House of Sweden

Structural System and Existing Conditions Report

2900 K St. NW
Washington, DC 20007

The Pennsylvania State University
Department of Architectural Engineering

Senior Thesis 2008-2009

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EXECUTIVE SUMMARY

Technical Report 1 is the Structural System and Existing Conditions Report. Research was performed into the structural system of the building and the methods used to design the system. This report uses the current standards to check the design; however, the standards used in the actual design were taken into account.

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. 7 levels exist in the north building and 6 in the south. The primary structural system is a two-way post-tensioned slab with drop panels. Shear walls exist in the north building for lateral support, but the south building is strictly 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. 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 over turning moment is the controlling lateral loads for both the north and south buildings.

Spot checks were performed. A post-tensioned slab spot check was performed on the fourth floor of the north building. The calculated slab design was fairly close to the actual design and the discrepancies likely resulted from the author's unfamiliarity with a post-tensioned slab system. Column capacity checks were performed on the vertical column line 11N at levels 5, 3, and the garage. These levels are where the concrete changed strength. All the columns passed the capacity check. Discrepancies occurred in the calculation of the axial load on the columns and most likely stem from the use of live load reduction factors.

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INTRODUCTION

This Structural System and Existing Conditions Report contains a description of the physical conditions currently existing in the House of Sweden, including a material strengths summary and required loading discussions. It provides a synopsis of the structural components including gravity and lateral load systems, foundation, and supports. This report also discusses the applicable design codes and the design practices used in this building through analysis of the House of Sweden's structural 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 foundation with a parking garage level and then two separate towers rise out of the site (Figure 1A. – Appendix A). 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 (Figure 2A. – Appendix A).

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 (Figure 3A. – Appendix A). 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 (Figure 4A. – Appendix A). 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.
- *Post-tensioned Concrete Floors* authored by Sami Khan and Martin Williams published in 1995 by Butterworth-Heinemann Ltd
- *Notes on ACI 318-08 Building Code Requirements for Structural Concrete* published in 2005 by the Portland Cement Association
- *Two-Way Post-Tensioned Design Example* published by the Portland Cement Association
- Construction Documents originally dated October 28, 2003 by VOA and TCE

STRUCTURAL SYSTEM DISCUSSION

Foundation

Cast-in-place piles support a 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.

Columns

A fairly regular rectangular column layout exists in the north building (Figure 1.). The gravity columns are rectangular reinforced, normal-weight concrete and range from 30” x 30”, 24” x 30”, 26”, 24”, and 12”, and 16” x 12”. At the 4th floor where the concrete frame turns into a concrete moment frame, the 28” x 20” columns are introduced and the 30” x 30” columns do not continue. Typical column strengths are 6 ksi from the garage to the third floor and 4 ksi from the third floor to the penthouse.

A rectangular column layout also exists in the south building (Figure 2.). Many of the gravity columns are 26” Φ columns with 8” square notches for pipe to travel vertically through the slab or corner L-shaped columns 20” to a side and 8” wide. Other repeating column sizes are 24” Φ, 28” x 28”, 24” x 44” or 24”, and 22” x 12”. At the 4th floor, 22” x 22” columns are introduced and the 26” Φ notched columns do not continue. Typical column strengths are 8 ksi from the garage to the third floors, 6 ksi from the third floor to the fifth floor and 4 ksi from the fifth floor to the penthouse.

Floor 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 flat plate concrete slabs with drop panels.

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.

Roof System

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

Shear walls make up the lateral system of the north building from the garage to the fourth floor (Figure 1). 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. These shear walls stop below the fifth floor where the structure becomes a concrete moment frame. This system resists lateral loads in the north-south and east-west direction depending upon the orientation of the wall.

Shear walls exist in the garage under the south building and are 12” thick with a concrete strength of 6 ksi reinforced with #4 bars at 12” spacing in two curtains. However, these walls do not extend past the garage level, and the building lateral system becomes a concrete moment frame to resist lateral loads in both the north-south and east-west directions.

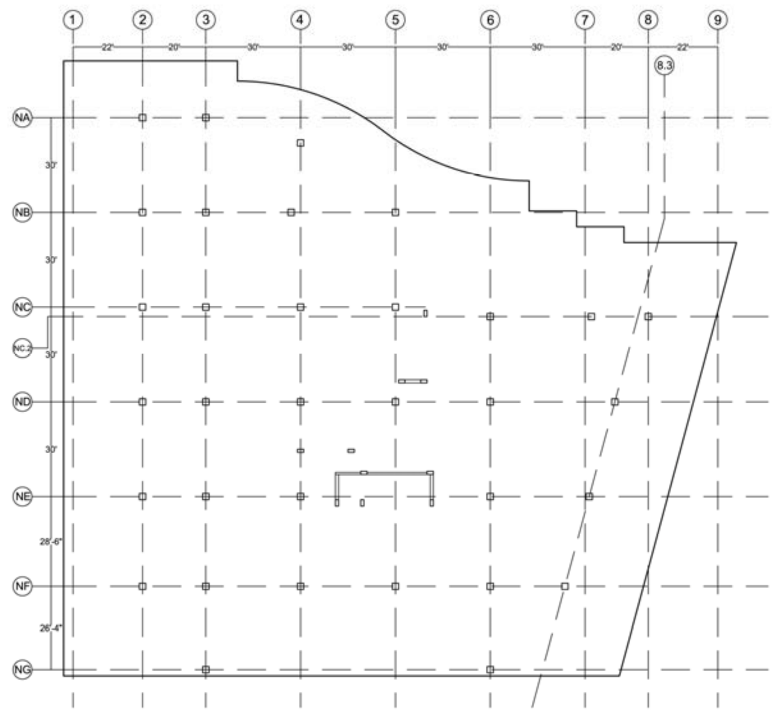


Figure 1. North Building Column and Shear Wall Layout

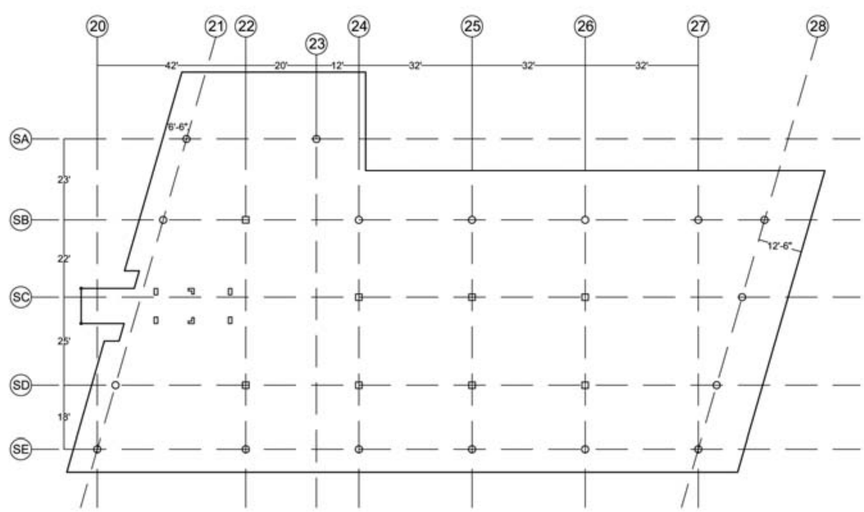


Figure 2. South Building Column Layout

MATERIAL STRENGTHS

Concrete

Mat Foundation.. f'_c = 4000 psi
Piles .... f'_c = 6000 psi
Slabs .... f'_c = 6000 psi
.... f'_c = 8000 psi

Columns (north building)

Parking to 3rd level.. f'_c = 8000 psi
3rd to 5th level.. f'_c = 6000 psi
5th to penthouse roof level .... f'_c = 4000 psi

Columns (south building)

Parking to 3rd level.. f'_c = 6000 psi
3rd to penthouse roof level .... f'_c = 4000 psi

Shear Walls .... f'_c = 6000 psi

Concrete Masonry

Concrete block units.. f'_m = 1900 psi

Stone

Limestone Panels.. f'_m = 8000 psi

Reinforcement

Rebar.. F_y = 60,000 psi
Steel wire.. F_y = 70,000 psi

Structural Steel

Wide flange shapes.. F_y = 50,000 psi
Channels, angles, plates, bars .... F_y = 36,000 psi
Cold formed, hollow structural sections.. F_y = 45,000 psi

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

Typical Live Load Deflection L/360

Typical Total Deflection L/240

Floor Dead Loads		
Occupancy	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		
Occupancy	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 B 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

Analyzing the lateral system of the House of Sweden is outside of the scope of this technical report and will be addressed at a later date. However, the lateral load applied to the building was calculated. These loads will be used in a later technical report focusing on the lateral system of the buildings. The following is a summary of the lateral loads. For more detailed calculations, please refer to the Appendix C.

Deflection Criteria

Lateral Deflection

Wind allowable inter-story drift H/500

Seismic allowable story drift H/400

Wind Load

Design wind load was calculated using ASCE 7-05 §6.5 Method 2 analysis. Method 2 does not take into account apparent shielding 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 C.

Factor	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 N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	77'-0"	30.0	30.0	2100.0
MR	59'-0"	46.7	76.7	2755.3
6	48'-2"	33.9	110.6	1633.0
5	37'-4"	32.7	143.3	1220.7
4	26'-6"	30.8	174.1	816.2
3	15'-8"	28.5	202.6	446.6
2	4'-10"	26.3	228.9	127.0
1	-6'-0"	0.0	228.9	0.0
			V = 228.9	**ΣM = 9099**

North Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	77'-0"	26.8	26.8	1876.0
MR	59'-0"	41.6	68.4	2454.4
6	48'-2"	30.0	98.4	1445.1
5	37'-4"	28.7	127.1	1071.4
4	26'-6"	26.7	153.8	707.6
3	15'-8"	24.3	178.1	380.8
2	4'-10"	22.2	200.3	107.2
1	-6'-0"	0.0	200.3	0.0
			V = 200.3	**ΣM = 8042**

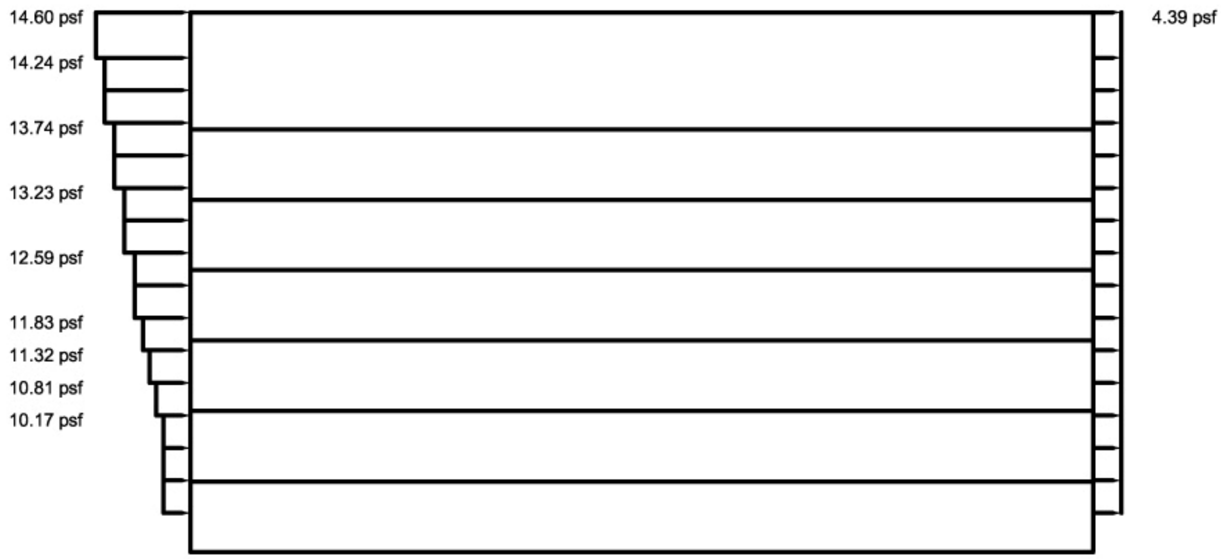


Figure 3. North Building Wind Pressure Diagram in the N-S Direction

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

South Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	15.5	15.5	1085.0
MR	52'-0"	23.8	39.3	1237.6
5	41'-6"	16.9	56.2	703.0
4	31'-0"	18.0	74.2	558.0
3	18'-0"	18.3	92.5	329.4
2	5'-0"	15.5	108.0	77.5
1	-6'-0"	0.0	108.0	0.0
			V = 108.0	**ΣM = 3991**

South Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	29.9	29.9	2093.0
MR	52'-0"	45.8	75.7	2381.6
5	41'-6"	32.5	108.2	1352.0
4	31'-0"	34.7	142.9	1075.7
3	18'-0"	35.1	178.0	631.8
2	5'-0"	29.7	207.7	148.5
1	-6'-0"	0.0	207.7	0.0
			V = 207.7	**ΣM = 7683**

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

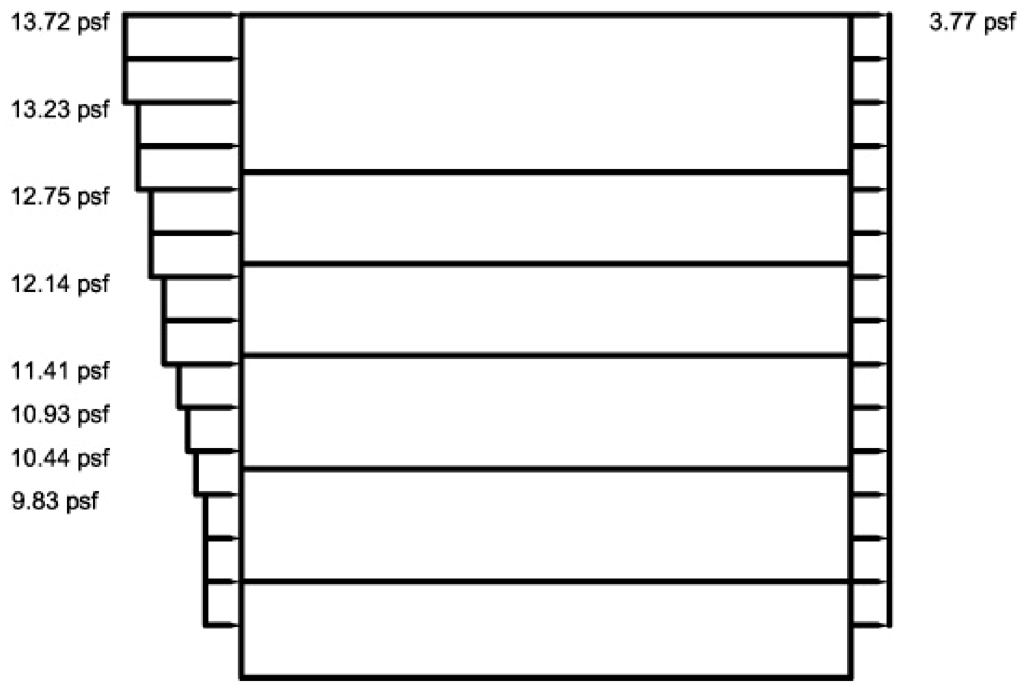


Figure 6. South Building Wind Pressure Diagram in the E-W Direction

North Building Wind Load Summary

- N-S Direction Base Shear: V = 228.9 K (Controls)
- N-S Direction Moment: ΣM = 9099 ft-K (Controls)
- E-W Direction Base Shear: V = 200.3 K
- E-W Direction Moment: ΣM = 8042 ft-K

South Building Wind Load Summary

- N-S Direction Base Shear: V = 108.0 K
- N-S Direction Moment: ΣM = 3991 ft-K
- E-W Direction Base Shear: V = 207.7 K (Controls)
- E-W Direction Moment: ΣM = 7683 ft-K (Controls)

Seismic Load

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 C.

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"	2380	126	126	0.239	10500
MR	65'-0"	2790	115	241	0.219	7480
6	54'-2"	3090	106	347	0.201	5740
5	43'-4"	3100	84.8	432	0.161	3670
4	32'-6"	2760	56.5	488	0.107	1840
3	21'-8"	1890	25.7	514	0.0488	557
2	10'-10"	1880	12.9	526	0.0241	140
$\Sigma w_i h_i^k$ =	865,194	**ΣF_x = V =**	526K		**ΣM =**	29,927ft-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"	828	55.7	55.7	0.0146	4290
MR	59'-0"	2230	110	166	0.0289	6490
5	48'-6"	2320	91.7	257	0.024	4450
4	38'-0"	2320	69.3	327	0.181	2630
3	25'-0"	1940	35.8	363	0.0938	895
2	12'-0"	2390	19	382	0.0497	228
$\Sigma w_i h_i^k$ =	835,443	**$\Sigma F_x = V =$**	382K		**ΣM =**	18,983ft-k

Figure 8. South Building Seismic Force Diagram

North Building Seismic Load Summary:
Base Shear: V = 526 K
Moment: ΣM = 29,927 ft-K

South Building Seismic Load Summary:
Base Shear: V = 382 K
Moment: ΣM = 18,983 ft-K

Seismic loads control the lateral design for both the north and south buildings.

STRUCTURAL DESIGN DISCUSSION

Two types of spot checks were performed for this technical report, one slab spot check and three column spot checks. All spot checks were performed on the north building structural system. Originally, it was the intent of the author to perform one slab spot check in each of the buildings and five column checks, one every time the vertical column line changed concrete strength. After more investigation into the structure, it was determined that the slabs of the south building participate in the lateral system. Analyzing the lateral system is outside of the scope of this technical report. Therefore, it was decided to strictly focus the spot checks on the north building in the areas where shear walls made up the lateral system. It is understood that the slab in the north building will also carry some lateral however, however the total amount of lateral load carried by the floor slab and columns in significantly reduced due to the presence of the shear walls.

Spot check one was performed on the fourth floor slab in between column lines 3-5 and NB-ND (Figure 9.). This area was chosen due to the single thickness of slab and the regular bay spacing. The equivalent frame method and moment distribution was used to determine the moments on the slab. Then, the strength of the slab and the amount of post-tensioning force necessary to balance 75% of the dead load was determined. An example for a flat plate was followed and the author attempted to modify the example to include drop panels. The final calculated design is an 8" post-tensioned slab with drop panels. In the E-W direction, 30 tendons are bundled to produce 810K. In the N-W direction, 30 tendons are uniformly distributed to total 810K. 7 #5 bars @12 o.c. top reinforcing is necessary at the supports and #5 bars @ 12" o.c. is necessary at the end spans.

For the column spot checks, three capacity checks of the existing vertical column line 11N were performed at each floor where the concrete strength changed, floors five and three and the garage (Figure 9.). Due to the simplifying assumption that the columns are not part of the lateral system the columns were assumed to experience no moment. Due to concrete being the structural material, the columns will see a moment and carry some of the lateral load. If a column design had been performed, the columns would have come out as an undersized design due to not including the lateral load. By checking capacity, the author was able to determine if the columns in place were suitable for the axial load and if there was room for bending moment to be introduced into the system. All columns were determined to be of adequate design with room to include a small moment.

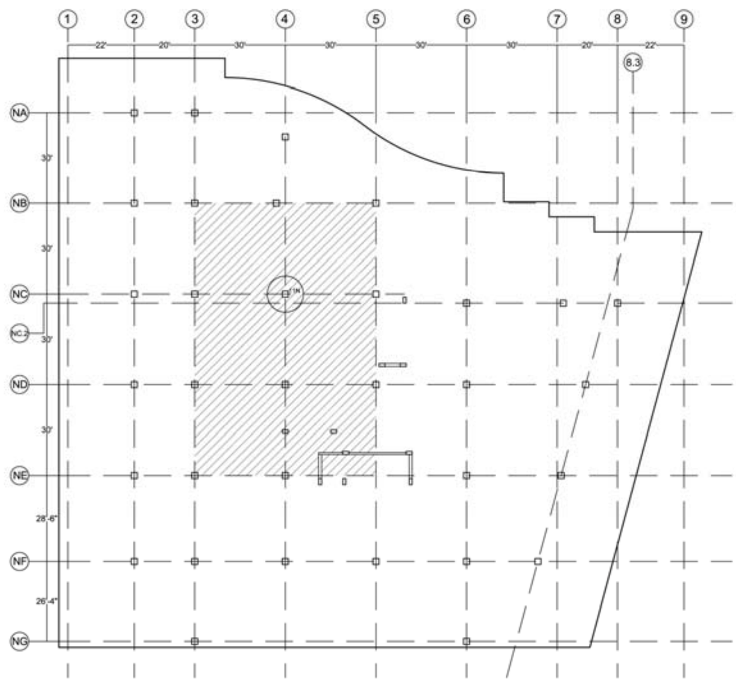


Figure 9. North Building Spot Check Identification

CONCLUSION

Spot check one was the most challenging of all the spot checks performed. Since it was approached as a design, there were an infinite amount of outcomes possible. The calculated outcome was fairly close to the original design. An 8" slab was calculated, versus the 7" slab in existence, but the amount of tendon force calculated is the same as the design. Discrepancies most likely occurred in the assumptions the author made about the drop panels in the equivalent frame method equations. Another difference is the layout of the tendons. Non-uniform tendon layout in the N-S direction was called for in the design. Since the author was just recently introduced to post-tensioned design, a uniform layout was done for this report. The structural engineer used the computer program RISA to design the slabs and columns; therefore, an economic design was easier to obtain.

As stated above, the column spot checks were performed as capacity checks. All the columns passed the capacity check and still have room to introduce a moment into the column. The main difference in the calculations and the design comes from the calculated load on the column. The most likely source of this discrepancy is the use of the code load when it was higher than the design load. Also, the author did not have access to the reduction factors for the live load, so discrepancies may also exist in that calculation.

APPENDIX A – PHOTOGRAPHS

PHOTOGRAPHS

Figure 1A. Rendering of the House of Sweden development

Figure 2A. Night view of the North Building

PHOTOGRAPHS

Figure 3A. Picture of the South Building

Figure 4A. Comparison of the North and South Building exterior cladding

APPENDIX B – GRAVITY LOAD CALCULATIONS

SNOW AND RAIN LOAD CALCULATIONS

Presented below are table summaries of the snow 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 C – LATERAL LOAD CALCULATIONS

WIND LOAD CALCULATIONS

Factor	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	$q_z = 0.00256K_zK_{zt}K_dV^2I$	Pressure Windward Wall pz = qzGfCp-qh(GCpi)	Pressure Leeward Walls ph = qhGfCp-qh(Gcpi)	Total
77	0.918	16.18	14.60	-4.39	18.99
70	0.89	15.69	14.24	-4.39	18.64
60	0.85	14.98	13.74	-4.39	18.13
50	0.81	14.28	13.23	-4.39	17.62
40	0.76	13.40	12.59	-4.39	16.98
30	0.7	12.34	11.83	-4.39	16.22
25	0.66	11.63	11.32	-4.39	15.71
20	0.62	10.93	10.81	-4.39	15.20
0-15	0.57	10.05	10.17	-4.39	14.56

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	0.837
$V_{\dot{z}}$	64.6
N_1	5.4
R_n	0.0483
R_h	0.17
R_B	0.0786
R_L	0.0734
R	0.0861
G_f	0.903

North Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	77'-0"	30.0	30.0	2100.0
MR	59'-0"	46.7	76.7	2755.3
6	48'-2"	33.9	110.6	1633.0
5	37'-4"	32.7	143.3	1220.7
4	26'-6"	30.8	174.1	816.2
3	15'-8"	28.5	202.6	446.6
2	4'-10"	26.3	228.9	127.0
1	-6'-0"	0.0	228.9	0.0
			V = 228.9	**ΣM = 9099**

North Building in the E-W Direction

Wind Pressures (north building E-W)					
Height (ft)	K_z	$q_z = 0.00256K_zK_{zt}K_dV^2I$	Pressure Windward Wall pz = qzGfCp-qh(Gcpi)	Pressure Leeward Walls ph = qhGfCp-qh(Gcpi)	Total
77	0.918	16.18	14.43	-1.41	15.84
70	0.89	15.69	14.08	-1.41	15.49
60	0.85	14.98	13.58	-1.41	14.99
50	0.81	14.28	13.08	-1.41	14.49
40	0.76	13.40	12.45	-1.41	13.86
30	0.7	12.34	11.70	-1.41	13.10
25	0.66	11.63	11.20	-1.41	12.60
20	0.62	10.93	10.69	-1.41	12.10
0-15	0.57	10.05	10.07	-1.41	11.47

Gust Factor (north building E-W)	
Factor	Design Value
g_q	3.4
g_v	3.4
g_r	4.18
$\dot{z}$	42
$I_{\dot{z}}$	0.288
$L_{\dot{z}}$	347
Q	0.818
$V_{\dot{z}}$	63.1
N_1	5.36
R_n	0.0485
R_h	0.178
R_B	0.0718
R_L	0.0768
R	0.0845
G_f	0.089

North Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	77'-0"	26.8	26.8	1876.0
MR	59'-0"	41.6	68.4	2454.4
6	48'-2"	30.0	98.4	1445.1
5	37'-4"	28.7	127.1	1071.4
4	26'-6"	26.7	153.8	707.6
3	15'-8"	24.3	178.1	380.8
2	4'-10"	22.2	200.3	107.2
1	-6'-0"	0.0	200.3	0.0
			V = 200.3	**ΣM = 8042**

South Building in the N-S Direction

Wind Pressures (south building N-S)					
Height (ft)	K_z	$q_z = 0.00256K_zK_{zt}K_dV^2I$	Pressure Windward Wall pz = qzGfCp-qh(Gcpi)	Pressure Leeward Walls ph = qhGfCp-qh(Gcpi)	Total
70	0.89	15.69	13.39	-3.84	17.23
60	0.85	14.98	12.92	-3.84	16.76
50	0.81	14.28	12.45	-3.84	16.29
40	0.76	13.40	11.86	-3.84	15.70
30	0.7	12.34	11.15	-3.84	15.00
25	0.66	11.63	10.68	-3.84	14.53
20	0.62	10.93	10.21	-3.84	14.06
0-15	0.57	10.05	9.62	-3.84	13.47

Gust Factor (south building N-S)	
Factor	Design Value
g_q	3.4
g_v	3.4
$\dot{z}$	42
$I_{\dot{z}}$	0.288
$L_{\dot{z}}$	347
Q	0.845
G_f	0.835

South Building N-S				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	15.5	15.5	1085.0
MR	52'-0"	23.8	39.3	1237.6
5	41'-6"	16.9	56.2	703.0
4	31'-0"	18.0	74.2	558.0
3	18'-0"	18.3	92.5	329.4
2	5'-0"	15.5	108.0	77.5
1	-6'-0"	0.0	108.0	0.0
			V = 108.0	**ΣM = 3991**

South Building in the E-W Direction

Wind Pressures (south building E-W)					
Height (ft)	K_z	$q_z = 0.00256K_zK_{zt}K_dV^2I$	Pressure Windward Wall pz = qzGfCp-qh(Gcpi)	Pressure Leeward Walls ph = qhGfCp-qh(Gcpi)	Total
70	0.89	15.69	13.72	-3.77	17.49
60	0.85	14.98	13.23	-3.77	17.01
50	0.81	14.28	12.75	-3.77	16.52
40	0.76	13.40	12.14	-3.77	15.91
30	0.7	12.34	11.41	-3.77	15.19
25	0.66	11.63	10.93	-3.77	14.70
20	0.62	10.93	10.44	-3.77	14.21
0-15	0.57	10.05	9.83	-3.77	13.61

Gust Factor (south building E-W)	
Factor	Design Value
g_q	3.4
g_v	3.4
$\dot{z}$	42
$I_{\dot{z}}$	0.288
$L_{\dot{z}}$	347
Q	0.89
G_f	0.861

South Building E-W				
Story	Height (ft)	Force (K)	Shear (K)	Moment (ft-K)
P	70'-0"	29.9	29.9	2093.0
MR	52'-0"	45.8	75.7	2381.6
5	41'-6"	32.5	108.2	1352.0
4	31'-0"	34.7	142.9	1075.7
3	18'-0"	35.1	178.0	631.8
2	5'-0"	29.7	207.7	148.5
1	-6'-0"	0.0	207.7	0.0
			V = 207.7	**ΣM = 7683**

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

SIESMIC LOAD CALCULATIONS

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

Factor **Reference**

Site Class D .... (Table 20.3.1)
S_s = 0.15 .... (Figure 22-1)
S_1 = 0.051 .... (Figure 22-2)
T_L = 8.. (Figure 22-15)
Occupancy Category II
S_{ms} = 0.24.. (Table 11.4.1)
S_{m1} = 0.1224 .... (Table 11.4.2)
S_{DS} = 0.16 .... (eq. 11.4-3)
S_{D1} = 0.0816.. (eq. 11.4-4)
SDC = B
TS = 0.51
North Building T = 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

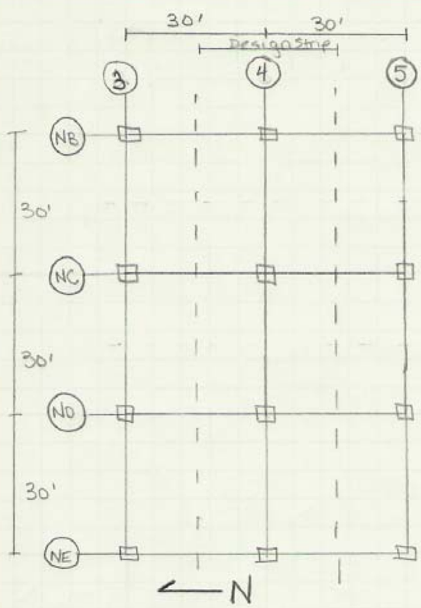
Vertical Distribution of Seismic Forces (north building)						
Level	Height h_x (ft)	Story Weight w_x (K)	Lateral Force Fx (K)	Story Shear Vx (K)	C_{vx}	Moment at Floor (ft-K)
P	83'-0"	2380	126	126	0.239	10500
MR	65'-0"	2790	115	241	0.219	7480
6	54'-2"	3090	106	347	0.201	5740
5	43'-4"	3100	84.8	432	0.161	3670
4	32'-6"	2760	56.5	488	0.107	1840
3	21'-8"	1890	25.7	514	0.0488	557
2	10'-10"	1880	12.9	526	0.0241	140
$\Sigma w_i h_i^k$ =	865,194	**$\Sigma F_x = V =$**	526K		**ΣM =**	29,927ft-k

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"	828	55.7	55.7	0.0146	4290
MR	59'-0"	2230	110	166	0.0289	6490
5	48'-6"	2320	91.7	257	0.024	4450
4	38'-0"	2320	69.3	327	0.181	2630
3	25'-0"	1940	35.8	363	0.0938	895
2	12'-0"	2390	19	382	0.0497	228
$\Sigma w_i h_i^k$ =	835,443	**$\Sigma F_x = V =$**	382K		**ΣM =**	18,983ft-k

APPENDIX D – SPOT CHECK CALCULATIONS

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 1



Columns : 24" x 24"
Story height : 10'-10"

$f'c$ = 6 ksi (slab), 6 ksi (columns)
* slab strength varies on plans but for this report, just the 6 ksi will be used

f_y = 60 ksi
f_{pu} = 270 ksi
f'_{ci} = 4500 psi
w = 150 pcf (N-W concrete)
LL = 100 psf (commercial occupancy)

PT: Unbonded tendons
½" ϕ, 7 wire strands, $A = 0.153\ in^2$
estimated prestress losses = 15 ksi
$f_{se} = 0.7(270) - 15 = 174$ ksi
$P_{eff} = Af_{se} = 0.153(174) = 26.6$ K/tendon

14" deep drop panels

Preliminary Slab Thickness

$$\frac{span}{depth} = \frac{L}{45} = \frac{360}{45} = 8''$$

Use Equivalent Frame Method

Shear Strength of Slab

d = 6.94" (¾" clear cover and #5 bars)

Factored Load

LL reduction: Interior Bay

$A_T = 30(30) = 900\ ft^2$
$K_{LL} = 1$
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 0.75(100) = 75\ psf$$

$$w_u = 1.2(8/12)(150) + 1.6(75) = 240\ psf$$

consider a 12" wide strip taken at distance d from face of support

$$V_u = 0.240(12.4) = 2.98\ K$$
$$\phi V_c = \phi 2\sqrt{f'c}\,bwd = 0.75(2)\sqrt{6000}(12)(20.9) = 29.1\ K > V_u \checkmark$$

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 2

Shear strength at distance $d/2$

$$V_u = 0.240[(30)(30) - 1.29^2] = 216\,K$$

$$\phi V_c = \phi 4\sqrt{f'_c}\, b_o d = 0.75(4)\sqrt{6000}\,(4 \cdot 15.47)(20.9)/1000 = 301\,K > V_u \checkmark$$

Frame members of Equivalent Force

- Flexural Stiffness of Slab-beams at both ends, K_{sb}

$$\frac{C_{N1}}{\ell_1} = \frac{24}{30(12)} = 0.07 \qquad \frac{C_{N2}}{\ell_2} = \frac{24}{30(12)} = 0.07$$

by Interpolation from Tables A6
with drop panels thickness = 1.75h

C_1/ℓ_1	C_2/ℓ_2	K_{NF}	C_{NF}	m_{NF}	K_{FN}	C_{FN}	m_{FN}
0.07	0.07	5.99	0.51	0.1093	4.65	0.63	0.0732

$$E_{cs} = 57000\sqrt{f'_c} = 57000\sqrt{6000} = 4.42 \cdot 10^6$$

$$I_s = \frac{\ell_2 h^3}{12} = \frac{30(12)(8)^3}{12} = 15360\ \text{in}^4$$

$$K_{sb} = K_{NF}\frac{E_{cs} I_s}{\ell_1} = \frac{5.99\,(4.42 \cdot 10^6 \cdot 15360)}{30(12)} = 1130 \cdot 10^6\ \text{in-lb}$$

- Flexural Stiffness of Column Members at both ends K_c

Table A7 $t_a = 18''$ $t_b = 18''$ $t_a/t_b = 1.0$
$H = 10'\text{-}10'' = 130''$ $H_c = 112''$ $H/H_c = 1.16$

K_{AB} 5.84
C_{AB} 0.604

$$I_c = \frac{c^4}{12} = \frac{24^4}{12} = 27648\ \text{in}^4$$

$$E_{cs} = 57000\sqrt{6000} = 4.42 \cdot 10^6$$

$$K_c = 5.84\, E_{cc} I_c/\ell_c = \frac{5.84\,(4.42 \cdot 10^6)(27648)}{130} = 5490 \cdot 10^6\ \text{in-lb}$$

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 3

- Torsional Stiffness of Torsional Members, K_t

$$K_t = \frac{9 E_{cs} C}{[\ell_2(1-c_2/\ell_2)^3]} = \frac{9(4.42\cdot10^6)(4224.7)}{360(1-{}^{24}/_{360})^3} = 574 \text{ in-lb}$$

\# so high due to drop panels

- Equivalent Column Stiffness, K_{ec}

$$K_{ec} = \frac{\Sigma K_c \bullet \Sigma K_t}{\Sigma K_c + \Sigma K_t} = \frac{2(5490\cdot10^6)(2)(574\cdot10^6)}{(2)(5490\cdot10^6)+2(574\cdot10^6)} = 1039\cdot10^6 \text{ in-lb}$$

- Slab beam joint Distribution Factors

at exterior joint

$$DF = \frac{1130}{1130+1039} = 0.521$$

at interior joint

$$DF = \frac{1130}{1130+1130+1039} = 0.343$$

COF = 0.509

Partial frame analysis of equivalent frame

$w_D = 100$ psf $w_L = 75$ psf $w_{bal} = 75$ psf

- FEM for slab beams

Dead load

$$m_{nF}\, w_u\, \ell_2\, \ell_1^2 = 0.1093(100)(30)(30)^2 = 295 \text{ ft-K}$$

Live Load

$$m_{nF} = 0.1093(75)(30)(30)^2 = 221 \text{ ft-K}$$

Balanced Load

$$m_{nF} = 0.1093(75)(30)(30)^2 = 221 \text{ ft-K}$$

Refer to Spreadsheet for Moment Distribution

SLAB SPOT CHECK

Dead Load Moments

Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	295	-295	295	-295	295	-295
DIST	-153.7					153.7
CO		-78.23			78.23	
DIST		128.02			-	
CO			65.16	-65.16		
DIST			-	123.54		
CO			62.88	-62.88		
DIST			-32.01	32.01		
CO		-16.29			16.29	
DIST		5.59			-5.59	
CO	2.84					-2.84
DIST	-1.48					1.48
CO		-0.75			0.75	
DIST		0.26			-0.26	
CO			0.13	-0.13		
DIST			-0.05	0.05		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	142.7	-256.4	267.6	-267.6	256.4	-142.7
M @ Midspan	137.95		69.91		137.95	

Live Load Moments

Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	221	-221	221	-221	221	-221
DIST	-115.1					115.1
CO		-58.61			58.61	
DIST		95.91			-	
CO			48.82	-48.82		
DIST			-	92.55		
CO			47.11	-47.11		
DIST			-	23.98		
CO		-12.20			12.20	
DIST		4.19			-4.19	
CO	2.13					-2.13
DIST	-1.11					1.11
CO		-0.57			0.57	
DIST		0.19			-0.19	
CO			0.10	-0.10		
DIST			-0.03	0.03		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	106.9	-192.1	200.5	-200.5	192.1	-106.9
M @ Midspan	103.63		52.66		103.63	

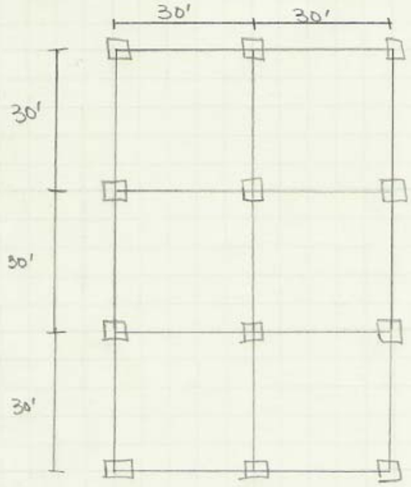
Balanced Load Moments

Joint	1	2		3		4
Member	1-2	2-1	2-3	3-2	3-4	4-3
DF	0.521	0.343	0.343	0.343	0.343	0.521
COF	0.509	0.509	0.509	0.509	0.509	0.509
FEM	221	-221	221	-221	221	-221
DIST	-115.1					115.1
CO		-58.61			58.61	
DIST		95.91			-95.91	
CO			48.82	-48.82		
DIST			-92.55	92.55		
CO			47.11	-47.11		
DIST			-23.98	23.98		
CO		-12.20			12.20	
DIST		4.19			-4.19	
CO	2.13					-2.13
DIST	-1.11					1.11
CO		-0.57			0.57	
DIST		0.19			-0.19	
CO			0.10	-0.10		
DIST			-0.03	0.03		
CO		-0.02			0.02	
DIST		0.01			-0.01	
CO	0.00					0.00
DIST	0.00					0.00
Neg. M	106.9	-192.1	200.5	-200.5	192.1	-106.9
M @ Midspan	103.63		52.66		103.63	

SLAB SPOT CHECK

Slab | Spot Check | North Building | 5

Look at the Slab between column lines 3-5 and NB-ND
on the 4th Floor
- Commercial Occupancy -



Loads

Framing Dead Load = selfweight
Live Load = 100 psf Commercial

Materials

Concrete: NW 150 pcf
$f'_c = 6000$ psi
$f'_{ci} = 4500$ psi

Rebar: $F_y = 60{,}000$ psi

PT: Unbonded tendons.
$1/2"\ \phi$, 7 wire strands, $A = 0.153\ in^2$
$f_{pu} = 270{,}000$ psi
Estimated prestress losses = 15 ksi
$f_{se} = 0.7(270) - 15 = 174$ ksi
$P_{eff} = Af_{se} = 0.153(174) = 27$ k/tendon

Determine Preliminary Slab Thickness

8" (Determined Previously)

Loading
DL = 100 psf
$L_o = 100$ psf

LL reduction : Interior Bay

$A_T = 30(30) = 900\ ft^2$
$K_{LL} = 1$
$$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL}A_T}}\right) = 0.75L_o = 75 \text{ psf}$$

Design of E-W Interior Frame

- Total Bay width between center lines = 30'
- No pattern loading

Calculate Section Properties

$A = bh = 360(8) = 2880\ in^2$
$S = bh^2/6 = 360(8)^2/6 = 3840\ in^3$

SLAB SPOT CHECK

Slab | Spot Check | North Building | 6

Set Design Parameters

At time of Jacking
f'_{ci} = 4500 psi
Compression = $0.60 f'_{ci}$ = 2700 psi
Tension = $3\sqrt{f'_{ci}}$ = 201 psi

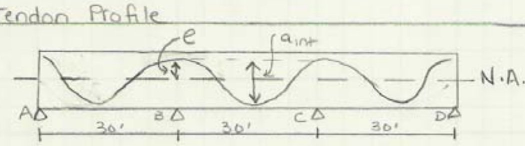
At service loads
f'_c = 6000 psi
compression = $0.45 f'_c$ = 2700 psi
Tension = $6\sqrt{f'_c}$ = 465 psi

Average Precompression Limits
P/A = 125 psi min
= 300 psi max

Target Load Balances
60% - 80% of DL for slabs
For this example $0.75\, w_{DL}$ = 75 psf

Cover Requirements
3/4" clear cover top and bottom

Tendon Profile



Tendon ordinate	Tendon (CG) location
Interior support - top	7.0"
Interior span - bottom	1.0"

a_{int} = 7.0" - 1.0" = 6.0"
Eccentricity, e, is the distance from the center of the tendon to the N.A. It varies along the span.

Prestress Force Required to Balance - 75% of the DL

$w_b = 0.80 w_{DL} = 0.75(100)(30) = 2250$ plf

Force needed in tendons to counteract load

$P = w_b L^2/8a = 2250(30)^2/8(\frac{3.75}{12}) = 810$ K

SLAB SPOT CHECK

Slab | Spot Check | North Building | 7

Check Precompression Allowance

Determine number of tendons needed
tendons = 810/27 = 30
Use 30 tendons

Actual Force for banded tendons

$P_{actual} = 30(27) = 810$ K

Adjust balanced load

$w_b = (810/810)(2250) = 2.25$ K/ft

Determine actual precompression stress

$P_{actual}/A = 810(1000)/2880 = 281$ psi > 125 psi min ✓
< 300 psi max ✓

$\boxed{P_{eff} = 810\text{K}}$

Check Slab Stresses

• Stage 1: Stresses immediately after Jacking (DL + PT)

Midspan Stresses
$f_{top} = (-M_{DL} + M_{bal})/S - P/A$
$f_{bottom} = (+M_{DL} - M_{bal})/S - P/A$

Interior Span
$f_{top} = (-69.91 + 52.66)(12)(1000)/3840 - 281$
$= -335$ psi (compression < $0.6f'_{ci}$ = 2700 psi ✓
$f_{bottom} = (69.91 - 52.66)(12)(1000)/3840 - 281$
$= -227$ psi (compression) < $0.6f'_{ci}$ = 2700 psi ✓

Exterior Span
$f_{top} = (-137.95 + 103.93)(12)(1000)/3840 - 281$
$= -387$ psi (compression) < $0.6f'_{ci}$ = 2700 psi ✓
$f_{bottom} = (137.95 - 103.93)(12)(1000)/3840 - 281$
$= -270$ psi (compression) < $0.6f'_{ci}$ = 2700 psi ✓

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 8

Support Stresses

$f_{top} = (+M_{DL} - M_{bal})/S - P/A$

$f_{bottom} = (-M_{DL} + M_{bal})/S - P/A$

* used difference of Moment Distribution Moments

$f_{top} = (524.02 - 392.56)(12)(1000)/3840 - 281$

$= 130$ psi (tension) $< 3\sqrt{f'_{ci}} = 201$ psi ✓

$f_{bottom} = (-524.02 + 392.56)(12)(1000)/3840 - 281$

$= -692$ psi (compression) $< 0.6 f'_{ci} = 2700$ psi

• Stage 2: Stresses at Service Load (DL + LL + PT)

Midspan Stresses

$f_{top} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{bottom} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

Interior Span

$f_{top} = (-69.91 - 52.66 + 52.66)(12)(1000)/3840 - 281$

$= -499$ psi (compression) $< 0.45 f'_c = 2700$ psi ✓

$f_{bottom} = (69.91 + 52.66 - 52.66)(12)(1000)/3840 - 281$

$= -62.5$ psi (compression) $< 0.45 f'_c = 2700$ psi ✓

Exterior Span

$f_{top} = (-137.95 - 103.63 + 103.63)(12)(1000)/3840 - 281$

$= -712$ psi (compression) $< 0.45 f'_c = 2700$ psi ✓

$f_{bottom} = (137.95 + 103.63 - 103.63)(12)(1000)/3840 - 281$

$= 150$ psi (tension) $< 6\sqrt{f'_c} = 465$ psi ✓

Support Stresses

$f_{top} = (+M_{DL} + M_{LL} - M_{bal})/S - P/A$

$f_{bottom} = (-M_{DL} - M_{LL} + M_{bal})/S - P/A$

$f_{top} = (524.02 + 392.56 - 392.56)(12)(1000)/3840 - 281$

$= 1360$ psi (tension) $> 6\sqrt{f'_c} = 465$ psi →

$f_{bottom} = (-524.02 - 392.56 + 392.56)(12)(1000)/3840 - 281$

$= -1920$ psi (compression) $< 0.45 f'_c = 2700$ psi ✓

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 9

Ultimate Strength

Primary post-tensioning moments, M_1

$M_1 = P * e$
$e = 0$ in. at the exterior support
$e = 3.0$ in at the interior support (N.A. to center of tendon)

$M_1 = (810)(3)/12 = 203$ ft-K

Secondary post-tensioning Moments, M_{sec}

$M_{sec} = M_{bal} - M_1$
$= 392.56 - 203 = 190$ ft-K at interior supports
$=$ at midspan

$M_u = 1.2M_{DL} + 1.6M_{LL} + 1.0M_{sec}$

At Midspan: $M_u = 1.2(137.95) + 1.6(103.63) + 1.0(95) = 426$ ft-K
At Support: $M_u = 1.2(-524.02) + 1.6(-392.56) + 1.0(190) = -1070$ ft-K

Determine minimum bonded reinforcement

- Positive Moment Region
  Interior Span: $f_t = 15\text{psi} < 2\sqrt{f'c} = 2\sqrt{6000} = 155\text{psi}$
  no positive reinforcement required

  Exterior Span: $f_t = 180\text{psi} > 155\text{psi}$

Minimum positive moment reinforcement required

$y = f_t/(f_t + f_c)h$
$= [180/(180+622)]8 = 1.80\text{in}$

$N_c = M_{DL+LL}/S + 0.5\, y\, l_2$
$= (137.95 + 103.63)(12)/3840 + 0.5(1.80)(30)(12))$
$= 245$ K

$A_{s,min} = N_c/0.5f_y$
$= 245/0.5(60)$
$= 8.17\ in^2$

$A_{s,min} = 8.17/30 = 0.272\ in^2/ft$

Use #5 @ 12" oc Bottom $= 0.31\ in^2/ft$

SLAB SPOT CHECK

4th Floor Slab | Spot Check | North Building | 10

• Negative Moment Region

A_{cf} = max 8 (30+30)/2 (12) = 2880
8 (30)/12 = 2880

Interior Supports

$A_{s\,min}$ = 0.00075 A_{cf} Try 7 #5 top ($A = 2.17 in^2$)
= 0.00075(2880)
= 2.16 in^2

Exterior Supports

A_{cf} = max 8(30/2)(12) = 1440
8(30)(12) = 2880 Try 7 #5 top ($A = 2.17 in^2$)

$A_{s\,min}$ = 0.00075 A_{cf}
= 0.00075(2880)
= 2.16 in^2

maximum bar spacing

= 1.5h = 1.5(8) = 12"

Check that minimum reinforcement is sufficient

$M_n = (A_s f_y + A_{ps} f_{ps})(d - a/2)$

d = effective depth
A_{ps} = 0.153(30) = 4.59 in^2
$f_{ps} = f_{se} + 10{,}000 + (f'_c bd)/300 A_{ps}$
$a = (A_s f_y + A_{ps} f_{ps})/0.85 f'_c b$

At Supports
d = 18" − 3/4" − 1/4" = 17"
f_{ps} = 174000 + 10000 + (6000 (30)(12)(17))/300(4.59)
= 210,667 psi
a = (2.17(60) + 4.59(211))/0.85 (6)(30)(12)
= 0.59

ϕM_n = 0.9 (2.17(60) + 4.59(211)) (17 − 0.59/2)
= 1376 ft-k > 1070 ft-k Minimum reinforcement ok

7 #5 @ 12" oc Top at supports

SLAB SPOT CHECK

At Midspan (end span)

$d = 8 - 1.5 - .25 = 6.25''$

$f_{ps} = 184000 + (6000(30)(12)(6.25))/300(4.59)$

$= 193{,}804$ psi

$a = [9.3(60) + 4.59(195)]/0.85(6)(30)(12) = 0.79$

$\phi M_n = 0.9[9.3(60) + 4.59(195)][6.25 - 0.79/2]/12$

$= 638$ ft-K > 426 ft-K Minimum reinforcement ok

#5 @ 12" oc Bottom at end spans

In Conclusion

Design: 8" post-tensioned slab with drop panels in the E-W direction, 30 tendons are bundled to give 810 K. In the N-W direction, 30 tendons are uniformly distributed to total 810 K.

Use #5 @ 12" oc Bottom reinforcing at end spans
Use 7#5 @ 12" oc Top reinforcing at supports

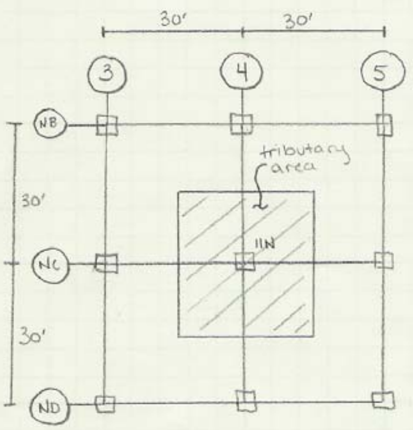
COLUMN SPOT CHECK

Concrete Weight					
Item	Volume (ft^3)	Factored Weight (K)	Item	Volume (ft^3)	Factored Weight (K)
Penthouse	525	94.5	Main Roof Column	48	8.64
Main Roof	600	108	Sixth Floor Column	43.3	7.8
Sixth Floor	525	94.5	Fifth Floor Column	43.3	7.8
Fifth Floor	525	94.5	Fourth Floor Column	43.3	7.8
Fourth Floor	525	94.5	Third Floor Column	43.3	7.8
Third Floor	600	108	Second Floor Column	43.3	7.8
Second Floor	525	94.5	First Floor Column	54.2	9.76
First Floor	788	142	Garage Floor Column	45	8.1

Loads						
Item	Area (ft^2)	Live Load (psf)	Reduced Live Load (psf)	Dead Load	Snow Load (psf)	Total Factored Load (K)
Penthouse	900	--	--	8	30	28.9
Main Roof	900	100	75	--	--	108
Sixth Floor	900	60	45	--	--	64.8
Fifth Floor	900	100	75	--	--	108
Fourth Floor	900	100	75	--	--	108
Third Floor	900	100	75	--	--	108
Second Floor	900	100	75	--	--	108
First Floor	900	100	75	30	--	140

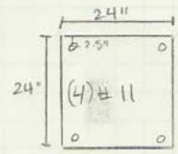
COLUMN SPOT CHECK

5th Floor Column 11N | Spot Check | North Building



Column: 24" x 24"
Story Height: 10'-10"

$f'_c = 4$ Ksi
$f_y = 60$ Ksi
(4) #11 bars
See tables for loads



Total Load $P_u = 519K$

Symetrical Column ∴ either bending axis produces the same result.

$A_s = 6.24\ in^2$ $\quad\phi\ 1.41"$

Assume at this level, column is not part of the lateral system ∴ $M = 0$

Pure Compression

$\epsilon_y = \frac{f_y}{E_s} = \frac{60}{29000} = 0.00207$

$\epsilon_c = \epsilon_u = 0.003$

$P_o = 0.85 f'_c A_c + A_s f_y$
$P_o = 0.85(4)(24)(24) + 6.24(60)$
$P_o = 2333\ K$

$\phi = 0.65$

$\phi P_o = 1516\ K > P_u = 519\ K$ ✓

Balanced Strain

$c = \frac{0.003}{0.003 + \epsilon_y} d_{max} = \frac{0.003}{0.003 + 0.00207}(21.5) = 12.7"$

$f_{s1} = \frac{0.003}{12.7}(12.7 - 2.5)\ 29000 = 69.9 > 60 \leftarrow$

$f_{s2} = \frac{0.003}{12.7}(12.7 - 21.5)\ 29000 = -60.3 > -60 \leftarrow$

COLUMN SPOT CHECK

5th Floor Column 11N | Spot Check | North Building

$$P_b = 0.85(4)(24)(0.85)(12.7) + 3.12(60) + 3.12(-60)$$
$$= 881\text{ K}$$
$$\phi P_b = 603\text{ K}$$
$$M_b = 0.85(4)(24)(0.85)(12.7)\left(\frac{24}{2} - \frac{0.85(12.7)}{2}\right) + 3.12(60)\left(\frac{24}{2} - 2.5\right) + 3.12(-60)\left(\frac{24}{2} - 21.5\right)$$
$$= 485\text{ ft-K}$$
$$\phi M_b = 332\text{ ft-K}$$

Pure Tension

$T_o = \Sigma A_{si} f_{si}$ all $f_{si} = -f_y$

$$= 3.12(-60) + 3.12(-60) = -374\text{ K}$$
$$\phi T_o = -337\text{ K}$$

Pure Bending

Assume $\epsilon_1 < \epsilon_y$ $\epsilon_2 = \epsilon_y$

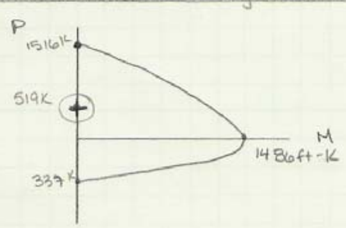
$$f_{s1} = \frac{0.003}{c}(c - 2.5)(29000)$$
$$f_{s2} = -60$$

$$0 = 0.85(4)(24)(0.85)c + (3.12)\frac{0.003}{c}(c-2.5)(29000) + (3.12)(-60)$$
$$0 = 69.36c + 271.44 - 678.6/c - 187.2$$
$$0 = 69.36c^2 + 84.24c - 678.6$$
$$c = 2.58''$$

$f_{s1} = 2.70$ $f_{s2} = -60$

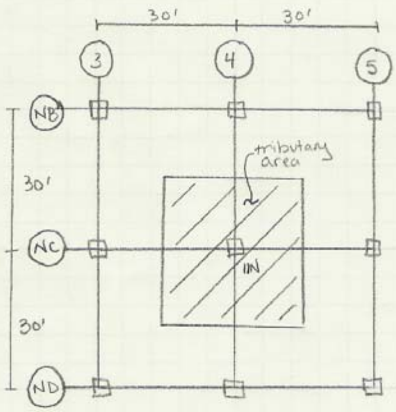
$$M_o = 0.85(4)(24)(0.85)(2.58)\left(\frac{24}{2} - \frac{0.85(2.58)}{2}\right) + 3.12(2.70)\left(\frac{24}{2} - 2.5\right) + 3.12(-60)\left(\frac{24}{2} - 21.5\right)$$
$$= 1651\text{ ft-K} \qquad \phi M_o = 1486\text{ ft-K}$$

Interaction Diagram



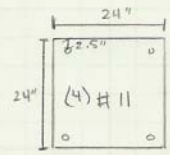
COLUMN SPOT CHECK

3rd Floor Column 11N | Spot Check | North Building



Column : 24" × 24"
Story Height : 10'-10"

$f'_c = 6\,ksi$
$f_y = 60\,ksi$
(4) #11 bars
see tables for loads



Total load $P_u = 940K$

Symetrical Column ∴ either bending axis produces the same results

$A_s = 6.24\,in^2$ ϕ 1.41"

Assume at this level, the column is not part of the lateral system ∴ M=0

Pure Compression

$\epsilon_y = 0.00207$

$P_o = 0.85 f'_c A_c + A_s f_y$
$P_o = 0.85(6)(24)(24) + 6.24(60)$
$P_o = 3323\,K$

$\phi P_o = 2160\,K > P_u = 940\,K$ ✓

Balanced Strain

$c = \frac{0.003}{0.003 + 0.00207}(21.5) = 12.7"$

$f_{s1} = \frac{0.003}{12.7}(12.7 - 2.5)(29000) = 69.9 > 60 \leftarrow$

$f_{s2} = \frac{0.003}{12.7}(12.7 - 21.5)(29000) = -60.3 > -60 \leftarrow$

$P_L = 0.85(6)(24)(0.75)(12.7) + 3.12(60) + 3.12(-60)$
$= 1166K$

$\phi P_L = 798\,K$

COLUMN SPOT CHECK

3rd Floor Column 11N | Spot Check | North Building

$$M_b = 0.85(6)(24)(0.75)(12.7)\left(\frac{24}{2} - \frac{0.75(12.7)}{2}\right) + 3.12(60)\left(\frac{24}{2} - 2.5\right) + 3.12(-60)\left(\frac{24}{2} - 21.5\right)$$

$$= 1000 \text{ ft-K}$$

$$\phi M_b = 684 \text{ ft-K}$$

Pure Tension

$T_o = \Sigma A_{si} f_{si}$ $f_{si} = -f_y$

$T_o = 6.24(-60) = -374 \text{ K}$

$\phi T_o = -337 \text{ K}$

Pure Bending

Assume $\epsilon_1 < \epsilon_y$ $\epsilon_2 = \epsilon_y$

$$f_{s1} = \frac{0.003}{c}(c-2.5)(29000)$$

$$f_{s2} = -60$$

$$0 = 0.85(5)(24)(0.75)c + 3.12\left(\frac{0.003}{c}\right)(c-2.5)(29000) + 3.12(-60)$$

$$0 = 76.5c + 271.44 - 678.6/c - 187.2$$

$$0 = 76.5c^2 + 84.24c - 678.6$$

$$c = 2.48''$$

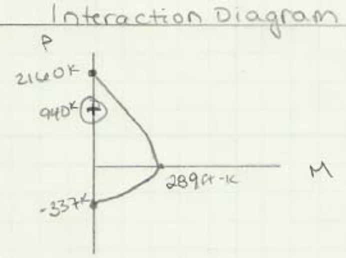
$f_{s1} = -0.70$ $f_{s2} = -60$

$$M_o = 0.85(5)(24)(0.75)(2.48)\left(\frac{24}{2} - \frac{0.75(2.58)}{2}\right) + 3.12(-0.70)\left(\frac{24}{2} - 2.5\right) + 3.12(-60)\left(\frac{24}{2} - 21.5\right)$$

$$= 321 \text{ ft-K}$$

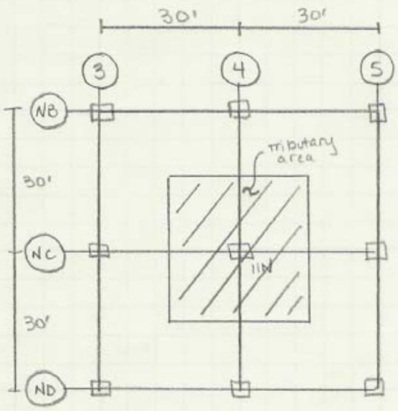
$$\phi M_o = 289 \text{ ft-K}$$

Interaction Diagram



COLUMN SPOT CHECK

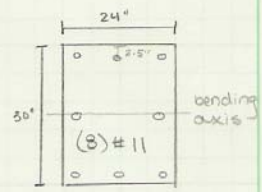
Garage Floor Column 11N | Spot Check | North Building



Column : 24" x 30"
Story Height : 9'-0"

$f'_c = 5$ ksi
$f_y = 60$ ksi
(8)#11 bars
See tables for loads

Total Load $P_u = 1666$ K



$A_{s1} = 4.68\ in^2$ $\quad d_1 = 2.5"$
$A_{s2} = 3.12\ in^2$ $\quad d_2 = 15"$ $\quad \phi\ 1.41"$
$A_{s3} = 4.68\ in^2$ $\quad d_3 = 27.5"$

Assume at this level, the column is not part of the lateral system $\therefore M = 0$

Pure Compression

$\epsilon_y = 0.00207$

$P_o = 0.85(5)(24)(30) + 12.48(60)$
$= 3809$ K

$\phi P_o = 2476\ K > P_u = 1666\ K$ ✓

Balanced Strain

$c = \dfrac{0.003}{0.003 + 0.00207}(27.5) = 16.3"$

$f_{s1} = \dfrac{0.003}{16.3}(16.3 - 2.5)(29000) = 73.7 > 60 \leftarrow$

$f_{s2} = \dfrac{0.003}{16.3}(16.3 - 15)(29000) = 6.94$

$f_{s3} = \dfrac{0.003}{16.3}(16.3 - 27.5)(29000) = -59.8$

COLUMN SPOT CHECK

Garage Floor Column 11N | Spot Check | North Building

$P_b = 0.85(5)(24)(0.7)(16.3) + 4.68(60) + 3.12(6.94) + 4.68(-59.8)$
$= 1186K$
$\phi P_b = 777K$

$M_b = 0.85(5)(24)(0.7)(16.3)\left(\frac{30}{2} - \frac{0.7(16.3)}{2}\right) + 4.68(60)\left(\frac{30}{2} - 2.5\right)$
$+ 3.12(6.94)\left(\frac{30}{2} - 15\right) + 4.68(-59.8)\left(\frac{30}{2} - 27.5\right)$

$= 1486$ ft-K
$\phi M_b = 973$ ft-K

Pure Tension

$T_o = \Sigma A_{si} f_{si}$ all $f_{si} = -f_y$

$T_o = 4.68(-60) + 3.12(-60) + 4.68(-60)$
$= -749K$
$\phi T_o = -674K$

Pure Bending

Assume $\epsilon_1 < \epsilon_y$ $\epsilon_2 = \epsilon_y$ $\epsilon_3 = \epsilon_y$

$f_{s1} = \frac{0.003}{c}(c-2.5)(29000)$ $f_{s2} = -60$ $f_{s3} = -60$

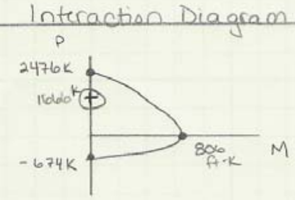
$0 = 0.85(5)(24)(0.7)c + (4.68)\frac{0.003}{c}(c-2.5)(29000) + 3.12(-60) + 4.68(-60)$

$0 = 71.4c + 407.16 - 1017.9/c - 187.2 - 280.8$
$0 = 71.4c^2 - 60.84c - 1017.9$
$c = 4.23''$

$f_{s1} = 35.6$ $f_{s2} = -223 > -60 \leftarrow$ $f_{s3} = -479 > -60 \leftarrow$

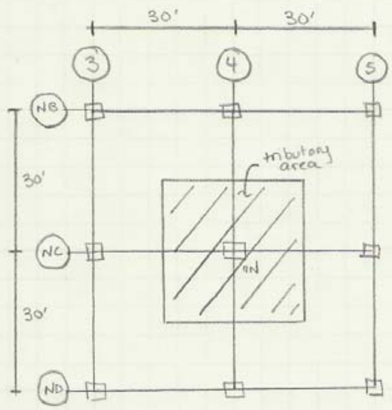
$M_o = 0.85(5)(24)(0.7)(4.23)\left(\frac{30}{2} - \frac{0.7(4.23)}{2}\right) + 4.68(35.6)\left(\frac{30}{2} - 2.5\right) + 3.12(-60)\left(\frac{30}{2} - 15\right)$
$+ 4.68(-60)\left(\frac{30}{2} - 27.5\right)$
$= 806$ ft-K
$\phi M_o = 725$ ft-K

Interaction Diagram



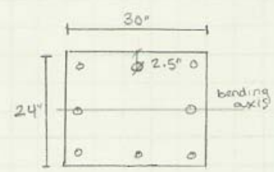
COLUMN SPOT CHECK

Garage Floor Column 11N | Spot Check | North Building



Column: 24" x 30"
Story Height: 9'

$f'_c = 5$ ksi
$f_y = 60$ ksi
(8) #11 bars
See tables for loads



Total Load $P_u = 1666K$

$A_{s1} = 4.68$ in^2 $d_1 = 2.5"$
$A_{s2} = 3.12$ in^2 $d_2 = 12"$ ϕ 1.41"
$A_{s3} = 4.68$ in^2 $d_3 = 21.5"$

Assume at this level, the column is not part of the lateral system $\therefore M = 0$

Pure Compression

$\epsilon_y = 0.00207$

$P_o = 0.85(5)(30)(24) + 12.48(60)$
$= 3809K$

$\phi P_o = 2476K > P_u = 1666K$

Balanced Strain

$c = \frac{0.003}{0.003+0.00207}(21.5) = 12.7"$

$f_{s1} = \frac{0.003}{12.7}(12.7-2.5)(29000) = 69.9 > 60 \leftarrow$

$f_{s2} = \frac{0.003}{12.7}(12.7-12)(29000) = 4.80$

$f_{s3} = \frac{0.003}{12.7}(12.7-21.5)(29000) = -60.3 > -60 \leftarrow$

COLUMN SPOT CHECK

Garage Floor Column 11N | Spot Check | North Building

$P_b = 0.85(5)(30)(0.7)(12.7) + 4.68(60) + 3.12(4.80) + 4.68(-60)$
$= 1148K$
$\phi P_b = 754K$

$M_b = 0.85(5)(30)(0.7)(12.7)\left(\frac{24}{2} - \frac{0.7(12.7)}{2}\right) + 4.68(60)\left(\frac{24}{2} - 2.5\right)$
$+ 3.12(4.80)\left(\frac{24}{2} - 12\right) + 4.68(-60)\left(\frac{24}{2} - 21.5\right)$

$= 1158 ft\text{-}K$
$\phi M_b = 760 ft\text{-}K$

Pure Tension

$T_o = \Sigma A_{si} f_{si}$ all $f_{si} = -f_y$

$T_o = 4.68(-60) + 3.12(-60) + 4.68(-60)$
$= -749K$
$\phi T_o = -674K$

Pure Bending

Assume $\epsilon_1 < \epsilon_y$ $\epsilon_2 = f_y$ $\epsilon_3 = f_y$

$f_{s1} = \frac{0.003}{c}(c-2.5)(29000)$ $f_{s2} = 60$ $f_{s3} = -60$

$0 = 0.85(5)(30)(0.7)c + (4.68)\frac{0.003}{c}((c-2.5)(29000)) + 3.16(-60) + 4.68(-60)$

$0 = 89.25c + 407.16 - {}^{1017.9}/c - 189.6 - 280.8$
$0 = 89.25c^2 - 63.24c - 1017.9$
$c = 3.75''$

$f_{s1} = 29$ $f_{s2} = -191 > -60 \leftarrow$ $f_{s3} = -412 > -60 \leftarrow$

$M_o = 0.85(5)(30)(0.7)(3.75)\left(\frac{24}{2} - \frac{0.7(3.75)}{2}\right) + 4.68(29)\left(\frac{24}{2} - 2.5\right) + 3.12(-60)\left(\frac{24}{2} - 12\right)$
$+ 4.68(-60)\left(\frac{24}{2} - 21.5\right)$

$= 3311 ft\text{-}K$
$\phi M_o = 2980 ft\text{-}K$

Interaction Diagram