figure 3: Typical South Building Column Layout

DEPTH STUDY – STRUCTURAL SYSTEM REDESIGN

Proposal

Problem Statement:

In its current design, the House of Sweden is a post-tensioned concrete multi-use facility. The post-tensioned design was a solution to the restricted building height in the Washington, D.C. Metro area. However, during Technical Report II, A Structural Study of Alternative Floor Systems, it was found that a composite deck with composite beam system might prove to be a viable alternative for the building. This system has comparable slab depth and overall cost with the original, and is more easily constructed than the post-tensioned concrete due to the elimination of formwork and curing and stressing time. Steel, as a solution, would also cut down on the floor weight by approximately half which leads to a reduction in seismic base shear and may possibly cause wind to control the design of the lateral system.

Another point of interest is the location of the mechanical room in the north building. The entire penthouse of this building is utilized as the mechanical space. It is noted in the background section of this report that the House of Sweden is located at the intersection of Rock Creek and the Potomac River in Georgetown, Washington, D.C. and the penthouse is the prime real estate in this particular building. An alternative area for the mechanical equipment will be proposed while attempting to keep the architectural layout of the rentable space in mind.

Proposed Solution:

As stated above, a proposed solution to the constructability of the design will be to re-design the north building in steel. This building is the tower with a twenty-two foot cantilever, so an economical solution to this will need to be considered during the re-design process. The gravity system will look at the use of castellated beams and lightweight or normal weight concrete with moment frames or braced frames for the lateral system. The most economical combination will be used. When this occurs, it is found that the floor-to-floor height that results is sufficient for the architectural requirements. A parking study will still be conducted for the ground floor parking garage to see if space can be created on that floor to house the mechanical system. If it cannot, a sub-basement for the mechanical equipment will be created. Then, the extra space that is created by this move will be analyzed as an extra apartment floor.

Implications of Redesign:

The weight of the building will most likely decrease and the wind load cases may control the design of the lateral system. The impact on the foundations will need to be considered, along with blast protection and progressive collapse mitigation because of the embassy security. It is possible that the mechanical system might be optimized now that the main mechanical room will be centered under the two towers as opposed to currently being housed at the top of the north tower. Scheduling and cost impacts should also be considered.

Gravity Loads

The following is a summary of the design gravity loads and criteria used to design and spot check the North Building gravity system. For more detailed calculations, please refer to Appendix B.

Deflection Criteria:

Floor Deflection – IBC 2006 Table 1604.3

Typical Live Load Deflection L/360

Typical Total Deflection L/240

Floor Dead Loads		
	Design Load	Reference
Normal Weight Concrete	150 pcf	ACI 318-08
Roof Pavers	25 psf	Structural Drawings
Ballast, Insulation, and waterproofing	8 psf	AISC 13th Edition
Glass Curtain Wall	6.4 psf	Glass Association of North America
Studs and Batt Insulation	4 psf	AISC 13th Edition
Superimposed MEP	12 psf	

Roof Live Loads		
	Design Load	Reference
Public Terrace	100 psf	ASCE7-05
Snow Load	30 psf	ASCE7-05

Floor Live Loads		
Occupancy	Design Load	Reference
Penthouse Machine Room	150 psf	Structural Drawings
Residential	80 psf + 20 psf for partitions	Structural Drawings
Stairways	100 psf	ASCE7-05
Corridors	100 psf	ASCE7-05
Commercial and Plaza Area	100 psf	Structural Drawings

Lateral Loads

Four different lateral systems were analyzed for this thesis report. The wind loads are based on the building geometry and since this geometry did not change from one alternative to another, the wind loads do not change and are summarized below. Seismic loads are based on the lateral system choice and the weight of the building; therefore, the seismic loads were different for each alternative although an R = 3 was used for each system so that seismically detailed connections were not necessary. Summarized below are example seismic loads used for the normal weight concrete braced frames. For more detailed calculations on both types of loads, as well as the seismic loads for the other alternatives, please refer to Appendix C.

Deflection Criteria:

Lateral Deflection

Allowable building deflection	H/400 – 1968 Structural Handbook
Wind allowable inter-story drift	h/400 to h/600 – ASCE 7-05 (Section CC.1.2)
Seismic allowable story drift	0.020h – ASCE 7-05 (Table 12.12-1)

Wind Loads:

Design wind load was calculated using ASCE 7-05 §6.5 Method 2 analysis. Method 2 does not take into account interference afforded by other buildings to reduce the wind velocity. For the purposes of this report, the House of Sweden will be considered a regular-shaped building. However, for later design purposes, a wind tunnel analysis of both buildings and their interactions with each other is recommended. Presented below is a summary of the wind load findings and story pressures. Figures 4. and 5.

illustrate the distribution of wind pressure on the building façades. For more detailed calculations, please refer to the Appendix C.

Factor (Both Buildings)	Design Value	Reference
K_{zt}	1	§6.5.7
K_d	0.85	Table 6-4
Exposure Category	B	§6.5.6
V	90	Figure 6-1
I	1	Table 6-1

North Building

Number of Floors: 7
Height: 77’
N-S Building Length: 192’
E-W Building Length: 206’
η_1: 0.97 (Flexible)

North Building N-S							
Story	Height (ft)	Force (K)		Shear (K)		Moment (ft-K)	
		N-S	E-W	N-S	E-W	N-S	E-W
PH	77'-0"	14	14	0	0	1071	1075
MR	59'-0"	31	34	14	14	1805	1996
6	48'-2"	30	33	44	48	1442	1613
5	37'-4"	29	35	74	81	1069	1293
4	26'-6"	81	97	103	116	2143	2579
3	15'-8"	75	90	184	213	1178	1404
2	4'-10"	18	22	259	303	85	107
1	-6'-0"	0	0	277	325	0	0
Total				**V = 277**	**V = 325**	**ΣM = 8792**	**ΣM = 10069**

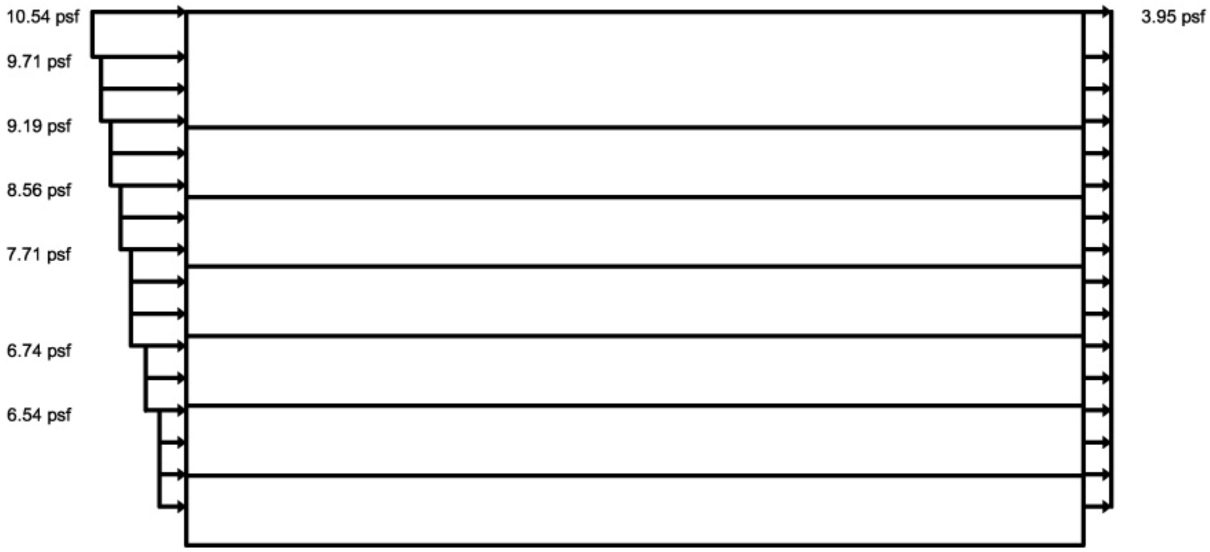


Figure 4: North Building Wind Pressure Diagram in the North – South Direction

Figure 5: North Building wind Pressure Diagram in the East – West Direction

North Building Wind Load Summary

N-S Direction Base Shear: V = 277 K
N-S Direction Moment: ΣM = 8,792 ft-K
E-W Direction Base Shear: V = 325 K (Controls)
E-W Direction Moment: ΣM = 10,069 ft-K (Controls)

Seismic Load:

Design seismic loads were calculated using ASCE 7-05 chapter 12. The Equivalent Lateral Force Procedure was determined as the procedure to use. Below is a summary of the base shear and moment for the NWC braced frame. Figure 6. illustrates the distribution of seismic forces and shears on the building façades. For more detailed calculations and for the seismic forces for the other types of frames, please refer to the Appendix C.

Vertical Distribution of Seismic Forces (NWC Braced Frame)					
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
P	83'-0"	1524	64	64	5308
MR	65'-0"	1604	47	111	3069
6	54'-2"	1972	45	156	2414
5	43'-4"	1968	32	188	1394
4	32'-6"	1769	19	207	619
3	21'-8"	1098	7	214	142
2	10'-10"	1076	2	216	26
$\Sigma w_i h_i^k$ =	3,119,645	**$\Sigma F_x = V =$**	216 K	**ΣM =**	12,972 ft-k

Figure 6: NWC Braced Frame Building Seismic Force Diagram

North Building Seismic Load Summary:

Base Shear: V = 216 K

Moment: ΣM = 12,972 ft-K

Wind loads control the lateral design for the north building. When the 1.6 factor is applied to the wind load, it is greater than the magnitude of the seismic load with the applied 1.0 factor. Therefore, the wind load governs and the lateral system spot checks will be performed with the wind loads only since this is the governing case. The results are summarized below.

Conclusion:

Wind loads (control):

Shear = 1.6*325 = 520 K
Moment = 1.6*10,069 = 16,110 ft-K

V_{wind} = 520 K > $V_{seismic}$ = 216 K

Seismic Loads:

Shear = 1.0*216 = 216 IK
Moment = 1.0*12,972 = 12,972 ft-K

M_{wind} = 16,100 K > $M_{seismic}$ = 12,972 K

Load Combinations

The following load combinations should be considered for combining factored loads for gravity and lateral load analysis. In gravity analysis, load case 2 normally governs. In lateral and gravity load analysis, load case 4 or 5 may govern depending on the magnitude of the lateral load.

1. 1.4(D+F)
2. 1.2(D+F+T) + 1.6(L+H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(L_r or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Design Goals

To determine if the changes investigated in this thesis should be recommended, a set of design criteria was formulated. The following criteria were decided upon to test the adequacy of the investigated changes and will be used to decide upon recommendations at the end of this report.

- Provide a steel structural solution that reduces the overall cost of the building.
- Provide a steel structural solution that does not interfere with the current architectural design due to the fact that the House of Sweden is a signature building for the Swedish Embassy.
- Reduce the structural erection schedule to complete the building faster than the original concrete design.
- Design for progressive collapse mitigation in the structural steel solution.
- Generate more revenue for the owner with the gain of an extra floor by moving the mechanical system.

Design Criteria

The girders, braces, and columns were all designed using the AISC steel manual and the LRFD method. The castellated beams were designed using programs and information from the CMC website. Both lateral systems were designed using seismic and wind loads. Due to the location of the building and the decrease in the weight of the building, wind loads governed the design of both lateral systems. However, many special provisions from both seismic design and wind design were taken into account. The following is a list of special provisions used in the design of the lateral systems of the structure:

- ASCE 7-05 (Figure 6-9) All the design wind loads cases were taken into account for the design of the structure. Please see Appendix C for a description of these load cases.
- ASCE 7-05 (Table 12.2-1) None of the frames were seismically detailed to cut down on cost, so an R=3 was used for design. As shown in the Lateral Load section of this report, even with R=3, wind still controls the design.
- ASCE 7-05 (12.8.2) In the seismic load calculations, originally C_uT_A was used, but then it was compared to the actual periods of the building and the loads were updated if necessary.
- ASCE 7-05 (Table 12.3-1,2) Structural Irregularities – There are no horizontal irregularities. Soft stories occur at the fifth floor of the moment frame systems but not in the braced frame systems, however, because the SDC=B, this does not affect the design of the structure. These calculations are not included in this report but are available upon request.
- ASCE 7-05 (12.3.4.2) There are only two braced frames in each direction so if there is a loss of a frame in either direction, there will be a loss of at least 50% of the stiffness, however, because of the SDC=B, the structure can still be designed with a ρ=1.0.
- ASCE 7-05 (12.7.3) Panel zone deformations and P-Delta effects were included in the model.

Evolution of Design

One of the first things considered at the start of the design process was the location of the lateral systems. The moment frame locations were easy decisions because they do not affect the placement of openings or the look of the façade. The biggest issue with the moment frames was torsion issues. An attempt was made to keep the center of rigidity close to the center of mass. This was done by placing the moment frames in as close to a square configuration as possible while trying to follow the geometry of the building. Also, the moment frames could only exist in structural frames that extended to the foundation of the building. The locations of the moment frames are denoted in the figure to the right. The frames in red are the locations of the moment frames. The frames in purple are possible locations for moment frames that were not used.

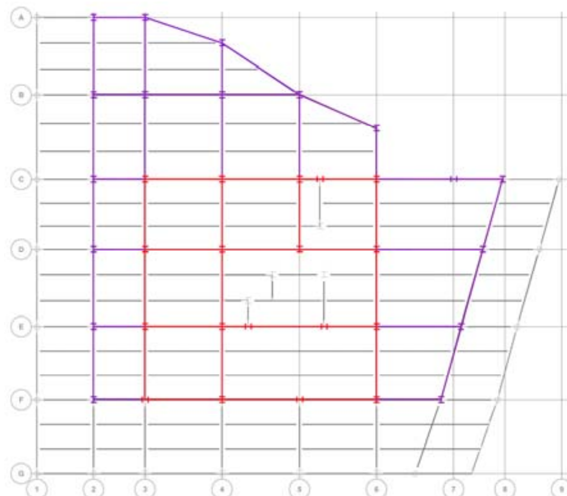


Figure 7: Location of Moment Frames

Architectural floor plans were studied carefully so that braced frames locations would not interfere with door openings or the exterior façade. This left very few positions for the frames. These locations are denoted by the figure to the left. The frames in red are the locations of the braced frames. The frames in purple are possible locations of braced frames that were not used. The final locations were chosen because they have minimal architectural impact. Although this layout causes more torsion than other layouts might, the torsion effects are still limited and again, these locations were architecturally driven. The only place these frames affect the layout of the floors is in the parking garage. Two parking spaces were eliminated due to one of the braced frames, however, a parking study was conducted and the lost spaces were made up in other parts of the garage. For more information on the parking study, please refer to the Breadth Study 1 section of this report.

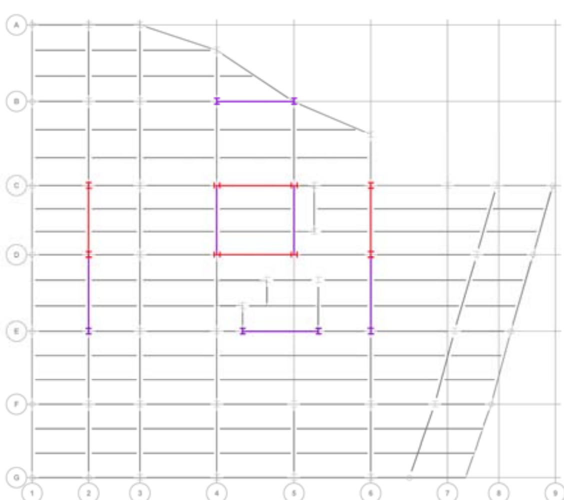


Figure 8: Location of Braced Frames

Another major design consideration was the use of castellated beams. There were a few factors in the evaluation of wide-flange beams or castellated beams. These factors were:

- Floor Depth
- Cost
- Constructability

The driving factor in looking at castellated beams in the first place was the small floor-to-floor height available for this design. The original slab depth was 14" overall with a floor depth of approximately 20". Very basic composite wide-flange designs came up with a slab and beam depth of 40" in some areas resulting in an overall floor depth of 52". Basic castellated beam designs came up with a slab and beam depth of 30" but the holes in the beams are large enough to allow the mechanical, electrical, and telecommunications systems to pass through so the overall floor depth is 30". This value was adjusted during the actual gravity and lateral design, but the floor depth did not increase significantly. The wide-flange calculations were done with the steel manual and can be found in Appendix D. The castellated beam designs were completed with a spreadsheet from CMC Steel and a sample of these calculations are in Appendix E. The spreadsheet can be found at http://www.cmcsteelproducts.com/design_progs.html.

The cost of a castellated beam is a function of the span. Larger spans are more economically constructed as castellated beams than wide-flange beams. The typical 30' spans in the House of Sweden are on the low side for castellated beams, so they are a bit more expensive than a wide-flange beam for the same span, but again, the floor depth savings was overriding.

Castellated beams are easily constructed. Pieces can be connected on the ground as with wide-flange beams and then lifted into place easily. The construction factor that could pose a problem is the connections. If the connections occur at a hole, special provisions need to be made. This will be looked at later in the report to try to alleviate any problems with the connections so that construction will not be an issue.

Computer Analysis

The RAM Modeling Process – MAE Requirement:

After all the loads were calculated and the design criterion was set, a RAM model was constructed to design the gravity and lateral structural systems. This model was created in RAM Structural System. The following modeling assumptions were taken into account:

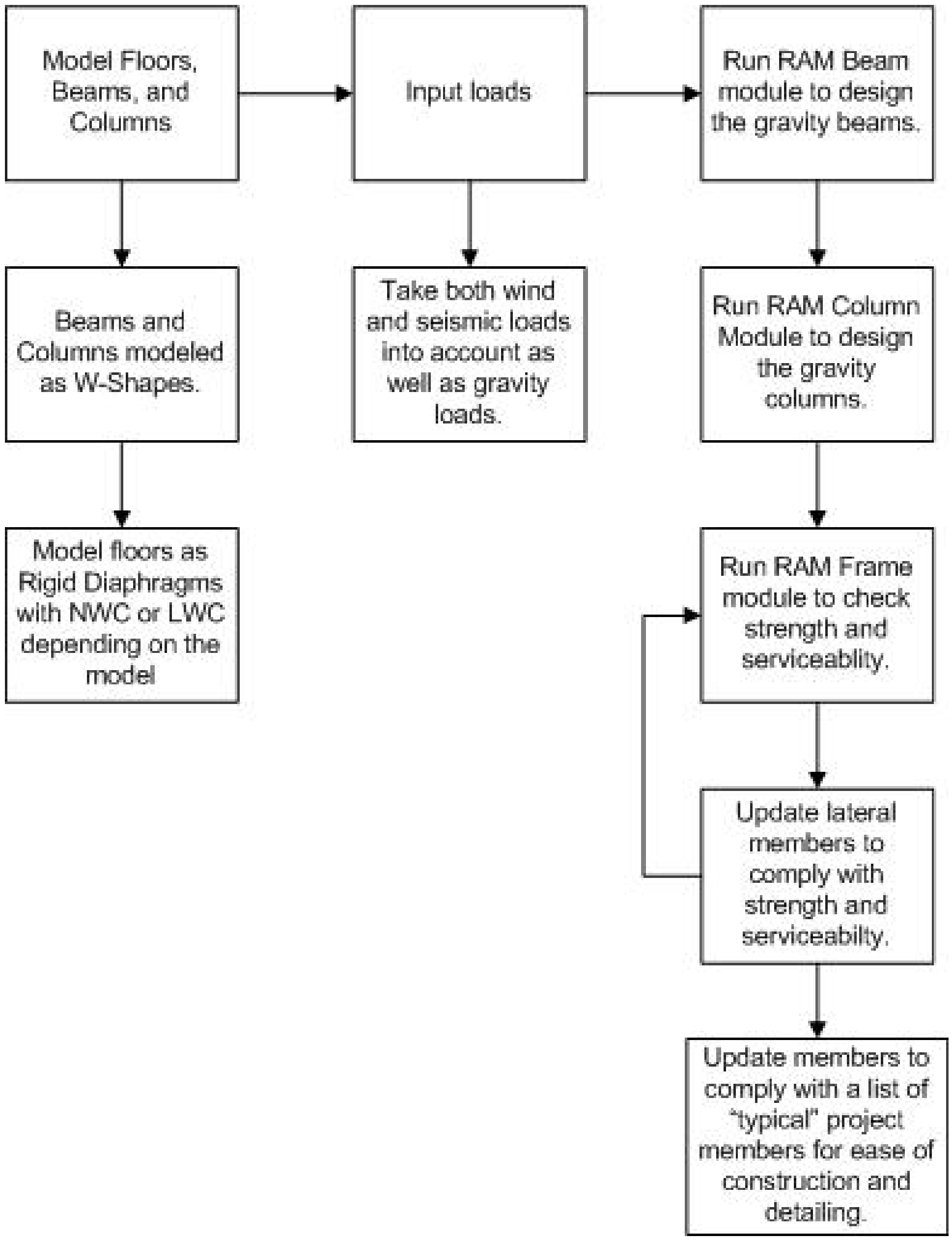
- Both the gravity and lateral resisting systems were modeled.
- Four different models were created, normal weight concrete moment frames, normal weight concrete braced frames, lightweight concrete moment frames, and lightweight concrete braced frames.
- The beams were modeled as wide-flange members because RAM will not allow lateral loads to be collected by castellated beams. Equivalent castellated beams were then chosen based on moment of inertia and shear area. Then, using the "other" material property, the castellated beam properties were modeled to reflect the change. The list of equivalent beams is listed below.

Wide-Flange	Equivalent Castellated Beam
W12x14	CB12x15
W14x22	CB15x19
W16x26	CB18x22
W21x48	CB27x46
W24x76	CB27x60
W27x84	CB27x76
W30x90	CB27x97
W30x108	CB27x119
W40x167	CB36x162
W40x324	CB50x201
W40x372	CB50x221

- A rigid diaphragm was assumed on each level designed as concrete on metal deck. A pseudo – rigid diaphragm was assumed at the first floor level because the material is reinforced concrete and a shear reversal will probably occur moving from the first floor to the basement floor below ground.
- Both inherent and accidental torsion effects were taken into account.

- Seismic forces were applied to the center of mass of each floor and then applied at a 5% offset to model torsion effects.
- Wind forces were applied to the center of pressure of each floor. These forces took into account each of the 4 load cases listed in ASCE7-05 involving both direct and torsion effects. For a list of these cases, please refer to Appendix C.
- Load combinations were generated from the ASCE7-05 code. Please see the section in this report entitled Load combinations for a list.
- The basement floor was modeled as the base with an infinite stiffness to ensure 0% drift at ground level. Due to the stiffness of the reinforced concrete first floor, the drift at the first floor is minimal, although it was not neglected.
- Braces were assumed to be pinned at both ends.
- Lateral beams were assumed to be fixed at both ends.
- The structure was assigned as a fixed base due to the mat foundation.
- The beam and column elements were designed taking into account panel zone deformations and both shear and axial deflections.
- P – Delta effects and rigid end offsets were considered and a dynamic analysis was performed to find a modal response.
- Wind drift was determined from the ASCE7-05 commentary stating that drift can be calculated from the load combination D+0.5L+0.7W.

Shown below is an outline of the modeling process.



Moment Frame Lateral Force Resisting System Cases:

A layout of the chosen moment frame locations has already been presented in the Evolution of Design section of this report. The following figures represent 3-D views of the moment frame RAM model and just the lateral force resisting system. These views represent both the normal and lightweight concrete models since the only difference between the two systems is the weight of the floors.

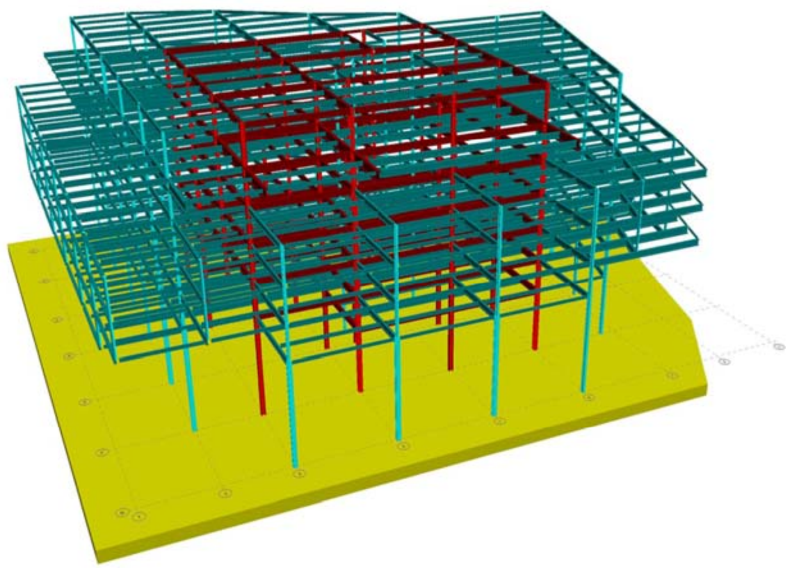


Figure 9: 3-D Moment Frame RAM Model

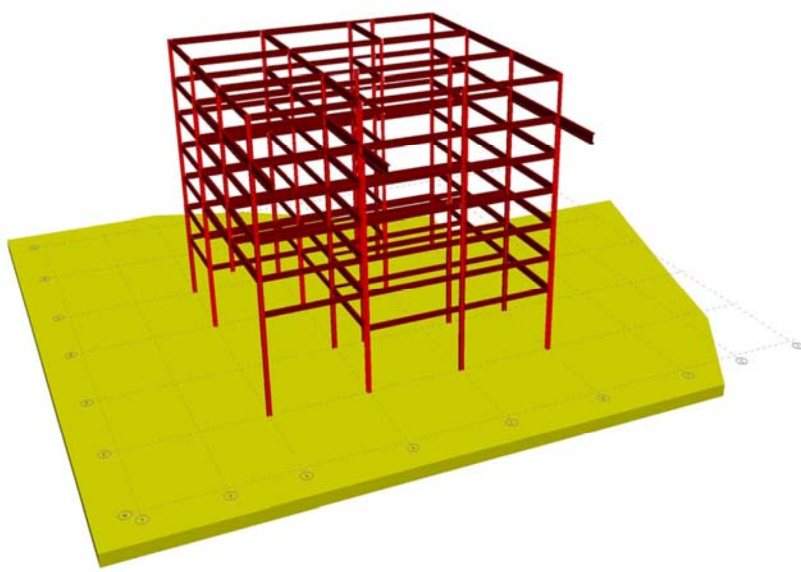


Figure 10: 3-D Moment Frame Lateral Force Resisting System RAM Model

Moment Frame Design Check:

A series of checks were performed to prove the adequacy of the moment frame lateral force resisting systems designed by RAM. The following table represents a summary of these checks performed and observations made.

Check	Comment	Observation
Story Drifts	Allowable story drifts for each level are met in each of the two orthogonal directions. The computed story drifts is at most 81% of the allowable.	OK
Torsion	Accidental Torsion = 5%. Inherent torsion is assumed by applying loads at the center of mass and being resisted by the center of rigidity of the structure.	OK
Redundancy	There are only three frames in each direction so each frame had to be designed to resist more than 25% of the total story shear, however, in SDC=B, ρ is still equal to 1.0.	OK
Modal Period	ASCE7-05 Approximate Period: 1.63 seconds RAM modal period: 2.224 seconds (NWC) RAM modal period: 1.843 seconds (LWC) The RAM model period is more than the conservative period approximation of the ASCE7-05 code.	OK
Member Spot Checks	Columns and beams are approximately 30% to 98% of their total design strength based off their interaction equations. This occurs due to member updates for size uniformity and drift improvement.	Some System Overdesign

Story Drifts

The following tables represent the story displacements based on the wind loads that control the design in the RAM model in normal weight concrete. These displacements are higher due to the lesser stiffness of the structure and are therefore used as a representation of both models. The story drift limit is h/400 for both the overall displacement and inter-story displacement.

N-S Direction (NWC) – H/400 Limit				
Story	h_x (ft)	Allowable Displacement (in)	Story Displacement (in)	Check
Roof	12.00	0.36	0.20	OK
Penthouse	10.83	0.32	0.23	OK
Fifth	10.83	0.32	0.26	OK
Fourth	10.83	0.32	0.29	OK
Third	10.83	0.32	0.25	OK
Second	10.83	0.32	0.12	OK
First	10.83	0.32	0.02	OK
Basement	10.83	--	0.00	OK

Total displacement: 1.37”   Total allowed displacement: 2.31”

E-W Direction (NWC) – H/400 Limit				
Story	h_x (ft)	Allowable Displacement (in)	Story Displacement (in)	Check
Roof	12.00	0.36	0.21	OK
Penthouse	10.83	0.32	0.09	OK
Fifth	10.83	0.32	0.24	OK
Fourth	10.83	0.32	0.23	OK
Third	10.83	0.32	0.26	OK
Second	10.83	0.32	0.14	OK
First	10.83	0.32	0.03	OK
Basement	10.83	--	0.00	OK

Total displacement: 1.20”   Total allowed displacement: 2.31”

Torsion

According to ASCE7-05 section 12.8.4.2, diaphragms that are not modeled as flexible are required to account for inherent torsion and accidental torsion.

Inherent Torsion

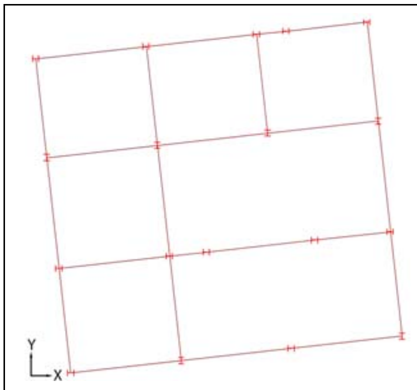
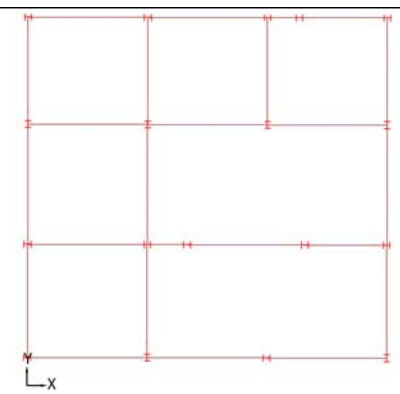
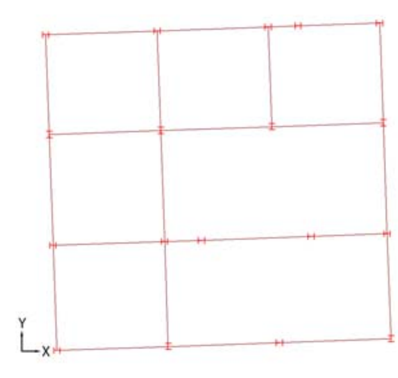
Since the lateral forces are applied to the center of mass and the center of rigidity is calculated in the RAM model, this will account for inherent torsion for seismic provisions. Wind load cases that involved torsion were also taken into account in the model. A visual inspection of the model verified the accuracy of the center of mass and the center of rigidity for each floor.

Accidental Torsion

The analysis was run with the seismic loads in the X and Y directions running through the center of mass, and then with a 5% accidental torsion. The worst case in deflections was found and the C_d factor, 3, was determined according to ASCE7-05 section 12.8.4.2. The amplification factor was determined to be equal to 1 in both the X and Y directions. These calculations are not included in this report because seismic deflections and loads do not control but they can be reviewed upon request.

Modal Period

Shown below are the first three modes for the NWC moment frame case. These periods were compared to the approximated periods calculated with ASCE7-05.

		
T_z = 3.265 seconds	T_y = 2.285 seconds	T_x = 2.224 seconds

Braced Frame Lateral Force Resisting System Cases:

A layout of the chosen braced frame locations has already been presented in the Evolution of Design section of this report. The following figures represent 3-D views of the braced frame RAM model and just the lateral force resisting system. These views represent both the normal and lightweight concrete models since the only difference between the two systems is the weight of the floors.

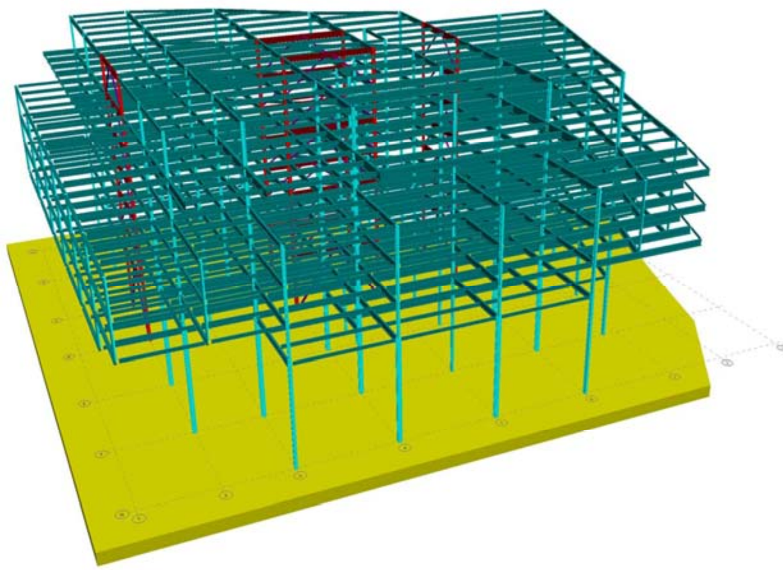


Figure 11: 3-D Braced Frame RAM Model

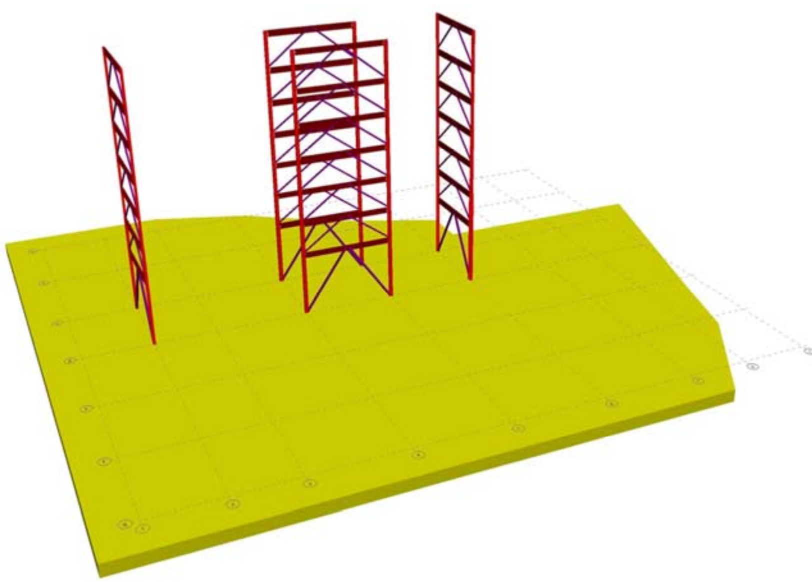


Figure 12: 3-D Braced Frame Lateral Force Resisting System Model

Braced Frame Design Check:

A series of checks were performed to prove the adequacy of the braced frame lateral force resisting systems designed by RAM. The following table represents a summary of these checks performed and observations made.

Check	Comment	Observation
Story Drifts	Allowable story drifts for each level are met in each of the two orthogonal directions. Although the computed story drifts is at most 38% of the allowable, this design was driven more by member strength instead of serviceability.	OK
Torsion	Accidental Torsion = 5%. Inherent torsion is assumed by applying loads at the center of mass and being resisted by the center of rigidity of the structure.	OK
Redundancy	There are only two frames in each direction so one resists at least 50% of the total story shear, however, in SDC=B, ρ is still equal to 1.0.	OK
Modal Period	ASCE7-05 Approximate Period: 1.63 seconds RAM modal period: 1.485 seconds (NWC) RAM modal period: 1.244 seconds (LWC) Since the RAM model period is less than the conservative period approximation, this period was then used to update the seismic loads in the model.	OK
Member Spot Checks	Columns and beams are approximately 32% to 96% of their total design strength based off their interaction equations. This occurs due to member updates for size uniformity.	Some System Overdesign

Story Drifts

The following tables represent the story displacements based on the wind loads that control the design in the RAM model in normal weight concrete. These displacements are higher due to the lesser stiffness of the structure and are therefore used as a representation of both models. The story drift limit is h/400 for both the overall displacement and inter-story displacement.

N-S Direction (NWC) – H/400 Limit				
Story	h_x (ft)	Allowable Displacement (in)	Story Displacement (in)	Check
Roof	12.00	0.36	0.08	OK
Penthouse	10.83	0.32	0.09	OK
Fifth	10.83	0.32	0.10	OK
Fourth	10.83	0.32	0.09	OK
Third	10.83	0.32	0.10	OK
Second	10.83	0.32	0.09	OK
First	10.83	0.32	0.09	OK
Basement	10.83	--	0.00	OK

Total displacement: 0.64” Total allowed displacement: 2.31”

E-W Direction (NWC) – H/400 Limit				
Story	h_x (ft)	Allowable Displacement (in)	Story Displacement (in)	Check
Roof	12.00	0.36	0.13	OK
Penthouse	10.83	0.32	0.09	OK
Fifth	10.83	0.32	0.10	OK
Fourth	10.83	0.32	0.11	OK
Third	10.83	0.32	0.10	OK
Second	10.83	0.32	0.10	OK
First	10.83	0.32	0.12	OK
Basement	10.83	--	0.00	OK

Total displacement: 0.75” Total allowed displacement: 2.31”

Torsion

According to ASCE7-05 section 12.8.4.2, diaphragms that are not modeled as flexible are required to account for inherent torsion and accidental torsion.

Inherent Torsion

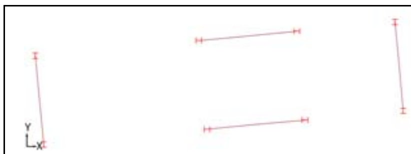
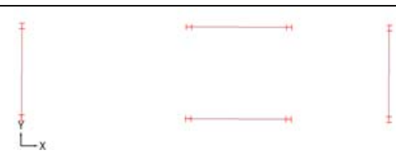
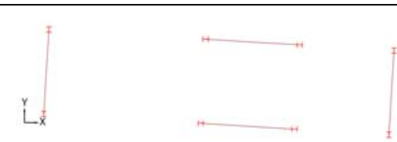
Since the lateral forces are applied to the center of mass and the center of rigidity is calculated in the RAM model, this will account for inherent torsion for seismic provisions. Wind load cases that involved torsion were also taken into account in the model. A visual inspection of the model verified the accuracy of the center of mass and the center of rigidity for each floor.

Accidental Torsion

The analysis was run with the seismic loads in the X and Y directions running through the center of mass, and then with a 5% accidental torsion. The worst case in deflections was found and the C_d factor, 3, was determined according to ASCE7-05 section 12.8.4.2. The amplification factor was determined to be equal to 1 in both the X and Y directions. These calculations are not included in this report because seismic deflections and loads do not control but they can be reviewed upon request.

Modal Period

Shown below are the first three modes for the NWC moment frame case. These periods were compared to the approximated periods calculated with ASCE7-05.

		
T_z = 2.184 seconds	T_y = 1.711 seconds	T_x = 1.485 seconds

Material Cost Evaluation

A basic material cost estimate was used to determine which alternative would be chosen for the structural system. This was based off of steel tonnage takeoffs from RAM and an estimated cost/lb of steel. The cost was estimated as $1.50/lb of steel. The summary is shown to the right.

Structural Frame Type	Steel Weight (lb)	Cost
NWC Braced Frame	1229639	$1,844,459
LWC Braced Frame	1176033	$1,764,050
NWC Moment Frame	1343073	$2,014,610
LWC Moment Frame	1302411	$1,953,617

Based off the table, the LWC braced frame is the cheapest option, however, there is approximately a 30% premium to get lightweight concrete instead of normal weight concrete. The total area of the composite steel deck is 185,147SF. For lightweight concrete, the deck is 4.5" deep and a total of 2,571 CY. For normal weight concrete, the deck is 5.5" and a total of 3,143 CY. The approximate savings in material is 18% if lightweight concrete is used. However, the savings between the LWC braced frame and the second cheapest option, the NWC braced frame is only $80,400. This is only a 5% savings. The total savings of 23% is not enough to offset the 30% premium for the lightweight concrete.

Based on the fact that LWC braded frame is not cheap enough to offset the 30% concrete premium, the NWC braced frame is the chosen alternative for this structural system.

Floor Plans and Brace Elevations

Shown below and on the following pages are a typical floor plan, the roof plan, and the brace elevations. Member sizes are called out along with the locations of the braces highlighted in orange and the splice locations shown as orange x's on the elevations.

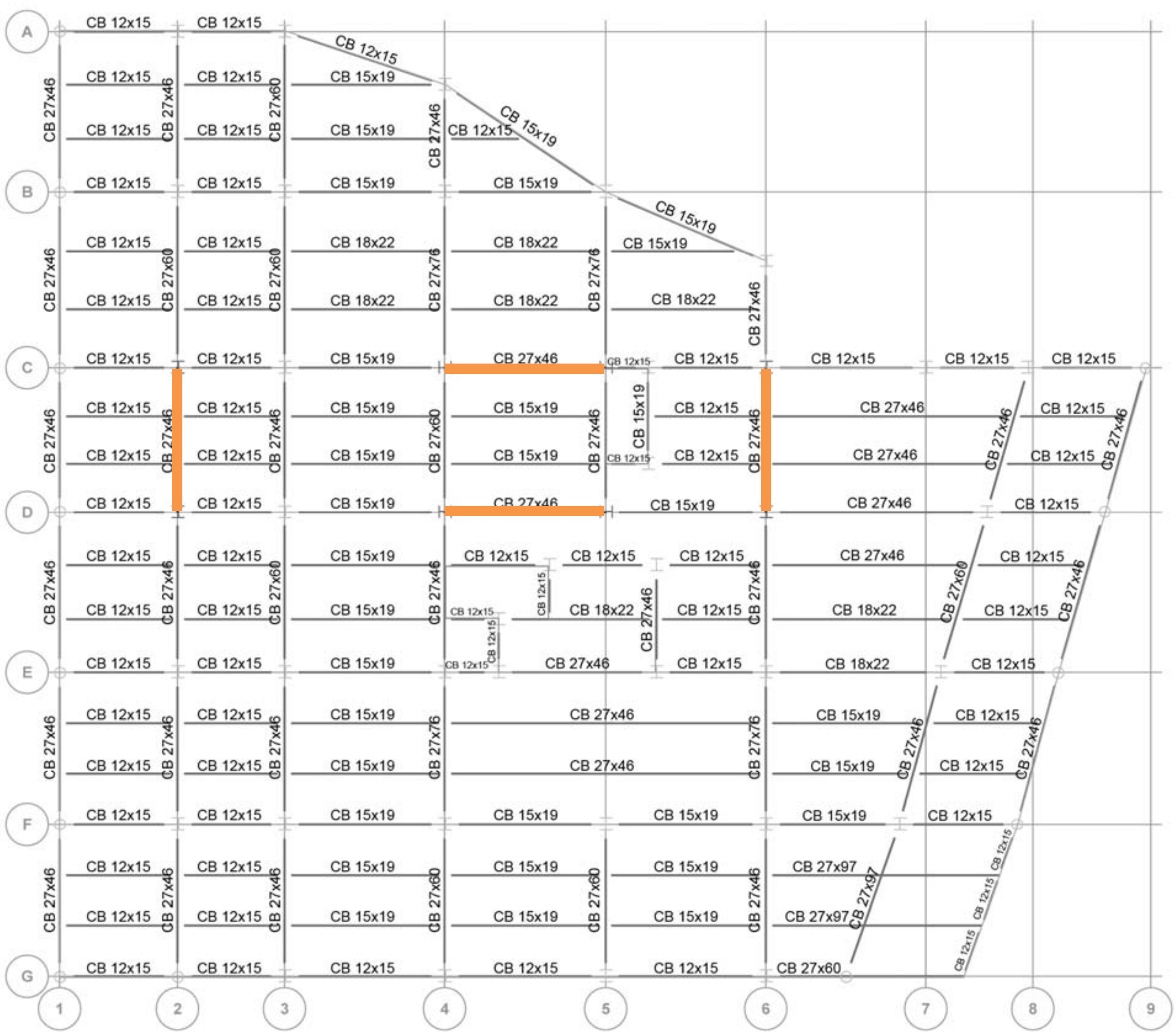


Figure 13: Typical Floor Plan

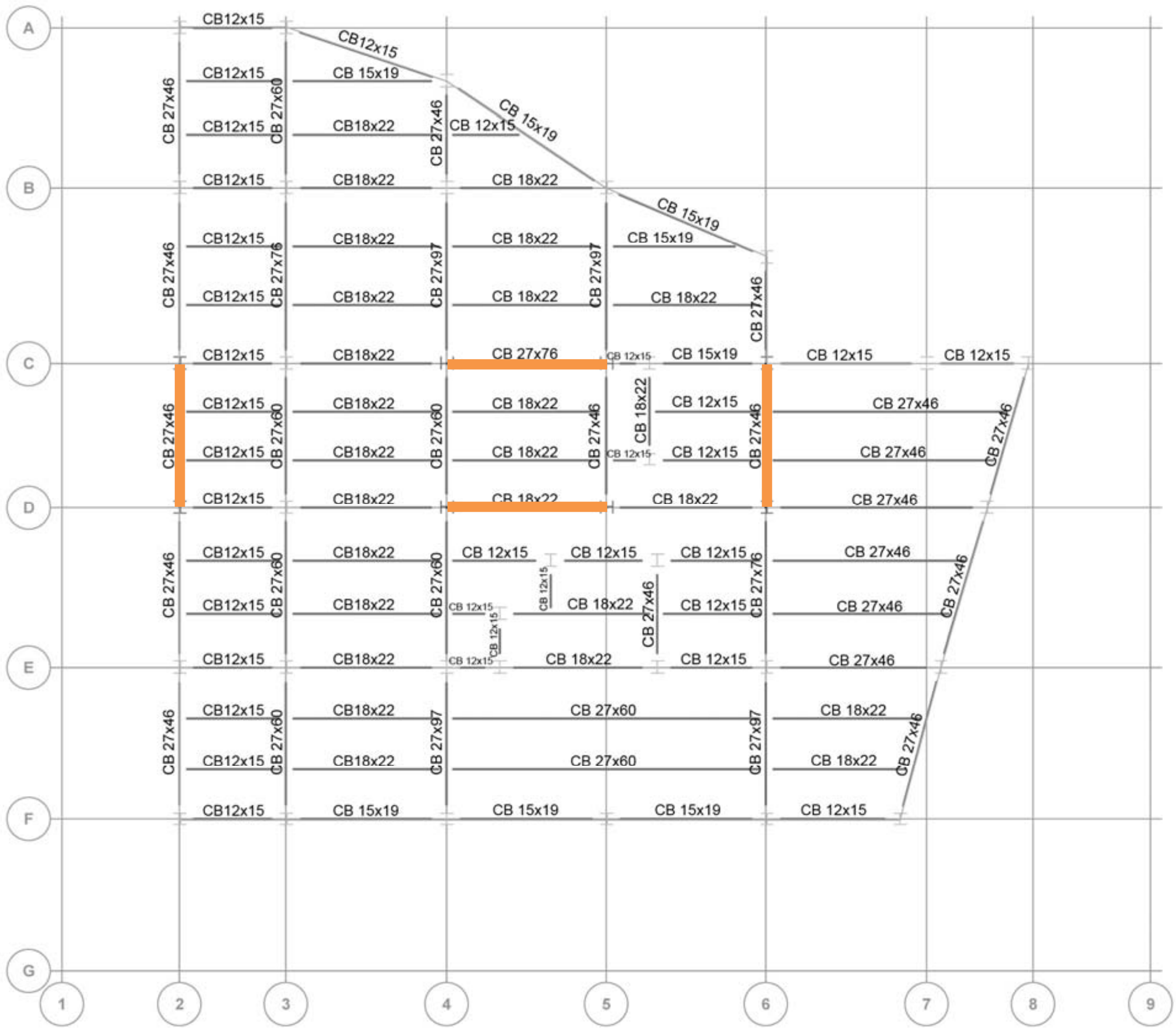


Figure 14: Roof Plan

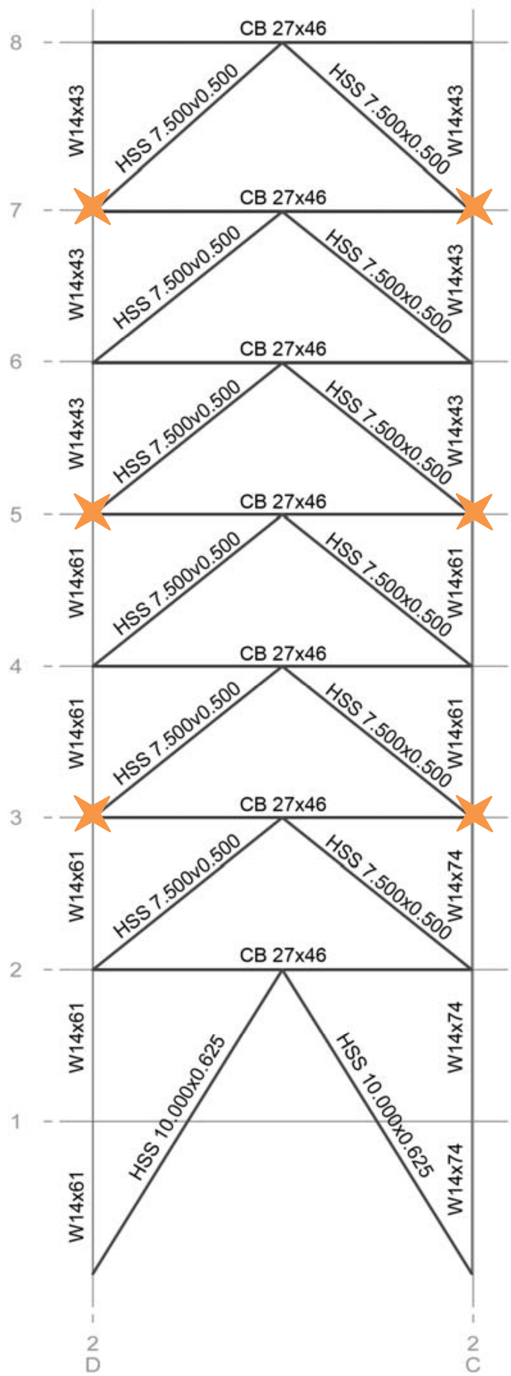


Figure 15: Braced Frame 1 Elevation

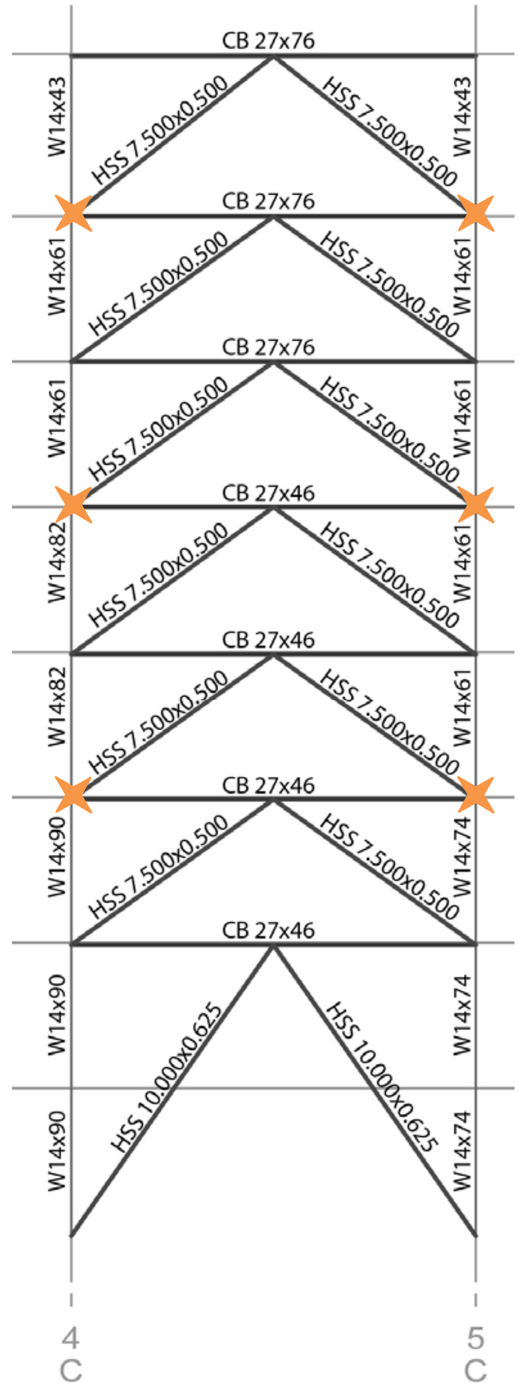


Figure 16: Braced Frame 2 Elevation

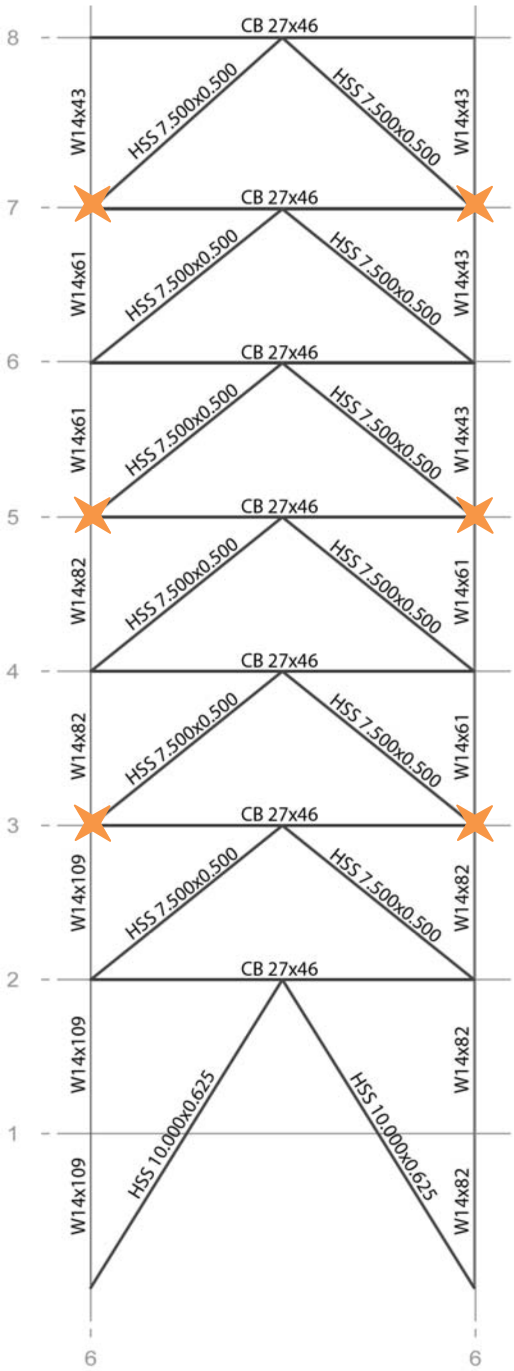


Figure 17: Braced Frame 3 Elevation

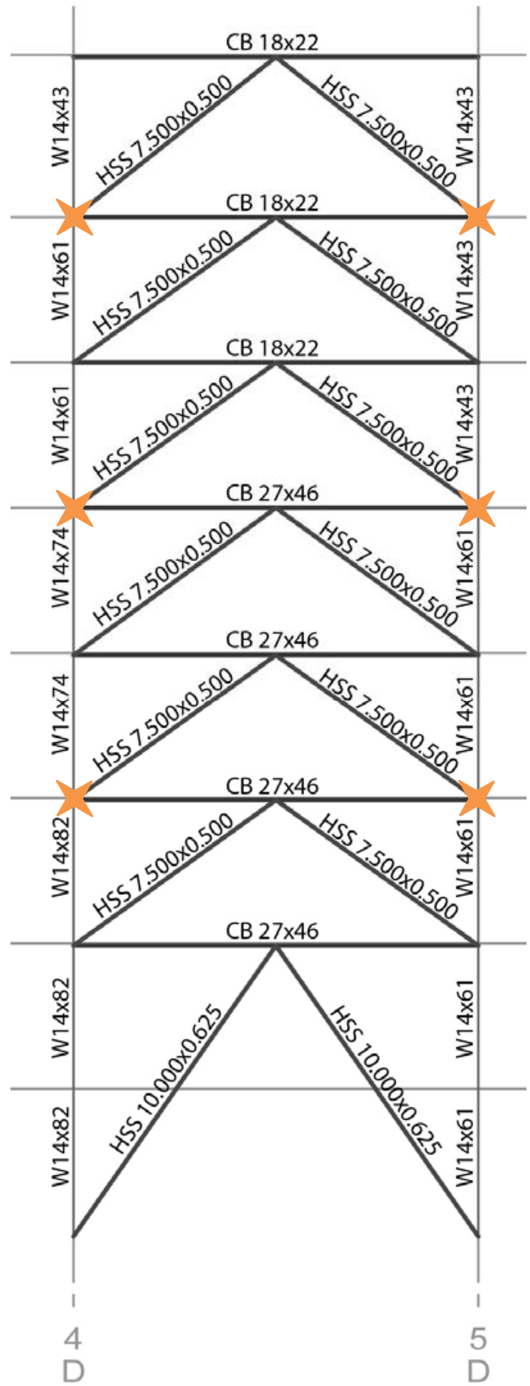


Figure 18: Braced Frame 4 Elevation

Cantilever Solution

House of Sweden is a signature building for the Swedes in America. As such, the architects designed the buildings to appear as if they were floating jewels above the Potomac. This was accomplished by having the building cantilever as you move up the façade. Some of these cantilevers are as long as 22'. To help minimize the depth of the steel members, a steel hanger system was devised to tie the cantilevers back to the perimeter columns. The cantilever at the penthouse is only 11' long so that was left as an actual cantilevered beam, and therefore, this member is deeper than the 22' long cantilever beams.

Not shown in the 3D computer models are the hangers. In the RAM model, the cantilevers were supported from the underside with HSS columns. The forces transferred to the columns from the cantilevers were then used to size the hangers. The forces that result from the hangers tying into the perimeter columns were then modeled as point loads in the RAM model. These hangers are at an angle of 46.1°. The final sizing for the normal weight concrete braced frame was HSS7.0 tubing, except for one hanger at a corner which was sized as HSS8.625x0.625. The hanger connections were not designed so the tension only members were designed with the Steel Manual and $A_e = 0.75A_g$ for rupture to control. The final sizes are shown below and the brace locations are shown in the section cut in Figure 19.

Hanger	Gravity Load	P_u (K)	Shape	Rupture ΦP_n (K)
A1	125.02	146.37	HSS 7.0x0.250	161
B1	237.93	278.56	HSS7.0x0.500	311
C1	227.81	266.71	HSS7.0x0.500	311
D1	217.48	254.62	HSS7.0x0.500	311
E1	222.61	260.63	HSS7.0x0.500	311
F1	193.71	226.79	HSS7.0x0.375	238
G1	93.5	109.47	HSS7.0x0.188	122
G2	160.64	188.07	HSS7.0x0.312	200
147	384.09	449.68	HSS8.625x0.6250	479
179, 28.33	143.9	168.47	HSS7.0x0.312	200
186.67, 56.83	223.32	261.46	HSS7.0x0.500	311
195.33, 86.83	217.28	254.39	HSS7.0x0.500	311
203, 113.83	112.32	131.50	HSS 7.0x0.250	161

These hangers induce a horizontal compression force in the beams on the fifth floor. These point loads were added into the RAM to see how they affected the design of these beams. Most of the beams at the south and west ends of the building perimeter had already been upsized for uniformity for construction so they were able to take the extra compression load. The beams at the north end of the building perimeter needed to be upsized to take the additional loads. The original shapes were CB15x19 so they had to be resized as CB18x22. This was the only floor where this has to occur for the hangers. The roof beam members are placed in tension, but the sizes that are already called out for the roof beams are adequate to take the load.

Also studied was the floor-to-ceiling height of the new structure to ensure that there was adequate space for the solution. As the depth at the cantilevers increases, the floor-to-ceiling height decreases at the perimeter of some floors. The average floor-to-ceiling height is 8' which is a decrease of 1' from the original floor-to-ceiling height of 9'. Some floors have an interior floor-to-ceiling height of 8.5' or more due to the reduced depth of the beams. These varying heights are shown in Figure 19. below. The concrete floor slab is denoted in purple, the heavy black line is the ceiling tile, and the castellated beams are colored orange.

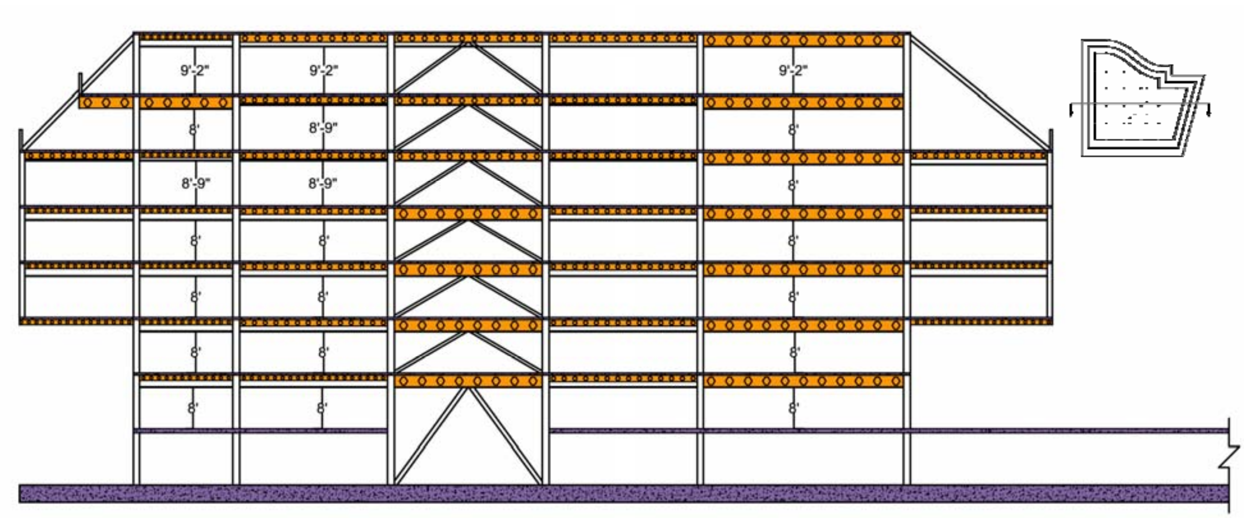


Figure 19: Clear Floor - to - Ceiling Heights

Standard connections are addressed below. They are shear tab connections for beam-to-beam and beam-to-column connections. They were taken from the standard details webpage of CMC Steel.

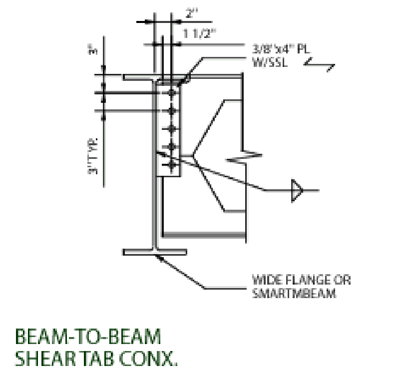


Figure 21: Beam-to-Beam Shear Tab Connection

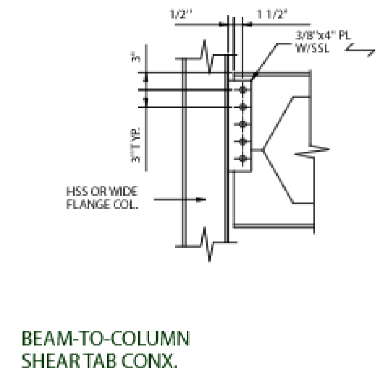


Figure 20: Beam-to-Column Shear Tab Connection

Implications of Redesign

The redesign of the north building of the House of Sweden will have many impacts on different systems involved in the structure. This redesign will affect things such as the garage level column design, the foundation, and progressive collapse security. Presented below are the impacts from the redesign and some ways they can be addressed.

Garage Level Column Design:

The first floor of both the north and south buildings are connected by a pedestrian plaza between the two towers. Therefore, for the purpose of this thesis, the first floor was left in its original design as a reinforced concrete flat slab. With this being the case, it is challenging to tie to first floor reinforced concrete flat slab to the new steel design of the north building. Instead of looking into ties from steel into concrete, it was decided to design reinforced concrete columns for the garage that encase the garage steel columns and hold up the first floor only. These columns will ease construction of the steel so that ties from the steel supporting the concrete will not have to be designed and placed in exactly the right locations. They will also help with blast protection and progressive collapse mitigation (see the same titled section later in this report).

The columns were designed as 30"Φ composite columns for the critical tributary area of 30'x30'. Spiral reinforcing (#4) was used for confinement purposes as outlined by GSA for blast protection and progressive collapse mitigation (again see the same titled section later in this report). For details on the design of this column, please see Appendix F. Below is the column geometry showing the placement of the #8 bars and the steel wide-flange column encased by the concrete.

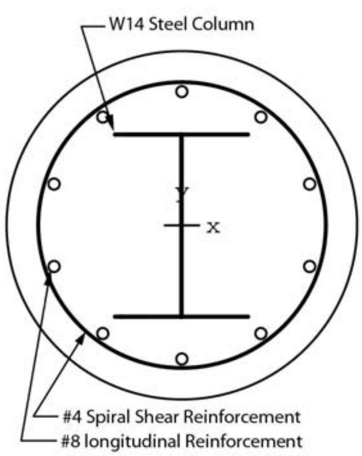


TABLE A.14
Size and pitch of spirals, ACI Code

Diameter of Column, in.	Out to Out of Spiral, in.	f'_c, psi 2500	3000	4000	5000
f_y = 40,000 psi					
14, 15	11, 12	$\frac{3}{8}$–2	$\frac{3}{8}$–1$\frac{3}{4}$	$\frac{1}{2}$–2$\frac{1}{2}$	$\frac{1}{2}$–1$\frac{3}{4}$
16	13	$\frac{3}{8}$–2	$\frac{3}{8}$–1$\frac{3}{4}$	$\frac{1}{2}$–2$\frac{1}{2}$	$\frac{1}{2}$–2
17–19	14–16	$\frac{3}{8}$–2$\frac{1}{4}$	$\frac{3}{8}$–1$\frac{3}{4}$	$\frac{1}{2}$–2$\frac{1}{2}$	$\frac{1}{2}$–2
20–23	17–20	$\frac{3}{8}$–2$\frac{1}{4}$	$\frac{3}{8}$–1$\frac{3}{4}$	$\frac{1}{2}$–2$\frac{1}{2}$	$\frac{1}{2}$–2
24–30	21–27	$\frac{3}{8}$–2$\frac{1}{4}$	$\frac{3}{8}$–2	$\frac{1}{2}$–2$\frac{1}{2}$	$\frac{1}{2}$–2
f_y = 60,000 psi					
14, 15	11, 12	$\frac{1}{4}$–1$\frac{3}{4}$	$\frac{3}{8}$–2$\frac{3}{4}$	$\frac{3}{8}$–2	$\frac{1}{2}$–2$\frac{3}{4}$
16–23	13–20	$\frac{1}{4}$–1$\frac{3}{4}$	$\frac{3}{8}$–2$\frac{3}{4}$	$\frac{3}{8}$–2	$\frac{1}{2}$–3
24–29	21–26	$\frac{1}{4}$–1$\frac{3}{4}$	$\frac{3}{8}$–3	$\frac{3}{8}$–2$\frac{1}{4}$	$\frac{1}{2}$–3
30	27	$\frac{1}{4}$–1$\frac{3}{4}$	$\frac{3}{8}$–3	$\frac{3}{8}$–2$\frac{1}{4}$	$\frac{1}{2}$–3$\frac{1}{4}$

Figure 22: Garage Level Column Geometry

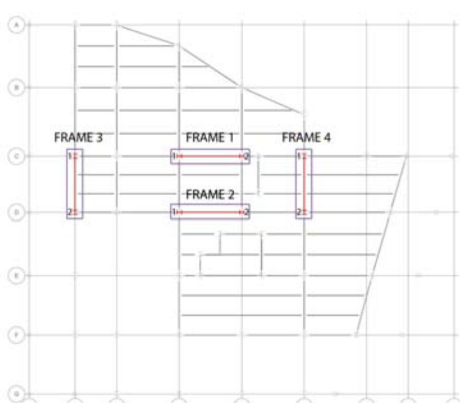
There is composite action between the steel and concrete columns so only the steel columns need to transfer their load into the mat foundation. The steel will sit on bearing plates, and the concrete column will be attached to the foundation with rebar. The design of these bearing plates and rebar attachments are outside the scope of this thesis, but if the owner desires to implement this new design, this is an area of the design where more investigation is required.

Foundation Impacts:

The goal of the foundation impact exploration was to see it the foundation could stay in its original form or possibly improve. To test this, a few different parameters were investigated. These parameters are:

- Necessity of Mat Foundation
- Thickness Based on Punching Shear
- Location of Embedded Sewer Pipes
- Overturning Moment

To check whether the mat foundation was even still necessary, a basic P/A evaluation was conducted. The bending moment induced in the foundation from the column loads was not taken into account. The basis for this decision was that the bending moment is going to add more stress in the mat foundation and therefore, more area than just looking at P/A will be needed. If a mat foundation is necessary just by looking at P/A, then there is no need to add the bending moment into the analysis. The analysis looked at the critical columns that are part of the braced frames. Using the soil bearing pressure of 2.2 ksf, the area needed to support the column force was found. From this area, the length of a side of a square footing was determined. A summary of the findings is presented below.



Frame	Column	P_{max} (K)	Area (ft^2)	Footing Dimensions (ft)
1	1	911.18	414.17	20'-5"
	2	726.03	330.01	18'-3"
2	1	729.74	331.70	18'-3"
	2	541.36	246.07	15'-9"
3	1	525.22	238.74	15'-6"
	2	445.43	202.47	14'-3"
4	1	674.18	306.45	17'-7"
	2	945.19	429.63	20'-9"

Figure 23: Location of the Braced Frames and Footing Size Check

As shown by the chart, the size of the footings is quite large. The largest space between these footings is only 13' and the smallest spacing is 7.5'. Therefore, the foundation is still more practical as a mat foundation.

To check the thickness of the mat foundation, punching shear was considered. The critical section is d/2 from the edge of the column. First, it was determined if the column loads in the north building controlled the thickness of the mat. The critical column was identified and a d necessary for $\Phi V_c = V_u$ was found, with Φ=0.75 from the ACI 318-08 code. This d was found to be 43" then, when the 3" clear cover and 1.27"Φ steel bars was added on, the total thickness was determined to be 48". It is therefore assumed that the north building column loads drove the design of the thickness of the mat.

Then, the critical column in the braced frames was identified and the thickness of the new design was calculated and the determined d was 36.6". The overall total thickness is 42" which is an easy dimension for excavation and construction. The south building thickness was also checked to assure that the north building column loads still control. The d for the south building was found to be 31" and the overall depth is 36". From these calculations, it is shown that the north building still controls and it might be possible to reduce the thickness of the mat to 42". To review the calculations, please refer to Appendix G.

With respect to embedded pipes in the foundation, there are very few. Based on the existing conditions plans, there are no existing pipes or obstructions that need to be taken into account for the thickness of the mat. Based on the plumbing plans, the largest pipe embedded in the mat is only 6" in diameter. It is possible to place these pipes in the mat, even if 8" is taken off the thickness of the foundation.

Replacing a concrete moment frame with a lighter steel braced frame system is also going to have an impact on the overturning moment versus the resisting moment. The proposed system is approximately 38% less weight than the concrete moment frame system. This being said, a check should be performed to ensure that the thinner mat foundation can resist the overturning moment from the wind load. It is assumed that the dead load of the slab will contribute to resisting the overturning moment over half of its length in the specified direction. The results are summarized in the table on the next page:

Overturning Moment Resistance	N-S Direction	E-W Direction
Height	77 ft	77 ft
Length	192 ft	206 ft
Applied Wind Load	277 K	325 K
Overturning Moment	21,329 ft-K	25025 ft-K
Resisting Dead Load	8,944 K	8,944 K
Resisting Moment	858,624 ft-K	921,232 ft-K
	$M_R > M_{OT}$	$M_R > M_{OT}$

As shown in the table to the left, the applied wind loads create an overturning moment at the mat foundation. The self-weight of the mat is more than adequate to resist the overturning moment created by these loads.

Overall, based on the parameters checked, the slab can be reduced by 6" from 48" to 42". This provides a 12.5% savings on the amount of concrete necessary for the mat. If the owner would like to take this reduction in mat foundation depth, some things to explore further would be the amount of reinforcement necessary for the new design versus the old design and also, how much bending moment is induced in the foundation and if that changes the depth savings at all. For this thesis, the four points listed above were investigated to show proof of concept that the original mat foundation could be used or even improved upon and that the foundation would not worsen.

	Original Design	New Design	Ratio New: Original	Savings
Building Weight	17,883 K	11,032 K	0.62	38%
Depth, d	43"	37"	0.86	14%
Steel Reduction	1-(1/1.38)*1.14			17%

A brief estimate of the savings on the foundation was conducted. The overall weight of the building was reduced by 38%. In turn, this should reduce the overall moment by approximately 38%. However, the depth, d, was only reduced by 14%. Therefore, there should be a reduction of reinforcing steel by 17%. These results are summarized to the left.

Foundation Cost Estimate			
Steel Rebar:			
Cost: $830/ton	Total Original Tonnage	358.63	$297,663
Contractor Cost	Total New Tonnage	304.84	$253,013
		Total Steel Savings:	-$44,650 (-15%)
4000 psi NW Concrete:			
Cost: $115/CY	Total Original Volume (CY)	6,156	$707,974
Contractor Cost	Total New Volume (CY)	5,387	$619,477
		Total Concrete Savings:	-$88,497 (-13%)
460 HP Dozer, 150' Haul, Clay Soil Excavation:			
Cost: $3.18/CY	Total Original Volume (CY)	10,006	$31,820
RS Means Estimate	Total New Volume (CY)	9,234	$29,365
		Total Excavation Savings:	-$2,455 (-7.7%)
		Total Original Cost:	$1,037,457
		Total New Cost:	$901,855
		Total Savings:	-$135,602 (-13%)

This estimate includes material and labor. Overall, the total foundation and excavation cost savings is $135,600, or approximately 13% from the original cost of the mat foundation and 6.1% of the original $22.1 million budget. Excavation was taken into account for this estimate, but a conservative number was used from RS Means. Due to the high water table at the site next to Rock Creek, the savings on excavation is likely higher than what was estimated above and additional savings can be obtained from a more in-depth cost estimate.

Blast Protection and Progressive Collapse Analysis:

House of Sweden houses, above all else, part of the Swedish Embassy. Even though this is not the main embassy for Sweden, security is still a top concern for the owners and engineers alike. As shown in the Special Systems section of the Building Overview, no expense was spared in outfitting the building with intrusion detection equipment; however, the owners and engineers conducted no real exploration into blast protection and progressive collapse mitigation. For this thesis, a brief look into blast protection and progressive collapse mitigation was completed.

Blast protection is an immediate problem with this building. There are three main issues with the building with respect to blast protection:

- There is commercial space in both buildings and they are open to the public, as is the embassy itself. There are no metal detects and little in the way of security guard personnel to help detect a blast threat from a person off the street.
- The location of the embassy is right next to the street, with only the sidewalk and a small walkway between the building and the street. There is no separation between the street and the building in the way of bollards or other structures that can obstruct the pathway of a moving vehicle intent on running into the building.
- The parking garage below both buildings is open to the public using the commercial space. Most of the parking spaces are adjacent to a structural column. With little to no hassle, a car bomb will be able to detonate in the garage and take out at least one of the columns.

These are major concerns that are not easily mitigated with the existing conditions. The building could tighten security by adding metal detectors or more guards, but these measures defeat the purpose of the open and welcoming commercial spaces and embassy atmosphere. The site itself does not afford the possibility of creating a larger barrier between the street and the buildings due to the tight site and the location of Rock Creek right behind the buildings. Therefore, mitigation of progressive collapse becomes a bigger issue since the possibility of a structural attack is high.

As mentioned above, the most prominent places for an attack on the building is at or below grade. The ductility of the steel at grade will be able to resist some of the impact of a blast from a car impact or personal bomb. Also, a redundancy can be designed into the building for an attack on the exterior columns (excluding the corner columns) by embedding steel cables in the floor system and attaching them to these columns. This is somewhat newer technology in progressive collapse mitigation techniques and is being tested at the University of California at Berkeley. For more information on this technology, please refer to the paper *Use of Catenary Cables to Prevent Progressive Collapse of Buildings*. The citation for this paper can be found in the Document and Code Review section of this report. Embedding these cables will help ensure that if a column is removed from the structure, the gravity loads are redistributed to other structural elements. A shear failure is also not likely with steel. A flexural failure is more likely and will not fail in a fast, disastrous manner. If a column fails, and the cable supports are called upon, there will likely be compression crushing of the concrete and tension cracks through the floor, but the floor designed for House of Sweden is a total of 5.5" including the ribs, which is 1" thinner than the composite floor used in the test, but as long as the cables are embedded in the ribs, there should not be an issue with cable blow-out.

Floor	# of Cables	Length	Weight (lb)	Cost
PH	6	563	2256	$1,466
5	8	563	3008	$1,955
4	11	563	4136	$2,688
3	13	563	4888	$3,177
2	15	563	5640	$3,666
1	16	563	6016	$3,910

Based on the largest load on a column at the perimeter and the amount of load acceptable on a cable (53 K), the total number of cables needed at the perimeter of the first floor is 16. This number can be reduced on each floor going up the building and an estimate is summarized to the left. The contractor cost of a cable is $0.65/lb. The overall cost of the cables is $16, 864, or approximately 0.08% of the original budget of $22.1 million.

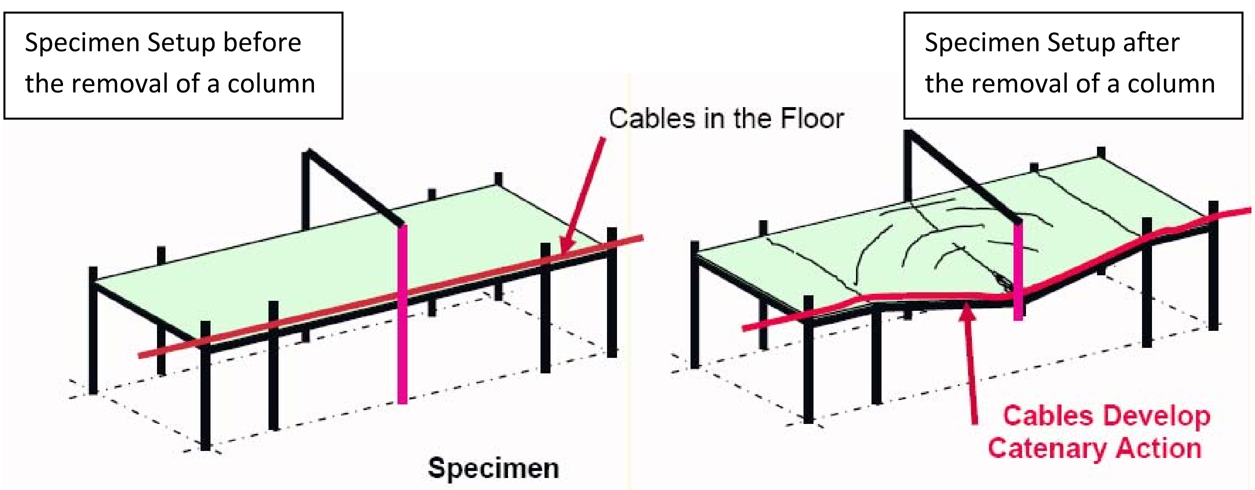


Figure 24: Schematic View of Catenary Cable Action Taken from Astaneh-Asl et.al.

The parking garage is more susceptible to progressive collapse since a column could be taken out at the base of the structure, and an interior column can be taken out more easily than coming at the building with a car from the street. There is a mixture of steel and concrete columns in the parking garage holding up a reinforced concrete floor. The GSA makes recommendations in their *Progressive Collapse Analysis and Design Guidelines* for the design of reinforced concrete in a structure. These structural components should be:

- Designed with redundancy – This promotes a more robust structure to ensure that alternate load paths are available in the event of a structural failure. As stated above, the structure can be designed with steel cables embedded in the floors. Also, the reinforced concrete structure, by nature, will redistribute the gravity loads if a structural component fails.
- Designed with structural continuity and ductility – This means that the primary structural components (slab, beams, and girders) are able to span at least two full spans. The reinforced concrete floor was placed in three pour sequences. This means that the floor extends over at least two spans, if not more. The garage level columns were designed as reinforced concrete encased wide-flange columns. The reinforced concrete was designed with spiral reinforcing to aid confinement and add strength. For additional information on these columns, please see the section entitled Garage Level Column design.
- Designed to resist load reversals – This makes redistribution of the loads easier throughout the structural elements. The reinforced concrete floor was designed as a flexible diaphragm and is therefore subject to a shear reversal from the columns above to the column below.
- Designed to resist shear failure – This will help prevent a non-ductile, sudden failure of the structure. This is the only provision that was not looked at specifically. Without re-designing and detailing the floor, this provision cannot be confirmed. It is assumed that the correct amount of shear reinforcing was provided in the floor to assure that flexural failure occurs before shear failure. If the owner ever wanted a more comprehensive study of these circumstances conducted, it is recommended that this provision is the place to start.

BREADTH STUDY 1 – PENTHOUSE REDESIGN AND MECHANICAL EQUIPMENT RELOCATION

Problem Statement

In the original design, the penthouse is entirely taken up by the mechanical system. As the building is located in Georgetown near the Potomac River, the penthouse is the prime real estate in the building. This loss of the penthouse floor is a loss in revenue for the owner as apartment units on this penthouse floor can be sold at a premium because of the view of the river and of Georgetown.

Goals

- Move the mechanical room to the basement parking garage area without losing the required number of parking spaces.
- Create apartments in the new space created in the penthouse so that more revenue can be generated for the owner by charging a premium for these units.
- Look at the impacts of this move on the cost and schedule of the project.

Zoning Impacts

Before any mechanical equipment could be moved, it had to first be determined if zoning would allow any more residential space than what was already in the building. Based off the site area of 61,260 SF the allowable office and residential areas are summarized in the table below. As shown, it is allowed by zoning to create more residential space.

Zoning: W-2			
FAR	Allowed Square Footage	Original Provided Square Footage	Thesis Provided Square Footage
Total: 4.0	245,040	167,298	185,426
Office: 2.0	122,520	122,520	122,520
Residential: 2.0	122,520	54,778	62,906

Parking Study

Using a variety of resources including *Architectural Graphics Standards*, a parking study was completed for the below grade parking level. For the tables used for this study, please refer to those resources listed in the Document and Code Review section of this report. The goal of this study was to create space in the parking garage for some or all of the mechanical equipment from the penthouse could be moved to the basement and more apartment space could be created. Shown below and on the next page are tables showing the amount of spaces provided and the original and new layouts of the parking level. Orange denotes normal sized spaces, purple denotes compact spaces, and blue denotes handicapped spaces.

Original Parking Count			
Building Use	Requirements	Parking Required	Parking Provided
General Office 122,520 SF	One space per 1,800 SF over 2,000 SF	67 Spaces	67 Spaces
Residential 23 Units	One space per 3 residential units	8 Spaces	8 Spaces
	Total Spaces Required	75 spaces	75 Spaces
	Handicapped Spaces Required	3 Spaces	4 Spaces
	Allowable Compact Spaces (40% of Total)	30 Spaces Max.	30 Spaces

Figure 25: Old Parking Level Layout

New Parking Count			
Building Use	Requirements	Parking Required	Parking Provided
General Office 122,520 SF	One space per 1,800 SF over 2,000 SF	67 Spaces	67 Spaces
Residential 26 Units	One space per 3 residential units	9 Spaces	9 Spaces
	Total Spaces Required	79 spaces	79 Spaces
	Handicapped Spaces Required	4 Spaces	4 Spaces
	Allowable Compact Spaces (40% of Total)	30 Spaces Max.	30 Spaces

Figure 26: New Parking Level Layout

A space was created for the chillers and boilers to be placed in the parking level. As shown on the plans, the chillers were placed outside of the wall so that it is easier for air to be drawn. This area where the mechanical equipment was placed is underneath the plaza separating the two towers of the House of Sweden, therefore, noise from the chillers and boilers do not affect residences or offices, however, the chillers were placed next to a "scenic walkway" at the back of the building.

Figure 27: Location of Walkway under Whitehurst Freeway

As shown on the site plan on the above, this scenic walkway goes right underneath the Whitehurst Freeway. The noise from the chillers will be masked by the noise from the freeway and will not impact that walkway.

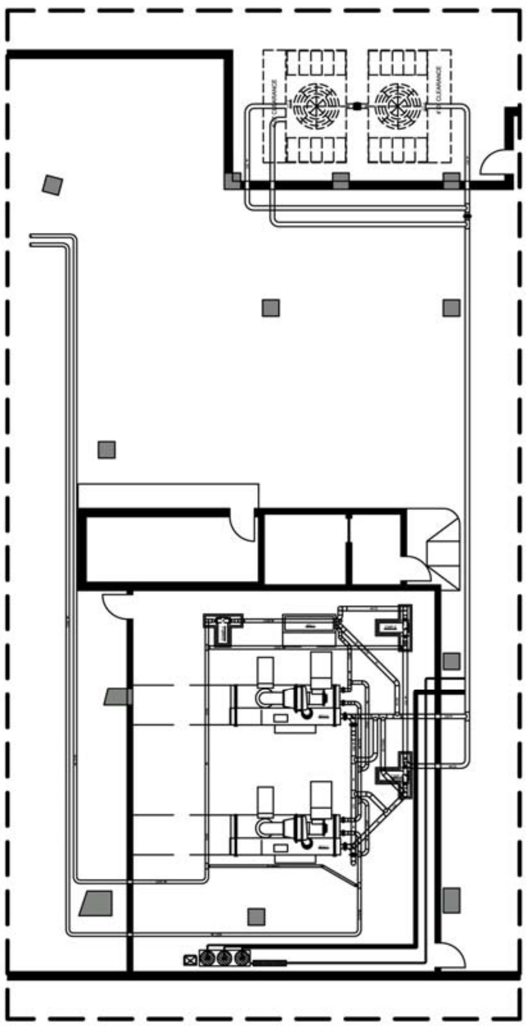


Figure 28: Layout of New Mechanical Room

Waterproofing

With moving the chillers and boilers to the basement, waterproofing becomes a focus of the parking level. This level is below the water table of the site, so it will be a challenge to make sure not only that water does not infiltrate to the interior but that any condensation or water overflow can be removed. Waterproofing details are very important, but for this job, the details are just shown as waterproofing detail 1-waterproofing detail 31. It is clear these were standard details that had not even been updated to the current job. These details have been updated to the standards set forth in the *Building Envelope Design Guide* and can be found in Appendix H. A set of good practice guidelines have also been generated from discussions in the Building Failures course, from internship experience, and from the *Building Envelope Design Guide* and are also presented in Appendix H.

Penthouse Redesign

The penthouse was redesigned and the new space created from the mechanical move was divided into three new apartments. These layouts are shown on the next page. Purple represents the area taken up by mechanical equipment. Orange represents dead space that was not even used as storage on the plans. Blue represents the new apartment spaces.

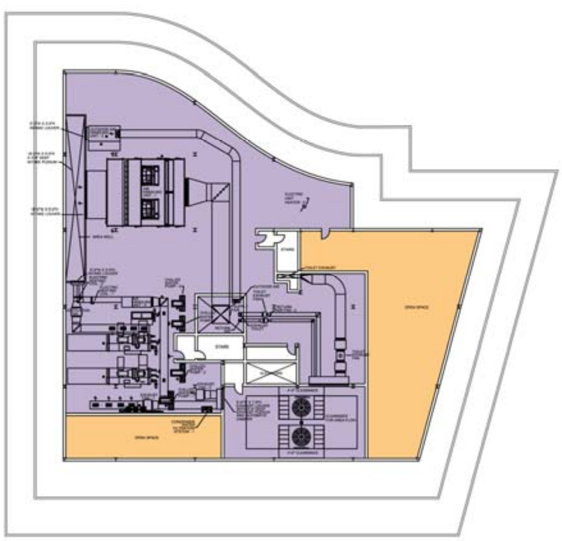


Figure 29: Original Penthouse Layout

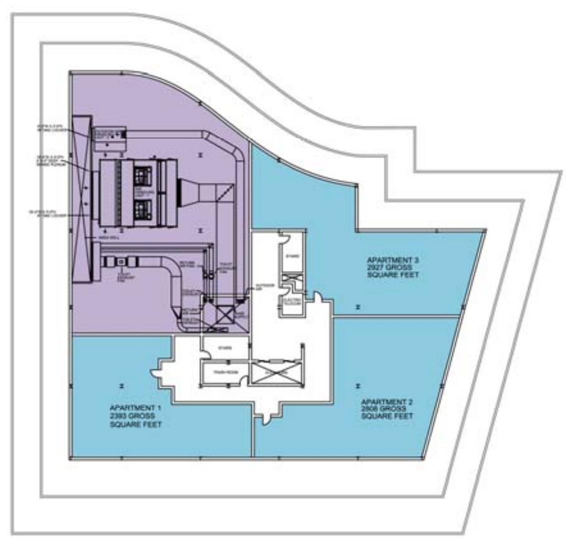


Figure 30: New Penthouse Layout

Acoustics Study

Noticeable in the plans are the fact that two of the apartments and the mechanical room share walls. An acoustics study was done for these walls to determine if the noise from the air handling units would not disturb the residents.

Transmission Loss (dB)						
Construction	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
8" painted concrete block wall	34	40	44	49	59	64
4" Airspace Improvement in TL	10	12	24	30	35	35
4" concrete block + 4" airspace + 4" concrete block with 2" glass fiber in airspace	44	52	68	79	94	99

Sound Pressure Level (dB)						
	125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
Sound in Source Room	83	85	86	84	83	81
Background Noise Level (RC-25)	40	35	30	25	20	15
Required Noise Reduction	43	50	56	59	63	66
Provided Noise Reduction	44	52	68	79	94	99
Acceptable	Yes	Yes	Yes	Yes	Yes	Yes

A wall construction of 4” concrete block, a 4” air space with glass fiber in the air space, and 4” concrete block will provide the necessary TL coefficients to ensure enough noise reduction in the apartment units. The tables used for this study are presented in Appendix I. The next section will look at the cost and schedule impacts from this move and then conclusions will be drawn.

BREADTH STUDY 2 – COST AND SCHEDULE ANALYSIS

Problem Statement

In the original design, the overall schedule for the north building lasts from February 2005-February 2006 which is 12 months in duration. The structural schedule lasts from February 2005-October 2005 which is 8 months in duration. This is a total of 67% of the overall schedule.

The overall cost of the project is $22,084,233 and the total structural cost is $6,751,194. This is 31% of the entire budget.

Goals

- Decrease the overall structural cost based on percentage of the total budget.
- Decrease the schedule duration of the structural system.
- Look at the impacts of the penthouse redesign on the cost and schedule.

Cost Analysis

Detailed takeoffs were completed for the various structural building elements for the revised structural system to determine how the structural system redesign would affect the overall cost of the building. For the sake of cost comparison, the thesis cost values were adjusted for 2004 when this job was bid and construction started. These costs are presented on the next few pages. More detailed structural cost breakdowns can be found in Appendix J.

Item	Amount	Units	Material Unit Cost	Material Cost	Labor Unit Cost	Labor Cost	Total Cost
Columns	134.3	Ton	$838	$112,530	$370	$49,691	$162,221
Beams	480.5	Ton	$1,461	$701,770	$370	$177,785	$879,555
Braces	41.8	Ton	$2,899	$121,178	$370	$15,466	$136,644
Brace Connections	84	EACH	$0	$0	$200	$16,800	$16,800
Shear Connections	1880	EACH	$0	$0	$100	$188,000	$188,000
Shear Studs	11865	EACH	$0	$3,441	$1	$7,712	$11,153
Metal Deck	185147	SF	$4	$740,588	$1	$185,147	$925,735
Concrete (4000 psi)	3143	CY	$85	$267,155	$79	$248,297	$515,452
Welded Wire Fabric	1851.47	CSF	$18	$34,160	$22	$39,807	$73,966
Concrete (5000 psi)	1506	CY	$92	$138,552	$79	$118,974	$257,526
Rebar	54.3	Ton	$230	$12,489	$600	$32,580	$45,069
Fireproofing	50374	SF	$2	$100,748	$2	$100,748	$201,496
New Foundation	Refer to the Foundation Impacts Section of this Report						$901,855
						Subtotal	$4,315,473
						O&P	15%
						Total	$4,962,794

Original Structural Cost: $6,751,194
New Structural Cost: $4,962,794
Total Structural Savings: -$1,788,400 (-26%)

The table above illustrates the significant savings from redesigning the structure in steel instead of post-tension concrete.

Next, the extra cost involved with adding three new apartment units on the penthouse level. These results are summarized below.

Interior Cost Analysis	Number of Units	3
	Average size	2709 SF
	Size Modifier	0.93
	Cost Per Unit	$196,500
	Modified Cost Per Unit	$182,745
	Total Cost	$548,235

As shown, the added cost of the new units is minimal overall. This adds only a 2.5% increase to the overall budget for the building. The potential profit is $4,500,000 which will offset the cost of the new units.

Potential Profit From Added Units	**# of Units Added**	**3**
	Average Cost of Unit	$1,500,000.00
	Total Possible Profit	$4,500,000.00

Cost Comparison:

Original Total Budget: $22,084,233
New Overall Budget: $20,844,068
Total Overall Savings: -$1,240,165 (-6%)

Total New Structural Cost: $4,962,794
Percentage of Overall Budget: 24%

Schedule Analysis

To complete the evaluation of the structural system and penthouse redesign, the scheduling impact of the proposed changes were considered. Small changes in schedule are not extremely critical for this project, as it is a signature building and therefore, quality and appearance mean more than cost. However, drastic scheduling delays would have an impact on the cash flow to the owner due to renting costs of the units and commercial offices. The structural schedule presented below is based off of discussions with the general contractor and with Baltimore Steel, a prominent steel erector in the Washington, D.C./Baltimore Metro Area.

Structural Schedule by Floor	Item	Duration (Days)
	Shop Drawings	40 (total)
	Drawing Review	10 (total)
	Fabrication	80 (total)
	Steel	14
	Embeds	3
	MEP Rough-in	1 (2 for Residential Floors)
	Concrete	2

The durations listed are per floor, except for the upfront durations for shop drawings, drawing review, and fabrication. Fabrication will overlap steel erection, and the shop drawing production and review are standard for any type of building, so no extra upfront time will be added to the critical path. The total duration of a floor on the critical path is 8 days for the beams and columns until the roof. Then, the entire floor is on the critical path. Total duration for the new part of the building is 85 days. This duration, added to the excavation and first floor duration of 60 days (this design reminded fairly constant) gives a total structural duration of 145 days. This is a decrease of the critical path by 15 days

For the penthouse redesign, the time it takes to fit-out the new apartment units must be taken into account for scheduling. The durations were taken from the original schedule for the mechanical ductwork and for the fit-out of the residential floors. This schedule is presented below.

	Item	Duration (Days)
Interior Schedule	Layout	2
	Mechanical Ducts/Shafts	2
	Vertical Plumbing Risers	2
	Vertical Fire Protection Risers	3
	Plumbing Rough-In	5
	Sprinkler Rough-In	5
	Duct Rough-in	15
	Electrical Rough-In	7
	CMU Walls	9
	Mechanical Controls Rough-In	3
	Set Mechanical Equipment	20
	Mechanical and Plumbing Insulation	5
	Metal Stud Framing	2
	Shaftwall Fireproofing	2
	In-Wall Electrical Rough-In	3
	Inspections	1
	Hang Drywall	2
	Finish Drywall	1
	Prime Paint	2
	Point Up	1
	Hang Doors	1
	Set Light Fixtures	5
	Finish Hardware	2
	Mechanical Trim-Out	1
	Electrical Trim-Out	1
	Punch Out	5

Total duration of the original penthouse floor was 115 days. New duration with the moved mechanical equipment and the apartment units is 107 days. This interior work is almost all on the critical path so this reduces by 8 days.

Overall with the critical path is decreased by a total of 23 days (-13%) for a total schedule duration of 252 days. The original schedule was not included in this report due to length but can be viewed upon request for comparison as can the gantt chart that was formulated for the new schedule.

CONCLUSIONS AND RECOMMENDATIONS

Structural Redesign Conclusions

To evaluate the success of the redesign, the results were compared to the original design goals set forth in this report. These goals are relisted below with arguments as to why they were met or not met.

- ✓ Provide a steel structural solution that reduces the overall cost of the building.
    - The new design is a steel braced frame lateral system with composite steel beams for the gravity system.
- ✓ Provide a steel structural solution that does not interfere with the current architectural design due to the fact that the House of Sweden is a signature building for the Swedish Embassy.
    - During the steel redesign, the architecture of the building was continually consulted. The braces were placed where they would not interfere with the layout and the steel column grid followed the original concrete column grid.
- ✓ Reduce the structural erection schedule to complete the building faster than the original concrete design.
    - The original structural schedule duration was 115 days. The new structural schedule adds 12 days to the critical path, but moving most of the mechanical system to the basement removes 8 days from the critical path so the overall critical path extension is 4 days. This is almost a week of extra time that is added to the schedule on paper, but as discussed in the construction management breadth, my switching to steel and using a crawler crane instead of the tower crane for the north building, this will save a month of negotiations with the neighboring property owner.
- ✓ Design for progressive collapse mitigation in the structural steel solution.
    - Solutions were set forth for mitigating progressive collapse with Catenary cables. The new structural system also tries to help increase the blast protection of the columns in the garage.

- ✓ Generate more revenue for the owner with the gain of an extra floor by moving the mechanical system.
    - The entire mechanical system was not able to be used, but three new apartment units were created on the penthouse floor and can generate possible revenue of approximately $4.5 million.

Based on these criteria, the structural redesign was a success. There was an area of issue which is the reduced floor-to-ceiling height. If the restricted building height was not imposed, this would be a better structural solution for this building than the original post-tensioned design, but even with the 8' floor-to-ceiling height, this solution should be considered as a solution.

Penthouse Redesign and Mechanical Equipment Relocation Conclusions

- ✓ Move the mechanical room to the basement parking garage area without losing the required number of parking spaces.
    - No parking spaces were lost in the redesign and
- ✓ Create apartments in the new space created in the penthouse so that more revenue can be generated for the owner by charging a premium for these units.
    - As addressed above, three new apartments were created and can generate possible revenue of $4.5 million.
- ✓ Look at the impacts of this move on the cost and schedule of the project.
    - These impacts are addressed and mitigated and can be reviewed in the Penthouse Redesign Section of this report.

Based on these criteria, the penthouse redesign was a success and can help generate more revenue for the owner with very little impact on the budget or schedule.

Cost and Schedule Analysis Conclusions

- ✓ Decrease the overall structural cost based on percentage of the total budget.
  - The overall structural cost decreased by approximately 44% from $6.8 million to $3.8 million. The overall budget decreased by approximately 11% so the structural savings was able to offset the extra apartment fit-out costs. This savings can also offset the extra cost of the Catenary cables. The owner could even retain a waterproofing consultant to ensure that the details are drawn and installed correctly and there would still be a decrease in the budget.
- ✓ Decrease the schedule duration of the structural system.
  - This criterion is already addressed under the structural redesign conclusions and it was shown that this condition was met.
- ✓ Look at the impacts of the penthouse redesign on the cost and schedule.
  - Moving some of the mechanical equipment to the basement decreases the critical path by 8 days. The cost to add the three new apartments is only about $548,235 which is only a 2.5% increase of the overall budget. So the potential profit from these additional units are able to offset the small additional cost of these units. The additional cost of these units is also offset by the savings from the new steel structural system.

Based on these criteria, the overall project was a success and can help save the owner money without increasing the schedule by a significant amount and even possibly generating more revenue from the extra apartments.

DOCUMENT AND CODE REVIEW

The following documents were either furnished for review or otherwise considered for this report:

- ACI 318-08 *Building Code Requirements for Structural Concrete* published in January 2008 by the American Concrete Institute.
- AISC 13th Edition *Steel Construction Manual* published in December 2005 by the American Institute of Steel Construction, Inc.
- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers.
- Astaneh-Asl, Ph.d., P.E., Abolhassen, Erik A. Madsen, Charles Noble, Roger Jung, David B. McCallen, Ph.D., Matthew S. Hoehler, Wendy Li, and Ricky Hwa. *Use of Catenary Cables to Prevent Progressive Collapse of Buildings*. Tech. no. UCB/CEE-STEEL-2001/02. Berkeley: University of California at Berkeley, 2002.
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- Hill, J. D., and G. Rhodes, eds. *Car Park Designers' Handbook*. Nashville: Thomas Telford Limited, 2005.
- IBC 2006 *International Building Code* published in January 2006 by the International Code Council, Inc.

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- Postma, Mark. *Building Envelope Design Guide - Plazas, Tunnels, Vaults* | *Whole Building Design Guide.* WBDG - The Whole Building Design Guide. 14 Mar. 2006. National Institute of Building Sciences. 16 Mar. 2009 <http://www.wbdg.org/design/env_bg_plaza.php>.

- Ramsey, Charles George, and Harold Reeve Sleeper. *Architectural Graphic Standards*. Ed. Bruce Bassler. 11th ed. New York: Wiley, 2008.

APPENDIX A – PHOTOGRAPHS

PHOTOGRAPHS

Figure 1A: Rendering of the House of Sweden Development

Figure 2A: Night View of the North Building

PHOTOGRAPHS

Figure 3A: Main Entrance of the North Building

Figure 4A: Comparison of the North and South building Exterior Cladding

APPENDIX B – GRAVITY LOAD CALCULATIONS

SNOW AND RAIN LOAD CALCULATIONS

Presented below are table summaries of the snow load calculations performed for the north building. Hand calculations can be reviewed upon request.

Roof Snow Load		
Factor	Design Value	Code Section
Ground Snow Load, P_g	25 psf	Figure 7-1
Exposure Factor, C_e	1.0	Table 7-2
Thermal Factor, C_t	1.0	Table 7-3
Importance Factor, I	1.0	Table 7-4
Flat Roof Snow Load, P_f	17.5 psf	§7.3
Minimum Flat Roof Snow Load P_f	20 psf	§7.3.4

Snow Drift (North Building)		
Factor	Design Value	Code Section
γ	17.25 psf	§7.7.1
h_b	1.16'	
h_c	10.84'	
h_c/h_b	9.34'	
l_u N-S top	148'	
Leeward Drift, h_d N-S top	4.03'	Figure 7-9
l_u N-S lower	11'	
Leeward Drift, h_d N-S lower	1.56'	Figure 7-9
l_u E-W top	162'	
Leeward Drift, h_d E-W top	4.20'	Figure 7-9
l_u E-W lower	11'	
Leeward Drift, h_d E-W lower	1.56'	Figure 7-9
l_u N-S top	11'	
Windward Drift, h_d N-S top	1.17'	Figure 7-9
l_u N-S lower	11'	
Windward Drift, h_d N-S lower	1.17'	Figure 7-9
l_u E-W top	11'	
Windward Drift, h_d E-W top	1.17'	Figure 7-9
l_u E-W lower	11'	
Windward Drift, h_d E-W lower	1.17'	Figure 7-9
$w=4*h_d$, N-S top	16.12'	
$p_d=h_d\gamma$, N-S top	69.5 psf	§7.7
$w=4*h_d$, N-S lower	6.24'	
$p_d=h_d\gamma$, N-S lower	26.9 psf	§7.7
$w=4*h_d$, E-W top	16.8'	
$p_d=h_d\gamma$, E-W top	72.5 psf	§7.7
$w=4*h_d$, E-W lower	6.24'	
$p_d=h_d\gamma$, E-W lower	26.9 psf	§7.7

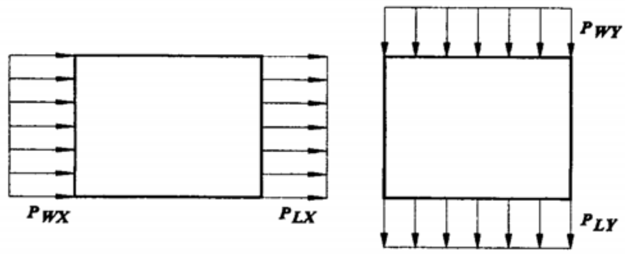
APPENDIX C – LATERAL LOAD CALCULATIONS

WIND LOAD CALCULATIONS

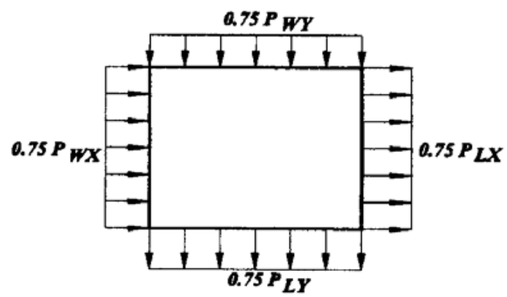
Static Load Cases

The load cases below were considered for wind loading of the structure. They were taken from ASCE7-05 Figure 6-9.

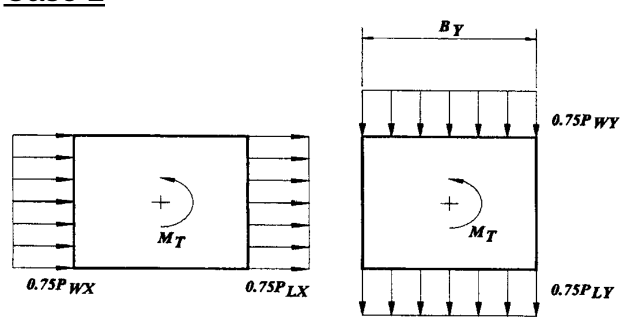
Case 1



Case 3



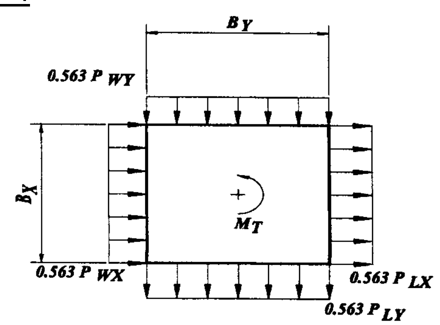
Case 2



$M_T = 0.75\ (P_{WX}+P_{LX})B_X e_X$ $\quad e_X = \pm\ 0.15\ B_X$

$M_T = 0.75\ (P_{WY}+P_{LY})B_Y e_Y$ $\quad e_Y = \pm\ 0.15\ B_Y$

Case 4



$M_T = 0.563\ (P_{WX}+P_{LX})B_X e_X + 0.563\ (P_{WY}+P_{LY})B_Y e_Y$

$e_X = \pm\ 0.15\ B_X \qquad e_Y = \pm\ 0.15\ B_Y$

WIND LOAD CALCULATIONS

Factor (Both Buildings)	Design Value	Reference
K_{zt}	1	§6.5.7
K_d	0.85	Table 6-4
Exposure Category	B	§6.5.6
V	90	Figure 6-1
I	1	Table 6-1

North Building in the N-S Direction

Wind Pressures (North Building N-S)						
Height (ft)	K_z	q_z (psf)	Windward Wall (psf)	Leeward Walls (psf)	Total (psf)	Length in E-W Direction (ft)
77	0.918	16.18	10.54	-3.95	14.49	160
59	0.846	14.91	9.71	-3.95	13.66	190
48.17	0.801	14.12	9.19	-3.95	13.14	206
37.33	0.746	13.15	8.56	-3.95	12.51	206
26.5	0.672	11.84	7.71	-3.95	11.66	206
15.67	0.587	10.35	6.74	-3.95	10.69	206
4.83	0.57	10.05	6.54	-3.95	10.49	162

Gust Factor (North Building N-S)	
Factor	Design Value
g_q	3.4
g_v	3.4
g_r	4.18
$\dot{z}$	46.2
$I_{\dot{z}}$	0.284
$L_{\dot{z}}$	358
Q	0.80
$V_{\dot{z}}$	64.6
N_1	5.4
R_n	0.05
R_h	0.17
R_B	0.07
R_L	0.02
R	0.08
G_f	0.814

North Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
PH	77'-0"	14	0.0	1071
MR	59'-0"	31	14	1805
6	48'-2"	30	44	1442
5	37'-4"	29	74	1069
4	26'-6"	81	103	2143
3	15'-8"	75	184	1178
2	4'-10"	18	259	85
1	-6'-0"	0.0	277	0.0
			V = 277	**ΣM = 8792**

North Building in the E-W Direction

Wind Pressures (North Building E-W)						
Height (ft)	K_z	q_z (psf)	Windward Wall (psf)	Leeward Walls (psf)	Total (psf)	Length in N-S Direction (ft)
77	0.918	16.18	10.57	-6.61	17.18	135.5
59	0.846	14.91	9.74	-6.61	16.35	176.5
48.17	0.801	14.12	9.22	-6.61	15.83	192
37.33	0.746	13.15	8.59	-6.61	15.20	192
26.5	0.672	11.84	7.74	-6.61	14.35	192
15.67	0.587	10.35	6.76	-6.61	13.37	163.5
4.83	0.57	10.05	6.56	-6.61	13.17	163.5

Gust Factor (North Building E-W)	
Factor	Design Value
g_q	3.4
g_v	3.4
g_r	4.18
$\dot{z}$	46.2
$I_{\dot{z}}$	0.28
$L_{\dot{z}}$	358
Q	0.81
$V_{\dot{z}}$	64.6
N_1	5.40
R_n	0.05
R_h	0.17
R_B	0.07
R_L	0.02
R	0.08
G_f	0.817

North Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
PH	77'-0"	14	0.0	1075
MR	59'-0"	34	14	1996
6	48'-2"	33	48	1613
5	37'-4"	35	81	1293
4	26'-6"	97	116	2579
3	15'-8"	90	213	1404
2	4'-10"	22	303	107
1	-6'-0"	0.0	325	0.0
			V = 325	**ΣM = 10069**

Presented above are table summaries of the wind load calculations performed for the north building. Hand calculations were also performed and can be reviewed upon request.

SEISMIC LOAD CALCULATIONS

Presented below are summaries of the seismic load factors and tables summaries of the loads for both the north and south buildings. Hand calculations were also performed as well as manual calculations of story weights and can be reviewed upon request.

Factor **Reference**

Site Class D .... (Table 20.3.1)
S_s = 0.15 .... (Figure 22-1)
S_1 = 0.051 .... (Figure 22-2)
T_L = 8.. (Figure 22-15)
Occupancy Category II
S_{ms} = 0.24.. (Table 11.4.1)
S_{m1} = 0.1224 .... (Table 11.4.2)
S_{DS} = 0.16 .... (eq. 11.4-3)
S_{D1} = 0.0816.. (eq. 11.4-4)
SDC = B
TS = 0.51
North Building T_L = 0.816 s
North Building R = 3 .... (Table 12.2-1)
North Building Moment Frame C_UT_A = 1.63 s
North Building Moment Frame C_s = 0.01669
North Building Normal Weight Concrete Braced Frame C_UT_A = 1.39 s
North Building Normal Weight Concrete Braced Frame C_s = 0.01957
North Building Lightweight Concrete Braced Frame T = 1.244 s (the calculated building period was less that C_UT_A therefore, the calculated period was used for the calculations)
North Building Lightweight Concrete Braced Frame C_s = 0.02186

SEISMIC LOAD DISTRIBUTIONS

Normal Weight Concrete:

Vertical Distribution of Seismic Forces (Moment Frame)					
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
P	83'-0"	1533	58	58	4775
MR	65'-0"	1613	41	99	2679
6	54'-2"	1982	38	137	2061
5	43'-4"	1995	27	164	1169
4	32'-6"	1782	15	179	498
3	21'-8"	1109	5	184	109
2	10'-10"	1098	5	186	18
$\Sigma w_i h_i^k$ =	5,103,746	**$\Sigma F_x = V =$**	186 K	**ΣM =**	11,330 ft-k

Vertical Distribution of Seismic Forces (Braced Frame)					
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
P	83'-0"	1524	64	64	5308
MR	65'-0"	1604	47	111	3069
6	54'-2"	1972	45	156	2414
5	43'-4"	1968	32	188	1394
4	32'-6"	1769	19	207	619
3	21'-8"	1098	7	214	142
2	10'-10"	1076	2	216	26
$\Sigma w_i h_i^k$ =	3,119,645	**$\Sigma F_x = V =$**	216 K	**ΣM =**	12,972 ft-k

SEISMIC LOAD DISTRIBUTIONS

Lightweight Concrete:

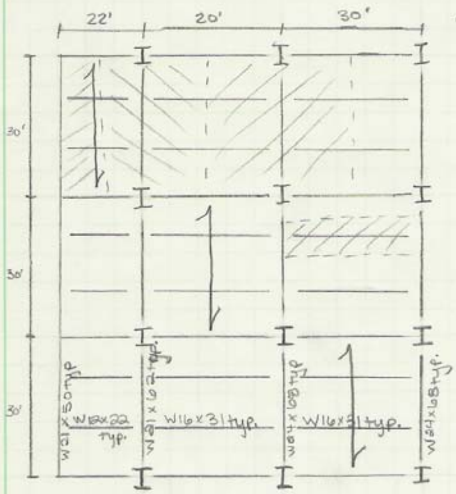
Vertical Distribution of Seismic Forces (Moment Frame)					
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
P	83'-0"	1014	38	39	3280
MR	65'-0"	1094	28	67	1831
6	54'-2"	1336	26	93	1399
5	43'-4"	1328	18	111	784
4	32'-6"	1202	10	121	339
3	21'-8"	778	4	125	77
2	10'-10"	747	1	126	12
$\Sigma w_i h_i^k$ =	3,423,048	**$\Sigma F_x = V$ =**	126 K	**ΣM =**	7,623 ft-k

Vertical Distribution of Seismic Forces (Braced Frame)					
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
P	83'-0"	1006	47	47	3936
MR	65'-0"	1086	36	83	2334
6	54'-2"	1314	33	117	1807
5	43'-4"	1312	24	141	1044
4	32'-6"	1185	14	155	466
3	21'-8"	761	5	160	111
2	10'-10"	727	2	162	19
$\Sigma w_i h_i^k$ =	2,084,780	**$\Sigma F_x = V$ =**	162 K	**ΣM =**	9,718 ft-k

APPENDIX D – Wide-Flange Beam Preliminary Design

WIDE-FLANGE BEAM DESIGN

Composite Steel Deck | Composite Beams



10' beam spacing
Assume fully laterally braced

• Volcraft Steel Roof + Floor Deck Catalog
Loading: 80 psf + 20 psf partitions
Dead load of Structure
$w_u = 1.6(100) = 160$ psf

Materials: $f'c = 3000$ psi
$f_y = 50$ ksi (beams + girders)

2" metal deck, Volcraft
18 gauge

5¼" concrete (3¼" above rib)
LWC - dead weight of deck + concrete = 42 psf

recommended welded wire fabric 6×6 - W1.4 × W1.4

Superimposed live load @ 10' clearspan = 176 psf > 160 psf ✓

Design the Beams

• Interior Beam

Tributary width: worst case 10'

$L_r = 100(.25 + 15/\sqrt{2(30)(10)}) = 86.2$ psf

$w_u = 1.6(82.6) + 1.2(42) = 188$ psf

$w = 188(10) = 1.88$ K/ft

$M_n = \frac{wl^2}{8} = \frac{1.88(30)^2}{8} = 212$ ft-K $\quad V_n = \frac{wl}{2} = \frac{1.88(30)}{2} = 28.2$K

Steel Construction Manual - Table 3-19

assume $a = 1.5"$

$Y_2 = 3\frac{1}{4}" - a/2 = 3\frac{1}{4} - 1.5/2 = 2.5"$

W12×26 PNA location 3 $\phi M_n = 225$ ft-K $> M_n = 212$ ft-K ✓

$\Sigma Q_n = 259$ K

$a = \frac{259}{0.85(3)(90)} = 1.13" < 1.5"$ ✓

Table 3-21

3/4" stud, perpendicular deck, 1 stud weak, 3ksi LWC = 17.2K

# of studs = 259K/17.2K = 16 studs per side = 32 studs

WIDE-FLANGE BEAM DESIGN

Composite Steel Deck | Composite Beams

Table 3-19

W14×26 PNA location 6 $\phi M_n = 220$ ft-K $> M_n = 212$ ft-K ✓
$\Sigma Q_n = 135$ K

$a = \frac{135}{0.85(3)(66)} = 0.80'' < 1.5''$ ✓

# of studs $= \frac{135}{17.2} = 8$ studs per side $= 16$ studs

• Exterior "Cantilevered" Beam
tributary width : 10'
$w = 1.6(100) + 1.2(42) = 210$ psf
$w = 210(10) = 2.10$ K/ft

$M_n = \frac{wl^2}{8} = \frac{2.10(22)^2}{8} = 127$ ft-K $\quad V_n = \frac{wl}{2} = \frac{2.10(22)}{2} = 23.1$ K

Steel Construction Manual - Table 3-19
assume $a = 1.5''$
$Y_2 = 3'1/4'' - a/2 = 3'1/4'' - 1.5/2 = 2.5''$

W12×19 PNA location 7 $\phi M_n = 130$ ft-K $> M_n = 127$ ft-K ✓
$\Sigma Q_n = 69.7$ K

$a = \frac{69.7}{0.85(3)(66)} = 0.41'' < 1.5''$ ✓

Table 3-21
$Q_n = 17.2$ K

# of studs $= 69.7/17.2 = 5$ studs per side $= 10$ studs

WIDE-FLANGE BEAM DESIGN

Composite Steel Deck | Composite Beams

Check Deflections in Beams

- Interior Beam (worst case span)

Table 3-20
$I = 424\ in^4$

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1)(30)^4(12)^3}{384(29000)(424)} = 1.48'' > 1''$ No!

$I_{req} = 629\ in^4$

Try W16×31 PNA location 6 $\phi M_n = 294\ ft\text{-}K > M_n = 212\ ft\text{-}K$
$\Sigma Q_n = 164\ K$

$a = \frac{164}{0.85(3)(66)} = 0.97'' < 1.5''$ ✓

# of studs = 164/17.2 = 10 studs per side
= 20 studs

$\Delta_{LL} = \frac{5(1)(30)^4(12)^3}{384(29000)(638)} = 0.98'' < 1''$ ✓

$\Delta_{max} = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{5(1.42)(30)^4(12)^3}{384(29000)(638)} = 1.40'' < 1.5''$ ✓

- Exterior "Cantilevered" Beam

Table 3-20
$I = 212\ in^4$

$\Delta_{max} = l/360 = 22(12)/(360) = 0.73''$

$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1)(22)^4(12)^3}{384(29000)(212)} = 0.86'' > 0.73''$ No!

$I_{req} = 249\ in^2$

Try W12×22 PNA location 7 $\phi M_n = 153\ ft\text{-}K > M_n = 127\ ft\text{-}K$
$\Sigma Q_n = 81.0$

$a = \frac{81.0}{0.85(3)(66)} = 0.48'' < 1.5''$ ✓

# of studs = 81/17.2 = 5 studs per side
= 10 studs

$\Delta_{LL} = \frac{5(1)(22)^4(12)^3}{384(29000)(253)} = 0.72'' < 0.73''$ ✓

$\Delta_{max} = l/240 = 22(12)/240 = 1.1''$

$\Delta_{D+L} = \frac{5(1.42)(22)^4(12)^3}{384(29000)(253)} = 1.02'' < 1.1''$ ✓

WIDE-FLANGE BEAM DESIGN

Composite Steel Deck | Composite Beams

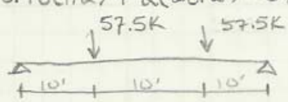
Design the Girders

Interior Girder

Tributary width : worst case 30'

beam self-weight : 31 plf (30) = 0.93

0.93(1.2) + 2(28.2) = 57.5K



V_n = 57.5K

M_n = Pa = 57.5(10) = 575 ft-K

Steel Construction Manual - Table 3-2

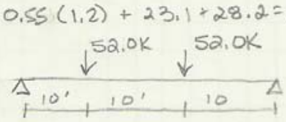
W21×68 ϕM_n = 600 ft-K > M_n = 575 ft-K ✓ ϕV_n = 273K > V_n = 57.5K ✓

Exterior Girder

Tributary width : 11' + 10'

beam self-weight : 31 plf (10) + 22 plf (11) = 0.552K

0.55 (1.2) + 23.1 + 28.2 = 52.0K



V_n = 52.0K

M_n = Pa = 52.0(10) = 520 K

Steel Construction Manual - Table 3-2

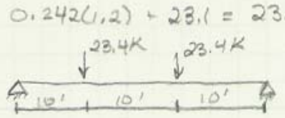
W21×62 ϕM_n = 540 ft-K > M_n = 520K ✓ ϕV_n = 252K > V_n = 52.0 K ✓

"Cantilevered" Girder

Tributary width : 11'

beam self-weight : 22 plf (11) = 0.242 K

0.242(1.2) + 23.1 = 23.4K



V_n = 23.4 K

M_n = Pa = 23.4 (10) = 234 ft-K

Steel Construction Manual - Table 3-2

W18×35 ϕM_n = 249 ft-K > M_n = 234 ft-K ✓ ϕV_n = 159K > V_n = 23.4K ✓

WIDE-FLANGE BEAM DESIGN

Composite Steel Deck | Composite Beams

Check Deflections in Girders.

Interior Girder (worst case 30')

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1480)} = 1.16'' > 1''$ No!

Try a W24x68

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{30(30)^3(12)^3}{28(29000)(1830)} = 0.94'' < 1''$ ✓

$\Delta_{max} = l/240 = 30(12)/(240) = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{43.9(30)^3(12)^3}{28(29000)(1830)} = 1.38'' < 1.5''$ ✓

Exterior Girder

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{21.9(30)^3(12)^3}{28(29000)(1330)} = 0.95'' < 1''$ ✓

$\Delta_{max}: l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{30.5(30)^3(12)^3}{28(29000)(1330)} = 1.32'' < 1.5''$ ✓

"Cantilevered" Girder

$\Delta_{max} = l/360 = 30(12)/360 = 1''$

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{15(30)^3(12)^3}{28(29000)(510)} = 1.69'' > 1''$ No!

Try W21x50

$\Delta_{LL} = \frac{Pl^3}{28EI} = \frac{15(30)^3(12)^3}{28(29000)(984)} = 0.88'' < 1''$ ✓

$\Delta_{max} = l/240 = 30(12)/240 = 1.5''$

$\Delta_{D+L} = \frac{Pl^3}{28EI} = \frac{15.9(30)^3(12)^3}{28(29000)(984)} = 0.93'' < 1.5''$ ✓

APPENDIX E – Castellated Beam Preliminary Design

Exterior Beam – CB 15x19

CASTELLATED BEAM INFORMATION		
Job Name	NWC	
Beam Mark #	Exterior	
Span	20.000	ft
Spac. Left	10.000	ft
Spac. Right	10.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	8.114	in
Duct W x H	4.500 in	7.980 in
Castellated Beam	CB15X19	
Root Beams (T/B)	W10X19	W10X19
d	10.24	10.24
bf	4.02	4.02
tf	0.395	0.395
tw	0.25	0.25
CASTELLATION PARAMETERS:		
e	5.000	in
b	2.500	in
dt	3.000	in
S	15.000	in
dg	14.480	in
phi	59.475	deg
ho	8.480	in
wo	10.000	in

CMC Steel Products

LOADING INFORMATION				
Uniform Distributed Loads				
Live Load	1000	plf	Pre-comp %	0%
Dead Load	660	plf	Pre-comp %	80%
Concentrated Point Loads				
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION			
Concrete & Deck:		**Shear Studs:**	
conc. strength - fc' (psi)	4000	stud dia. (in)	5/8"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	3 1/2	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	6	**Stud Spacing:**	
		N=26,Uniformly Dist.	

RESULTS			WARNINGS
Failure Mode	Interaction	Status	
Bending	0.726	<=1.0 OK!!	
Web Post	0.914	<=1.0 OK!!	
Shear	0.800	<=1.0 OK!!	
Concrete	0.340	<=1.0 OK!!	
Pre-Comp.	0.458	<=1.0 OK!!	
Overall	0.914	<=1.0 OK!!	
Pre-Composite Deflec.	0.361"	=L/665	
Live Load Deflection	0.178"	=L/1351	

EXPAND'D. SXN. PROP'S		
Avg. wt.	19.0	plf
Anet	4.556	in^2
Agross	6.676	in^2
Ix net	201.85	in^4
Ix gross	214.55	in^4
Sx net	27.88	in^3
Sx gross	29.63	in^3
rx min	5.67	in
Iy	4.29	in^4
Sy	2.14	in^3
COMPOSITE SXN. PROP'S		
n	7.89	
beffec.	60.00	in
Actr	26.607	in^2
N.A. ht.	16.63	in Conc.
Itr	698.79	in^4
Ieffec.	698.79	in^3
Sxconc	208.34	in^3
Sxsteel	42.03	in^3
CONSTRUCTION BRIDGING		
End Connection type	Double clip	
Min. No. Of Bridging Rows	0	
Max. Bridging. Spacing (ft)	28	

Interior Beam – CB 21x26

CASTELLATED BEAM INFORMATION		
Job Name	NWC	
Beam Mark #	Interior	
Span	30.000	ft
Spac. Left	10.000	ft
Spac. Right	10.000	ft
Mat. Strength-Fy	50	ksi
Round Duct Diam.	11.184	in
Duct W x H	6.250 in	11.161 in
Castellated Beam	CB21X26	
Root Beams (T/B)	W14X26	W14X26
d	13.91	13.91
bf	5.025	5.025
tf	0.42	0.42
tw	0.255	0.255
CASTELLATION PARAMETERS:		
e	5.500	in
b	4.000	in
dt	3.500	in
S	19.000	in
dg	20.820	in
phi	59.935	deg
ho	13.820	in
wo	13.500	in

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LOADING INFORMATION				
Uniform Distributed Loads				
Live Load	862	plf	Pre-comp %	0%
Dead Load	660	plf	Pre-comp %	80%
Concentrated Point Loads				
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Percent DL (%)	Percent Pre-Comp.
P1	0.00	0.00	0%	0%
P2	0.00	0.00	0%	0%
P3	0.00	0.00	0%	0%
P4	0.00	0.00	0%	0%

COMPOSITE INFORMATION			
Concrete & Deck:		**Shear Studs:**	
conc. strength - fc' (psi)	4000	stud dia. (in)	5/8"
conc. wt. - wc (pcf)	150	stud ht. (in)	5
conc. above deck - tc (in)	3 1/2	studs per rib	1
rib height - hr (in)	2	composite %	100%
rib width - wr (in)	6	**Stud Spacing:**	
		N=32,Uniformly Dist.	

RESULTS			WARNINGS
Failure Mode	Interaction	Status	
Bending	0.886	<=1.0 OK!!	
Web Post	0.955	<=1.0 OK!!	
Shear	0.874	<=1.0 OK!!	
Concrete	0.322	<=1.0 OK!!	
Pre-Comp.	0.544	<=1.0 OK!!	
Overall	0.955	<=1.0 OK!!	
Pre-Composite Deflec.	0.661"	=L/544	
Live Load Deflection	0.333"	=L/1081	

EXPAND'D. SXN. PROP'S		
Avg. wt.	26.0	plf
Anet	5.869	in^2
Agross	9.393	in^2
Ix net	560.22	in^4
Ix gross	616.31	in^4
Sx net	53.82	in^3
Sx gross	59.20	in^3
rx min	8.10	in
Iy	8.90	in^4
Sy	3.54	in^3
COMPOSITE SXN. PROP'S		
n	7.89	
beffec.	90.00	in
Actr	39.910	in^2
N.A. ht.	22.76	In Deck
Itr	1626.88	in^4
Ieffec.	1626.88	in^3
Sxconc	456.37	in^3
Sxsteel	71.49	in^3
CONSTRUCTION BRIDGING		
End Connection type	Double clip	
Min. No. Of Bridging Rows	0	
Max. Bridging. Spacing (ft)	33	

Exterior Girder – CB 21x83

BEAM INFORMATION		
Job Name:	NWC	
Beam Mark #	Exterior	
Span	30.000	ft
Unbraced Length	10.000	ft
Mat. Strength-Fy	50	ksi
Castellated Beam	CB30X83	
Root beam	W21X83	
d	21.4	in
bf	8.36	in
tf	0.835	in
tw	0.515	in
Castellation Parameters:		
e	6.000	in
b	5.500	in
dt	6.000	in
S	23.000	in
dg	30.800	in
phi	59.668	deg
ho	18.800	in
wo	17.000	in

LOADING INFORMATION			
Uniform Distributed Loads			
Live Load	0	plf	
Dead Load	0	plf	
Concentrated Point Loads			
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Perc. DL (%)
P1	40.00	10.00	0%
P2	40.00	20.00	0%
P3	0.00	0.00	0%
P4	0.00	0.00	0%
RESULTS			
Failure Mode	Interaction	Status	
Bending	0.939	<=1.0 OK!!	
Shear	0.580	<=1.0 OK!!	
Web Post	0.630	<=1.0 OK!!	
Overall	**0.939**	**<=1.0 OK!!**	
Live Load Deflection		0.685"	=L/526
Dead Load Deflection		0.016"	=L/22959
WARNINGS			

EXPAND'D. SXN. PROP'S		
Anet	19.281	in^2
Agross	28.963	in^2
Ix net	3910.423	in^4
Ix gross	4195.6	in^4
Sx net	253.924	in^3
Sx gross	272.441	in^3
rx net	14.241	in
rx gross	12.036	in
Iy	81.429	in^4
Sy	19.481	in^3
ry	2.055	in
rT	2.274	in
deffec	28.310	in
CONSTRUCTION BRIDGING		
End Connection type		Shear Tab
Min No. Of Bridging Rows		0
Max. Bridging. Spacing (ft)		43
MAXIMUM PASSABLE DUCTS		
Round (Diam.(in)	Rectangular Width (in) x Height (in)	
14.173	8.000	14.027

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Interior Girder – CB 24x94

BEAM INFORMATION		
Job Name:	NWC	
Beam Mark #	Interior	
Span	30.000	ft
Unbraced Length	10.000	ft
Mat. Strength-Fy	50	ksi
Castellated Beam	CB36X94	
Root beam	W24X94	
d	24.3	in
bf	9.07	in
tf	0.875	in
tw	0.515	in
Castellation Parameters:		
e	7.000	in
b	7.000	in
dt	6.000	in
S	28.000	in
dg	36.600	in
phi	60.356	deg
ho	24.600	in
wo	21.000	in

LOADING INFORMATION			
Uniform Distributed Loads			
Live Load	0	plf	
Dead Load	0	plf	
Concentrated Point Loads			
Load # (#)	Magnitude (kips)	Dist from Lft. End (ft)	Perc. DL (%)
P1	46.00	10.00	0%
P2	46.00	20.00	0%
P3	0.00	0.00	0%
P4	0.00	0.00	0%
RESULTS			
Failure Mode	Interaction	Status	
Bending	0.982	<=1.0 OK!!	
Shear	0.585	<=1.0 OK!!	
Web Post	0.646	<=1.0 OK!!	
Overall	**0.982**	**<=1.0 OK!!**	
Live Load Deflection		0.528"	=L/682
Dead Load Deflection		0.012"	=L/30365
WARNINGS			

EXPAND'D. SXN. PROP'S		
Anet	21.151	in^2
Agross	33.820	in^2
Ix net	6243.032	in^4
Ix gross	6881.9	in^4
Sx net	341.149	in^3
Sx gross	376.062	in^3
rx net	17.180	in
rx gross	14.265	in
Iy	108.929	in^4
Sy	24.020	in^3
ry	2.269	in
rT	2.485	in
deffec	34.228	in
CONSTRUCTION BRIDGING		
End Connection type		Shear Tab
Min No. Of Bridging Rows		0
Max. Bridging. Spacing (ft)		46
MAXIMUM PASSABLE DUCTS		
Round (Diam.(in)	Rectangular Width (in) x Height (in)	
17.751	10.000	17.950

CMC Steel Products

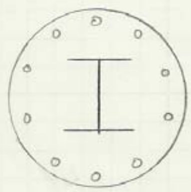
APPENDIX F – GARAGE LEVEL COLUMN DESIGN

GARAGE LEVEL COLUMN DESIGN

Column Design | Reinforced Concrete | Garage Level | 1/3

Critical Column Tributary Area: 30' x 30'

Column Dimensions: 24" ϕ $\quad$ $f'c = 5000$ psi



Thickness of Slab: 10"

150 pcf $\cdot$ 10/12 = 125 psf

Super imposed DL: 12 psf
Live Load: 100 psf

Live Load Reduction:

$$LL = LL_o\left(0.25 + \frac{15}{\sqrt{K_{A}A_T}}\right) = 100\left(0.25 + \frac{15}{\sqrt{4(900)}}\right)$$

$$= 100(0.5 < 0.6) = 100(0.60) = 60\text{ psf}$$

Loads from the Steel:
P = 1037.59 K
M_{major} = 56.75 K-ft
M_{minor} = 10.14 K-ft

P = 1.2(125+12)(900) + 1.6(60)(900) + 1037.59 K = 1272 K
Y-axis: 56.75 K-ft
X-axis: 10.14 K-ft

Use PCA column to investigate column designs

- Start with 6 #9 bars
- Analyze in the X and Y direction - works (see PCA printouts)

By ACI code, 6 bars is the least amount of reinforcing that can be confined by spiral ties

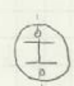
Use spiral reinforcing

- Size and Pitch of spirals based on Table A.14 of Design of Concrete Structures and the ACI code

for f_{yt} = 60,000 psi
$f'c$ = 5000 psi
24" ϕ column

use #4 spiral reinforcing at a 3" pitch

Will the reinforcing fit with the wide flange? -



ϕ	cc	ϕ bar	ϕ spiral	spacing	W shape	
24" -	2(3") -	2(1.128) -	2(0.5) -	4(2") -	14.8"	= -8"

GARAGE LEVEL COLUMN DESIGN

Column Design | Reinforced Concrete | Garage Level | 2/3

Try a 30" column with 10 #8 bars

$$\underbrace{30''}_{\phi} - \underbrace{2(3'')}_{cc} - \underbrace{2(1.0)}_{\phi bar} - \underbrace{2(0.5)}_{\phi spiral} - \underbrace{4(1.5'')}_{spacing} - \underbrace{14.8''}_{W shape} = 0.2'' \checkmark$$

for $f_{yt} = 60{,}000$ psi
$f'_c = 5000$ psi
30" ϕ column

use #4 spiral reinforcing at a 3¼" pitch

- See PCA column printouts for the column interaction diagrams

Transfer of the load from the steel column $P = 1037.59\ K$

3/4" ϕ stud, no deck $Q_n = 26.1\ K$

# of studs needed to transfer the load

$1037.59 / 26.1 = 39.8 = 40$ studs

~10' column, 20 studs per web side, 2 per foot

Check as the Composite Column Section

$A_s = 42.7\,in^2$, $A_{SR} = 10(0.79) = 7.90\,in^2$, $A_c = \pi(15)^2 - 42.7 - 7.90 = 656\,in^2$

$$\frac{42.7}{(\pi(15)^2)} = 0.60 > 0.01 \checkmark$$

$P_o = A_s f_y + A_{sr} F_{yr} + 0.85 f'_c A_c = 5397\ K$

y-axis is weak assume $K = 1$

$I_s = I_y = 677\,in^4$
$I_{SR} = 2(0.79(9.5)^2) + 2(0.79(6)^2) = 199\,in^4$
$I_c = \frac{\pi r^4}{4} = \frac{\pi(15)^4}{4} - 677 - 199 = 38885\,in^4$

$$C_1 = 0.1 + 2\left[\frac{A_s}{(A_c + A_s)}\right] \le 0.3$$

$$= 0.1 + 2\left[\frac{42.7}{42.7 + 656}\right] = 0.22 < 0.3 \checkmark$$

GARAGE LEVEL COLUMN DESIGN

Column Design | Reinforced Concrete | Garage Level | 3/3

$EI_{eff} = 29000(677) + 0.5(29000)(199) + 0.22(3904)(38885)$
$= 55{,}900{,}000 \text{ k-in}$

$P_e = \dfrac{\pi^2(55{,}900{,}000)}{(10.833 \cdot 12)^2} = 32{,}648 \text{ K}$

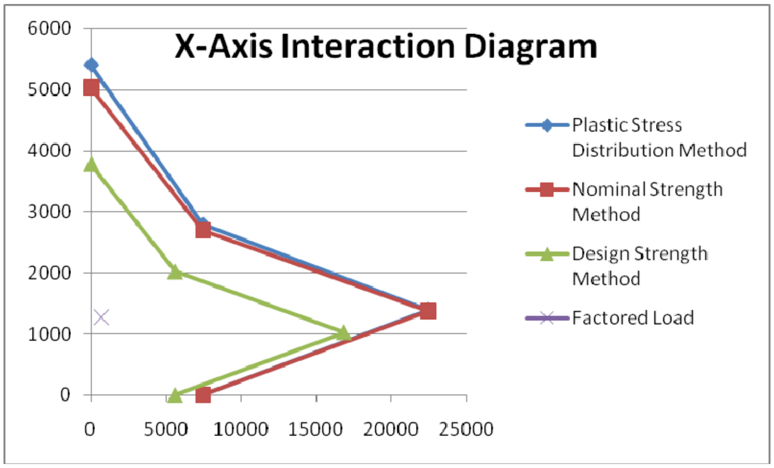
$0.44P_o = 2375\text{K} < P_e$

$P_n = P_o\left[0.658^{P_o/P_e}\right]$
$= 5397\left[0.658^{5397/32648}\right] = 5036\text{K}$

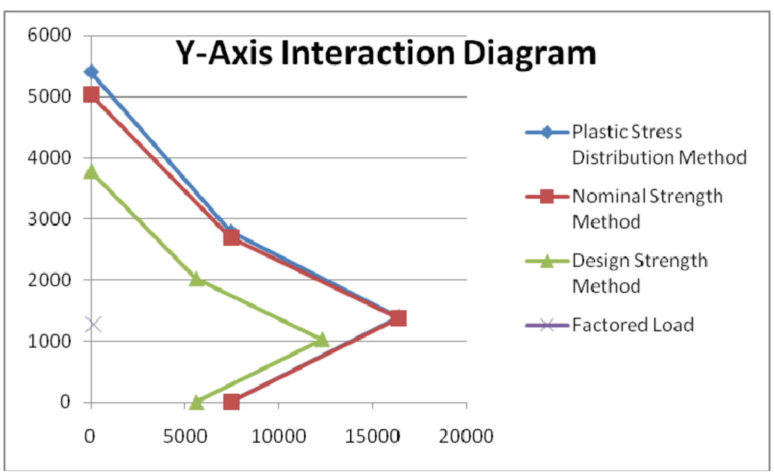
$\phi P_n = 0.75(5036\text{ K}) = 3777\text{K}$

INTERACTION DIAGRAMS

X-Axis	Plastic Stress Distribution Method		Nominal Strength Method		Design Strength Method	
Point	P (K)	M (in-K)	P (K)	M (in-K)	P (K)	M (in-K)
A	5397	0	5036	0	3777	0
C	2788	7448	2690	7448	2018	5586
D	1394	16389	1369	16389	1027	12292
B	0	7448	0	7448	0	5586



Y-Axis	Plastic Stress Distribution Method		Nominal Strength Method		Design Strength Method	
Point	P (K)	M (in-K)	P (K)	M (in-K)	P (K)	M (in-K)
A	5397	0	5036	0	3777	0
C	2788	7448	2690	7448	2018	5586
D	1394	22470	1369	22470	1027	16852
B	0	7448	0	7448	0	5586



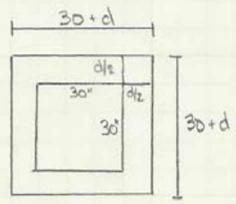
APPENDIX G – FOUNDATION CHECKS

FOUNDATION CHECKS

Foundation

Determine if the north building was the driving factor for the 48" thick mat foundation.

Critical Column: 43N   30" x 30"
$P = 2320\text{ K}$ from the structural drawings



Critical Section is at d/2 from the column

$$b_o = 4(30 + d) = 120 + 4d$$

$$\phi V_c \geq V_u$$

$$\phi V_c = \phi\, 4\sqrt{f'_c}\; b_o d = 0.85(4)\sqrt{4000}\,(120 + 4d)d$$

$$\Rightarrow \frac{0.75(4)\sqrt{4000}\,(120 + 4d)d}{1000} \geq 2320$$

$$(120 + 4d)d \geq 12227.47$$
$$d \approx 42.3"$$

Use d = 43"

with a minimum cover of 3" over the steel reinforcement and 1.27" ϕ steel bars;

Total thickness:

$$h = 43" + 3" + 1.27" = 47.27" \approx 48"$$

North Building Columns were probably the driving force behind the 48" thick mat;

The largest embedded sewer pipes are only 6" and there are no existing conditions sewer pipes to impact the size of the mat foundation.

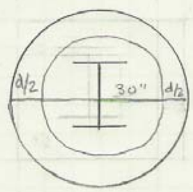
FOUNDATION CHECKS

Foundation

Determining the thickness of the mat slab for a critical braced frame column in the North Building.

Critical Column: Frame 4, column 2 W14×109
$P = 1323.27$ K

Critical Section is at $d/2$ from the column



$b_o = \pi D = \pi(30+d)$

$\phi V_c \geq V_u$

$\phi V_c = \phi\, 4\sqrt{f'_c}\, b_o d = 0.75(4)\sqrt{4000}\,\pi(24+d)\,d$

$$\Rightarrow \frac{0.75(4)\sqrt{4000}\,\pi(30+d)d}{1000} \geq 1323.27$$

$$(30+d)d \geq 2219.97$$
$$d \approx 34.4"$$

Use $d = 35"$

with a minimum cover of 3" over the steel reinforcement and 1.27" ϕ steel bars:

Total thickness:

$h = 35" + 3" + 1.27" = 39.27" \approx 40"$

use 42" for ease of excavation

Original Mat: 48" deep

New Mat: 42" deep under the North Building

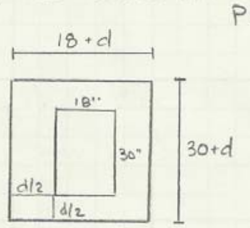
See if mat can be thinned under the South Building

FOUNDATION CHECKS

Foundation

Determine the thickness of the mat slab for a critical column in the South Building.

Critical column: S5 18" x 30"
P=1277 K from the structural drawings



Critical section is at d/2 from the column

$b_o = 2(18+d) + 2(30+d) = 96 + 4d$

$\phi V_c \geq V_u$

$\phi V_c = \phi 4\sqrt{f'c}\, b_o d = 0.85(4)\sqrt{4000}(96+4d)d$

$\Rightarrow \dfrac{0.75(4)\sqrt{4000}(96+4d)d}{1000} \geq 1277$

$(96+4d)d \geq 6730.38$

$d \approx 30.7''$

Use d = 31"

with a minimum cover of 3" over the steel reinforcement and 1.27" ϕ steel bars:

Total thickness:

$h = 31'' + 3'' + 1.27'' = 35.27'' \approx 36''$

Original Mat: 48" deep

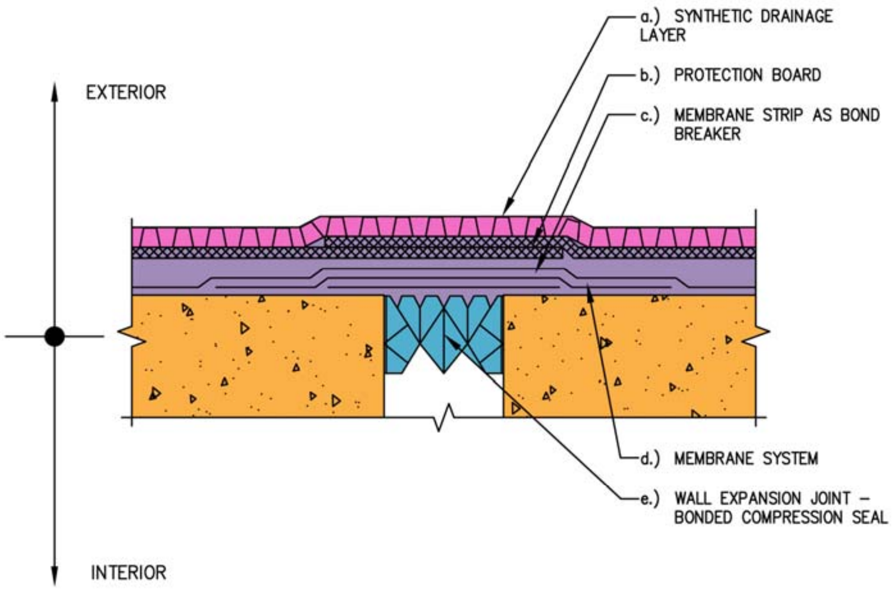
New Mat: 36" deep under the South Building

∴ New mat foundation is 42" deep

$\boxed{\text{Save 6" of excavation}}$

APPENDIX H – WATERPROOFING

FOUNDATION WALL DETAILS



General Notes:

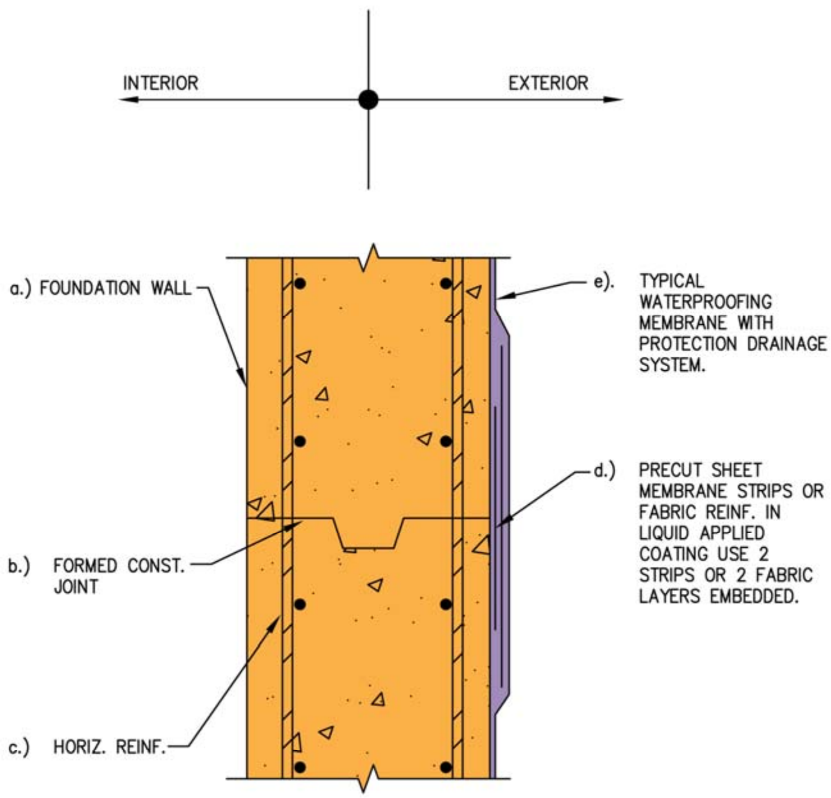
1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

a. Primary flow path for invasive surface water.
b. Serves as a base for geogrid. Protects membrane.
c. Provides expansion capability at joint opening.
d. Prevents water leaking through wall and into occupied spaces.
e. Absorbs expansion/contraction of concrete wall.

FOUNDATION WALL - WALL EXPANSION JOINT

FOUNDATION WALL DETAILS



FOUNDATION WALL PARTIAL PLAN

General Notes:

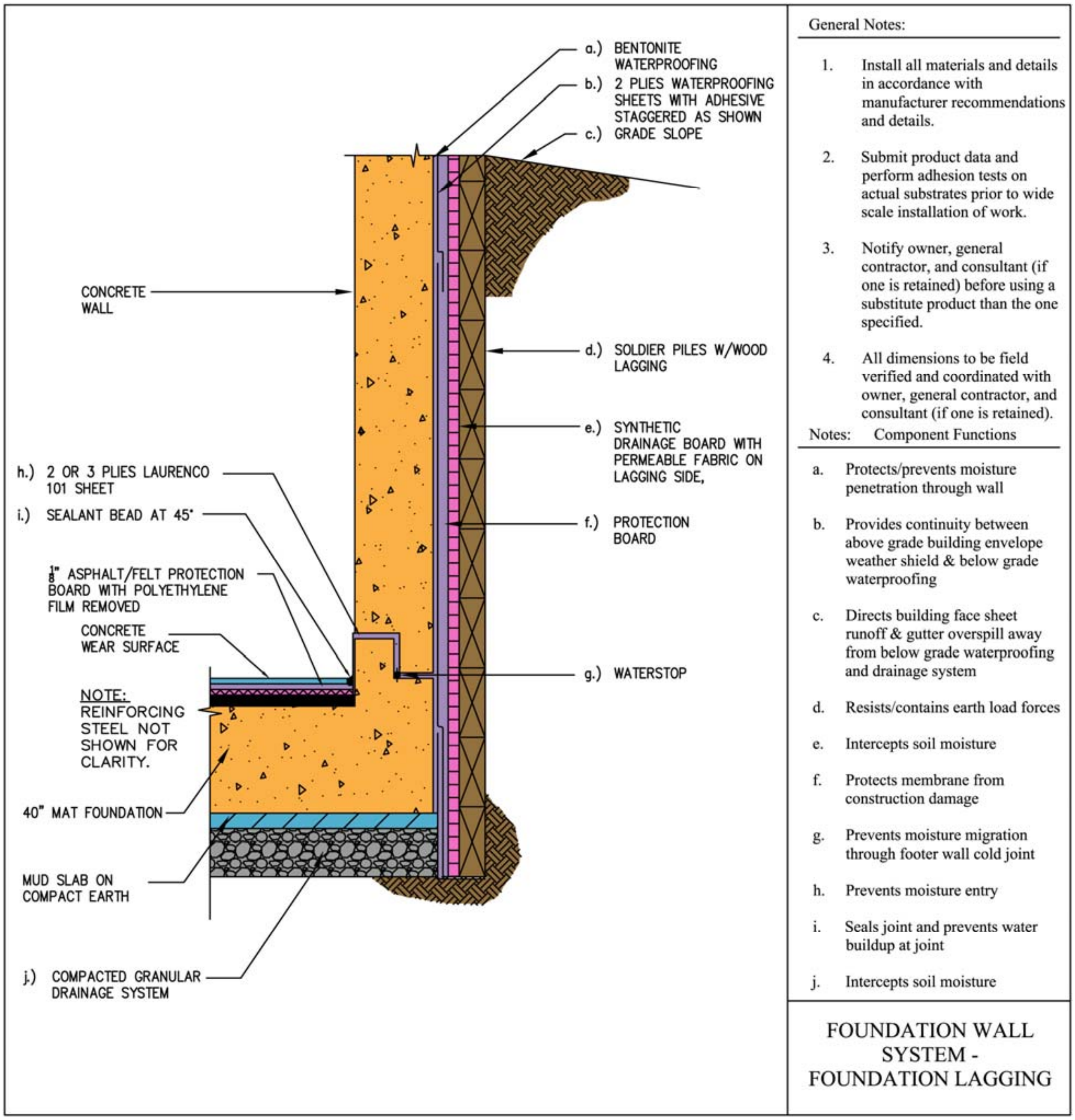
1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

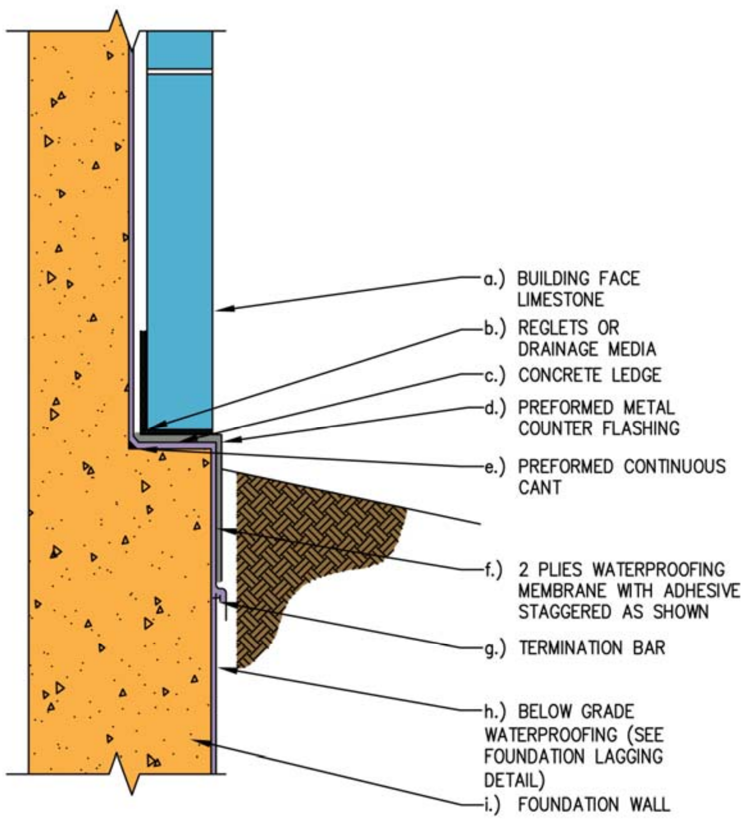
a. Load transfer
b. Facilitates const. operations.
c. Provide reinforcing continuity.
d. Prevents leakage into building at wall const. joint.
e. Prevents leaking thru wall cracks.

FOUNDATION WALL - CONSTRUCTION JOINT

FOUNDATION WALL DETAILS



FOUNDATION WALL DETAILS



General Notes:

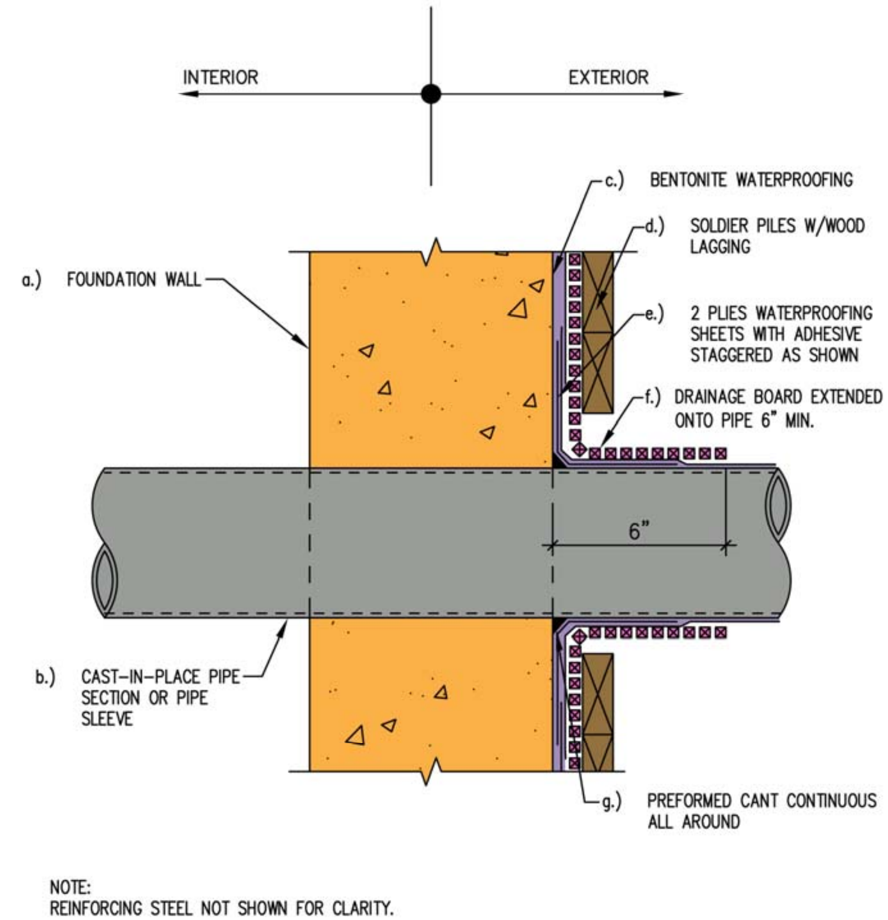
1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

a. Exterior building cladding
b. Allows cavity wall drainage
c. Supports facade masonry
d. Protects membrane from physical damage
e. Reduces stress concentration for membrane flashing
f. Prevents upper wall leaks
g. Prevents wall waterproofing pull off
h. Prevents wall leaks
i. Load transfer

FOUNDATION WALL - FACADE TRANSITION

FOUNDATION WALL DETAILS



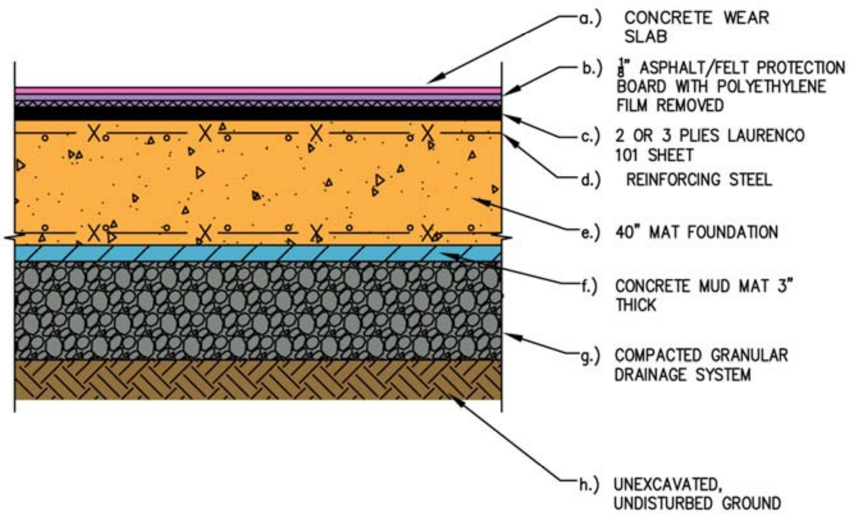
General Notes:

1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes:	Component Functions
a.	Resists earth loads transfers bldg loads to footer
b.	Conveys fluids to occupied space
c.	Protects/prevents moisture penetration through wall
d.	Resists/contains earth load forces
e.	Protects joint/wall against water leakage
f.	Remove vagrant water to subdrain system
g.	Reduces stress concentration for membrane flashing

FOUNDATION WALL - PIPE PENETRATION DETAIL

SLAB DETAILS



General Notes:

1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

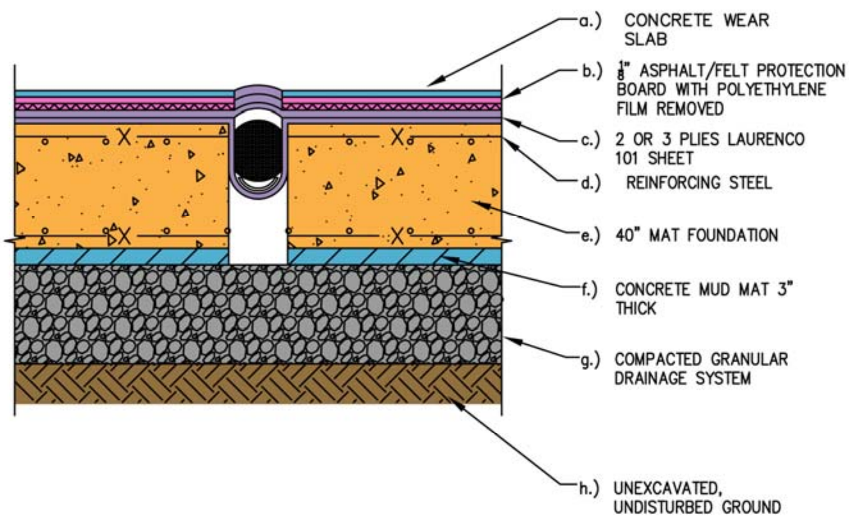
a. Completes aesthetic affect.
b. Protects membrane
c. Prevents moisture entry
d. Provides structural support to wear surface
e. Provides structural support to wear surface
f. Provides uniform slab base
g. Promotes water flow to subdrain pipes or sumps
h. Slab system foundation material

BELOW GRADE MAT - WATERPROOF SYSTEM

SLAB DETAILS

JOINT INSTALLATION:

1. COAT BOTH SIDES OF CONSTRUCTION JOINT WITH ADHESIVE
2. LOOP IN 1 PLY OF WATERPROOFING SHEET INTO JOINT
3. COAT WITH ADHESIVE
4. INSERT NEOPRENE RUBBER ROD $1\frac{1}{2}$ TIMES THE SIZE OF THE JOINT, SQUEEZE TO INSERT AND USE WET ADHESIVE
5. COAT WITH ADHESIVE
6. INSTALL FLASHING OVER JOINT
7. COAT WITH ADHESIVE
8. APPLY CONTINUOUS SHEETS OF WATERPROOFING OVER JOINT
9. INSTALL SEALANT OVER WATERPROOFING TO PROVIDE WEARING SURFACE



General Notes:

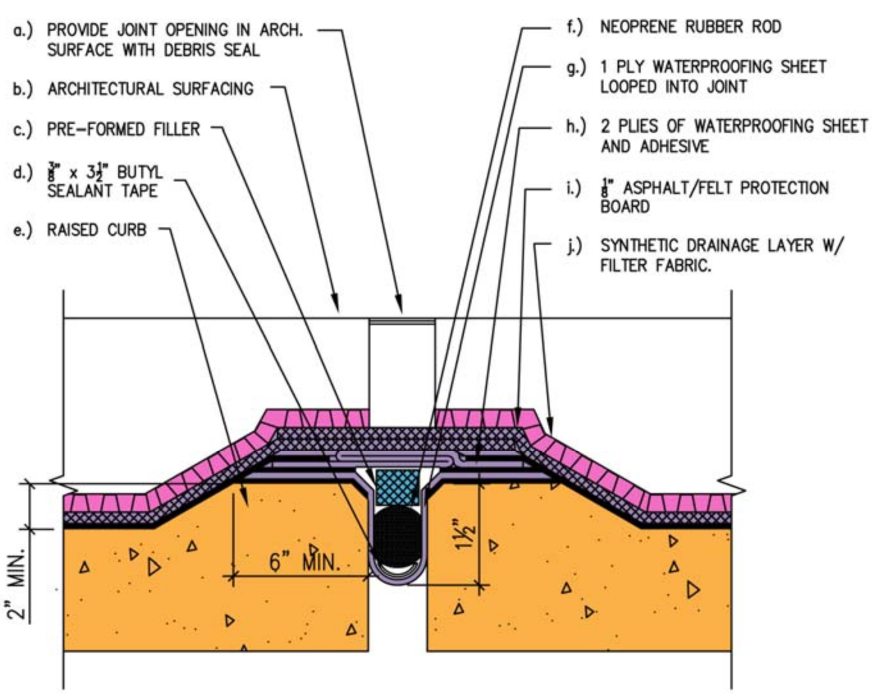
1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

a. Completes aesthetic affect.
b. Protects membrane
c. Prevents moisture entry
d. Provides structural support to wear surface
e. Provides structural support to wear surface
f. Provides uniform slab base
g. Promotes water flow to subdrain pipes or sumps
h. Slab system foundation material

BELOW GRADE MAT - WATERPROOF SYSTEM

PLAZA DETAILS



General Notes:

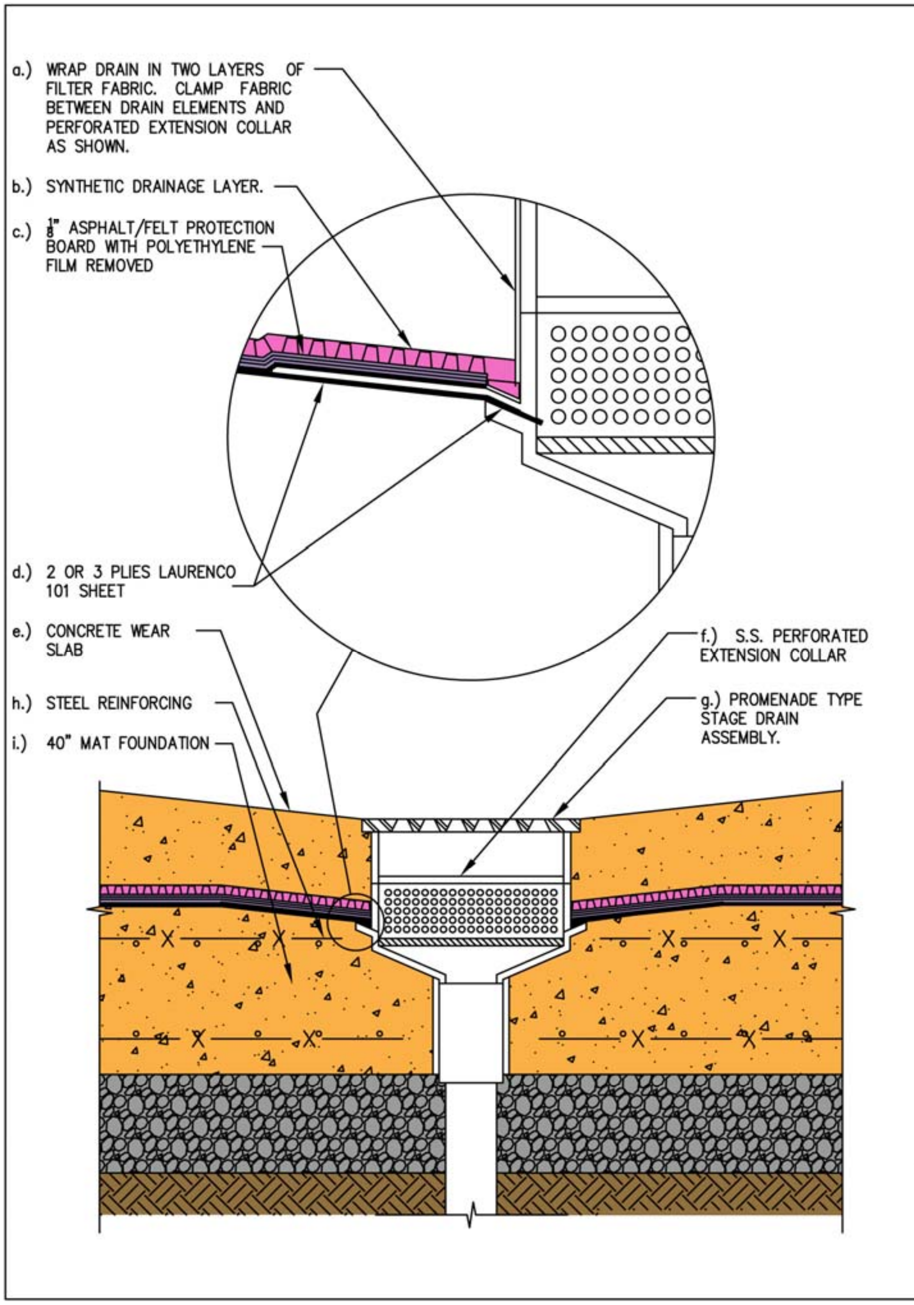
1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

a. Opening to accommodate topping slab movement
b. Finished surfacing
c. Ensure that waterproofing layers do not allow water to collect
d. Keep rubber rod in place
e. Raised concrete at expansion joint to keep water away from joint
f. Provides expansion capability at joint opening.
g. Provide back-up moisture protection
h. Provides moisture protection to occupied spaces. Also provides primary drainage to drainage basins
i. Protection board
j. Provides flow path to drainage basins

EXPANSION JOINT - PLAZA AREA

PLAZA DETAILS



General Notes:

1. Install all materials and details in accordance with manufacturer recommendations and details.
2. Submit product data and perform adhesion tests on actual substrates prior to wide scale installation of work.
3. Notify owner, general contractor, and consultant (if one is retained) before using a substitute product than the one specified.
4. All dimensions to be field verified and coordinated with owner, general contractor, and consultant (if one is retained).

Notes: Component Functions

a. Prevents soil and backfill from entering drain and causing settlement/voids around drain
b. Primary flow path for invasive surface water
c. Protects membrane
d. Provides moisture protection to occupied spaces
e. Finished surface
f. Provides effective drainage into drain
g. Provides surface and subsurface moisture removal
h. Provides structural support to wear surface
i. Provides structural support to wear surface

FLOOR DRAIN -
PLAZA AREA

WATERPROOFING CHECKLIST

1. **Hire a building envelop consultant to review the waterproofing details.** On most projects, architects normally deal with waterproofing details, but there is no one in the field checking the work. Most waterproofing details in construction documents are just standard details that have not been tailored for specific jobs. A consultant can perform a document review of the details and point out problem areas and this service normally only costs around $5,000. This may seem costly, but it can save time and money later in the project when waterproofing details either need to be clarified, or are installed incorrectly and need to be taken out and reinstalled.

2. **Hire a consultant to oversee correct installation of the waterproofing during the construction of the building.** This is an expansive endeavor, but it is cheaper than hiring the consultant a few years after the final fit-out of the building when leaks start to occur and all the waterproofing has to be ripped out and reinstalled.

3. **Hire experienced construction firms.** There is an organization called the National Organization of Waterproofing and Structural Repair Contractors. This organization is a professional trade association whose members are required to uphold a strict standard of practice and cannon of ethics. These documents can be reviewed on their website http://nawsrc.org. It is also possible to locate members and suppliers in the area of the construction project who are required to do the best possible job of waterproofing the construction job.

4. **Ensure that the waterproofing is continuous around the entire building.** This is one of the most important details. Even a small tear in the waterproofing can allow enough water to penetrate to the interior of the building that an identifiable leak can be found. Ideally, there should be no penetrations in the waterproofing, but this is impossible as windows and doors are a necessary part of design. Unnecessary penetrations as part of installation should be avoided. These include nail holes, tears in the waterproofing sheets, or outlet penetrations to name a few. If these occur, a new sheet of waterproofing should be installed, or at the very least, they should be repaired with mastic.

5. **Create a mock-up of the system and/or perform tests during construction.** It is possible to hire testing firms to come in and test curtain walls, brick panels, and other water sensitive areas to find trouble areas before the fit-out of the building when they will become harder and more costly to repair. These tests can cost approximately $10,000/day, but they will again be cheaper than trying to fix the problem areas later during the lifetime of the building when leaks occur.

6. **Perform regular building maintenance.** Replacing all the sealant on a building every 5 years is cheaper than removing all the curtain walls, ripping out the steel that is now corroded because of water infiltration, and then replacing all the steel and the curtain walls every 10 years.

APPENDIX I –ACOUSTICS STUDY

ACOUSTICS STUDY

TL DATA FOR COMMON BUILDING ELEMENTS*

Building Construction	Transmission Loss (dB) 125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz	STC Rating	IIC Rating†
Walls[2-6]‡								
Monolithic:								
1. 3/8-in plywood (1 lb/ft²)	14	18	22	20	21	26	22	
2. 26-gauge sheet metal (1.5 lb/ft²)	12	14	15	21	21	25	20	
3. 1/2-in gypsum board (2 lb/ft²)	15	20	25	31	33	27	28	
4. 2 layers 1/2-in gypsum board, laminated with joint compound (4 lb/ft²)	19	26	30	32	29	37	31	
5. 1/32-in sheet lead (2 lb/ft²)	15	21	27	33	39	45	31	
6. Glass-fiber roof fabric (37.5 oz/yd²)	6	9	11	16	20	25	16	
Interior:								
7. 2 by 4 wood studs 16 in oc with 1/2-in gypsum board both sides (5 lb/ft²)	17	31	33	40	38	36	33	
8. Construction no. 7 with 2-in glass-fiber insulation in cavity	15	30	34	44	46	41	37	
9. 2 by 4 staggered wood studs 16 in oc each side with 1/2-in gypsum board both sides (8 lb/ft²)	23	28	39	46	54	44	39	
10. Construction no. 9 with 2 1/4-in glass-fiber insulation in cavity	29	38	45	52	58	50	48	
11. 2 by 4 wood studs 16 in oc with 5/8-in gypsum board both sides, one side screwed to resilient channels. 3-in glass-fiber insulation in cavity (7 lb/ft²)	32	42	52	58	53	54	52	
12. Double row of 2 by 4 wood studs 16 in oc with 3/8-in gypsum board on both sides of construction. 9-in glass-fiber insulation in cavity (4 lb/ft²)	31	44	55	62	67	65	54	
13. 6-in dense concrete block, 3 cells, painted (34 lb/ft²)	37	36	42	49	55	58	45	
14. 8-in lightweight concrete block, 3 cells, painted (38 lb/ft²)	34	40	44	49	59	64	49	
15. Construction no. 14 with expanded mineral loose fill in cells	34	40	46	52	60	66	51	
16. 6-in lightweight concrete block with 1/2-in gypsum board supported by resilient metal channels on one side, other side painted (26 lb/ft²)	35	42	50	64	67	65	53	
17. 2 1/2-in steel channel studs 24 in oc with 5/8-in gypsum board both sides (6 lb/ft²)	22	27	43	47	37	46	39	
18. Construction no. 17 with 2-in glass-fiber insulation in cavity	26	41	52	54	45	51	45	
19. 3 5/8-in steel channel studs 16 in oc with 1/2-in gypsum board both sides (5 lb/ft²)	26	36	43	51	48	43	43	
20. Construction no. 19 with 3-in mineral-fiber insulation in cavity	28	45	54	55	47	54	48	
21. 2 1/2-in steel channel studs 24 in oc with two layers 5/8-in gypsum board one side, one layer other side (8 lb/ft²)	28	31	46	51	53	47	44	
22. Construction no. 21 with 2-in glass-fiber insulation in cavity	31	43	55	58	61	51	51	
23. 3 5/8-in steel channel studs 24 in oc with two layers 5/8-in gypsum board both sides (11 lb/ft²)	34	41	51	54	46	52	48	
24. Construction no. 23 with 3-in mineral-fiber insulation in cavity	38	52	59	60	56	62	57	

ACOUSTICS STUDY

Airspace (in)	Improvement in TL (dB) 125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
2	5	7	19	25	30	30
4	10	12	24	30	35	35

Type of Space (and Listening Requirements)	Preferred Range of Noise Criteria	Equivalent dBA Level *
Concert halls, opera houses, broadcasting and recording studios, large auditoriums, large churches, recital halls (for excellent listening conditions)	< NC-20	< 30
Small auditoriums, theaters, music practice rooms, large meeting rooms, teleconference rooms, audiovisual facilities, large conference rooms, executive offices, small churches, courtrooms, chapels (for very good listening conditions)	NC-20 to NC-30	30 to 38
Bedrooms, sleeping quarters, hospitals, residences, apartments, hotels, motels (for sleeping, resting, relaxing)	NC-25 to NC-35	34 to 42
Private or semiprivate offices, small conference rooms, classrooms, libraries (for good listening conditions)	NC-30 to NC-35	38 to 42
Large offices, reception areas, retail shops and stores, cafeterias, restaurants, gymnasiums (for moderately good listening conditions)	NC-35 to NC-40	42 to 47
Lobbies, laboratory work spaces, drafting and engineering rooms, general secretarial areas, maintenance shops such as for electrical equipment (for fair listening conditions)	NC-40 to NC-45	47 to 52
Kitchens, laundries, school and industrial shops, computer equipment rooms (for moderately fair listening conditions)	NC-45 to NC-55	52 to 61

* Do not use A-weighted sound levels (dBA) for specification purposes. Spectrum shapes and noise characteristics can vary widely for background noises with identical A-weighted sound levels (see Chap. 1).

Curve	Sound Pressure Level (dB) 125 Hz	250 Hz	500 Hz	1000 Hz	2000 Hz	4000 Hz
RC-50	65	60	55	50	45	40
RC-45	60	55	50	45	40	35
RC-40	55	50	45	40	35	30
RC-35	50	45	40	35	30	25
RC-30	45	40	35	30	25	20
RC-25	40	35	30	25	20	15
Threshold*	22	13	8	5	3	..

*Approximate threshold of hearing for continuous noise by listeners with normal hearing.

APPENDIX J –SUPPLEMENTAL COST INFORMATION

STRUCTURAL COST INFORMATION

Column Takeoff	Column	Length (ft)	Cost/ft	Cost
	W14x43	1800.50	$29.90	$53,834.95
	W14x61	715.00	$40.83	$29,193.45
	W14x74	335.90	$47.52	$15,961.97
	W14x82	216.60	$52.25	$11,317.35
	W14x90	260.00	$58.58	$15,230.80
	W14x109	162.50	$71.06	$11,547.25
	W14x120	65.00	$77.76	$5,054.40
	W14x132	65.00	$85.04	$5,527.60
	W14x145	32.50	$112.75	$3,664.38
			Total Cost:	$151,332.14
			Adjusted Cost:	$112,529.03

Beam Takeoff	Beam	Length (ft)	Cost/ft	Cost
	CB12x15	6863.50	$32.77	$224,916.90
	CB15x19	5383.45	$24.57	$132,271.37
	CB18x26	2592.00	$26.00	$67,392.00
	CB27x46	6671.07	$42.23	$281,719.29
	CB27x60	2070.14	$51.03	$105,639.24
	CB27x76	877.00	$65.83	$57,732.91
	CB27x97	379.59	$81.97	$31,114.99
	CB27x119	160.55	$98.35	$15,790.09
	CB36x162	139.50	$125.81	$17,550.50
	CB50x221	50.00	$193.45	$9,672.50
			Total Cost:	$943,799.78
			Adjusted Cost:	$701,799.84

STRUCTURAL COST INFORMATION

Brace Takeoff

Brace	Length (ft)	Cost/ft	Cost
HSS7.5x0.5	865.30	$75.46	$65,295.54
HSS10.0x0.625	207.50	$114.30	$23,717.25
		Total Cost:	$89,012.79
		Adjusted Cost:	$66,189.00

Steel Deck Takeoff

Floor	Area (ft^2)	Cost/ft^2	Cost
Roof	16269	$1.10	$17,895.90
Penthouse	25914	$1.10	$28,505.40
Sixth	32427	$1.10	$35,669.70
Fifth	32427	$1.10	$35,669.70
Fourth	32427	$1.10	$35,669.70
Third	28646	$1.10	$31,510.60
Second	17037	$1.10	$18,740.70
		Total Cost:	$185,765.80
		Adjusted Cost:	$138,133.54

Concrete Takeoff

Floor	Area (ft^2)	Thickness (ft)	Volume (yd^3)	Cost/yd^3	Cost
Roof	16269	0.46	276	$85.00	$23,474.56
Penthouse	25914	0.46	440	$85.00	$37,391.34
Sixth	32427	0.46	550	$85.00	$46,788.96
Fifth	32427	0.46	550	$85.00	$46,788.96
Fourth	32427	0.46	550	$85.00	$46,788.96
Third	28646	0.46	486	$85.00	$41,333.35
Second	17037	0.46	289	$85.00	$24,582.71
				Total Cost:	$267,148.83