House of Sweden

Lateral System Analysis and Confirmation Design Report

2900 K St. NW
Washington, DC 20007

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

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TABLE OF CONTENTS

EXECUTIVE SUMMARY

Technical Report 3 is the Lateral System Analysis and Confirmation Design Report. This report was generated to confirm the design of the lateral system of the house of Sweden. This report uses the current standards to check the design, as well as taking into account the standards used to design the building.

House of Sweden is located in Georgetown, Washington, DC. This development is a single foundation with two towers rising from the site. It is a multi-use facility housing the Swedish Embassy, along with office, commercial, and residential spaces. Seven levels exist in the north building and six in the south. The primary structural system is a two-way post-tensioned slab with drop panels and the lateral system is primarily a concrete moment frame.

For this report, seismic and wind loads were calculated using ASCE 7-05. Seismic loads were calculated using the equivalent lateral force procedure and wind loads were calculated using method 2 in §6.5 of the standard. After the loads were found, it was determined that the seismic base shear and overturning moment is the controlling load for the north building, but the wind and seismic loads are relatively comparable for the south building so both should be taken into account in the design.

Through the use of Excel spreadsheets, hand calculations, and Etabs computer models, the loads were distributed to the lateral system and analyzed. Relative stiffness was used to distribute the loads based on the center of mass and center of rigidity for torsional effects. Building and story drift and deflection were taken into account when checking serviceability of the system and strength spot check of the lateral system were performed for the columns of one frame.

It was noted that some of the columns on the lower floors did not pass the spot checks but the author feels this is due to the frame being analyzed in 2-D as opposed to using some of the slab in 3-D to take some of the moment. The drift analysis was within code limits for serviceability and façade requirements. Overturning was also analyzed. Overall, the lateral system passed the checks and the discrepancies had explanations.

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INTRODUCTION

This Lateral System Analysis and Confirmation Design Report contains a description of the physical conditions currently existing in the House of Sweden, including discussions on the gravity and lateral systems and required loading. It provides a synopsis of the lateral system including drift and deflection of the building, relative stiffness of the frames, and distribution of the lateral loads. Confirmation of the existing design of the lateral system of the House of Sweden is given through analysis of its serviceability and strength.

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 then two separate towers rise out of the site. The south building consists of 5 stories and a mechanical penthouse; the north building is 6 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 where the design of the south building was commissioned as a competition in Sweden.

Wingardh Arkitektkontor AB completed the winning design for the south building and houses the Swedish Embassy along with an exhibit hall, convention center, rooftop terrace, and apartments. They designed this building to be “a shimmering jewel in the surrounding parkland.” To accomplish this goal, the base of the building is clad in light stone, while the upper floors are 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.

Housed in the north building are offices and apartments, which incorporate 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 lets the north building relate to the south building, yet keep its own identity.

Both building envelopes are steel stud construction with faced blanket insulation and gypsum wallboard attached. A standoff system is used on the north building to attach light stone panels to the podium of the building and metal paneling to the upper floors. This same standoff system is used on the south building to attach 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.

DOCUMENT AND CODE REVIEW

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

- ASCE/SEI 7-05 *Minimum Design Loads for Buildings and Other Structures* published in 2006 by the American Society of Civil Engineers
- IBC 2006 *International Building Code* published in January 2006 by the International Code Council, Inc.
- 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.
- Construction Documents originally dated October 28, 2003 by VOA and TCE

STRUCTURAL SYSTEM DISCUSSION

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 will be 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 6 levels above grade. The first floor slab is a 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 sixth 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 5 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 has 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 1. for a layout of the columns and shear walls that contribute to the lateral load resisting system in the north building. Refer to Figure 2. for a layout of the columns that contribute to the lateral load resisting system in the south building.

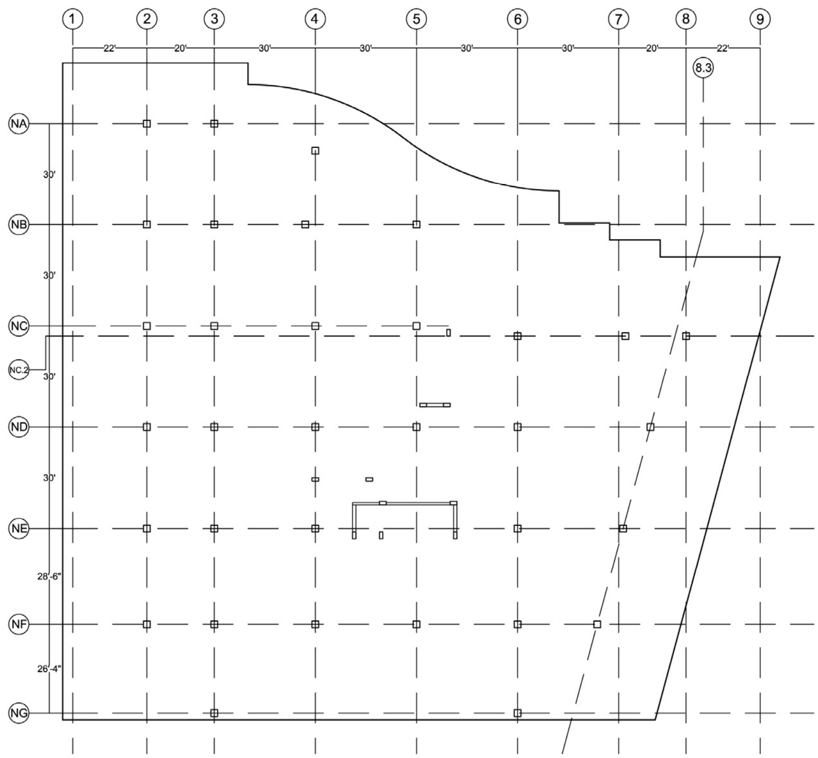


Figure 1. Typical North Building Column and Shear Wall Layout

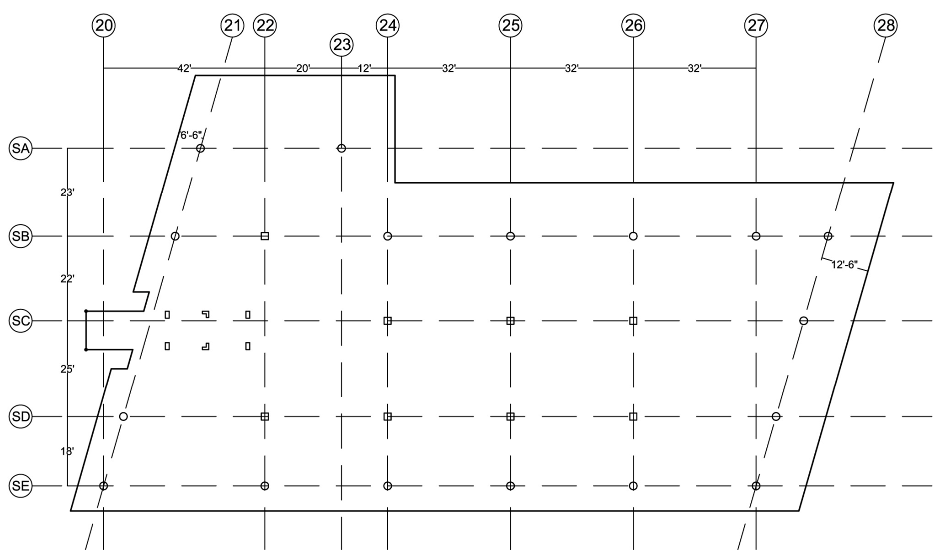


Figure 2. Typical South Building Column Layout

GRAVITY LOAD DISCUSSION

To analyze the gravity system of the House of Sweden, the static and dynamic loading on the structure had to be determined. The following is a summary of the approximate design gravity loads and criteria used to spot check the House of Sweden's gravity system. Load references are listed in the tables.

Deflection Criteria

Floor Deflection – IBC 2006 Table 1604.3

Typical Live Load Deflection L/360

Typical Total Deflection L/240

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

Roof Live Loads		
	Design Load	ASCE7-05 Load
Public Terrace	100 psf	100 psf
Snow Load**	30 psf*	20 psf*
Rain Load**	---	41.6 psf

**Snow drift will accumulate around the penthouse and on the lower roof of the north building. This load was calculated and can be found in the Appendix A along with the flat roof snow load and rain load calculations.

Floor Live Loads		
Occupancy	Design Load	ASCE7-05 Load
Penthouse Machine Room	150 psf*	Not listed specifically, but light storage warehouses - 125 psf*
Residential	40 psf + 20 psf for partitions*	40 psf*
Stairways	100 psf	100 psf
Corridors	100 psf	100 psf
Commercial and Plaza Area	100 psf*	Offices - 50 psf, Corridors above 1st floor - 80 psf, Lobby - 100 psf*
Elevator Machine Room	300 lbs of concrete load on 4 square inches	300 lbs of concrete load on 4 square inches
Loading Dock	400 psf	Not listed specifically
Parking Garage	50 psf and 2000 lbs of concrete load on 20 square inches*	40 psf and 3000 lbs of concrete load on 20 square inches*

*For load discrepancies, worst case scenario loading was used.

LATERAL LOAD DISCUSSION

To analyze the lateral system of the House of Sweden, the wind and seismic lateral loading on the structure had to be determined. The following is a summary of the approximate wind and seismic loads and criteria used to spot check the House of Sweden's lateral system. Load references are listed in the tables. For more detailed calculations, please refer to the Appendix B.

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)

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. For the north building, when the 1.6 factor is applied to the wind load found below, it is still less than the magnitude of the seismic load found below. Therefore, the seismic load governs in this case and the member spot checks will be performed with the seismic loads only since this is the governing case.

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

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 technical 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 3. through 6. illustrate the distribution of wind pressure on the building façades. For more detailed calculations, please refer to the Appendix B.

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)
PH	77'-0"	14	0.0	1071
MR	59'-0"	31	14	1805
6	48'-2"	30	44	1442
5	37'-4"	29	74	1069
4	26'-6"	81	103	2143
3	15'-8"	75	184	1178
2	4'-10"	18	259	85
1	-6'-0"	0.0	277	0.0
			V = 277	**ΣM = 8792**

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

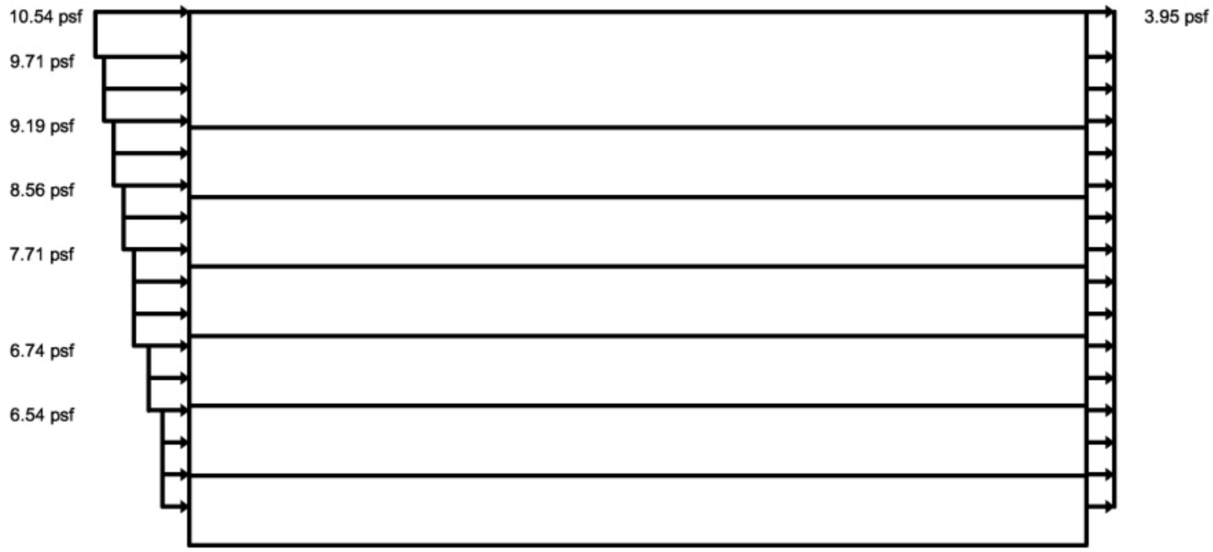


Figure 3. North Building Wind Pressure Diagram in the N-S Direction

Figure 4. North Building Wind Pressure Diagram in the E-W Direction

South Building

Number of Floors: 6
Height: 70'
N-S Building Length: 100'
E-W Building Length: 210'
η_1: 1.07 (Rigid)

South Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	31	0.0	2146
MR	52'-0"	48	31	2477
5	41'-6"	33	78	1383
4	31'-0"	32	112	990
3	18'-0"	88	144	1591
2	5'-0"	82	232	408
1	-6'-0"	0.0	314	0.0
			V = 314	**ΣM = 8994**

South Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	15	0.0	1067
MR	52'-0"	24	15	1232
5	41'-6"	17	39	688
4	31'-0"	33	56	1035
3	18'-0"	85	89	1525
2	5'-0"	78	174	390
1	-6'-0"	0.0	252	0.0
			V = 252	**ΣM = 5937**

Figure 5. South Building Wind Pressure Diagram in the N-S Direction

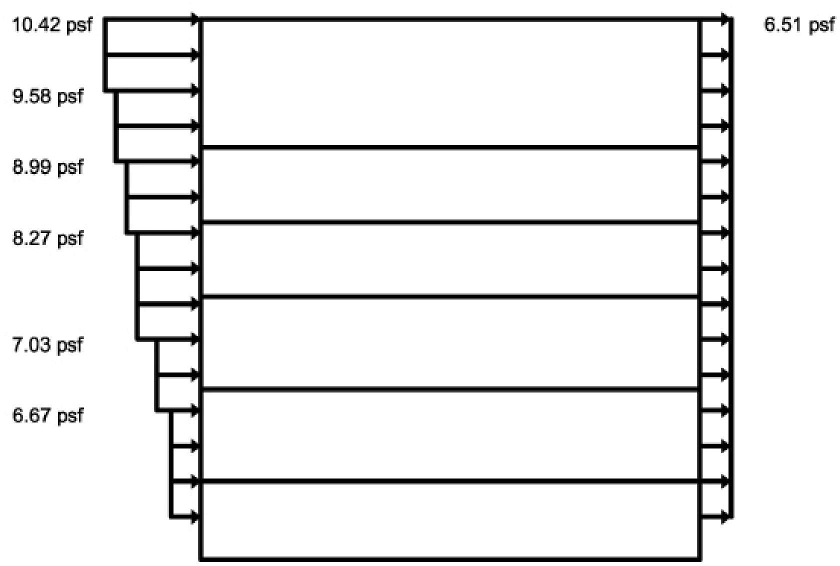


Figure 6. South Building Wind Pressure Diagram in the E-W Direction

North Building Wind Load Summary

N-S Direction Base Shear: V = 277 K = 1.6(277 K) = 433 K

N-S Direction Moment: ΣM = 8792 ft-K = 1.6(8792 ft-K) = 14067 ft-K

E-W Direction Base Shear: V = 325 K = 1.6(325 K) = 520 K

E-W Direction Moment: ΣM = 10069 ft-K = 1.6(10069 ft-K) = 16110 ft-K

South Building Wind Load Summary

N-S Direction Base Shear: V = 314 K = 1.6(314 K) = 502 K

N-S Direction Moment: ΣM = 8994 ft-K = 1.6(8994 ft-K) = 14390 ft-K

E-W Direction Base Shear: V = 252 K = 1.6(252 K) = 403 K

E-W Direction Moment: ΣM = 5937 ft-K = 1.6(5937 ft-K) = 9499 ft-K

Seismic Loads

Design seismic load was calculated using ASCE 7-05 chapter 12. The Equivalent Lateral Force Procedure was determined as the procedure to use. An approximate story weight was used because of the varying thicknesses of the slab. When the thickness varied, the largest thickness was applied over the total area of the slab. However, this approximation was done to estimate the weight of the cladding. Below is a summary of the base shear and moment. Figures 7. And 8. illustrate the distribution of seismic forces and shears on the building façades. For more detailed calculations, please refer to the Appendix B.

Vertical Distribution of Seismic Forces (North Building)						
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	C_{vx}	Moment at Floor (ft-K)
P	83'-0"	2379	127	127	0.239	10530
MR	65'-0"	2793	116	243	0.219	7561
6	54'-2"	3087	107	350	0.201	5795
5	43'-4"	3094	86	436	0.161	3707
4	32'-6"	2759	57	493	0.107	1854
3	21'-8"	1885	26	519	0.0488	561
2	10'-10"	1756	12	531	0.0241	130
$\Sigma w_i h_i^k$ =	863,359	**ΣF_x = V =**	531K		**ΣM =**	30,138ft-k

Figure 7. North Building Seismic Force Diagram

Vertical Distribution of Seismic Forces (South Building)						
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	C_{vx}	Moment at Floor (ft-K)
P	77'-0"	756	55	55	0.0146	4239
MR	59'-0"	2237	120	175	0.0289	7079
5	48'-6"	2308	99	274	0.024	4795
4	38'-0"	2325	75	349	0.181	2858
3	25'-0"	1938	39	388	0.0938	969
2	12'-0"	2300	20	408	0.0497	237
$\Sigma w_i h_i^k$ =	823,173	**$\Sigma F_x = V =$**	408K		**ΣM =**	20,178ft-k

Figure 8. South Building Seismic Force Diagram

North Building Seismic Load Summary:
Base Shear: V = 531 K
Moment: ΣM = 30,138 ft-K

South Building Seismic Load Summary:
Base Shear: V = 408 K
Moment: ΣM = 20,178 ft-K

Seismic loads control the lateral design for both the north building. Wind controls the south building for shear design and seismic controls the south building for the overturning moment. It is recommended that both wind and seismic load cases should be analyzed for the south building.

LATERAL ANALYSIS

A simplified analysis of the lateral system in the north building was performed with spreadsheets, computer models, and hand calculations. The goal of this analysis was to determine how loads are distributed throughout the lateral system of the building. This distribution will then be used to perform strength and serviceability spot checks of the lateral system components.

Relative Stiffness

To find the relative stiffness of each frame, the frames were modeled individually in Etabs as beam-slab and column frames. For simplifying purposes, frames were assumed to act in a single direction, even though they would actually add stiffness to the orthogonal direction as well. The bases of the frames were modeled as fixed restraints as a mat slab should be modeled. A 1 kip horizontal load was applied to the top of each frame. Deflection at the top of the frame was read from the program analysis output and the stiffness of each frame was easily calculated through P/Δ. Stiffness values for each frame on a single floor were summed and the relative stiffness was calculated for each frame on that floor. The following chart presents a summary of the relative stiffness values for each frame on each floor in both the north-south and east-west directions.

Relative Stiffness in the North-South Direction (North Building)							
Level	Frame NA	Frame NB	Frame NC	Frame ND	Frame NE	Frame NF	Frame NG
PH	-	0.12	0.21	0.15	0.38	0.14	-
MR	0.08	0.11	0.20	0.13	0.35	0.13	-
6	0.05	0.07	0.12	0.08	0.22	0.08	0.38
5	0.05	0.07	0.12	0.08	0.22	0.08	0.38
4	0.05	0.07	0.12	0.08	0.22	0.08	0.38
3	0.05	0.07	0.12	0.08	0.22	0.08	0.38
2	0.05	0.07	0.12	0.08	0.22	0.08	0.38
1	0.05	0.07	0.12	0.08	0.22	0.08	0.38

Relative Stiffness in the East-West Direction (North building)						
Level	Frame 2	Frame 3	Frame 4	Frame 5	Frame 6	Frame 7
PH	0.15	0.15	0.23	0.34	0.13	-
MR	0.14	0.13	0.21	0.31	0.12	0.09
6	0.14	0.13	0.21	0.31	0.12	0.09
5	0.14	0.13	0.21	0.31	0.12	0.09
4	0.14	0.13	0.21	0.31	0.12	0.09
3	0.14	0.13	0.21	0.31	0.12	0.09
2	0.14	0.13	0.21	0.31	0.12	0.09
1	0.14	0.13	0.21	0.31	0.12	0.09

As shown in the charts, in the north-south direction, frame NA takes only a small portion of the load, however, it is only a three column frame so this outcome is expected. In the east-west direction, frame 7 only takes a small fraction of the load as well, and it is also one of the smaller frames. The frames that extend the whole length of the building in either the north-south or east-west directions take a fairly comparable amount of load.

Center of Rigidity and Center of Mass

The center of rigidity (COR) was calculated using the relative stiffness of each frame from the charts above. A reference origin was taken as the north-east corner of the building shown as the top left corner on the plans. Since the building is fairly symmetrical in setbacks, the same center of rigidity was used on all floors to simplify the calculations. The averaged center of rigidity was found to be at 122.37 feet in the X-direction (north-south direction) and 64.17 feet in the Y-direction (east-west direction).

The Y-direction is fairly comparable to the Etabs averaged center of rigidity at 70.79 feet, however, the Etabs averaged center of rigidity for the x-direction is 85.98 feet. The author attributes this discrepancy to the simplifying assumptions made for finding the relative stiffness of the frames when assuming that the frames only act in a single direction. Frames in the center of the building should be stiffer than calculated due to the fully restrained slab that will help resist more of the lateral loads than the slabs at the edges of the building.

The center of mass was taken from Etabs due to the complicated floor geometry of the House of Sweden. Presented below is the Etabs output for the center of mass. Remember again that the X is the north-south direction and Y is the east-west direction.

Center of Mass in X Direction (ft) From Etabs	Center of Mass in Y Direction (ft) From Etabs
72.85	82.28
86.04	91.52
87.06	70.18
86.79	72.94
87.03	70.82
83.19	90.72
85.78	91.14

As is shown in the chart, the center of mass is fairly close to the center of rigidity. This outcome was expected due to the fact that the building is a concrete moment frame and all columns and slabs contribute to the lateral force resisting system. Due to this fact, the moment due to torsion should not be too great in magnitude.

Distribution of Direct and Torsional Shear

Spot checks were completed on frame ND spanning in the north-south direction. This frame was constructed in Etabs as a 2-D slab-beam and column frame as shown below (Figure 9.). The circled column line is column line 16N. PCA column printouts for this column line are included in this report in Appendix C. As with the relative stiffness frames, this frame was fully restrained at the base. Joint loads were applied to the left side of the frame at every floor. The loads were determined as follows.

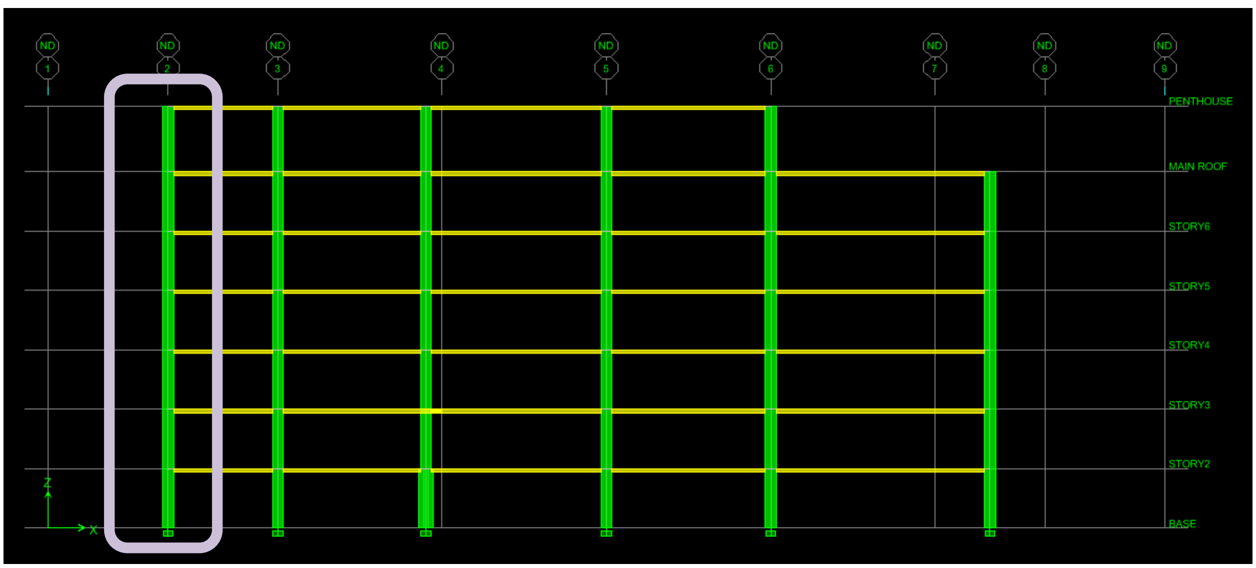


Figure 9. Frame ND as constructed in Etabs

The direct force on each level of frame ND was determined using the relative stiffness values calculated above. Seismic loading was determined to control for the north building, so the seismic load was distributed using the formula $F_{ix}=F(K_{ix}/\sum K_{ix})$.

The force from the torsional moment on each level was also determined. This was done by first determining the total torsional moment. Torsional moment is the moment that is induced on a floor level when the center of rigidity is not at the same point as the center of mass. Seismic loads are applied at the center of mass, but the building attempts to rotate about the center of rigidity. Torsional moment was then distributed to frame ND using relative stiffness and the equation $F_{ix}=M(K_i*x_i)/I_p$ to find the force from the moment. I_p is the polar moment of inertia to which all the frames contribute. This value is 6,952,438 ft^2 for the north building. Forces applied to each level of the frame are summarized in the table below.

Story	Total Seismic Force (K)	Direct Force on Frame ND (K)	Total Torsional Moment (ft-K)	Force From Torsional Moment on Frame ND (K-ft)	Total Force on Each Story (K)
PH	127	10.60	5092	0.65	11.25
MR	116	9.68	3579	0.46	10.14
6	107	8.93	5585	0.71	9.64
5	86	7.18	4252	0.54	7.72
4	57	4.76	2938	0.37	5.13
3	26	2.17	823	0.10	2.27
2	12	1.00	375	0.05	1.05

Spot Checks

Strength spot checks were performed for every column in frame ND. Lateral axial forces and moments on each column were determined from the Etabs model of frame ND. These were combined with the gravity axial forces obtained from the structural engineer. A summary of the loads on each column can be produced upon request but have not been attached as part of this report. Column geometries, reinforcing, and factored loads were input into PCA column. Column interaction diagrams were produced and the factored load point was plotted. A representative group of column interaction diagrams can be found in Appendix C.

All the columns on the fourth floor and above passed. Most columns below the fourth floor failed. In every case, this failure occurred due to the high amount of moment from the lateral loading. These high moments were not included within the interaction diagram for any column so all the columns would fail even if not axial force was present. The author contributes this failure to the fact that frame ND was analyzed as a 2-D frame when, in reality, it is a 3-D frame. Some of the moment on the column would actually be transferred into the slab connected orthogonally to the column. This method was a fairly simplified method so the author feels that the columns should pass the design loads even though they are not shown to do so in the interaction diagrams.

Building and Interstory Drift

Drift is a building serviceability consideration and should be limited as much as possible to comply with code and standard recommendations. Limiting of the drift of a building creates a more pleasant environment for building occupants if they are unable to notice the sway of the building.

Unless a building requires special deflection considerations, the deflection due to wind is recommended to be limited to the total height of the building divided by 400. It should be noted that this is not a code value, it is a recommended value, however, it has been an industry standard for a long enough period of time, that most designers use this number as their guide. In the case of the House of Sweden north building, this total deflection limitation is

$$\Delta_{max}=(77'*12")/400-2.31"$$

A 3-D Etabs model was constructed to calculate the displacements and story drifts for both wind and seismic load cases. It was a simplified model since only the lateral system will contribute to oppose lateral movement. Many assumptions were made in the construction of this model. These assumptions are that no openings were modeled and slabs were modeled as effected beams instead of rigid diaphragms so that the debugging process would become easier. The base was modeled with fixed constraints due to the presence of a mat foundation. The shear walls that exist in the north building were not modeled. As stated in the lateral system description above, the reason that the shear walls were introduced into this building was due to the large amount of sloped columns that exist and transfer more horizontal loads into the slabs. After discussions with the structural engineer, the columns were all modeled as straight columns and the author assumed the shear walls were no longer necessary. The slab-beams were all modeled with 8ksi for slabs with a varying ksi. This was done after discussions with the concrete subcontractor where they stated that is how the building was actually built in the field to simplify construction. Finally, the diaphragms were modeled with no property assignments and the mass assignments were manually calculated using the weight of the building stories from the seismic calculations. Shown on the next page is a 3-D picture of the Etabs model (Figure 10.).

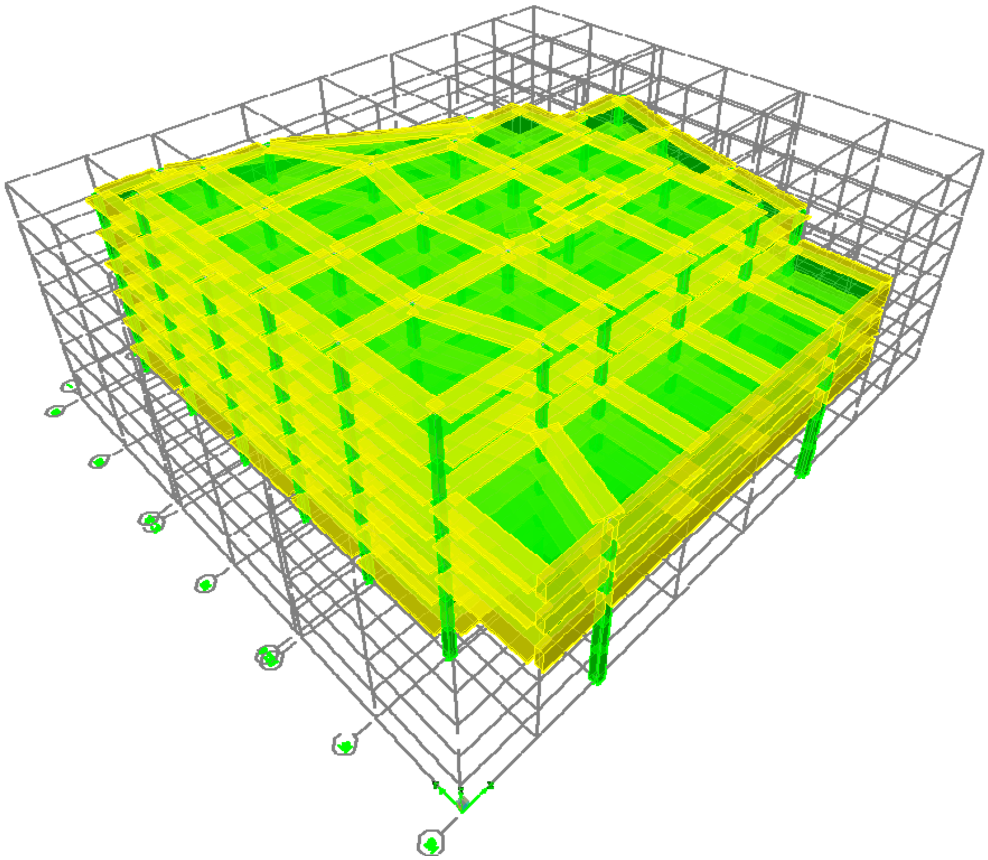


Figure 10. 3-D model as constructed in Etabs

Wind loads were applied at the center of pressure for the building. Seismic loads were applied at the center of mass. The building and story deflections were taken at the center of mass as required from §12.8.6 of ASCE 7-05. Total building deflection due to wind as calculated by Etabs is 0.35" in the north-south direction and 0.55" the east-west direction. Even though total seismic deflection is not kept to this same maximum deflection, the total building deflection due to seismic as calculated by Etabs is 1.16" in the north-south direction and 1.69" in the east-west direction. These displacements are well under the recommended maximum deflection limitation. The displacement values for each story are shown in the tables below.

Interstory drift is another serviceability requirement to ensure that no one story moves more than would be comfortable for occupants. Wind drift limitations are h/400 to h/600. Seismic limitations are 0.020h. These are code values, not just recommendations, so the building must comply with these values in order to be built. As shown in the tables below, the Interstory drifts are well below the maximum allowable Interstory drifts.

Wind Story Displacement and Drift						
Story	Load	X Displacement (in)	Y Displacement (in)	Drift X (in/in)	Drift Y (in/in)	Maximum Drift (in)
PH	N-S	0.3553	0.027	0.000305	0.000089	0.24
PH	E-W	-0.0235	0.5468	0.000028	0.000343	0.24
MR	N-S	0.3428	-0.002	0.000493	0.000206	0.22
MR	E-W	-0.0327	0.5149	0.000085	0.000502	0.22
6	N-S	0.2651	-0.0032	0.000672	0.000292	0.22
6	E-W	-0.0071	0.459	0.000127	0.000691	0.22
5	N-S	0.2232	-0.002	0.000806	0.000337	0.22
5	E-W	-0.0063	0.3806	0.000173	0.000897	0.22
4	N-S	0.1637	-0.0017	0.000854	0.000337	0.22
4	E-W	-0.0007	0.2811	0.000221	0.001079	0.22
3	N-S	0.1094	0.0006	0.000739	0.000228	0.22
3	E-W	-0.0077	0.1623	0.000245	0.001066	0.22
2	N-S	0.0397	-0.0003	0.000386	0.000093	0.22
2	E-W	-0.0025	0.0547	0.000134	0.00055	0.22

Seismic Story Displacement and Drift						
Story	Load	X Displacement (in)	Y Displacement (in)	Drift X (in/in)	Drift Y (in/in)	Maximum Drift (in)
PH	XSEISMIC	1.1622	0.0526	0.001453	0.000211	2.88
PH	YSEISMIC	0.0134	1.6913	0.00018	0.001836	2.88
MR	XSEISMIC	1.0211	-0.007	0.001843	0.000497	2.60
MR	YSEISMIC	0.0215	1.4299	0.000247	0.002017	2.60
6	XSEISMIC	0.7645	-0.0083	0.002253	0.000776	2.60
6	YSEISMIC	0.0008	1.1886	0.000207	0.002276	2.60
5	XSEISMIC	0.5881	-0.0047	0.002314	0.000778	2.60
5	YSEISMIC	0.0019	0.909	0.000185	0.0024	2.60
4	XSEISMIC	0.3927	-0.0032	0.002015	0.000655	2.60
4	YSEISMIC	-0.0013	0002 a	0.000155	0.002274	2.60
3	XSEISMIC	0.2328	0.0002	0.001413	0.000273	2.60
3	YSEISMIC	0.001	0.327	0.000101	0.00179	2.60
2	XSEISMIC	0.0804	-0.0004	0.000674	0.000066	2.60
2	YSEISMIC	-0.0003	0.1026	0.00004	0.000816	2.60

Overturning Moment

Overturning moment is important to consider because of the effects it could have on foundations. House of Sweden sits on a 4' deep mat foundation and the structural system is a concrete moment frame, therefore overturning is not an issue with this particular building.

Overturning moment caused by wind was calculated by multiplying the story shears by the mid height of each level. Seismic overturning moments were calculated by multiplying the story shears by the floor height. The differences in height account for why the seismic overturning moment is so much higher than the wind for the north building. The overturning moments due to wind and seismic loads have been calculated and are shown in both the wind and seismic lateral loading sections and in Appendix B.

CONCLUSION

After carefully conducting a lateral analysis of the north building of the house of Sweden, a better understanding has been gained into the distribution of lateral loads throughout the building. It was found that seismic was the controlling load for the north building, but the south building should take both wind and seismic loads into account for design purposes. Lateral loads applied to the building are resisted by the building as a whole through the presence of a concrete moment frame that distributes the loads to all columns. The relative stiffness of the frames proved that the entire building worked similarly in resisting the lateral loads. This relative stiffness of the frames was used to distribute direct and torsional shears to each frame so they could be analyzed.

In general, torsional shear does not appear to prove an issue in the north building due to the fact that the center of mass and center of rigidity are fairly close to one another. Overall deflection and story drift did not prove an issue in this building as they were well within the allowable code limits. The strength spot checks did not all prove satisfactory, but the author feels that analyzing these frames as 2-D elements is not a sufficient method due to the fact that the slab is involved in resisting the lateral loads as well as the columns.

APPENDIX A – GRAVITY LOAD CALCULATIONS

SNOW AND RAIN LOAD CALCULATIONS

Presented below are table summaries of the snow and rain load calculations performed for both the north and south buildings. Hand calculations were also performed and can be reviewed upon request.

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

Snow Drift (South Building)		
Factor	Design Value	Code Section
γ	17.25 psf	§7.7.1
h_b	1.16'	
h_c	16.84'	
h_c/h_b	14.5'	
l_u N-S	11'	
Leeward Drift, h_d N-S	1.56'	Figure 7-9
l_u E-W	141'	
Leeward Drift, h_d E-W	3.94'	Figure 7-9
l_u N-S	57'	
Windward Drift, h_d N-S	1.89'	Figure 7-9
l_u E-W	48'	
Windward Drift, h_d E-W	1.73'	Figure 7-9
$w=4*h_d$, N-S	7.56'	
$p_d=h_d\gamma$, N-S	32.6 psf	§7.7
$w=4*h_d$, E-W	6.92'	
$p_d=h_d\gamma$, E-W	29.8 psf	§7.7

SNOW AND RAIN LOAD CALCULATIONS

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

Rain Load		
Factor	Design Value	Code Section
d_s	8"	§8.3
d_h	0	§8.3
R	41.6 psf	§8.3

APPENDIX B – LATERAL LOAD CALCULATIONS

WIND LOAD CALCULATIONS

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

North Building in the N-S Direction

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

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

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

North Building in the E-W Direction

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

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

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

South Building in the N-S Direction

Wind Pressures (South Building N-S)						
Height (ft)	K_z	q_z (psf)	Windward Wall (psf)	Leeward Walls (psf)	Total (psf)	Length in E-W Direction (ft)
70	0.890	15.69	10.14	-6.08	16.22	210
52	0.818	14.42	9.32	-6.08	15.40	210
41.5	0.768	13.54	8.75	-6.08	14.83	210
31	0.706	12.44	8.04	-6.08	14.12	210
18	0.600	10.58	6.83	-6.08	12.91	210
5	0.570	10.05	6.49	-6.08	12.57	210

Gust Factor (South Building N-S)	
Factor	Design Value
g_q	3.4
g_v	3.4
$\dot{z}$	42
$I_{\dot{z}}$	0.29
$L_{\dot{z}}$	347
Q	0.80
G_f	0.81

South Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
PH	70'-0"	31	0.0	2146
MR	52'-0"	48	31	2477
5	41'-6"	33	78	1383
4	31'-0"	32	112	990
3	18'-0"	88	144	1591
2	5'-0"	82	232	408
1	-6'-0"	0.0	314	0.0
			V = 314	**ΣM = 8994**

South Building in the E-W Direction

Wind Pressures (South Building E-W)						
Height (ft)	K_z	q_z (psf)	Windward Wall (psf)	Leeward Walls (psf)	Total (psf)	Length in N-S Direction (ft)
70	0.890	15.69	10.42	-6.51	16.93	100
52	0.818	14.42	9.58	-6.51	16.09	100
41.5	0.768	13.54	8.99	-6.51	15.50	100
31	0.706	12.44	8.27	-6.51	14.78	100
18	0.600	10.58	7.03	-6.51	13.54	100
5	0.570	10.05	6.67	-6.51	13.18	100

Gust Factor (South Building E-W)	
Factor	Design Value
g_q	3.4
g_v	3.4
$\dot{z}$	42
$I_{\dot{z}}$	0.29
$L_{\dot{z}}$	347
Q	0.84
G_f	0.83

South Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
PH	70'-0"	15	0.0	1067
MR	52'-0"	24	15	1232
5	41'-6"	17	39	688
4	31'-0"	33	56	1035
3	18'-0"	85	89	1525
2	5'-0"	78	174	390
1	-6'-0"	0.0	252	0.0
			V = 252	**ΣM = 5937**

Presented above are table summaries of the wind load calculations performed for both the north and south buildings. Hand calculations were also performed and used with Excel spreadsheets and can be reviewed upon request.

SEISMIC LOAD CALCULATIONS

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

Factor	**Reference**
Site Class D	(Table 20.3.1)
$S_s = 0.15$	(Figure 22-1)
$S_1 = 0.051$	(Figure 22-2)
$T_L = 8$	(Figure 22-15)
Occupancy Category II	
$S_{ms} = 0.24$	(Table 11.4.1)
$S_{m1} = 0.1224$	(Table 11.4.2)
$S_{DS} = 0.16$	(eq. 11.4-3)
$S_{D1} = 0.0816$	(eq. 11.4-4)
SDC = B	
$T_S = 0.51$	
North Building T = 0.550 s	(§12.8.2)
South Building T = 0.798 s	(§12.8.2)
North Building R = 5	(Table 12.2-1)
South Building R = 3	(Table 12.2-1)
North Building $C_s = 0.0297$	
South Building $C_s = 0.0341$	

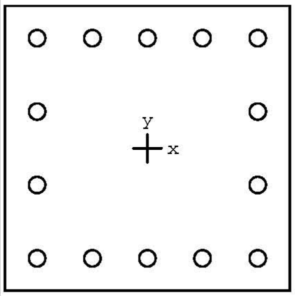
SEISMIC LOAD DISTRIBUTIONS

Vertical Distribution of Seismic Forces (North Building)						
Level	Height h_x (ft)	Story Weight w_x (K)	$w_x h_x^k$ (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
PH	83'-0"	2379	206419	127	127	10530
MR	65'-0"	2793	189273	116	243	7561
6	54'-2"	3087	174054	107	350	5795
5	43'-4"	3094	139197	86	436	3707
4	32'-6"	2759	92829	57	493	1854
3	21'-8"	1885	42116	26	519	561
2	10'-10"	1756	19471	12	531	130
$\Sigma w_i h_i^k$ =	863,359		**$\Sigma F_x = V =$**	531K	**ΣM =**	30,138ft-k

Vertical Distribution of Seismic Forces (South Building)						
Level	Height h_x (ft)	Story Weight w_x (K)	$w_x h_x^k$ (K)	Lateral Force Fx (K)	Story Shear Vx (K)	Moment at Floor (ft-K)
PH	77'-0"	756	111155	55	55	4239
MR	59'-0"	2237	242269	120	175	7079
5	48'-6"	2308	199631	99	274	4795
4	38'-0"	2325	151881	75	349	2858
3	25'-0"	1938	78278	39	388	969
2	12'-0"	2300	39959	20	408	237
$\Sigma w_i h_i^k$ =	823,173		**$\Sigma F_x = V =$**	408K	**ΣM =**	20,178ft-k

APPENDIX C – PCA COLUMN CHECK PRINTOUTS

COLUMN 16N-1/2 SPOT CHECK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

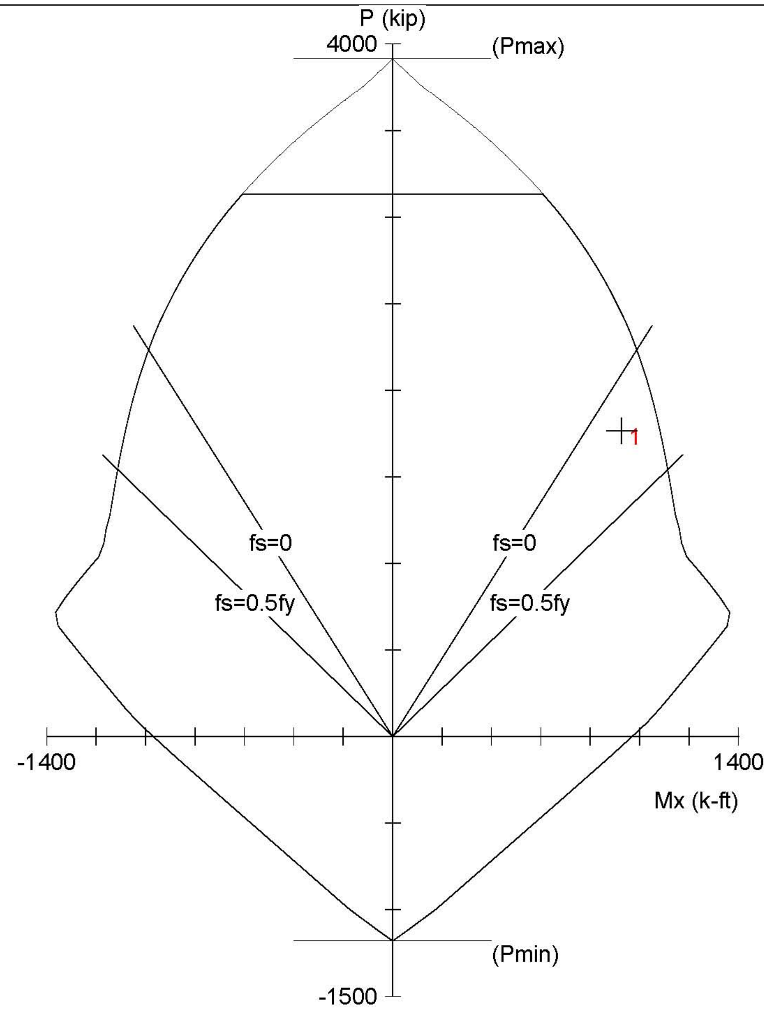
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 13:59:47

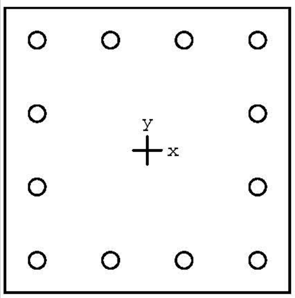


pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D

File: F:\PCA Column\16N\16N-1.col
Project: House of Sweden

Column: 16N-1		Engineer: KM	
f'c = 10 ksi	fy = 60 ksi	Ag = 576 in^2	14 #11 bars
Ec = 5700 ksi	Es = 29000 ksi	As = 21.84 in^2	Rho = 3.79%
fc = 8.5 ksi	fc = 8.5 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.65		Clear spacing = 3.24 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

COLUMN 16N-3 SPOT CHECK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

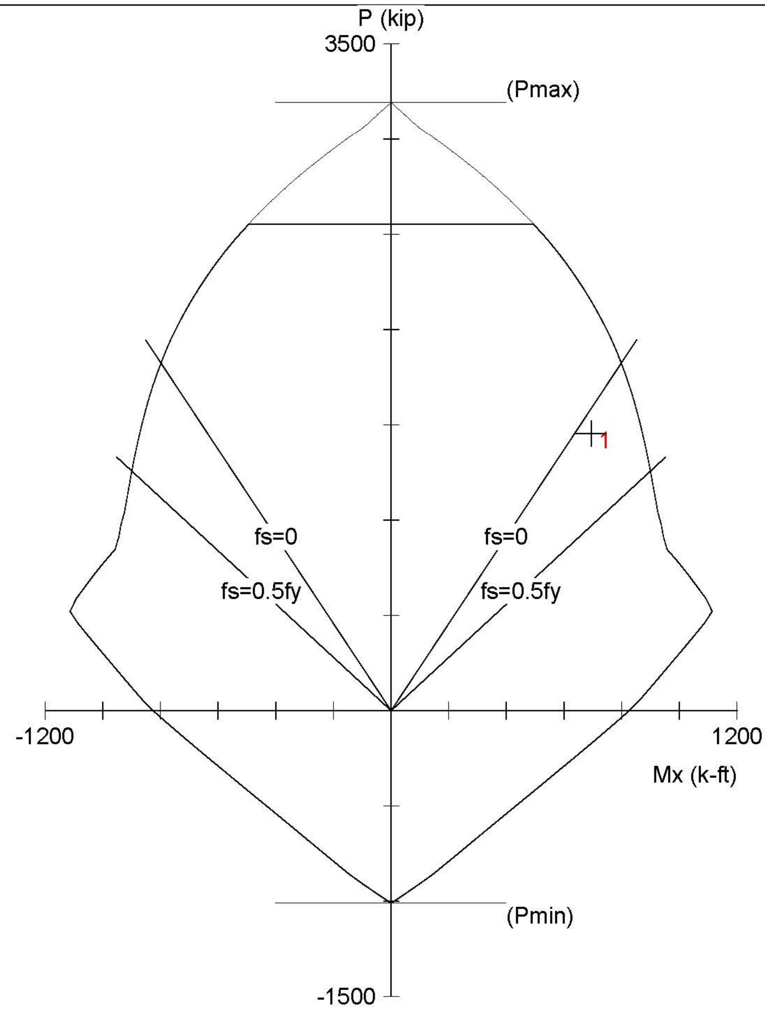
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 14:01:23



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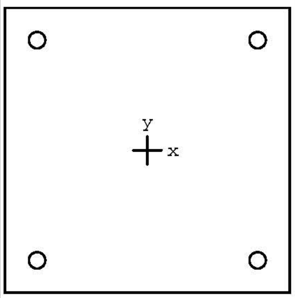
File: F:\PCA Column\16N\16N-3.col

Project: House of Sweden

Column: 16N-3		Engineer: KM	
f'c = 8 ksi	fy = 60 ksi	Ag = 576 in^2	12 #11 bars
Ec = 5098 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 3.25%
fc = 6.8 ksi	fc = 6.8 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.65		Clear spacing = 4.79 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

COLUMN 16N-4 SPOT CHECK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

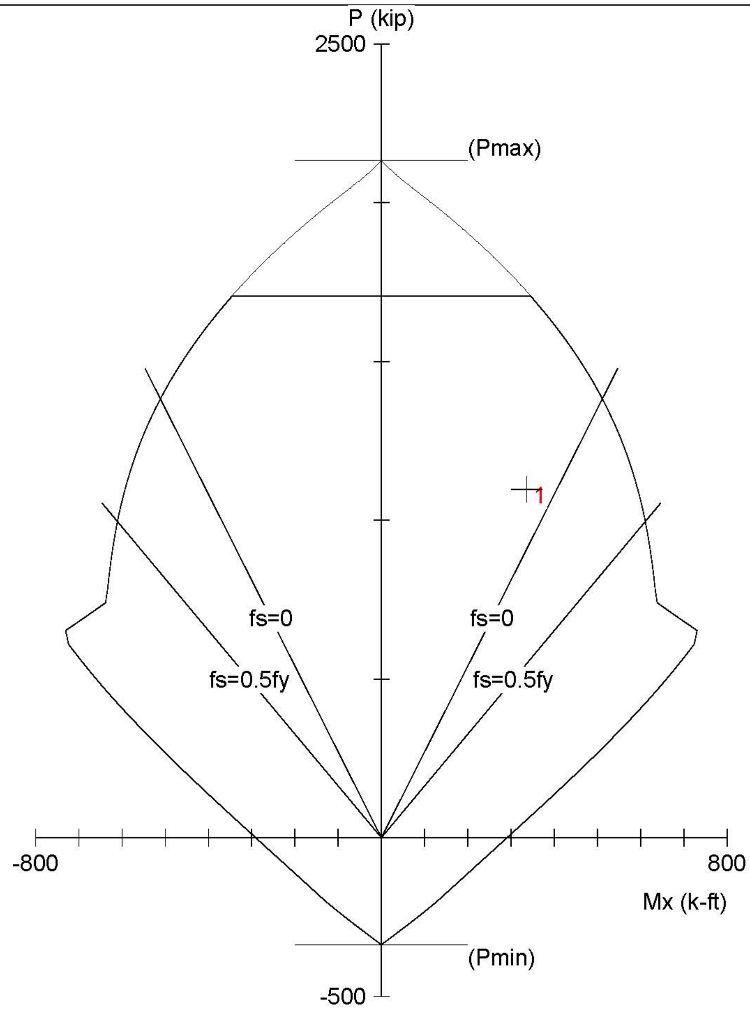
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 14:01:56



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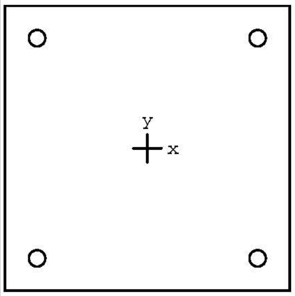
File: F:\PCA Column\16N\16N-4.col

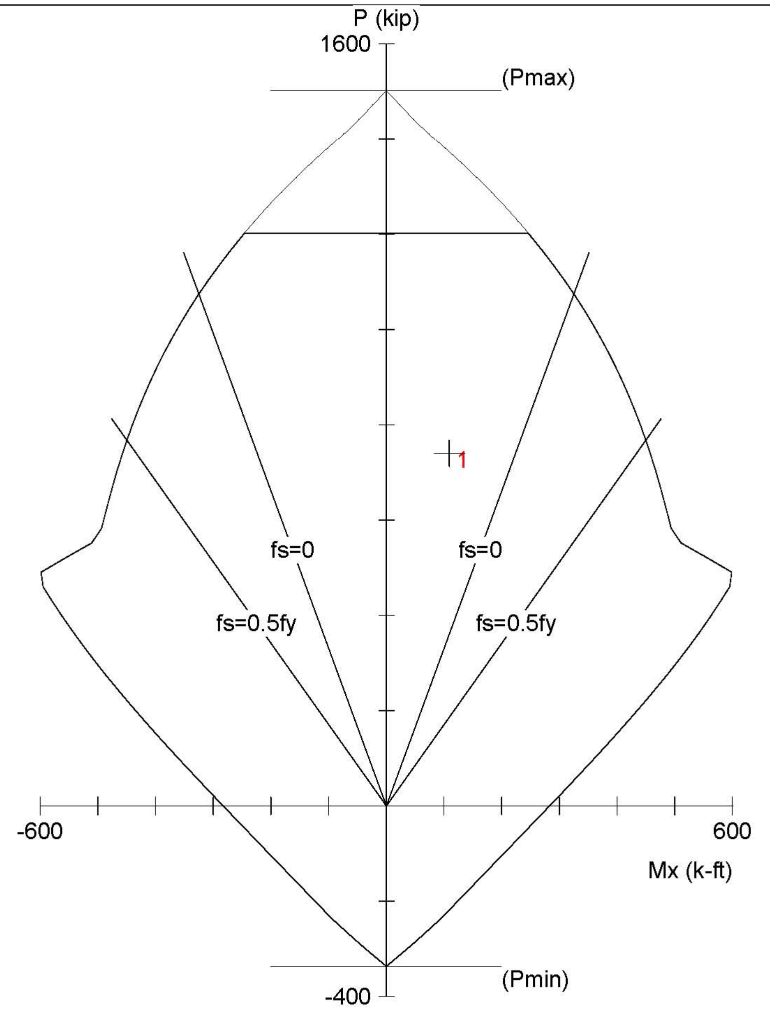
Project: House of Sweden

Column: 16N-4		Engineer: KM	
f'c = 6 ksi	fy = 60 ksi	Ag = 576 in^2	4 #11 bars
Ec = 4415 ksi	Es = 29000 ksi	As = 6.24 in^2	Rho = 1.08%
fc = 5.1 ksi	fc = 5.1 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.75		Clear spacing = 17.18 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

COLUMN 16N-5 SPOT CHECK



24 x 24 in



Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 14:03:25

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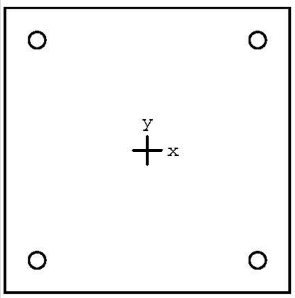
File: F:\PCA Column\16N\16N-5.col

Project: House of Sweden

Column: 16N-5		Engineer: KM	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	4 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 6.24 in^2	Rho = 1.08%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 17.18 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

COLUMN 16N-6 SPOT CHECK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

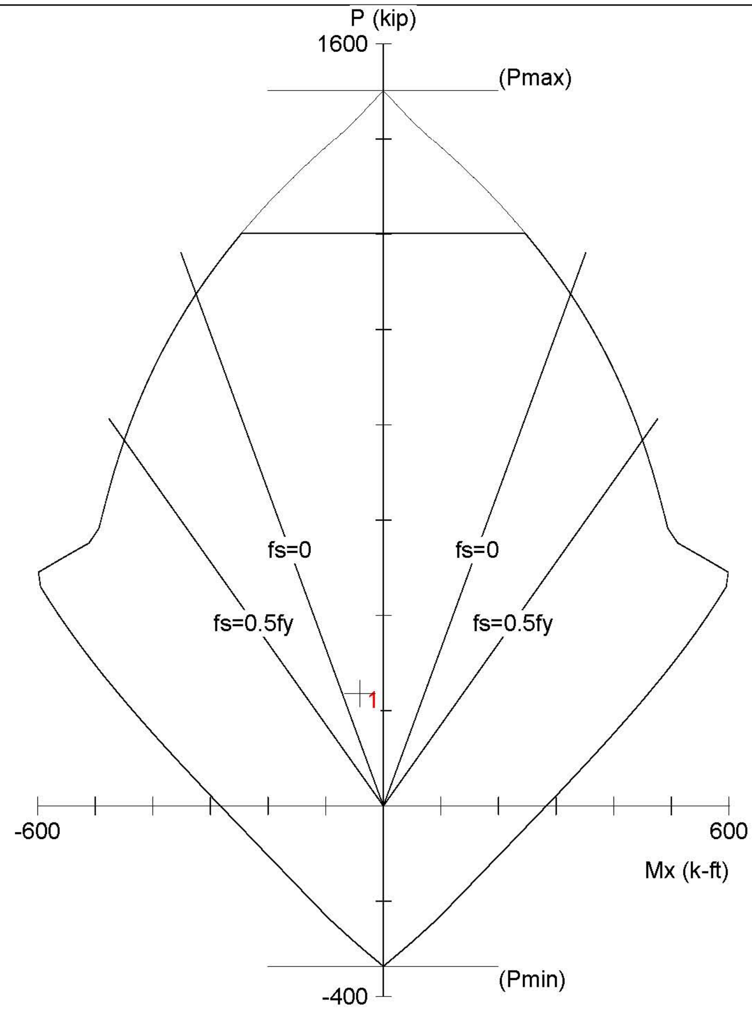
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 14:03:59



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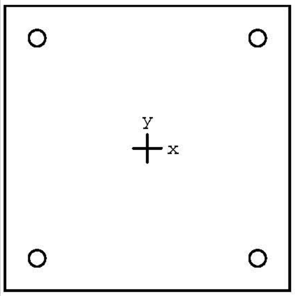
File: F:\PCA Column\16N\16N-6.col

Project: House of Sweden

Column: 16N-6		Engineer: KM	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	4 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 6.24 in^2	Rho = 1.08%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 17.18 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

COLUMN 16N-MR SPOT CHECK



24 x 24 in

Code: ACI 318-02

Units: English

Run axis: About X-axis

Run option: Investigation

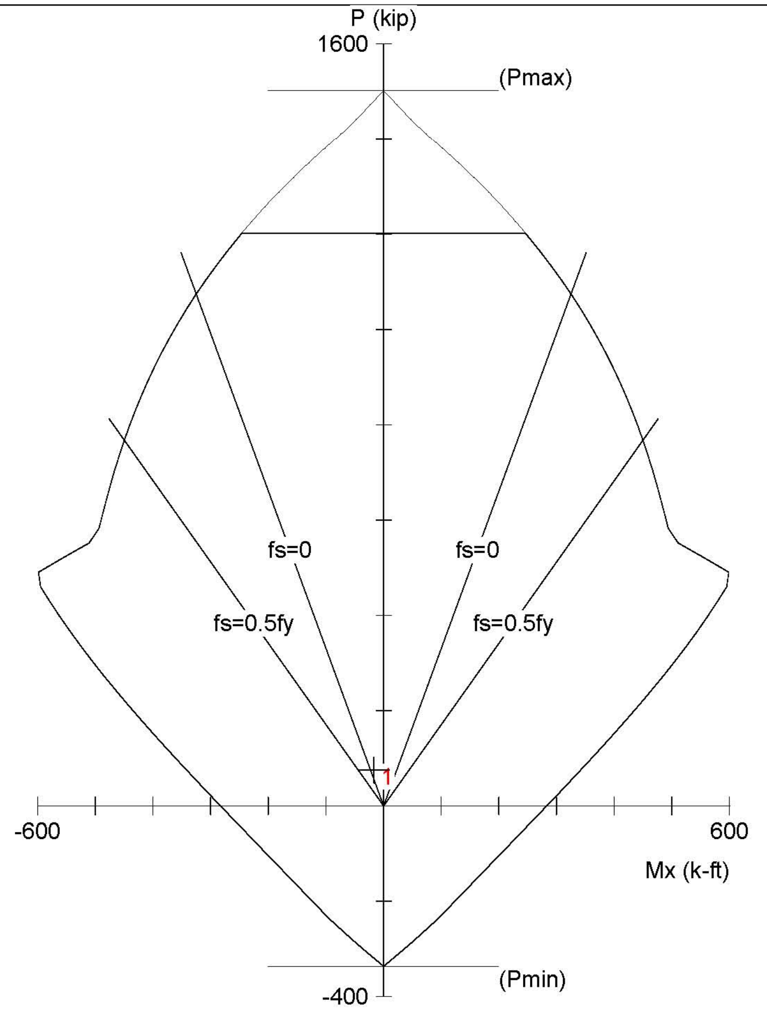
Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 11/19/08

Time: 14:04:36



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File: F:\PCA Column\16N\16N-MR.col

Project: House of Sweden

Column: 16N-MR		Engineer: KM	
f'c = 4 ksi	fy = 60 ksi	Ag = 576 in^2	4 #11 bars
Ec = 3605 ksi	Es = 29000 ksi	As = 6.24 in^2	Rho = 1.08%
fc = 3.4 ksi	fc = 3.4 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.85		Clear spacing = 17.18 in	Clear cover = 2.00 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		