

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

April 7, 2009

Prepared by:
Kimberlee McKitish – Structural Option

Prepared for:
Dr. Andres Lepage



HOUSE OF SWEDEN

Georgetown,
Washington, D.C.



Kimberlee McKitish

Structures Option

Building Statistics

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

Architecture

- o Built on a single foundation with two separate towers rising out of the site
- o 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
- o The north building is clad in glass and metal paneling with a light stone base
- o The roofing is rigid insulation topped with ballast over monolithic EDPM waterproofing membrane

Mechanical and Electrical Systems

- o 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
- o 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

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

Structural System

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

Special Systems

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



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

Pictures and renderings published with permission from Wingardh Arkitektkontor AB (Designer) and VOA Incorporated (Architect of Record).

TABLE OF CONTENTS

Executive Summary	4
Acknowledgements	5
Introduction	6
Background	6
Building Overview	7
Architecture	7
Building Envelope	7
Mechanical System	8
Electrical System	8
Lighting System	8
Telecommunications System	9
Special Systems	9
Structural System Overview	10
Foundation	10
Framing System	10
Lateral System	11
Depth Study - Structural System Redesign	13
Proposal	13
Problem Statement	13
Proposed Solution	13
Implications of Redesign	14
Gravity Loads	14
Deflection Criteria	14
Floor Dead Loads	14
Roof Live Loads	14
Floor Live Loads	15
Lateral Loads	15

Deflection Criteria	15
Wind Loads	15
Seismic Loads	18
Load Combinations.....	20
Design Goals	20
Design Criteria.....	21
Evolution of Design.....	22
Computer Analysis	24
The RAM Modeling Process – MAE Requirement	24
Moment Frame Lateral Force Resisting System Cases.....	27
Moment Frame Design Check	28
Braced Frame Lateral Force Resisting System Cases	31
Braced Frame Design Check.....	32
Material Cost Evaluation.....	35
Floor Plans and Brace Elevations.....	36
Cantilever Solution	40
Implications of Redesign	42
Garage Level Column Design.....	42
Foundation Impacts	43
Blast Protection and Progressive Collapse Analysis.....	47
Breadth Study 1: Penthouse Redesign	50
Problem Statement.....	50
Goals	50
Zoning Impact.....	50
Parking Study	51
Waterproofing	54
Penthouse Redesign	54
Acoustics Study	55

Breadth Study 2: Cost and Schedule Analysis	56
Problem Statement.....	56
Goals.....	56
Cost Analysis.....	56
Cost Comparison.....	58
Schedule Analysis.....	58
Conclusions and Recommendations	61
Document and Code Review	64
Appendix A – Photographs	66
Appendix B – Gravity Load Calculations	69
Appendix C – Lateral Load Calculations	71
Appendix D – Wide-Flange Beam Preliminary Design	78
Appendix E – Castellated Beam Preliminary Design	84
Appendix F – Garage Level Column Design	87
Appendix G – Foundation Checks	92
Appendix H – Waterproofing	96
Appendix I – Acoustics Study	107
Appendix J – Supplement Cost Information	110

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 RSMMeans 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.

Oncore Construction:

Cesar Casanova
Tony Horos
Ray Sowers

Armada Hoffler:

Brad Hunter

Tadjer Cohen Edelson:

Yehuda Nordman

VOA:

Warren Wick
Doug Jeoung

CMC:

Robert Anderson

The Pennsylvania State University:

Dr. Louis Geschwindner
Dr. Linda Hanagan
Robert Holland
Dr. Andres Lepage
M. Kevin Parfitt

A special thanks to family, friends, and classmates for their much needed support including: Katy Andaloro, Jess Laurito, Kristen Lechner, Nicki Lucas, Ryan Solnosky, Larry Warner

House of Sweden

Structural System and Existing Conditions Report

2900 K St. NW
Washington, DC 20007

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

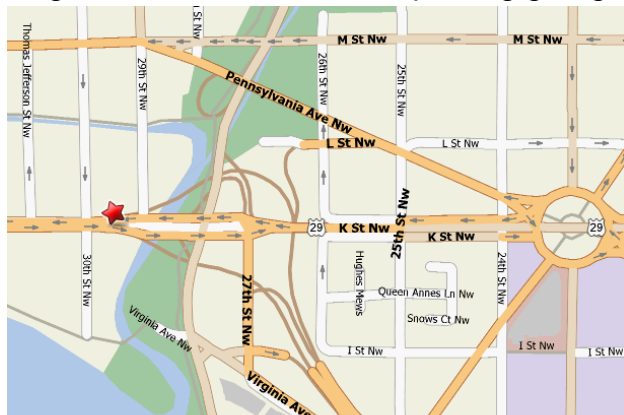


Figure 1: Site Location of the House of Sweden

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.

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.

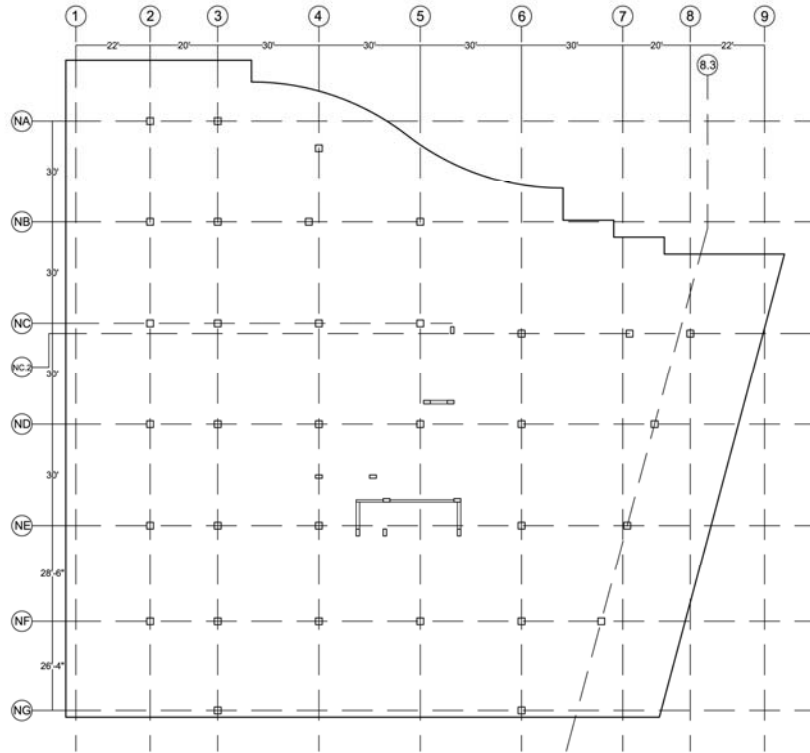


Figure 2: Typical North Building Column and Shear Wall Layout

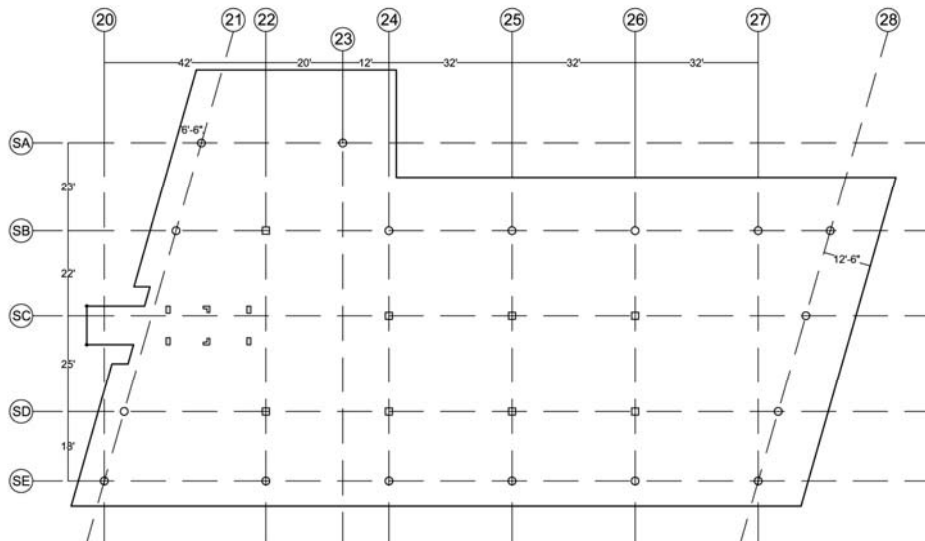


Figure 3: Typical South Building Column Layout

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 13 th Edition
Glass Curtain Wall	6.4 psf	Glass Association of North America
Studs and Batt Insulation	4 psf	AISC 13 th 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

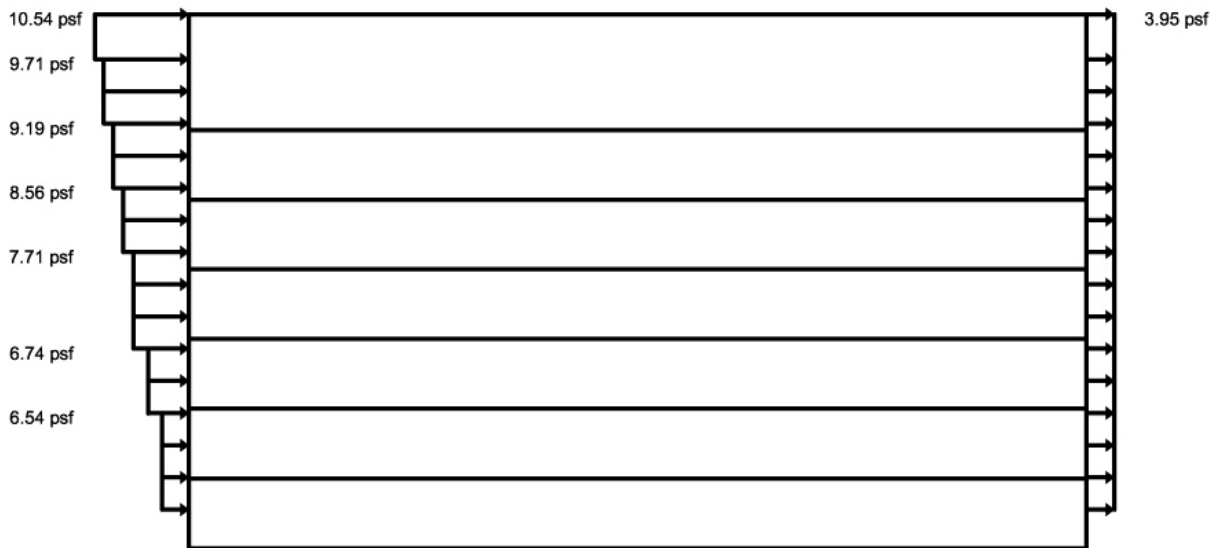


Figure 4: North Building Wind Pressure Diagram in the North – South Direction



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: $\Sigma M = 8,792$ ft-K
- E-W Direction Base Shear: $V = 325$ K (Controls)
- E-W Direction Moment: $\Sigma M = 10,069$ 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 F_x (K)	Story Shear V_x (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	$\Sigma M =$	12,972 ft-k



Figure 6: NWC Braced Frame Building Seismic Force Diagram

North Building Seismic Load Summary:

Base Shear: $V = 216 \text{ K}$

Moment: $\Sigma M = 12,972 \text{ ft-K}$

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 = $1.6 \cdot 325 = 520 \text{ K}$

Moment = $1.6 \cdot 10,069 = 16,110 \text{ ft-K}$

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

Seismic Loads:

Shear = $1.0 \cdot 216 = 216 \text{ K}$

Moment = $1.0 \cdot 12,972 = 12,972 \text{ ft-K}$

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

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. $1.2(D+F+T) + 1.6(L+H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
4. $1.2D + 1.6W + L + 0.5(L_r \text{ or } S \text{ or } R)$
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 $C_u T_A$ 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 $p=1.0$.
- 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.

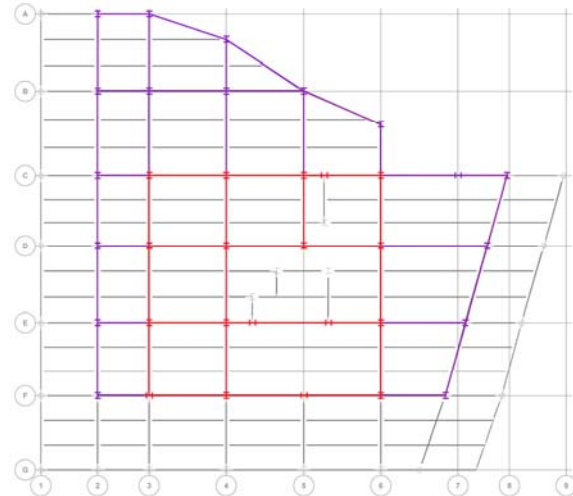


Figure 7: Location of Moment Frames

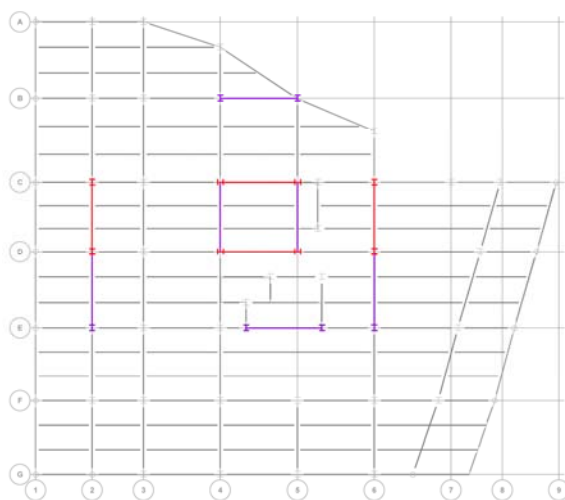


Figure 8: Location of Braced Frames

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.

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:

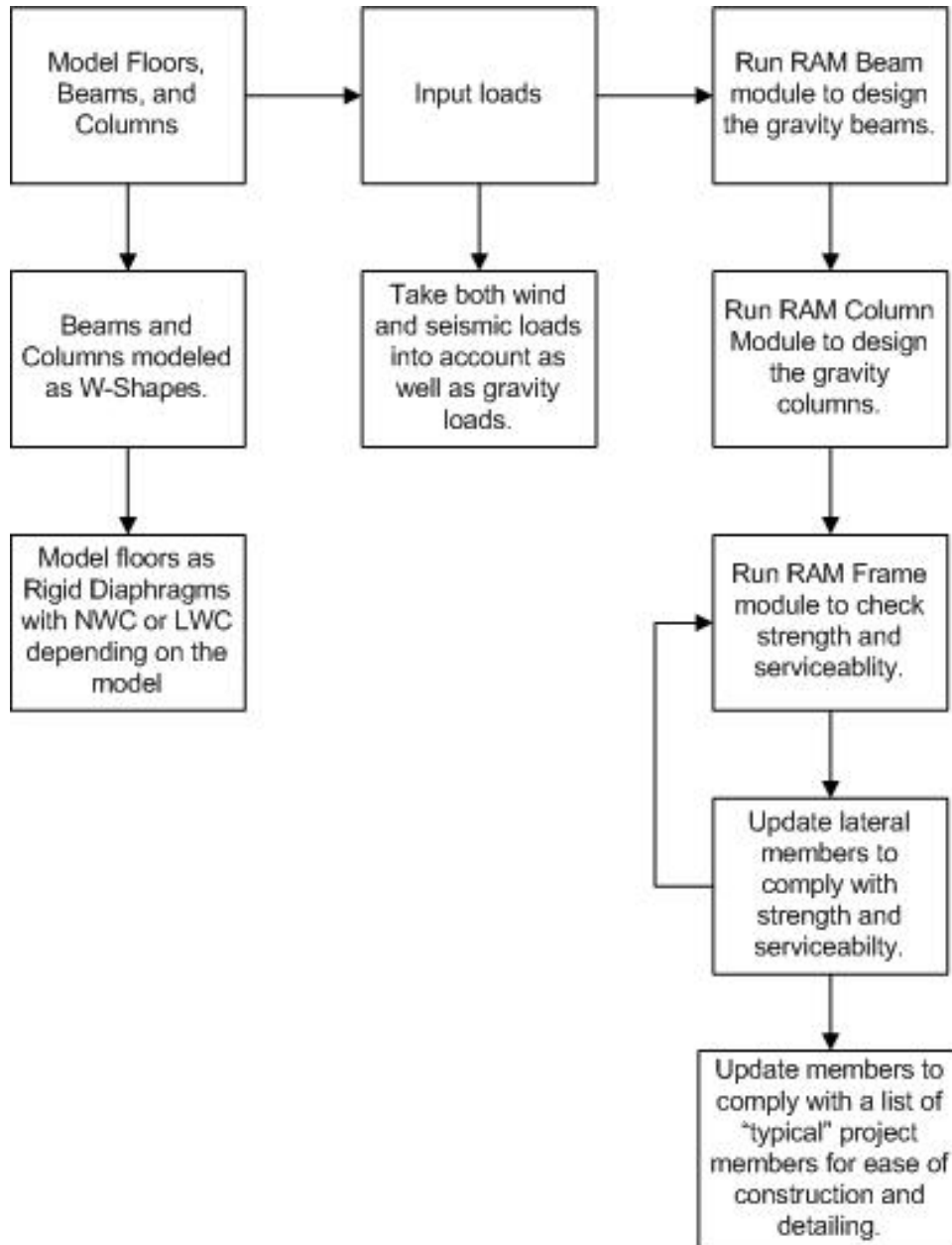
- Both the gravity and lateral resisting systems were modeled.
- Four different models were created, normal weight concrete moment frames, normal weight concrete braced frames, lightweight concrete moment frames, and lightweight concrete braced frames.
- The beams were modeled as wide-flange members 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.

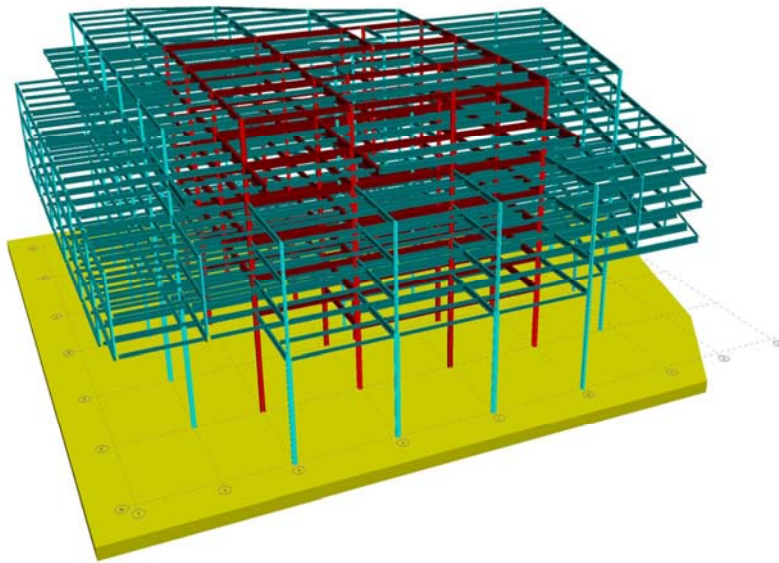


Figure 9: 3-D Moment Frame RAM Model

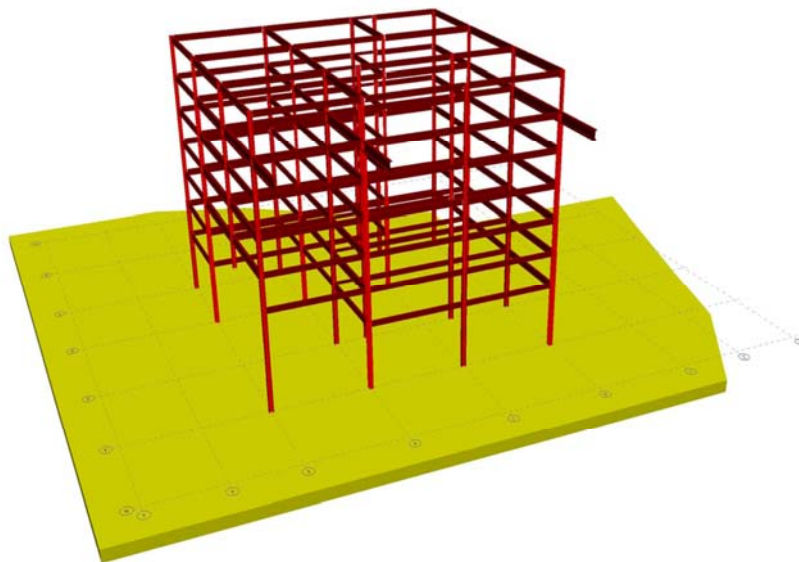


Figure 10: 3-D Moment Frame Lateral Force Resisting System RAM Model

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

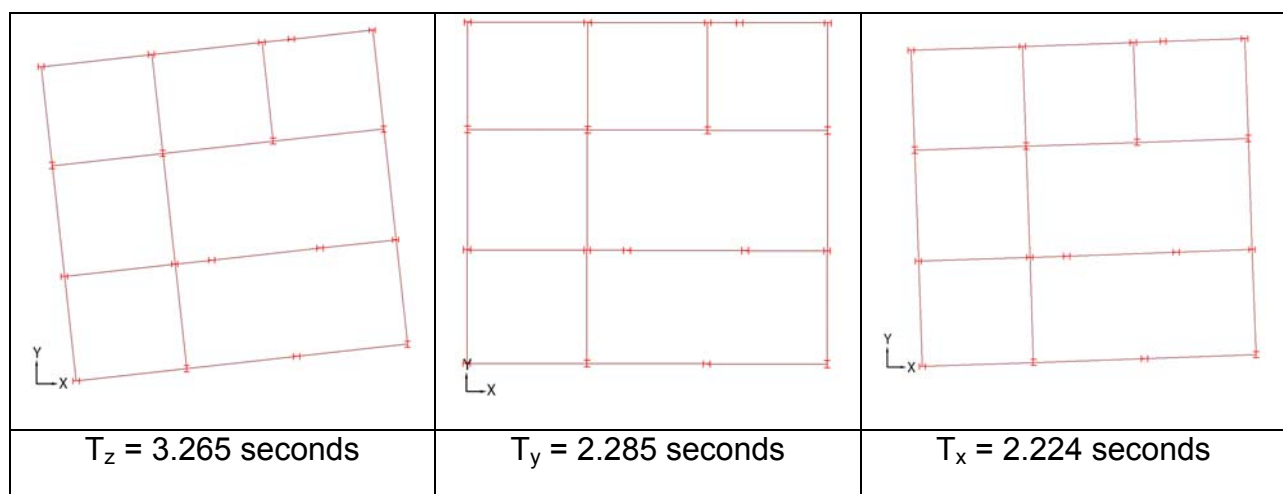
Since the lateral forces are applied to the center of mass and the center of rigidity is calculated in the RAM model, this will account for inherent torsion for seismic provisions. Wind load cases that involved torsion were also taken into account in the model. A visual inspection of the model verified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



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.

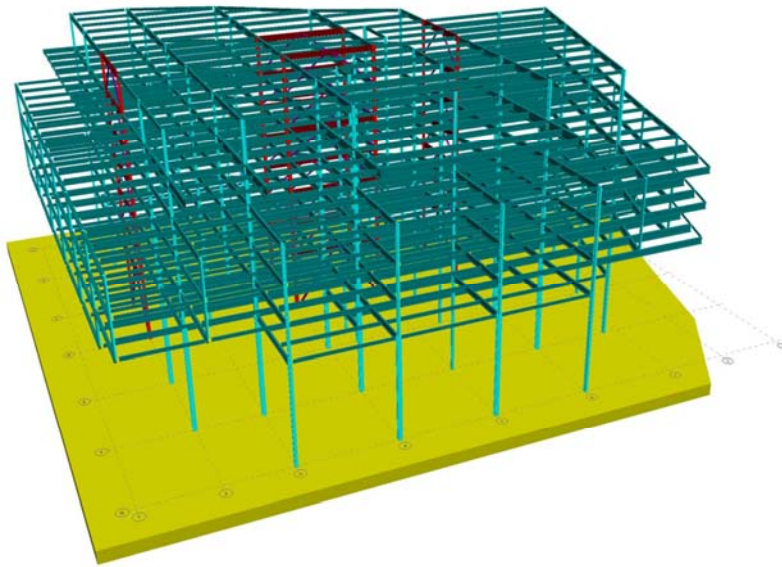


Figure 11: 3-D Braced Frame RAM Model

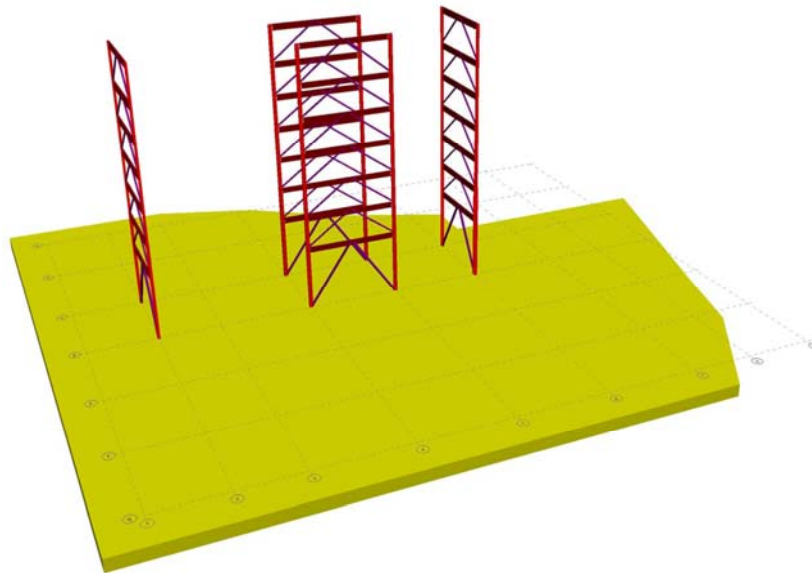


Figure 12: 3-D Braced Frame Lateral Force Resisting System Model

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

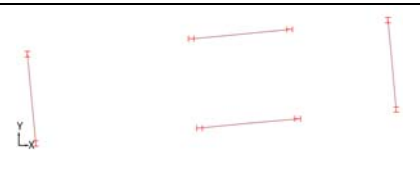
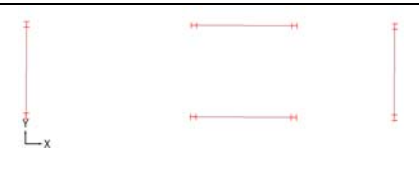
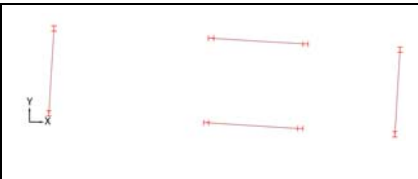
Since the lateral forces are applied to the center of mass and the center of rigidity is calculated in the RAM model, this will account for inherent torsion for seismic provisions. Wind load cases that involved torsion were also taken into account in the model. A visual inspection of the model verified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

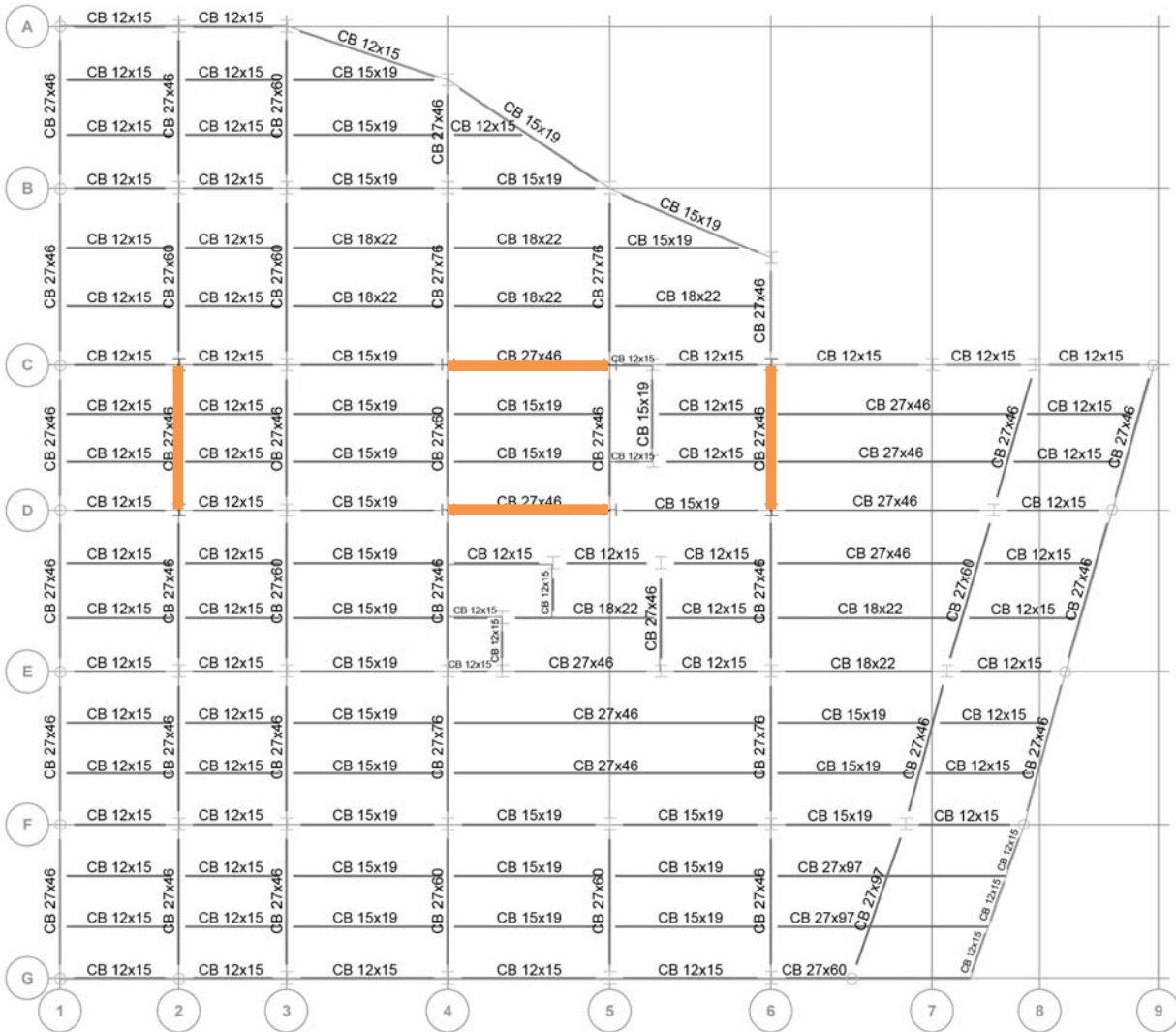


Figure 13: Typical Floor Plan

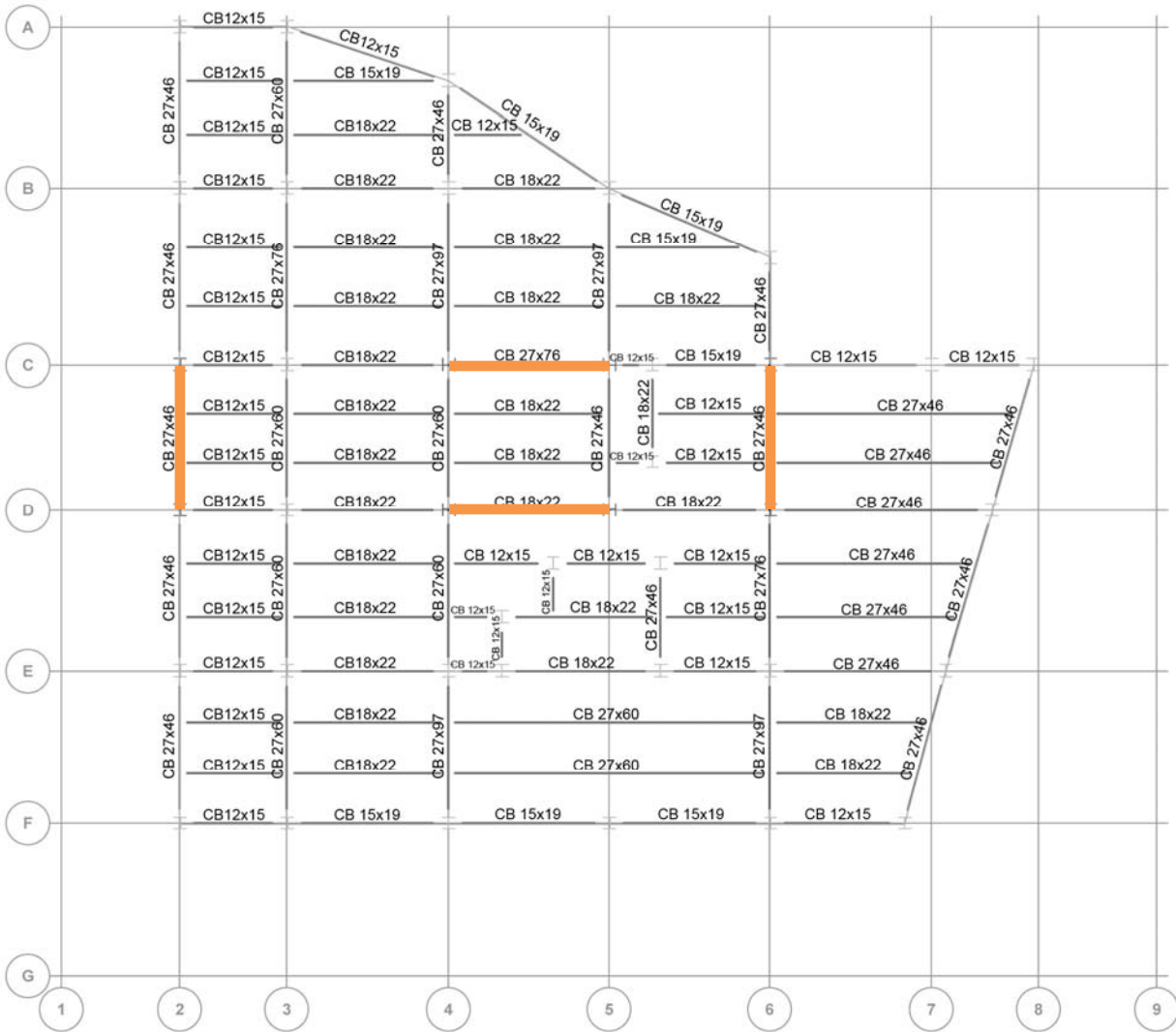


Figure 14: Roof Plan

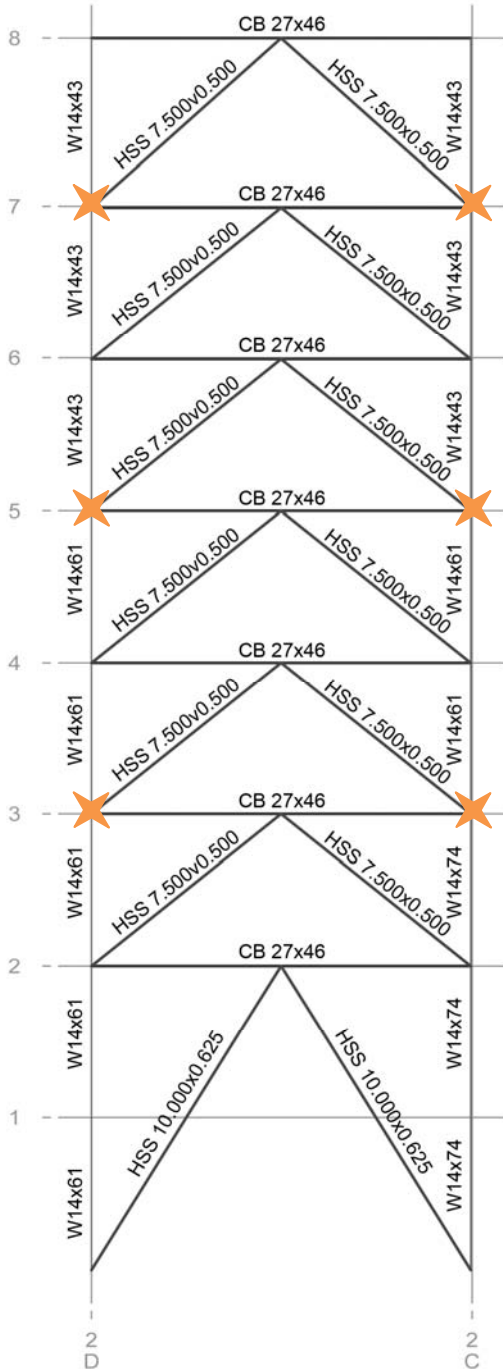


Figure 15: Braced Frame 1 Elevation

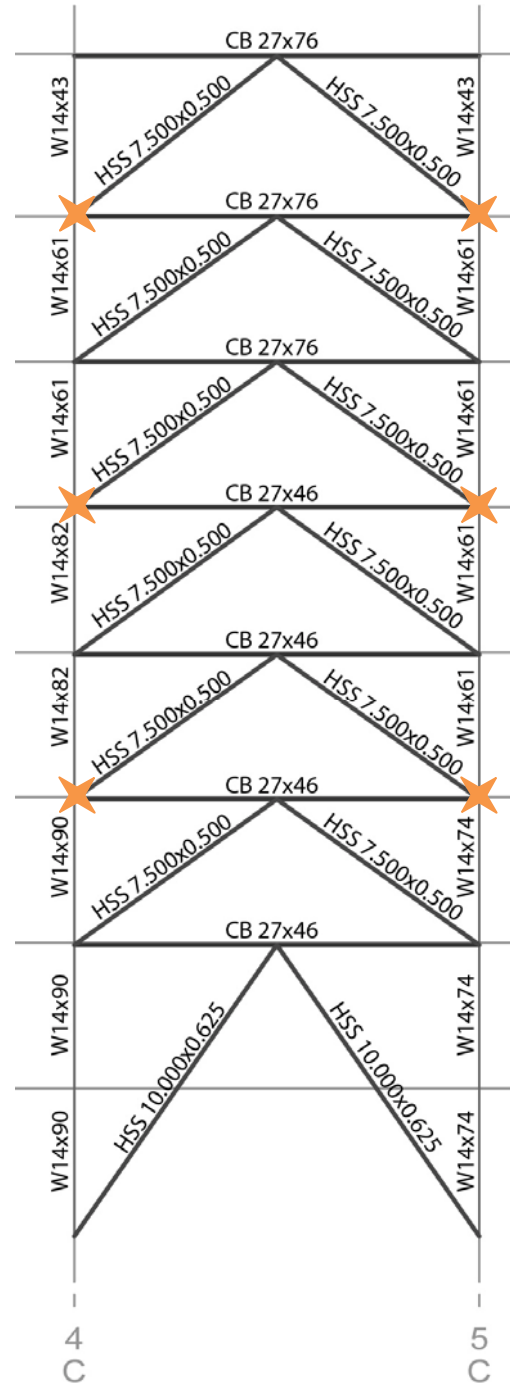


Figure 16: Braced Frame 2 Elevation

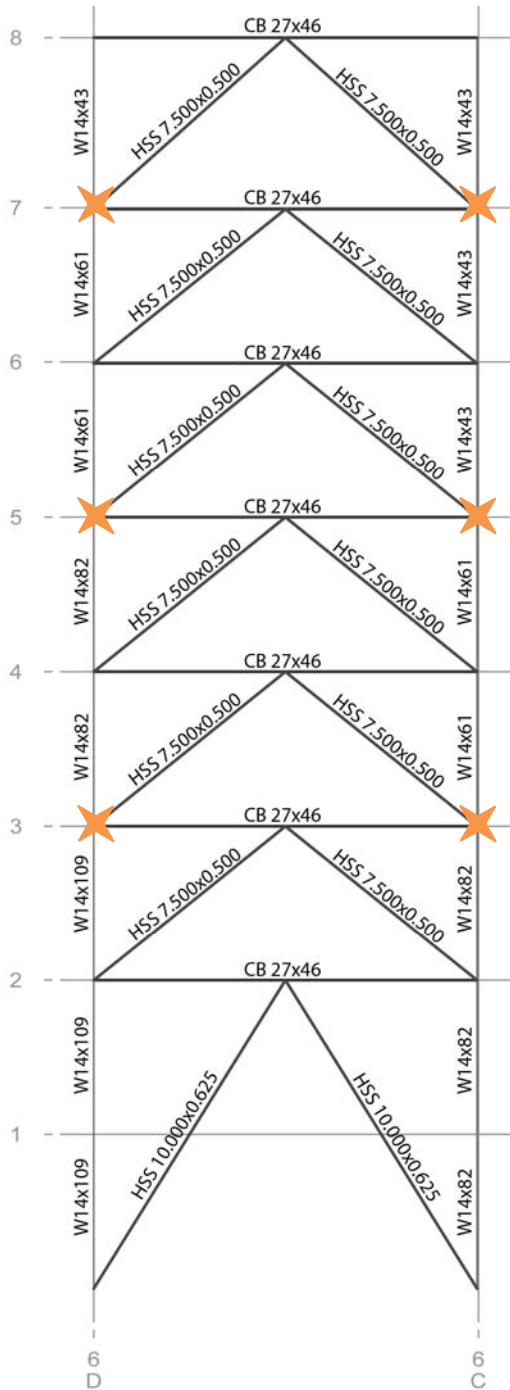


Figure 17: Braced Frame 3 Elevation

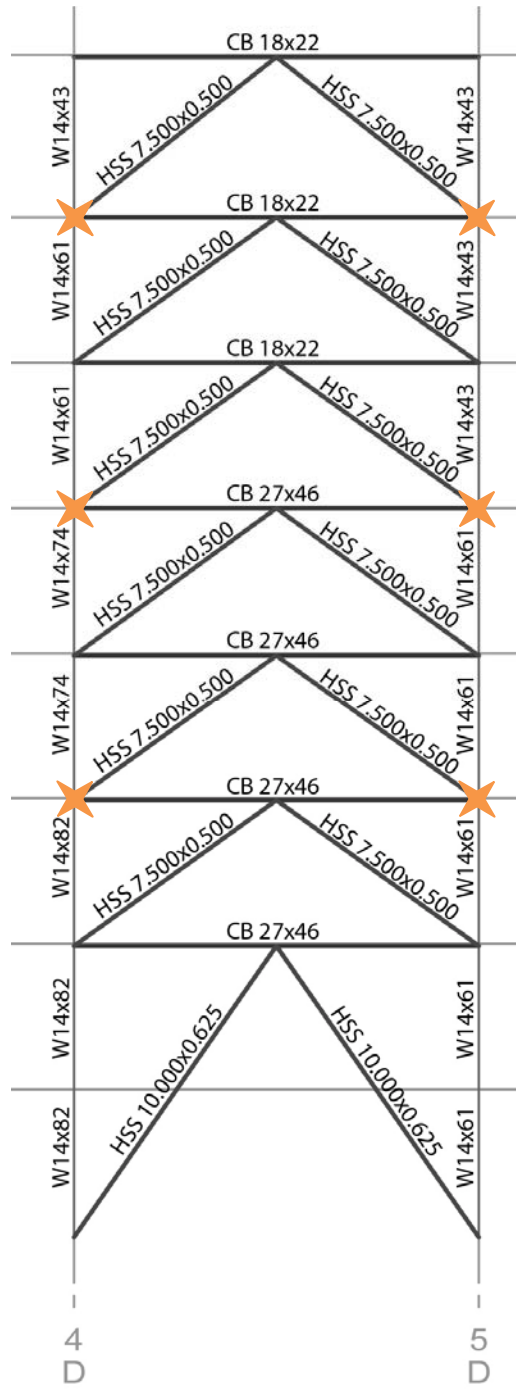


Figure 18: Braced Frame 4 Elevation

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.

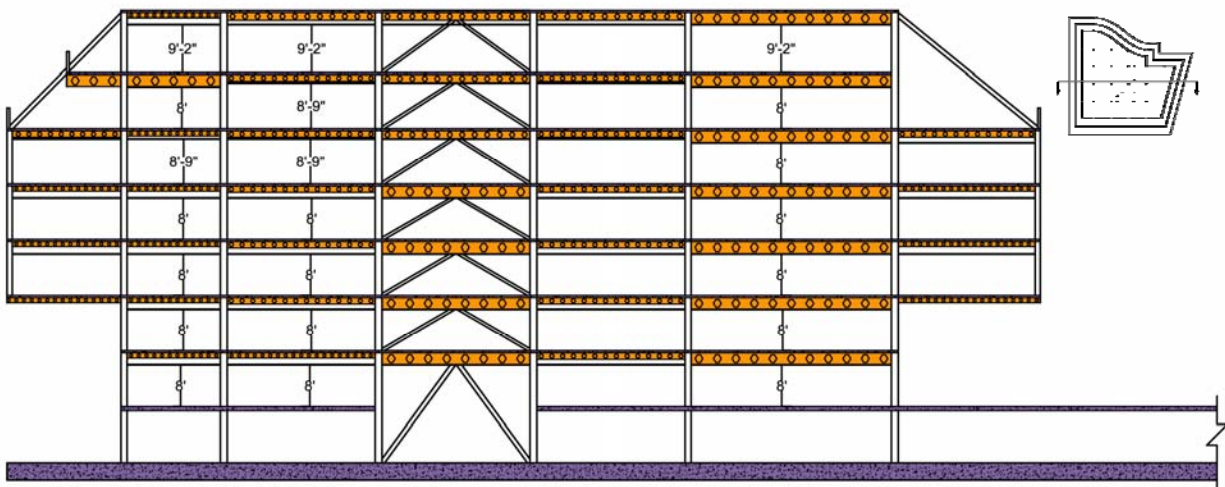
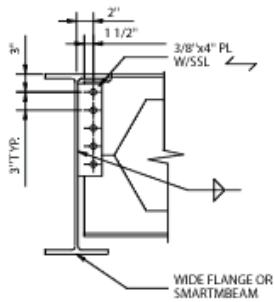


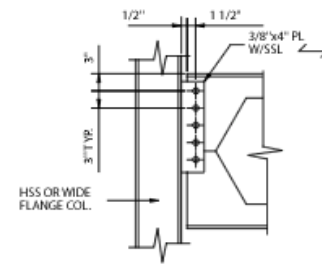
Figure 19: Clear Floor - to - Ceiling Heights

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.



BEAM-TO-BEAM
SHEAR TAB CONX.

Figure 21: Beam-to-Beam Shear Tab Connection



BEAM-TO-COLUMN
SHEAR TAB CONX.

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.

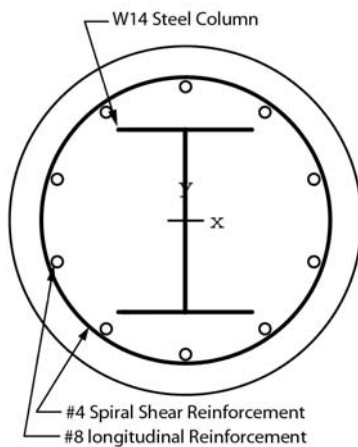


TABLE A.14

Size and pitch of spirals, ACI Code

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

Figure 22: Garage Level Column Geometry

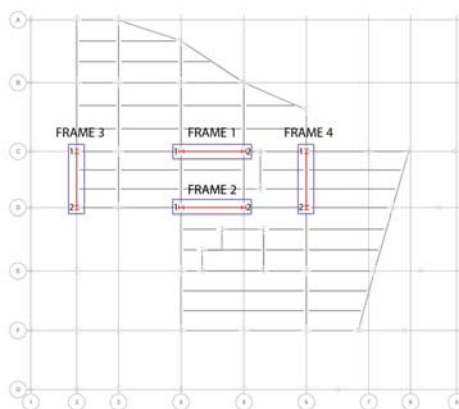
There is composite action between the steel and concrete columns so only the steel columns need to transfer their load into the mat foundation. The steel will sit on bearing plates, and the concrete column will be attached to the foundation with rebar. The design of these bearing plates and rebar attachments 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 if 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 $\Phi=0.75$ from the ACI 318-08 code. This d was found to be 43" then, when the 3" clear cover and 1.27Φ 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 detectors 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.

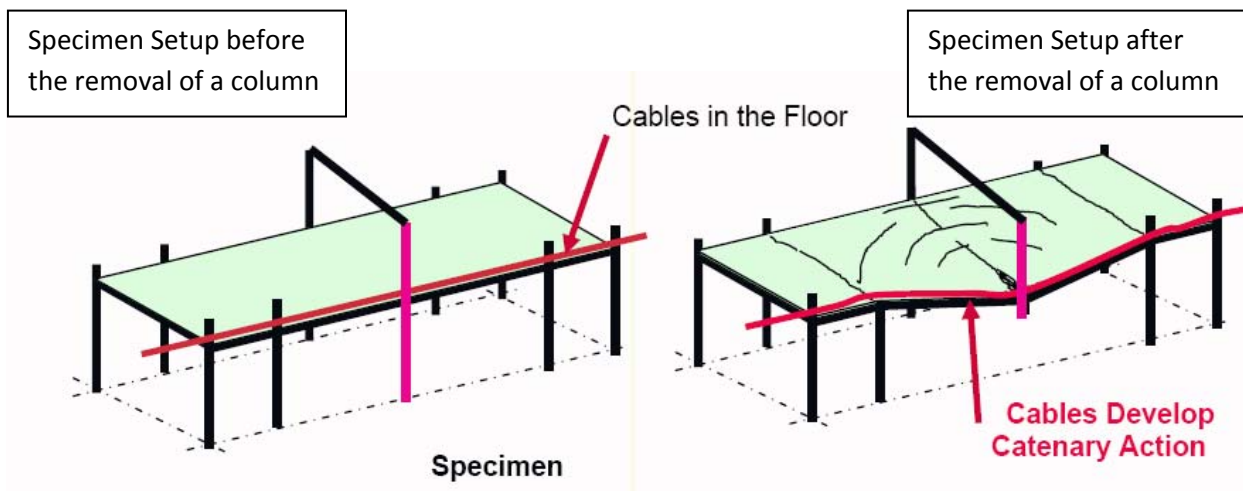


Figure 24: Schematic View of Catenary Cable Action Taken from Astaneh-Asl et.al.

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.

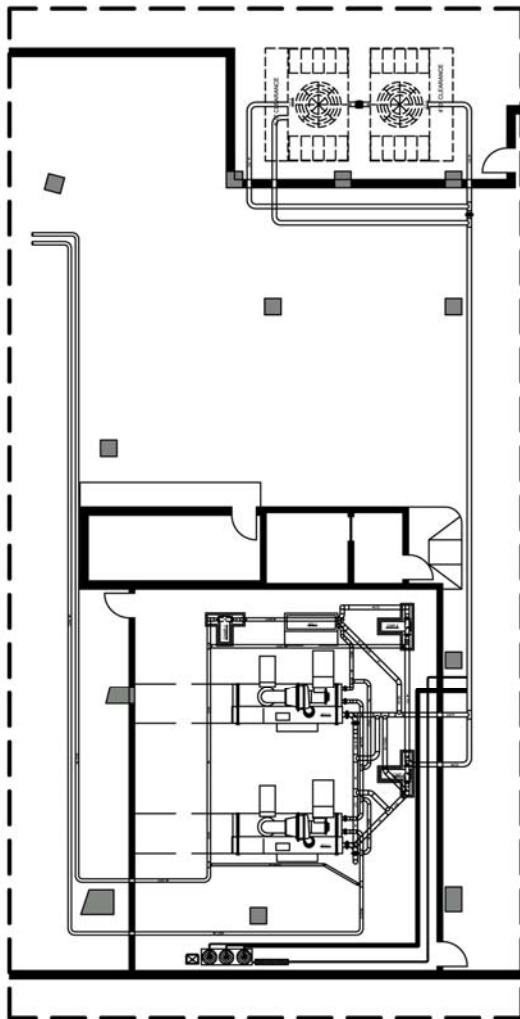


Figure 28: Layout of New Mechanical Room



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.

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.

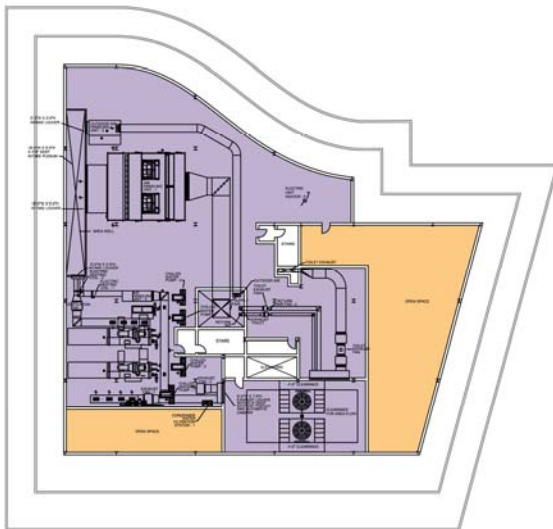


Figure 29: Original Penthouse Layout

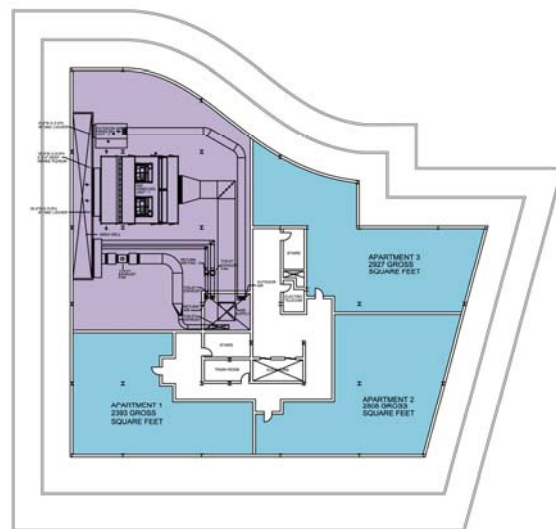


Figure 30: New Penthouse Layout

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.

Structural Cost Analysis	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.

Interior Schedule	Item	Duration (Days)
	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.
- Bangash, M.Y.H. *Impact and Explosion: Analysis and Design*. CRC Press: Boca Raton, FL, 1993.
- Childs, Mark C. *Parking Spaces*. New York: McGraw-Hill Professional, 1999.
- Construction Documents originally dated October 28, 2003 by VOA and TCE
- Das, Braja M. *Principles of Foundation Engineering*. 6th ed. Boston: Course Technology, 2006.
- General Services Administration. *Progress Collapse Analysis and Design Guidelines for New Federal Office Buildings and Major Modernization Projects*. U.S. General Services Administration, June 2003.
- 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.

- National Association of Waterproofing and Structural Repair Contractors, Inc. *National Association for Waterproofing and Structural Repair Contractors*. 1 Mar. 2005. 17 Mar. 2009 <<http://nawsrc.org/index.php>>.
- Nilson, Arthur H., David Darwin, and Charles W. Dolan. *Design of Concrete Structures*. 13th ed. New York: McGraw-Hill Companies, 2003.
- Postma, Mark. *Building Envelope Design Guide - Below Grade Systems | Whole Building Design Guide*. WBDG - The Whole Building Design Guide. 02 Jan. 2007. National Institute of Building Sciences. 16 Mar. 2009 <http://www.wbdg.org/design/env_bg_overview.php>.
- Postma, Mark. *Building Envelope Design Guide - Floor Slabs | 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_slab.php>.
- Postma, Mark. *Building Envelope Design Guide - Foundation Walls | 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_wall.php>.
- 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.