35 WEST 21ST STREET NEW YORK, NY

FINAL THESIS REPORT

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STRUCTURAL OPTION
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35 WEST 21ST STREET

NEW YORK, NY

GENERAL BUILDING DATA :

- BUILDING HEIGHT: 15 STORIES WITH A TOTAL HEIGHT OF 150 FT.
- SQUARE FOOTAGE: 162,000 SQ. FT.
- OCCUPANCY: RETAIL AND RESIDENTIAL
- CONSTRUCTION DATES: 01/02/08 TO PRESENT
- BUILDING COST: $20,000,000
- PROJECT DELIVERY: DESIGN-BID-BUILD
- OWNER: ROSLAND/STEMPLE ASSOCIATES, LLC
- ARCHITECT: SLCE*ARCHITECTS*
- STRUCTURAL ENGINEER: DESIMONE CONSULTING ENGINEERS
- MECHANICAL ENGINEER: COSENTINI ASSOCIATES
- CM: PLAZA CONSTRUCTION CORPORATION

ARCHITECTURE:

- T-SHAPE PLAN
- CLAD IN PRECAST CONCRETE PANELS WITH A CAST STONE, BRICK VENEER, OR GRANITE FINISH.
- PRECAST PANELS AND WINDOWS ARRANGED TO MATCH THE CHARACTER OF SURROUNDING BUILDINGS.

STRUCTURE:

- TWO WAY FLAT PLATE CONCRETE FLOOR SYSTEM WITHOUT DROPS.
- TYPICAL SLAB THICKNESS = 8"
- TYPICAL SLAB REINFORCEMENT CONSISTS OF #5@12" OC BOTTOM BARS E.W., AND #5@12" OC MIDDLE STRIP TOP BARS.
- REINFORCED CONCRETE COLUMN SIZES RANGE FROM 14"x16" TO 30"x18".
- LATERAL SYSTEM CONSISTS OF SHEAR WALL THAT EXTEND THE FULL HEIGHT OF THE BUILDING.
- COLUMN FOUNDATIONS CONSIST OF SPREAD FOOTS SUPPORTED BY MINI CAISSONS THAT GO DOWN TO BED ROCK

http://www.engr.psu.edu/ae/thesis/portfolios/2009/dpd5003/index.html

DANIEL P. DONECKER STRUCTURAL OPTION

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Executive summary

The purpose of this thesis is to optimize and redesign critical elements of the 16 story portion of the building structure. Past analysis conducted on 35 West 21st Street shows that there is a potential to increase the number levels above grade, while maintaining the existing height of the building. This means that the owner has the possibility of increasing the number of rentable units while maintaining upscale architectural spaces. With this added floor, revenue could easily be increased. The depth study focuses on structural issues created by adding an extra floor within the same height of the building, while breadth studies focus on the impacts of these structural changes on the architecture and mechanical systems.

The structural optimization begins with the column grid. The existing grid is replaced with a regular format in order to create a symmetrical layout of shear walls as well as improve constructability. The existing shear walls are replaced with a new shear wall layout that conforms to the new column grid with minimal effect on architecture and building period. Previous studies have revealed that by decreasing the typical floor thickness, the potential for adding another floor within the same building height significantly increases. It was discovered in previous studies that the most efficient way to reduce slab thickness is to utilize a post-tensioned two-way flat plate concrete slab. Although this will increase the initial cost of the building, increased revenue generated by the extra floor will quickly overcome this cost.

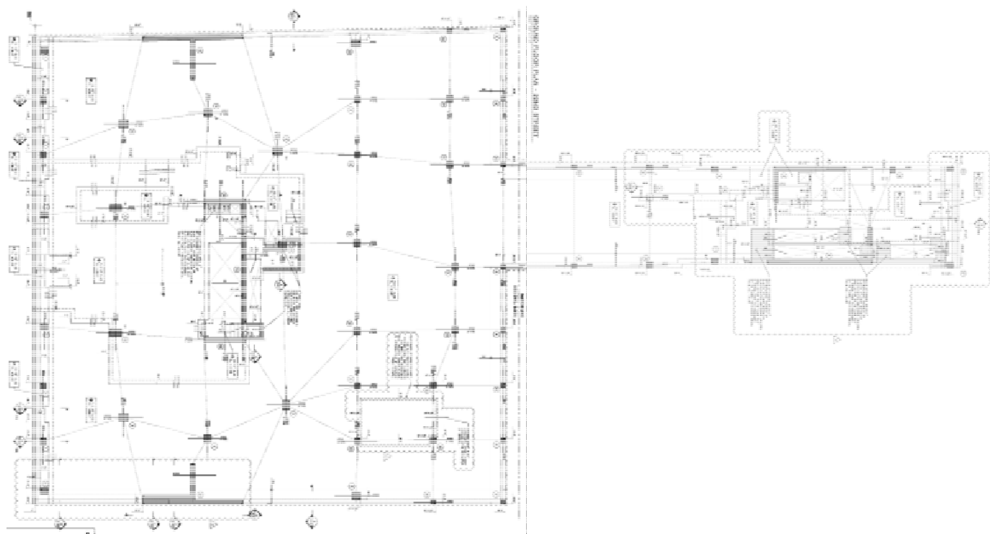
The architectural breadth focuses on the plan of the building. The objective is to maintain the same architectural spaces and net floor area while fitting the rooms to the new column grid. By rearranging the floor plans, the net area of the building increased due to a reduction in the number of columns provide by the new column grid. Closet space was the most improved area of the building, a welcome commodity in New York City. The mechanical breadth focuses on decreasing the ceiling cavity of the building in order decrease the height of each story and therefore, increasing the potential for the addition of another floor without increasing the overall building height. By using individual air-to-air heat pumps for each apartment, there is no need for a universal air-handling system which requires a ceiling cavity.

Introduction

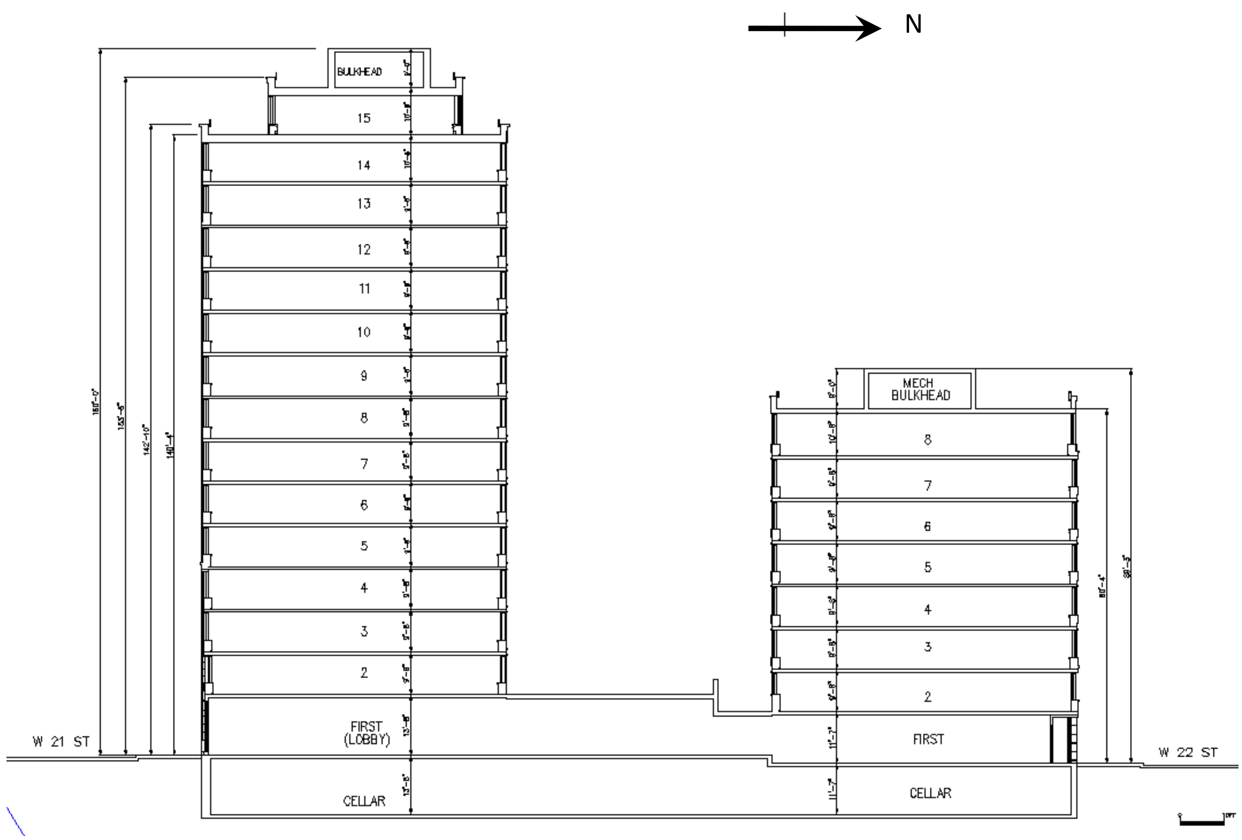
35 West 21st Street is shaped by the surrounding buildings and its site. With adjacent 4-12 story buildings, the plan takes on a T-shape to maximize the footprint. The stem of the T-shape is an eight story residential tower facing the north, while the top of the T-shape is a fifteen story residential tower facing the south with retail space at grade. Over 162,000 sq. feet of residential and retail space are provided.

35 West 21st Street is located in the Flatiron District within the Ladies' Mile Historic District. The area is zoned as C6-4A which allows for commercial, light manufacturing, and residential construction. The predominant historical requirements of Ladies' Mile consist of street walls a minimum of 60 feet tall that are in character with the surrounding area. Therefore, the building has a classic stone facade with infill glass windows.

The columns of the superstructure are continuous from the foundation to the top of the building with no transfers throughout the building. The columns are arranged in a semi regular pattern where most bays are rectangular in plan. The arrangement of columns allows for open residential and retail floor plans while a two way flat plate concrete floor system allows for 8' high ceilings while maintaining a typical 9'-8" floor to floor height. The residential units in the upper floors have large personal balconies which overlook the surrounding city and allow for a spacious outdoor room in crowded New York City.



Plan showing relationship of the two towers.



Section showing relationships of the two towers

Structural Systems Overview

Floor System

35 West 21st Street is a typical reinforced concrete residential structure. The floor system is a two way flat plate slab without drop panels or beams. Typical residential floors are 8 inches thick. The bottom is typically reinforced with #5 deformed bars at 12 inches on center each way. Middle strips are typically reinforced with #5 deformed bars at 12 inches on center at the top of the slab, while column strip top bars vary according to span lengths which range from 13' to 18'. In areas of high shear, slab supports also have studrails to help prevent punch through shear. Typical columns are gravity only, and run the entire height of the building without transfers. On the fifteenth floor, columns lining the exterior balconies are transferred to the 14" slab and then transferred to nearby columns that go down to the foundations. Typical columns are 16"x18" with 8-#7 longitudinal bars and #3 ties at 12 inches on center. Minimum concrete compressive strength is 5 ksi for slabs above ground, and 5.95 ksi for columns. The slab also provides a two hour fire rating.

Basement

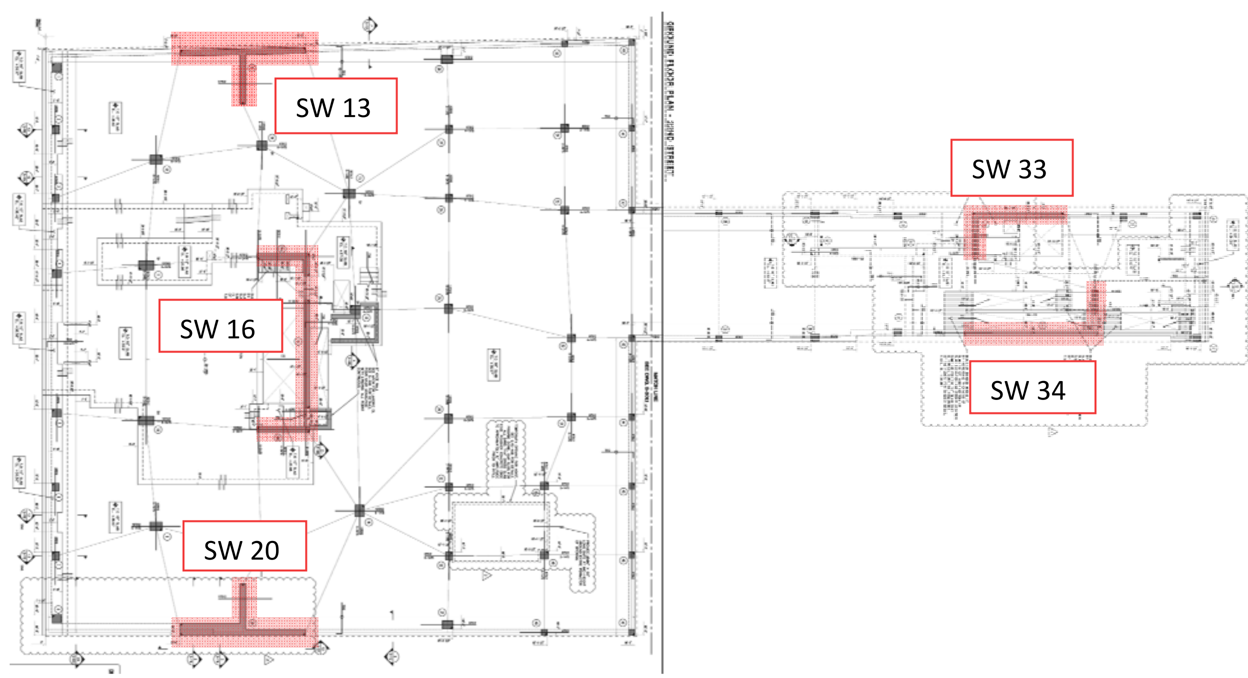
The basement floor is a slab on grade reinforced with 6" WWF 6x6 – W2.0xW2.0. Typical slab on grade thickness is 6".

Roof system

The roof slab is 12 inches thick with typical reinforcing like that on all the residential floors. Cooling towers bear on dunnage that consists of 16"x16" concrete piers and galvanized W10x33 steel beams. The remaining mechanical equipment including elevator machines are housed in the bulkhead, which consists of shear wall 16 and three transfer columns. The loads from the concrete piers and columns are transferred through the 12" slab and into columns below that continue to the foundation.

Lateral System

The lateral system of 35 West 21st Street is comprised of shear walls in both the North-South and East-West directions of the building. The two towers of the building are built integrally with each other through the two way slab at the basement, ground and second floor. However, at the second floor, the 15 story south tower steps back to allow for an outdoor courtyard, thus breaking the connection between the two towers. Because the connection of the two towers only exists on the first two floors, the towers' lateral systems were designed separately from each other. It is assumed that the two buildings act separately, and thus do not transfer any torsional moment between the two lateral systems. Typical shear walls are 1'-0" wide and longitudinal reinforcement ranges from #10 at 12" on center at the base of the shear walls to #4 at 12" on center at the top of the building. Horizontal shear reinforcement typically consists of #4 at 12" on center closed loop bars.



Foundation

The foundation system consists of spread footings for typical concrete columns and large mat foundations for shear walls. On the east side of the building, 240 ton caissons spread loads from the footings to the bedrock below. The caissons are at a minimum drilled 9'-0" into bedrock and are typically 12 inches in diameter.

Codes and Load Combinations

Codes and References

- The 2006 International Building Code
- Building Code Requirements for Structural Concrete (ACI 318-05), American Concrete Institute
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05), American Society of Civil Engineers
- Nelson, Arthur H., et al, "Design of Concrete Structures".
- Naaman, Antoine E., "Prestressed Concrete Analysis and Design".

Load Combinations

The following load combinations from ACI 318-05 were used for this analysis:

- 1.4D
- 1.2D + 1.6L
- 1.2D + 1.6W + 1.0L
- 0.9D + 1.6W
- 0.9D + 1.0E

- See Appendix A for Load Calculations

Problem Statement

35 West 21st Street is composed of 15 stories of apartment units above the Ground Level. The typical floor to floor height is 9'-8" with an eight inch slab and 6" ceiling cavity, creating an 8'-6" floor to ceiling height. If the ceiling cavity is removed and the slab thickness is reduced to 6", there is an extra eight inches of space per floor (ten feet in total) that can be added to the top of the building in the form of another story with rentable apartments.

The existing design of 35 West 21st Street uses a two-way flat plate concrete slab reinforced with mild steel. Gravity analysis from Technical Report 2 revealed that a post-tensioned two-way flat plate concrete slab gravity system would decrease the thickness of the floor slab as well as the overall dead load of the superstructure. However the irregular column placement of the existing structure creates a potential problem for the layout of tendons. The recommended radius of curvature for banded tendons in plan is greater than ten feet. When Columns are not in a fairly straight line, as is the case with the existing column layout, high stress concentrations can be induced due to the change in direction of the tendons.

The lateral system will need to be optimized based on the new column grid layout. The shear walls need to be placed in such a way that they have minimal effect on the architecture, they do not disrupt the layout of banded tendons significantly, and they keep the building periods close to the existing periods so deflections do not affect components and cladding.

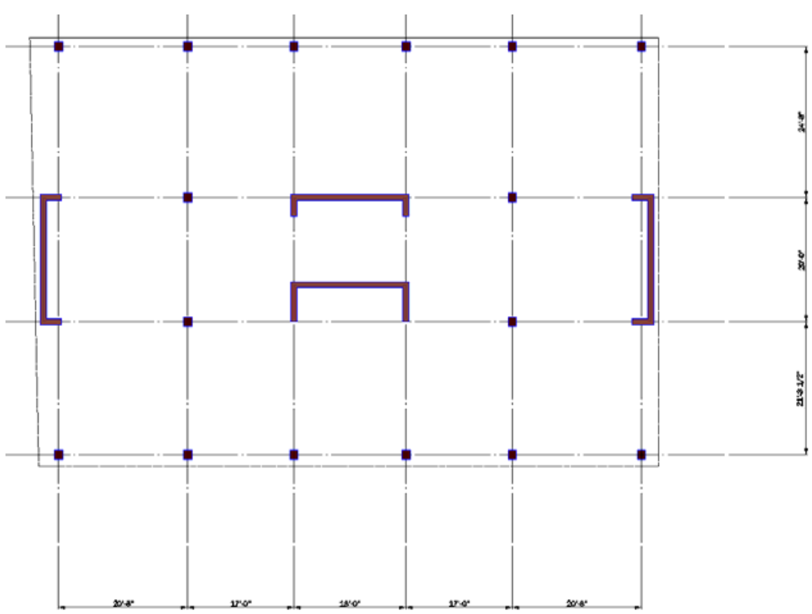
Solution Methods

Structural Analysis and Design

For analysis and design of the post-tensioned slab, the program PCA Slab will be used to determine service and design loads, while hand calculations based on ACI 318-05 and "Prestressed Concrete Analysis and Design" by Antoine E. Naaman will be used to design the tendons and check stresses and deflections. Alternate live loading will not be considered because 75% of the Dead Load is greater than the Live Load (per ACI 318-05 § 13.7.6). Columns will be designed using the Program PCA Column and the service moments from PCA Slab and a column load take down spreadsheet. These loads will be factored using PCA Column to determine the maximum effect on the column.

For analysis of the lateral system, the program ETABS will be used to determine the building periods, distribution of lateral forces, and drifts. Lateral loads will be determined in accordance with ASCE 7-05 provisions for seismic and wind. Shear reinforcement will be designed in accordance with ACI 318-05 Chapter 11 and 14. Chapter 21 of ACI need not be considered since the lateral force resisting system is ordinary reinforced concrete shear walls.

A comparison of the existing and redesigned structural system will be carried out to determine if the redesign of the structure to add an additional floor is a feasible solution to the problem.



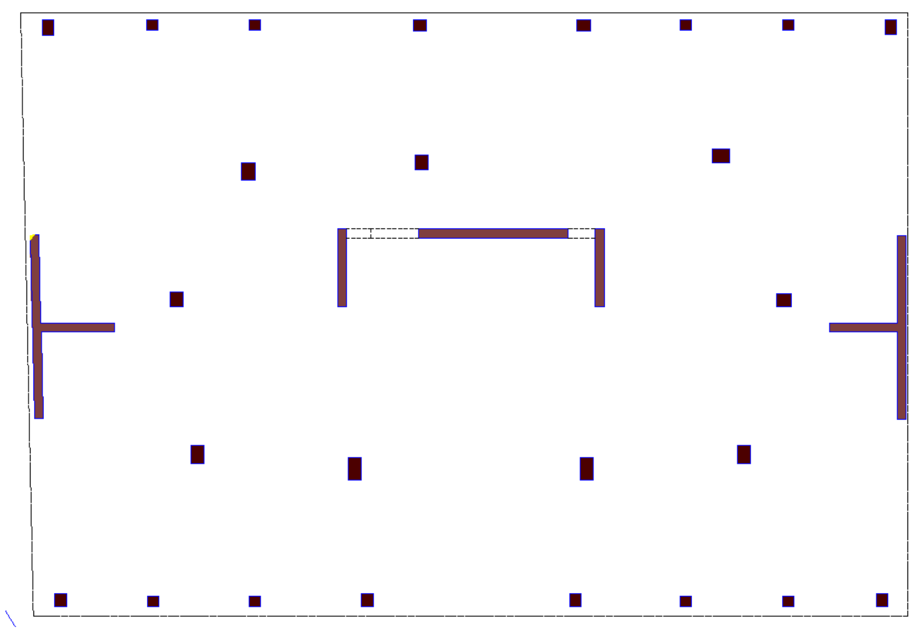
Possible Column and Shear Wall layout for 16 story tower

Depth Study: Building Optimization

The purpose of this thesis is to optimize the gravity and lateral system in order to reduce the floor-to-floor height of the apartment units for the second through 15th floor. By doing this, the building has the potential for an extra floor within the height restrictions. Gravity optimization will focus on reducing the slab thickness of the floors, while lateral optimization will focus on fitting the shear walls to the new column grid with the least amount of architectural interference and change in building period. Previous studies have shown that by decreasing floor to floor height, it may be possible to add an additional story without increasing the overall building height. Redesigning the gravity system and optimizing the lateral system should result in a more efficient structure that increases the Owner's revenue.

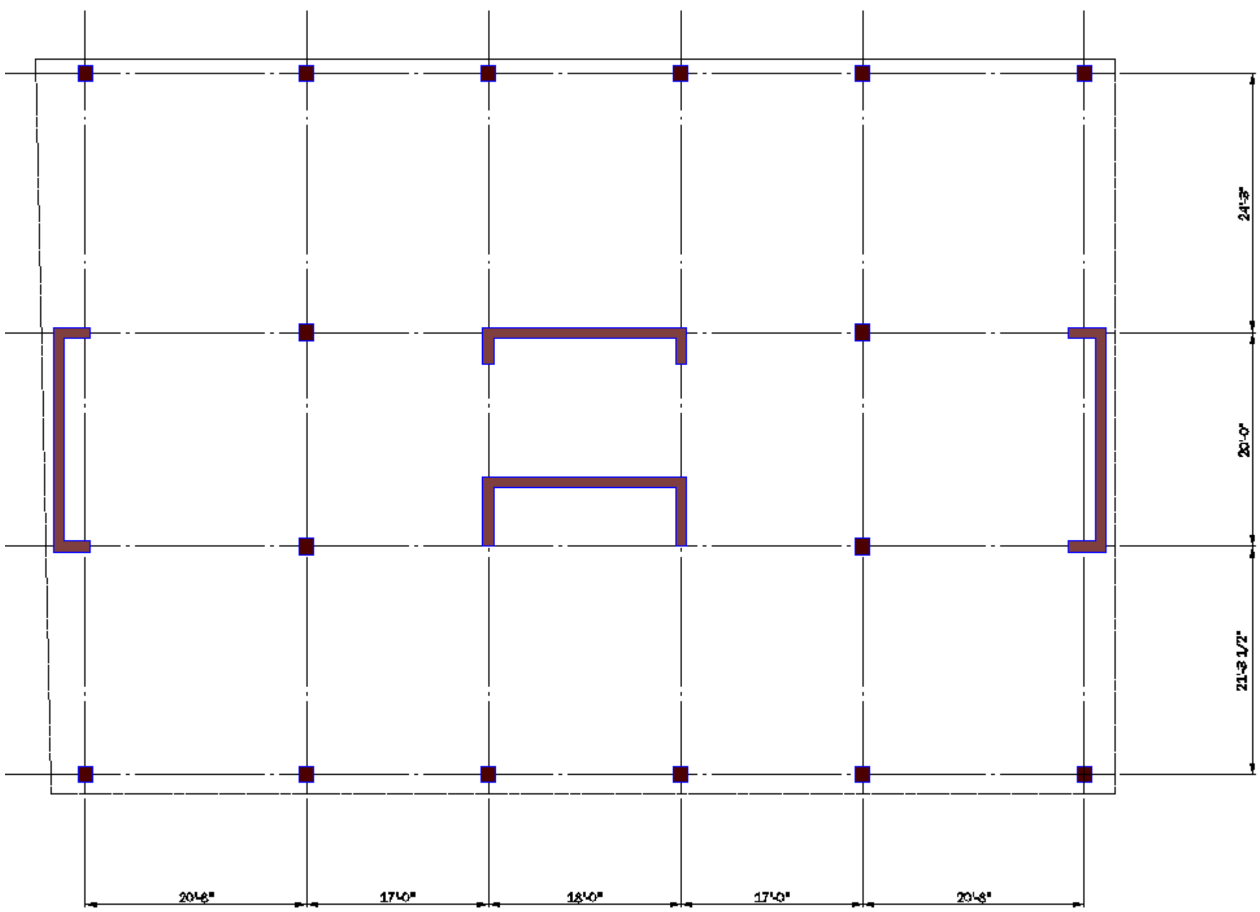
Gravity System: MAE Requirement (Please see Appendix-A for calculations)

The goal of this study is to design an efficient gravity system that will foster constructability and efficiency while decreasing the floor-to-floor height. The irregular column grid coupled with the large variation in column dimensions creates a very inefficient design. If the variation in column sizes was reduced, construction of formwork would present a much easier task during construction. However, in order to reduce the number of different column sizes, a regular grid with a fairly uniform spacing is needed. This way, columns will see similar loadings throughout the building allowing the use of typical column sizes. This regular column grid also lends itself to a more efficient flat plate slab design. In previous studies it was discovered that a post-tensioned two-way flat plate concrete slab has the potential for reducing the slab thickness. However, this reduced slab thickness will need to be carefully checked for punching shear.



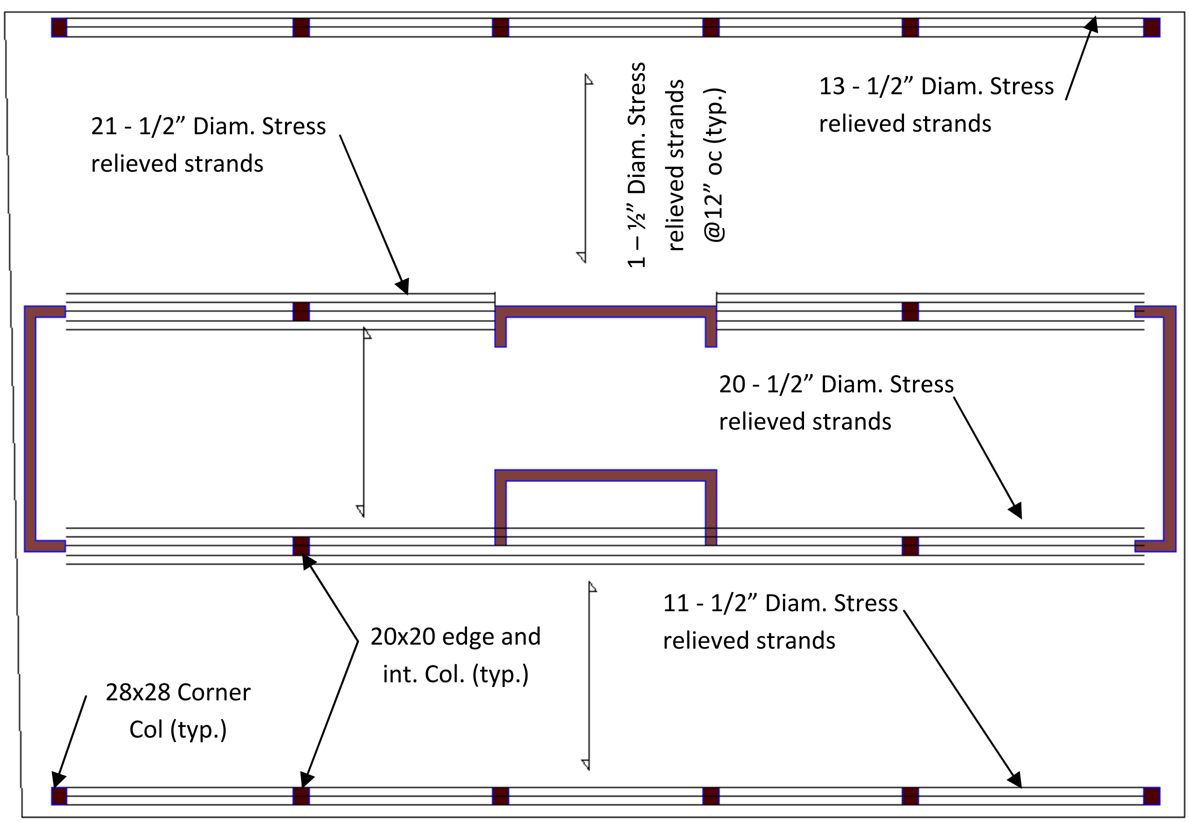
Existing Shear wall and column plan of 16 story tower

The gravity system depth study began with a rearrangement of the column grid. The new grid was designed to minimize effects on architecture while still providing a fairly uniform spacing. With the new grid, the number of columns is reduced from 31 to 16 and there is an approximately uniform grid with spacing of about 20 feet. Preliminary column sizes of 18x18 were chosen based on column-load-take-downs and the effects of member curvature per ACI 318-05 (§ 10.12.3). With the column sizes chosen and a new uniform grid developed, moments and shears of effective column strips were determined using PCA Slab for the design of the post-tensioned flat plate slab.



Proposed Column grid and shear wall layout for 16 story tower

As part of the MAE requirement for this thesis, the post-tensioned slab was designed by hand using the banded tendon method. For the banded tendon method all of the column and middle strip tendons needed, are banded close together in the column strip effectively creating a one-way slab supported by the banded tendons. ACI 318-05 chapter 11, 13, and 18, as well as "Prestressed Concrete Analysis and Design" by Antoine E. Naaman were used to design the slab by hand calculations. By balancing the total dead load, it was found that a six inch slab works well for service flexure stresses, ultimate strength, and deflection. However there was a problem with punching shear at the columns. Due to the increase in shear transferred by moment at the slab-column connection, the effective cross section considered in the stress check, was too small to resist the maximum factored shear. The solution to this problem was discovered to be a combination of increasing the column size to increase the effective cross section, and providing studrails to add to the strength of the concrete. A column size of 28x28 for corner columns and 20x20 for interior and edge columns was eventually worked out. It was also determined that for a 20x20 interior column, the concrete can effectively resist the punching shear load alone. With this design, the number of different column cross sections is reduced from 9 to 2. This reduction will greatly increase the constructability of 35West 21st Street.



Proposed tendon layout for 16 story tower based on hand calculation

Frame 2

Max Moments and Shears From PCA Slab

	Centerline Exterior Support	Exterior Span			Centerline First Interior Support	First Interior Span			Centerline Second interior Support	Second Interior Span	
		Face Left Support	Near Midspan	Face Right Support		Face Left Support	Midspan	Face Right Support		Face Left Support	Midspan
Service Load Moment (ft-k)		-44	60.59	-79.01		-64.23	30.76	-49.8		-55.28	41.59
Unbalanced (Live Load) Moment (ft-k)		-12.57	17.31	-22.57		-18.35	10.66	-14.23		-15.8	12.16
Factored Total Moment (1.2D+1.6L) (ft-k)		-57.82	79.63	-103.84		-84.42	41.18	-65.45		-72.66	54.77
Factored Transfer Moment (ft-k)	-57.82				-19.42				-7.21		
Factored Moment Orthagonal to Frame	-89.4				-89.64				-84.95		
Factored column Axial Force (k)	42.73				74.08				65.31		

Frame Properties	
Exterior Span length (in.)	248
First Interior Span Length (in.)	204
Second Interior Span Length (in.)	216
l_2 (in.)	248
Slab Thickness (in.)	6
β1	0.8
f'_c (psi)	5000
f'_{ci} (psi)	4000
E_c (psi)	4030509
1/2" Diameter Strand Area (in2)	0.153
f_{pu} (psi)	270000
f_{pe} (psi)	160000
Cover to center of Strand (in.)	1.25
d (in.)	5
fy (psi)	60000
fpy (psi)	216000
Zt (in3)	1488
Zb (in3)	1488

Loads	
SELF (psf)	75
SDL (psf)	25
LL (psf)	40

Preliminary Sags	
δ1 (in.)	2.4745
δ2 (in.)	3.5
δ3 (in.)	3.5

Load To Balance	
wb (plf)	2066.667

Balancing Force	
Fb (lbs)	535075.5

Number of Strands	
N	22

Actual Prestressing Force	
F (lbs)	538560

Actual Sags	
δ1 (in.)	2.4745
δ2 (in.)	1.66351
δ3 (in.)	1.864973

Allowable Stresses		
σ_c (psi)	2250	
σ_t (psi)	-141.421	Positive Moment
σ_t (psi)	-424.264	Negative Moment

Post-tensioned concrete slab design check using excel

The deflection of the post-tensioned slab was based on the assumption that no deflection is induced under dead load due to load balancing and, since the slab is level under load balancing, no additional long term deflection should theoretically be induced. Therefore, the deflection is calculated as the elastic deflection due to live load and is compared against the allowable limit of L/360.

Service Stress Checks		
Avg. Prestress (psi)	361.9355	OK
Max Compressive stress (psi)	543.9516	OK
Max Positive Moment Tensile Stress (psi)	222.3387	OK
Max Negative Tensile Stress (psi)	179.9194	OK

Bonded Reinf. @ Supports		
A_s (in^2)	1.116	
$A_{s\ provided}$ (in^2)	0.8	4-#4 bars

NO BONDED REINF.@ MID SPAN

ULTIMATE FLEXURAL STRENGTH (AT SUPPORTS)		
A_{ps} (in^2)	3.366	
A_s (in^2)	0.8	
d (in.)	5	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0.000645	
ω	0.007742	
ρ_p	0.002715	
f_{ps} (psi)	241353.3	
a	0.816314	
c	1.020393	
c/d	0.204079	
Φ	0.9	
ΦM_n (ft-k)	-296.3099	OK

ULTIMATE FLEXURAL STRENGTH (POSITIVE MOMENT)		
A_{ps} (in^2)	3.366	
A_s (in^2)	0	
d (in.)	2.91351	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0	
ω	0	
ρ_p	0.004658	
f_{ps} (psi)	223304.4	
a	0.713133	
c	0.891417	
c/d	0.30596	
Φ	0.9	
ΦM_n (ft-k)	144.1431	OK

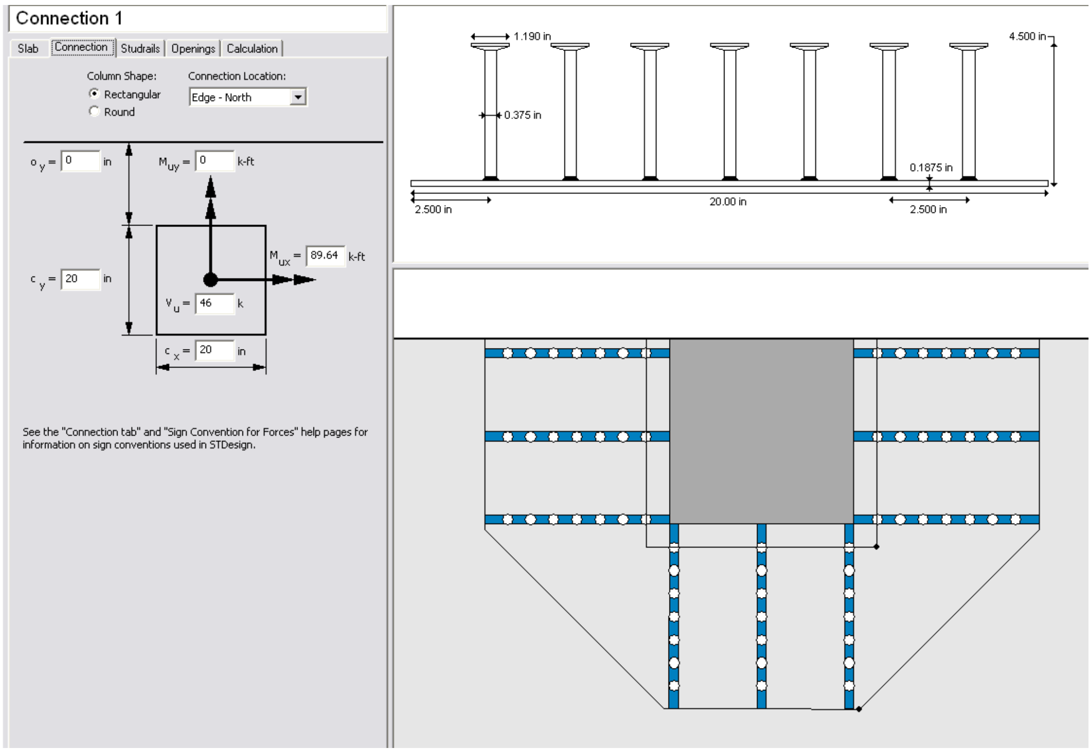
PUNCHING SHEAR CHECK AT INTERIOR COLUMN		
c_1 (in.)	20	
c_2 (in.)	20	
d (in.)	5	
b_o (in.)	100	
Ac (in^2)	500	
c_3 (in.)	12.5	
c_4 (in.)	12.5	
γ_v	0.4	
J_c (in^4)	52604.17	
M_u (ft-k)	89.64	
V_u (kips)	74.08	
v_{uMAX} (psi)	148.1685	
Can Prestress Shear Strength be Used?	YES	
α_s	40	
β_p	3.5	
$f_{cp,MIN}$	125	
v_n (psi)	282.5	
ϕv_n (psi)	211.875	OK,NO STUDRAILS NEEDED

Since the tensile stress generated by the maximum positive moment is greater than the allowable of -2sqrt(f'c), there is no bonded reinforcement required at mid span (Per ACI 318-05 §18.9.3.1).

DEFLECTION		
$\Delta_{allowable}$ (in.)	0.688889	
Δ_{actual} (in.)	0.132765	OK

Post-tensioned concrete slab check using excel (continued)

Studrails were designed using software downloaded from a manufacturer online and then check by hand using the provisions of ACI 318-08 Chapter 11. A typical edge column is shown below with added studrails. The manufacturer used is DECON.



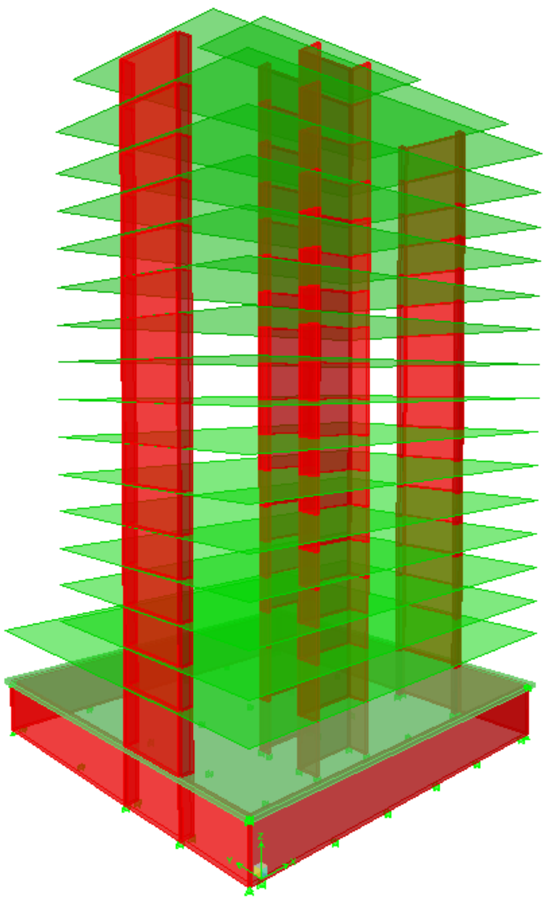
Studrail design of typical edge column using DECON Studrail software

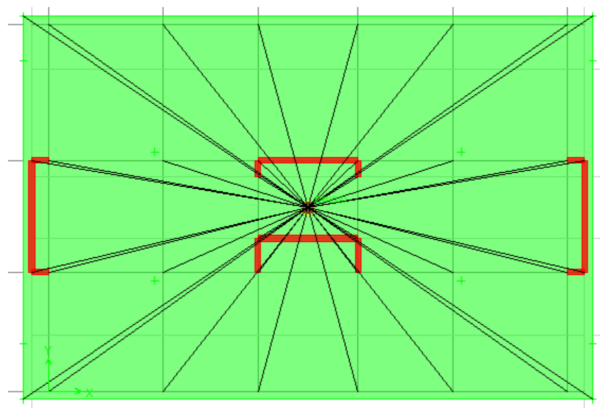
Lateral System (Please see Appendix B for calculations)

The Goal of the lateral study is to optimize the ordinary shear wall lateral system for the new column grid and added story of the South tower. This task poses many potential clashes between the architecture and structure. The shear walls need to be aligned in such a way that they do not problematically interfere with the layout of post-tensioning tendons, that they create reasonable building periods, and they need to fit the architectural floor plans. A number of configurations were considered and then modeled using the program ETABS. The configuration shown in the new column grid plans, above, was selected as the optimal configuration based on architectural impacts and structural impacts. This layout required the least amount of architectural adjustment while maintaining building periods close to those of the existing structure.

Existing Structure Period (South Tower)	
$T_{N/S}$ (sec.)	2.28
$T_{E/W}$ (sec.)	2.41
$T_{torsion}$ (sec.)	1.60

Optimized Structure Period (South Tower)	
$T_{N/S}$ (sec.)	2.30
$T_{E/W}$ (sec.)	2.53
$T_{torsion}$ (sec.)	1.58





New shear wall configuration modeled in ETABS

ETABS was used extensively model the effects of lateral loads as part of the M.A.E. requirement for this thesis. In previous studies of the lateral system, the basement was not modeled and the shear walls were assumed fixed at the ground level. This was done both as a simplifying assumption and because it was the assumption that the structural engineer made on the

Max Shear without basement (kips)	Max shear with basement (kips)
201	376

project. However, the author believes that the basement should be modeled as well as the ground floor diaphragm in order to properly represent the effects of shear reversal at the interface of the ground floor and basement ceiling. The 12" thick slab of the ground floor was modeled as a flexible diaphragm and meshed appropriately to accurately represent the distribution of forces. This model of building produced significant shear reversal in all of the walls. The load due to the shear reversal was found to be much higher than that found in any of the shear walls modeled without a basement and flexible ground floor. Floors 2 through the Bulkhead were modeled as rigid diaphragms in order accurately represent the distribution of lateral forces to each shear wall. Since it is more conservative to only allow shear walls to take in plane shears, the shear walls were modeled as membranes, since membranes do not have stiffness for out of plane loads.

Lateral loads were calculated using the provisions for wind and seismic in ASCE 7-05. Critical information for loads analysis is listed below.

Seismic

Spectral Response Acceleration

- S_s……………………………………………………0.363
- S_1……………………………………………………0.07

Site Class……………………………………………………C

Site Class Factors

- F_a……………………………………………………1.2
- F_v……………………………………………………1.7

S_{MS}……………………………………………………0.4356

S_{M1}……………………………………………………0.119

S_{DS}……………………………………………………0.2904

S_{D1}……………………………………………………0.0793

Seismic Design Category……………………………………B

Occupancy Category…………………………………………II

Importance Factor……………………………………………1.0

Response Modification Factor………………………………4.0

Wind

Basic Wind Speed **V**...110 mph
Exposure Category...B
Importance Factor..1.0
External Pressure Coefficient C_p...0.8

Shear wall design was based on the provisions of ACI 318-05 Chapters 11 and 14. Chapter 21 need not be considered because the lateral system is an ordinary reinforced concrete shear wall, not a special or intermediate system. By these provisions, the minimum horizontal and vertical shear reinforcement governed the design, not the requirement for ϕV_n. Typical shear reinforcement needed is 0.54 in^2/1.5ft, and typical shear reinforcement provided is #4's @12" oc. $A_{sprovided}$ = 0.6 in^2/1.5ft.

Fy (psi)	60000
f'c (psi)	5000
lw (in.)	252
hw (in.)	1920
h (in.)	12
N (kips)	2140
V (kips)	376
M (ft-k)	9863
Nu (lbs)	1926000
Vu (lbs)	601600
Mu (in-lb)	189369600

d (in.)	201.6	
Vc (kips)	799.4471	
1/2ΦVc (kips)	299.7927	< Vu, SHEAR REINF. PER 11.10.9.1
$S_{max\ horiz}$ (in.)	18	
$S_{max\ horiz}$ (in.)	18	
$\rho_{t,min}$	0.0025	
$\rho_{l,min}$	0.0025	
$A_{horiz,\ min}$ (in^2/S_{max})	0.54	
$A_{vert,\ min}$ (in^2/S_{max})	0.54	
$V_{s\ required}$ (kips)	2.686264	
$A_{horiz,\ required}$ (in^2/S_{max})	0.003997	
$A_{horiz\ provided}$ (in^2/S_{max})	0.6	#4 bars @ 12" EACH FACE
ΦV_n (kips)	901.9853	

Shear Reinforcement design per ACI 318-05 for Shear Wall 7 and 13.

The design of shear walls for flexure was carried out using PCA Column. Biaxial bending was considered using the moments from the shear wall returns and the in plane moment from the shear wall itself. Again the minimum reinforcement governed the design not the requirement for ϕM_n. To satisfy the minimum reinforcement ratio of 1% for compression members (ACI 318-05 § 10.9.1), #8 bars are provided at 12" oc. Each Face.

	SW 7	SW 9	SW 11	SW 13
Max In-Plane Wind Moment (ft-k)	9863	4815	9863	4796
Max out-of-plane Wind Moment (ft-k)	144	758	144	123
Max Wind Shear (kips)	376	176	376	150
Max In-Plane EQ Moment (k-ft)	7262	5328	7262	5287
Max out-of-plane EQ Moment (k-ft)	124	506	124	82
Max EQ Shear (kips)	267	182	267	157

Drift calculations were computed in accordance with ASCE 7-05. A C_d factor of 4.0 for amplified earth quake drifts is required for ordinary reinforced concrete shear walls. Allowable drifts for seismic applications were determined using table 12.12-1 in ASCE 7-05. The allowable drift is computed by 0.025h, where h is the height of the story below. Allowable drifts for wind loads were computed using the engineering standard of L/400.

Story	Story Height	EQ Load Building Drift N/S	EQ Story Drift N/S	Amplified Story Drifts	Allowable Story Drift	
BULKHEAD	9	2.4291	0.1771	0.7084	2.7	OK
MAIN ROOF	10.167	2.252	0.1957	0.7828	3.0501	OK
STORY16	10.167	2.0563	0.1953	0.7812	3.0501	OK
STORY15	9	1.861	0.1751	0.7004	2.7	OK
STORY14	9	1.6859	0.1736	0.6944	2.7	OK
STORY13	9	1.5123	0.1712	0.6848	2.7	OK
STORY12	9	1.3411	0.1678	0.6712	2.7	OK
STORY11	9	1.1733	0.163	0.652	2.7	OK
STORY10	9	1.0103	0.1569	0.6276	2.7	OK
STORY9	9	0.8534	0.1492	0.5968	2.7	OK
STORY8	9	0.7042	0.14	0.56	2.7	OK
STORY7	9	0.5642	0.129	0.516	2.7	OK
STORY6	9	0.4352	0.1162	0.4648	2.7	OK
STORY5	9	0.319	0.1015	0.406	2.7	OK
STORY4	9	0.2175	0.085	0.34	2.7	OK
STORY3	9	0.1325	0.0665	0.266	2.7	OK
STORY2	13.75	0.066	0.066	0.264	4.125	OK

Story	Story Height	Wind Load Building Drift N/S	Wind Story Drift N/S	Allowable Story Drift	
BULKHEAD	9	3.377	0.2403	0.27	OK
MAIN ROOF	10.167	3.1367	0.2656	0.30501	OK
STORY16	10.167	2.8711	0.2649	0.30501	OK
STORY15	9	2.6062	0.2379	0.27	OK
STORY14	9	2.3683	0.2364	0.27	OK
STORY13	9	2.1319	0.2338	0.27	OK
STORY12	9	1.8981	0.2301	0.27	OK
STORY11	9	1.668	0.2247	0.27	OK
STORY10	9	1.4433	0.2175	0.27	OK
STORY9	9	1.2258	0.2084	0.27	OK
STORY8	9	1.0174	0.1969	0.27	OK
STORY7	9	0.8205	0.1831	0.27	OK
STORY6	9	0.6374	0.1665	0.27	OK
STORY5	9	0.4709	0.147	0.27	OK
STORY4	9	0.3239	0.1243	0.27	OK
STORY3	9	0.1996	0.0989	0.27	OK
STORY2	13.75	0.1007	0.1007	0.4125	OK

Lateral Summary

The lateral study proved that an increase in the number of stories does not have a significant effect on the lateral system. This is mainly because even though the number of stories is increasing, the total height of the building remains the same. Since wind controls the design of the system, the height remains the same, and the building periods are approximately the same as the existing structure, the loads on the building are essentially the same as the existing building.

Impact on Foundations

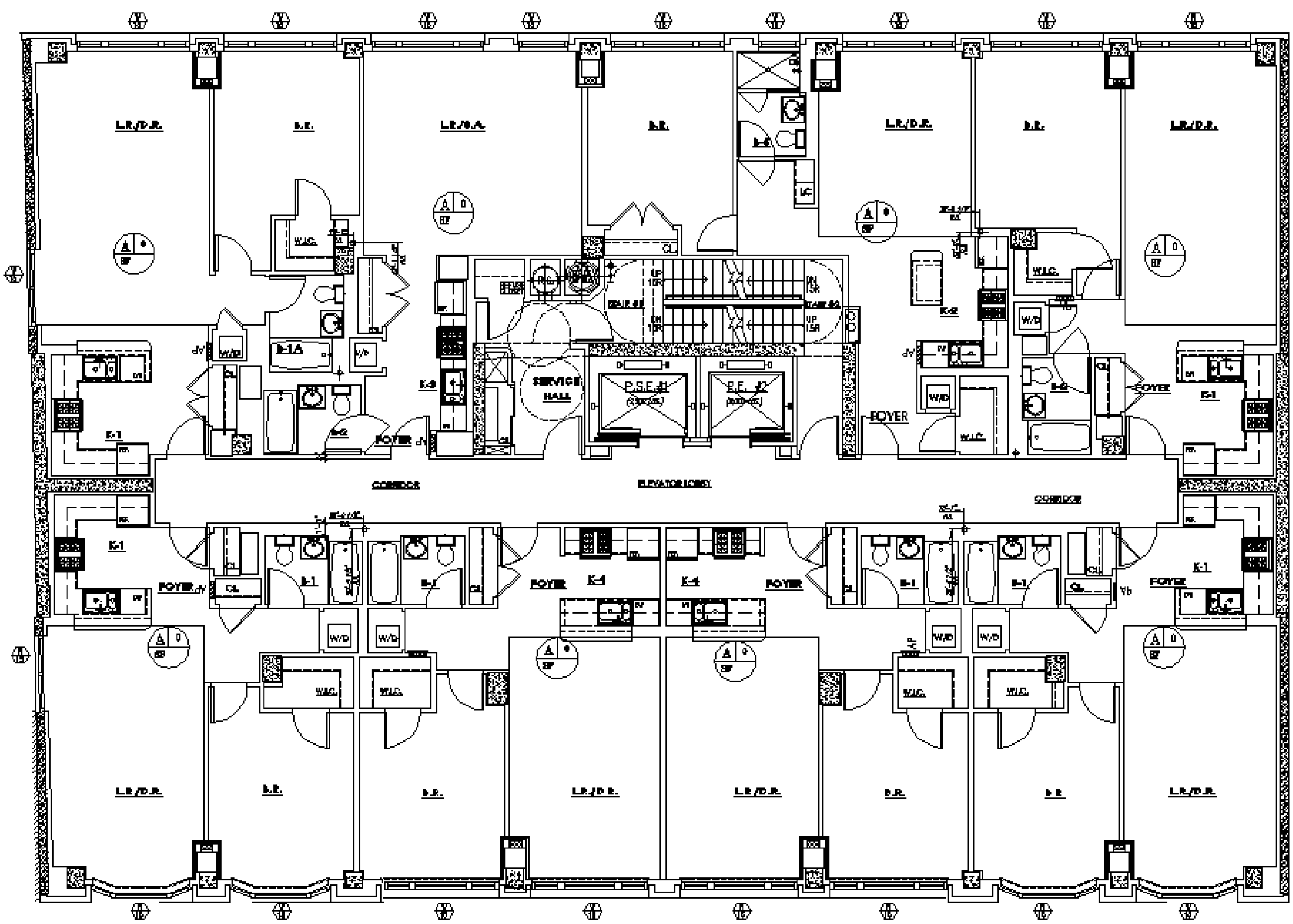
Because the column grid has been significantly changed as well as the thickness of the floor slabs, the column spread footings as well as the shear wall mat foundations will need to be completely redesigned. Previous studies have revealed that the building is susceptible to significant overturning moment which causes uplift on shear wall foundations. Because the height of the building hasn't changed (only the number of stories), and wind is the controlling load factor, the effects on the foundations shouldn't be significantly different from the effects of overturning on the existing structure. However, overturning should be checked and designed for accordingly.

Depth Study Summary

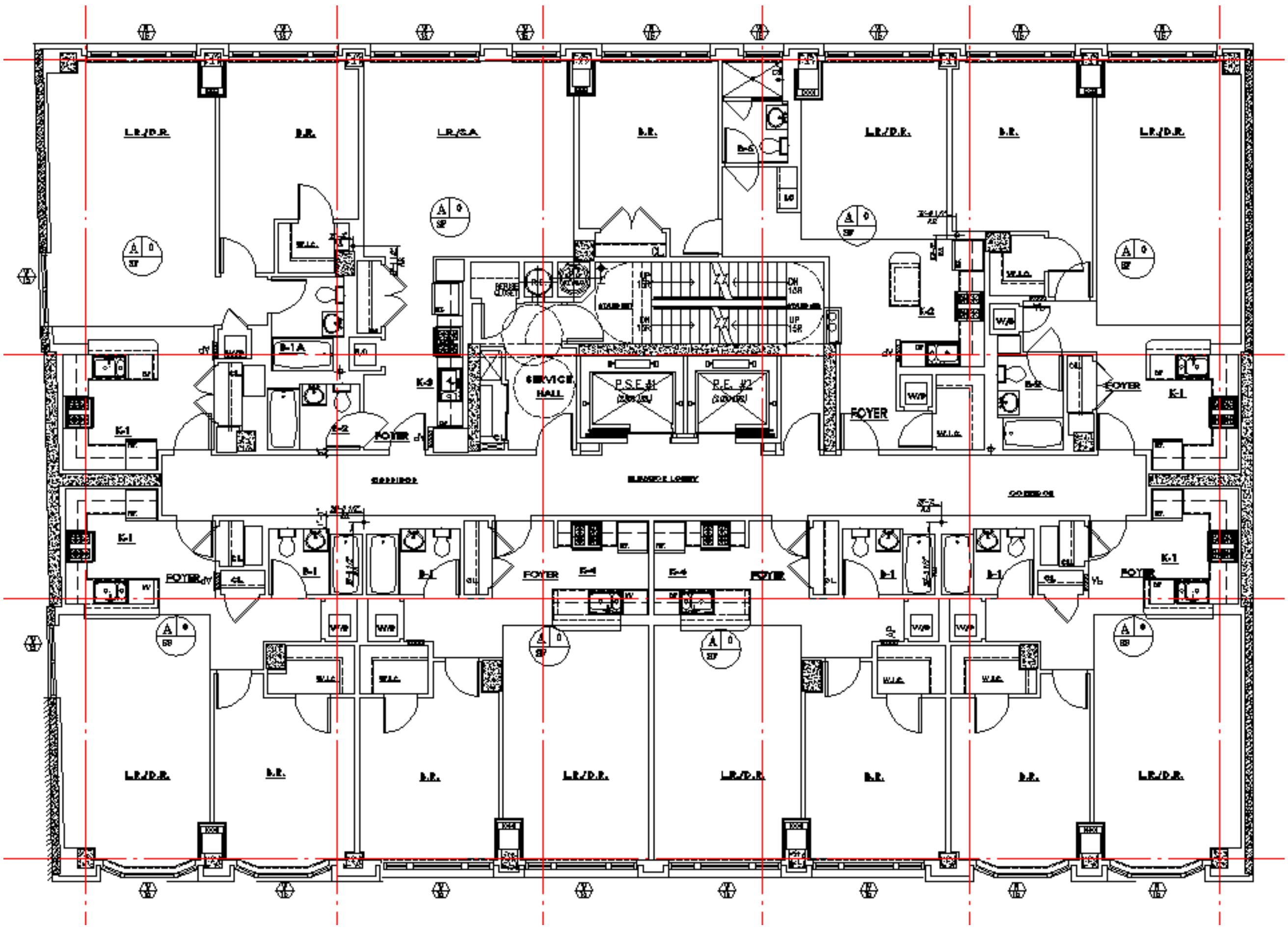
The intent of this depth was to determine the structural feasibility of adding another floor to the building without increasing the overall height. Gravity analysis proved that by creating a fairly uniform grid it is very possible to decrease the floor thickness from 8 inches to 6 inches by post-tensioning the two way flat plate. Studrails and increased column sizes are a key aspect of the new slab design. They stop the punching shear phenomenon created through direct shear and eccentricity of shear at the column support. Although the gravity system proved to be feasible, the lateral system needed to be designed within the limits of the new column grid as well as minimally impact the architectural plan and period of the building. It was determined that the new shear wall configuration did in fact do just what it needed to. Overall, utilization of the new column grid, post-tensioned floor, and new lateral system makes adding an additional floor within the same overall building height very feasible.

Architecture Breadth: Floor Plan Optimization

The purpose of this study is to integrate the change in the structural column grid with the architectural floor plan. The column grid was created to improve the efficiency of the structural system and minimally affect the architectural floor plan. However, there are changes that need to be taken into account. One of the most important changes is that of the exterior façade. By rearranging the column grid, the windows and vertical elements of the façade are affected directly. These elements need to be rearranged to fit the new column grid without drastically impacting the aesthetics of the façade and the layout of the floor plans.

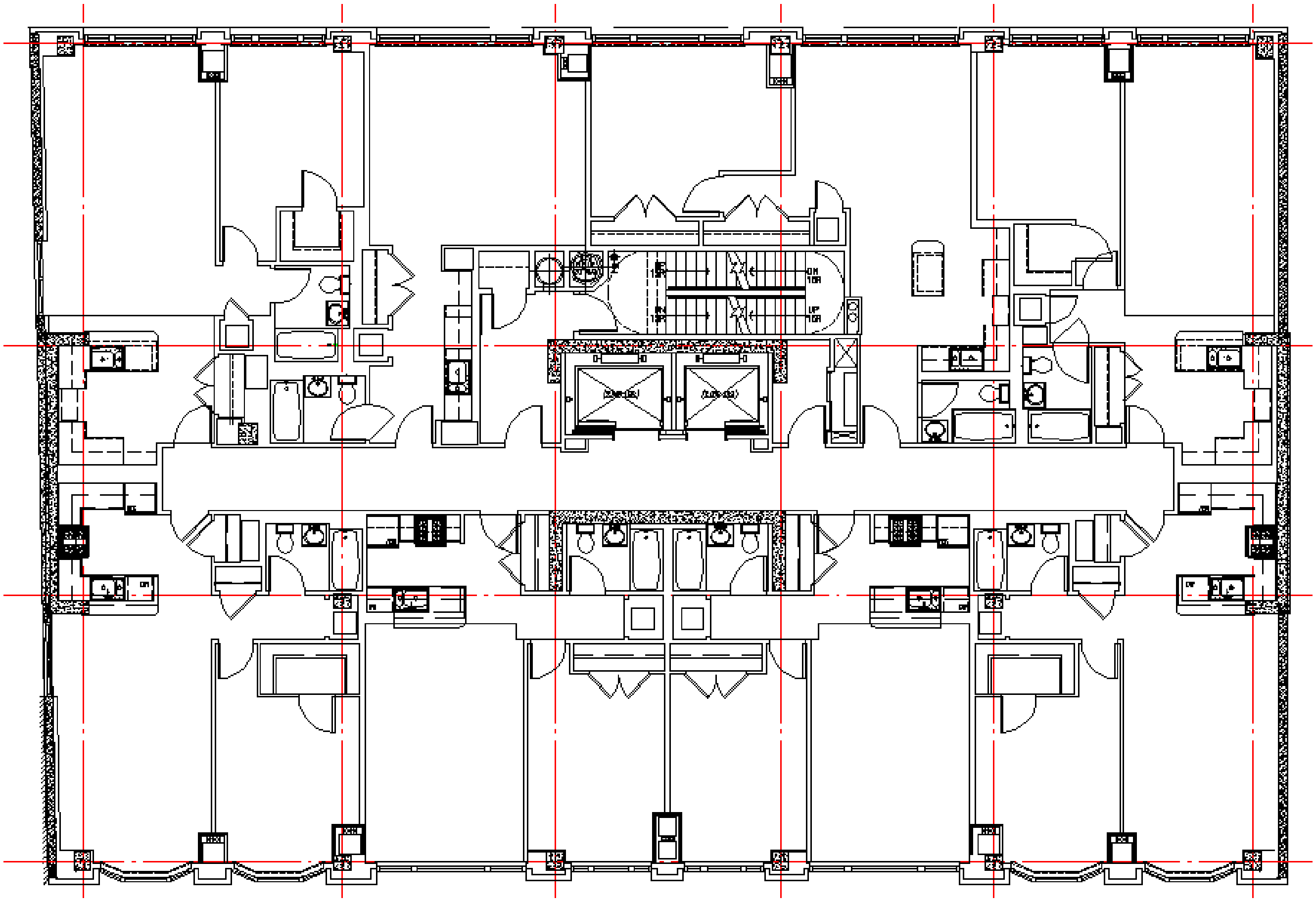


Existing architectural floor plan
(typical for stories 2-14)



Existing architectural floor plan with new column grid superimposed (Red)

As shown in the above figure, at the exterior walls, the new column grid doesn't match up exactly with the existing placement of exterior columns. In order to place the columns on the new grid, the architectural floor plan needs to be revised to match the spacing of the column grid. After some time, the floor plans below were determined to be the most feasible based on vertically aligning mechanical cores, modification of net floor area, modification of architectural flow, and modification of exterior façade.



The plan above represents the new architectural floor plan based on the column grid created for the structural system. The windows in the center bedrooms have decreased in size and the two living room windows in the center units have increased in size. The remaining windows are virtually untouched by the change in the column grid. The only other impact is that the precast architectural panels surrounding the columns have moved slightly in order to accommodate the placement of the columns.

Architecture Breadth summary

The purpose of the architecture breadth was to integrate the structure with the architecture of the building and determine if the structural changes created any drastic architectural changes. The slight rearrangements of the floor plans and façade that were made to match the structural grid prove that the new grid impacts the architecture very little. Overall, it is feasible to design the architecture per the structural requirements of adding an additional floor.

South Elevation on West 21st Street

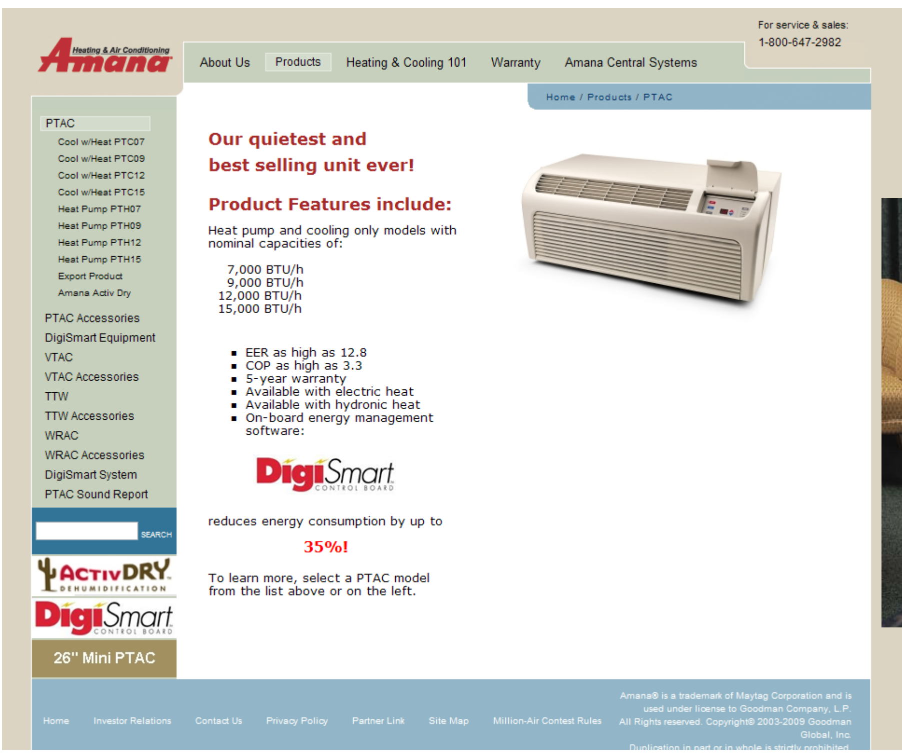
35 WEST 21ST STREET
NEW YORK, NEW YORK

Mechanical breadth: Air to Air Heat Pumps

The purpose of this breadth study is to determine the feasibility of changing the mechanical system for the apartment units from a universal mechanical system that utilizes a ceiling cavity to individual air-to-air air heat pumps for each apartment. It was determined that there are many products on the market that can do this job. An example is shown below.

By removing the ceiling cavity, an extra six inches of vertical space is obtained per floor. Coupled with the extra two inches per floor provided by the optimized structural system, this adds up to about nine feet of extra vertical space to add another level.

Sprinkler systems will be exposed when the ceiling cavity is removed. In order to maintain the clean architectural feel of the building, wall mounted sprinklers will be used with all the plumbing be contained in the wall.



http://www.amana-ptac.com/

Thesis Conclusion

The purpose of this thesis is to determine the feasibility of adding an additional floor to the building without increasing the overall building height. Depth studies of the major structural impacts were conducted and determined that a post-tensioned two-way flat plate concrete floor slab would reduce the thickness of the existing slab by 2 inches, and the increase in number of stories would not have a significant impact on the lateral design of the building. This is because the new lateral system had building periods that approximately matched that of the existing building and the overall height of the building did not change; therefore, the loads on the new structure are approximately the same as on the existing structure. Breadth studies were conducted on the architectural aspects and mechanical aspects of increasing the number of stories. It was determined that the impacts of the new structural system on the architecture were very minimal, while removing the ceiling cavity and using individual air-to-air heat pumps the height of each story can be decreased by six inches.

Overall, it is feasible to add another floor using the methods investigated in this thesis. It may also be more economical. Although the initial cost of the building and its design will increase, the revenue generated by the extra floor should allow for a short return period of the extra costs. It is recommended that solutions of this thesis be implemented in the building.

Acknowledgements

I would like to thank my fellow classmates for lending a helping hand whenever possible, even when we hadn't slept for days on end.

I would also like to thank the entire AE Faculty for all their guidance. Without you, none of this would be possible.

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Lastly, and most importantly, I would like to thank my family, especially my wife Lindley, for all of their support. I couldn't have done it without you pushing me to be the best I can.

Appendix – A

Gravity Calculations

Post-tensioned slab calcs

Frame 1

Max Moments and Shears From PCA Slab

	Centerline Exterior Support	Exterior Span			Centerline First Interior Support	First Interior Span			Centerline Second interior Support	Second Interior Span	
		Face Left Support	Near Midspan	Face Right Support		Face Left Support	Midspan	Face Right Support		Face Left Support	Midspan
Service Load Moment (ft-k)		-42.69	28.14	-45.17		-30.27	18.79	-29.48		-33.72	21.5
Unbalanced (Live Load) Moment (ft-k)		-12.2	8.04	-12.91		-8.65	5.37	-8.42		-9.63	6.14
Factored Total Moment (1.2D+1.6L) (ft-k)		-56.1	36.99	-59.36		-39.78	24.69	-38.75		-44.32	28.25
Factored Transfer Moment (ft-k)	-56.1				-19.58				-5.57		
Factored Moment Orthagonal to Frame	-89.4				-89.64				-84.95		
Factored column Axial Force (k)	27.6				40.85				38.02		

Frame Properties	
Exterior Span length (in.)	248
First Interior Span Length (in.)	204
Second Interior Span Length (in.)	216
l_2 (in.)	142
Slab Thickness (in.)	6
β1	0.8
f'_c (psi)	5000
f'_{ci} (psi)	4000
E_c (psi)	4030509
1/2" Diameter Strand Area (in2)	0.153
f_{pu} (psi)	270000
f_{pe} (psi)	160000
Cover to center of Strand (in.)	1
d (in.)	5
fy (psi)	60000
fpy (psi)	216000
Zt (in3)	852
Zb (in3)	852

Loads	
SELF (psf)	75
SDL (psf)	25
LL (psf)	40

Preliminary Sags	
δ1 (in.)	2.828
δ2 (in.)	4
δ3 (in.)	4

Load To Balance	
wb (plf)	1183.333

Balancing Force	
Fb (lbs)	268077.2

Number of Strands	
N	11

Actual Prestressing Force	
F (lbs)	269280

Actual Sags	
δ1 (in.)	2.828
δ2 (in.)	1.904987
δ3 (in.)	2.135695

Allowable Stresses		
σ_c (psi)	2250	
σ_t (psi)	-141.421	Positive Moment
σ_t (psi)	-424.264	Negative Moment

Service Stress Checks		
Avg. Prestress (psi)	316.0563	OK
Max Compressive stress (psi)	497.8873	OK
Max Positive Moment Tensile Stress (psi)	202.8169	OK
Max Negative Tensile Stress (psi)	134.2254	OK

Bonded Reinf. @ Supports		
A_s (in^2)	0.639	
$A_{s\ provided}$ (in^2)	0.8	4-#4 bars

NO BONDED REINF.@ MID SPAN

ULTIMATE FLEXURAL STRENGTH (AT SUPPORTS)		
A_{ps} (in^2)	1.683	
A_s (in^2)	0.8	
d (in.)	5	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0.0011268	
ω	0.0135211	
ρ_p	0.0023704	
f_{ps} (psi)	243729.62	
a	0.7592327	
c	0.9490409	
c/d	0.1898082	
Φ	0.9	
ΦM_n (ft-k)	-158.7784	OK

ULTIMATE FLEXURAL STRENGTH (POSITIVE MOMENT)		
A_{ps} (in^2)	1.683	
A_s (in^2)	0	
d (in.)	2.9049874	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0	
ω	0	
ρ_p	0.0040799	
f_{ps} (psi)	229103.91	
a	0.6389095	
c	0.7986369	
c/d	0.2749192	
Φ	0.9	
ΦM_n (ft-k)	74.770091	OK

PUNCHING SHEAR CHECK AT CORNER COLUMN		
c_1 (in.)	28	
c_2 (in.)	28	
d (in.)	5	
Distance to Discontinuous Edge (c1 side) (in.)	18	
Distance to Discontinuous Edge (c2 side) (in.)	0	
b_o (in.)	79	
Ac (in^2)	395	
c_3 (in.)	1.177532	
c_4 (in.)	29.32247	
c_5 (in.)	1.177532	
c_6 (in.)	29.32247	
c_7 (in.)	15.32247	
c_8 (in.)	15.32247	
γ_{v1}	0.4	
γ_{v2}	0.4	
J_{c1} (in^4)	42551.33	
J_{c2} (in^4)	42551.33	
M_{u1} (Orthagonal to Frame considederd) (ft-k)	89.4	
M_{u2} (In Direction of Frame considederd) (ft-k)	56.1	
v_{u1} (psi)	79.83798	
v_{u2} (psi)	146.0608	
v_{u3} (psi)	251.7846	
v_{uMAX} (psi)	251.7846	
Can Prestress Shear Strenth be Used?	NO	
α_s	20	
β	1	
v_n (psi)	230.9285	
ϕv_n (psi)	173.1964	NG, NEED STUDRAILS

PUNCHING SHEAR CHECK AT EXTERIOR EDGE COLUMN		
c_1 (in.)	20	
c_2 (in.)	20	
d (in.)	5	
Distance to Discontinuous Edge (in.)	0	
b_o (in.)	70	
Ac (in^2)	350	
c_3 (in.)	7.23214	
c_4 (in.)	15.2679	
c_5 (in.)	5.26786	
γ_v	0.38743	
J_c (in^4)	20131.1	
M_u (Orthagonal to Frame considederd) (ft-k)	89.64	
V_u (kips)	40.85	
v_{u1} (psi)	236.48	
v_{u2} (psi)	369.553	
v_{uMAX} (psi)	369.553	
Can Prestress Shear Strength be Used?	NO	
α_s	30	
β	1	
v_n (psi)	282.843	
ϕv_n (psi)	212.132	NG, NEED STUDRAILS

DEFLECTION		
$\Delta_{allowable}$ (in.)	0.68889	
Δ_{actual} (in.)	0.13276	OK

Frame 3

Max Moments and Shears From PCA Slab

	Centerline Exterior Support	Exterior Span			Centerline First Interior Support	First Interior Span			Centerline Second interior Support	Second Interior	
		Face Left Support	Near Midspan	Face Right Support		Face Left Support	Midspan	Face Right Support		Face Left Support	Midspan
Service Load Moment (ft-k)		-42.12	66.67	-86.51		-72.34	31.95	-54.93		-57.95	49.88
Unbalanced (Live Load) Moment (ft-k)		-12.03	19.05	-24.72		-20.69	9.54	-16.24		-16.21	16.21
Factored Total Moment (1.2D+1.6L) (ft-k)		-55.36	87.63	-113.7		-95.09	42.12	-72.41		-76.03	66.34
Factored Transfer Moment (ft-k)	-55.36				-18.61				-3.62		
Factored Moment Orthagonal to Frame	-89.4				-89.64				-84.95		
Factored column Axial Force (k)	49.4				80.7				63.1		

Frame Properties	
Exterior Span length (in.)	248
First Interior Span Length (in.)	204
Second Interior Span Length (in.)	216
l_2 (in.)	265
Slab Thickness (in.)	6
β1	0.8
f'_c (psi)	5000
f'_{ci} (psi)	4000
E_c (psi)	4030509
1/2" Diameter Strand Area (in2)	0.153
f_{pu} (psi)	270000
f_{pe} (psi)	160000
Cover to center of Strand (in.)	1
d (in.)	5
fy (psi)	60000
fpy (psi)	216000
Zt (in3)	1590
Zb (in3)	1590

Loads	
SELF (psf)	75
SDL (psf)	25
LL (psf)	40

Preliminary Sags	
δ1 (in.)	2.828
δ2 (in.)	4
δ3 (in.)	4

Load To Balance	
wb (plf)	2208.333

Balancing Force	
Fb (lbs)	500284.8

Number of Strands	
N	21

Actual Prestressing Force	
F (lbs)	514080

Actual Sags	
δ1 (in.)	2.828
δ2 (in.)	1.862186
δ3 (in.)	2.08771

Allowable Stresses		
σ_c (psi)	2250	
σ_t (psi)	-141.421	Positive Moment
σ_t (psi)	-424.264	Negative Moment

Service Stress Checks		
Avg. Prestress (psi)	323.3208	OK
Max Compressive stress (psi)	509.8868	OK
Max Positive Moment Tensile Stress (psi)	179.5472	OK
Max Negative Tensile Stress (psi)	136.7547	OK

Bonded Reinf. @ Supports		
A_s (in^2)	1.1925	
$A_{s\ provided}$ (in^2)	0.8	4-#4 bars

NO BONDED REINF.@ MID SPAN

ULTIMATE FLEXURAL STRENGTH (AT SUPPORTS)		
A_{ps} (in^2)	3.213	
A_s (in^2)	0.8	
d (in.)	5	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0.0006038	
ω	0.0072453	
ρ_p	0.0024249	
f_{ps} (psi)	244348.45	
a	0.7397039	
c	0.9246299	
c/d	0.184926	
Φ	0.9	
ΦM_n (ft-k)	-289.30029	OK

ULTIMATE FLEXURAL STRENGTH (POSITIVE MOMENT)		
A_{ps} (in^2)	3.213	
A_s (in^2)	0	
d (in.)	2.8621858	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0	
ω	0	
ρ_p	0.0042361	
f_{ps} (psi)	227538.31	
a	0.6491282	
c	0.8114102	
c/d	0.2834932	
Φ	0.9	
ΦM_n (ft-k)	139.14045	OK

PUNCHING SHEAR CHECK AT INTERIOR COLUMN		
c_1 (in.)	20	
c_2 (in.)	20	
d (in.)	5	
b_o (in.)	100	
Ac (in^2)	500	
c_3 (in.)	12.5	
c_4 (in.)	12.5	
γ_v	0.4	
J_c (in^4)	52604.17	
M_u (ft-k)	89.64	
V_u (kips)	80.7	
v_{uMAX} (psi)	161.4085	
Can Prestress Shear Strength be Used?	YES	
α_s	40	
β_p	3.5	
$f_{cp,MIN}$	125	
v_n (psi)	282.5	
ϕv_n (psi)	211.875	OK,NO STUDRAILS NEEDED

DEFLECTION		
$\Delta_{allowable}$ (in.)	0.688889	
Δ_{actual} (in.)	0.132765	OK

Frame 4

Max Moments and Shears From PCA Slab

	Centerline Exterior Support	Exterior Span: Face Left Support	Exterior Span: Near Midspan	Exterior Span: Face Right Support	Centerline First Interior Support	First Interior Span: Face Left Support	First Interior Span: Midspan	First Interior Span: Face Right Support	Centerline Second interior Support	Second Interior: Face Left Support	Second Interior: Midspan
Service Load Moment (ft-k)		-47.66	31.87	-50.71		-34	21.18	-33.01		-37.77	24.29
Unbalanced (Live Load) Moment (ft-k)		-13.62	9.11	-14.49		-9.71	6.05	-9.43		-10.79	6.94
Factored Total Moment (1.2D+1.6L) (ft-k)		-62.64	41.89	-66.65		-44.68	27.84	-43.39		-49.64	31.92
Factored Transfer Moment (ft-k)	-62.64				-21.97				-6.25		
Factored Moment Orthagonal to Frame	-89.4				-89.64				-84.95		
Factored column Axial Force (k)	31.58				46				42.72		

Frame Properties	
Exterior Span length (in.)	248
First Interior Span Length (in.)	204
Second Interior Span Length (in.)	216
l_2 (in.)	158
Slab Thickness (in.)	6
β1	0.8
f'_c (psi)	5000
f'_{ci} (psi)	4000
E_c (psi)	4030509
1/2" Diameter Strand Area (in2)	0.153
f_{pu} (psi)	270000
f_{pe} (psi)	160000
Cover to center of Strand (in.)	1
d (in.)	5
fy (psi)	60000
fpy (psi)	216000
Zt (in3)	948
Zb (in3)	948

Loads	
SELF (psf)	75
SDL (psf)	25
LL (psf)	40

Preliminary Sags	
δ1 (in.)	2.828
δ2 (in.)	4
δ3 (in.)	4

Load To Balance	
wb (plf)	1316.667

Balancing Force	
Fb (lbs)	298283

Number of Strands	
N	13

Actual Prestressing Force	
F (lbs)	318240

Actual Sags	
δ1 (in.)	2.828
δ2 (in.)	1.793536
δ3 (in.)	2.010747

Allowable Stresses		
σ_c (psi)	2250	
σ_t (psi)	-141.421	Positive Moment
σ_t (psi)	-424.264	Negative Moment

Service Stress Checks		
Avg. Prestress (psi)	335.6962	OK
Max Compressive stress (psi)	519.1139	OK
Max Positive Moment Tensile Stress (psi)	220.3797	OK
Max Negative Tensile Stress (psi)	152.2785	OK

Bonded Reinf. @ Supports		
A_s (in^2)	0.711	
$A_{s\ provided}$ (in^2)	0.8	4-#4 bars

NO BONDED REINF.@ MID SPAN

ULTIMATE FLEXURAL STRENGTH (AT SUPPORTS)		
A_{ps} (in^2)	1.989	
A_s (in^2)	0.8	
d (in.)	5	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0.0010127	
ω	0.0121519	
ρ_p	0.0025177	
f_{ps} (psi)	242507.29	
a	0.7897945	
c	0.9872431	
c/d	0.1974486	
Φ	0.9	
ΦM_n (ft-k)	-183.17268	OK

ULTIMATE FLEXURAL STRENGTH (POSITIVE MOMENT)		
A_{ps} (in^2)	1.989	
A_s (in^2)	0	
d (in.)	2.7935363	
f_{py}/f_{pu}	0.8	
γ_p	0.55	
ρ	0	
ω	0	
ρ_p	0.0045063	
f_{ps} (psi)	224829.64	
a	0.6659511	
c	0.8324388	
c/d	0.2979875	
Φ	0.9	
ΦM_n (ft-k)	82.524652	OK

	ULTIMATE FLEXURAL STRENGTH (POSITIVE MOMENT)		
	A_{ps} (in^2)	1.989	
	A_s (in^2)	0	
	d (in.)	2.793536	
	f_{py}/f_{pu}	0.8	
	γ_p	0.55	
	ρ	0	
	ω	0	
	ρ_p	0.004506	
	f_{ps} (psi)	224829.6	
	a	0.665951	
	c	0.832439	
	c/d	0.297987	
	Φ	0.9	
OK	ΦM_n (ft-k)	82.52465	OK

PUNCHING SHEAR CHECK AT CORNER COLUMN		
c_1 (in.)	28	
c_2 (in.)	28	
d (in.)	5	
Distance to Discontinuous Edge (c1 side) (in.)	18	
Distance to Discontinuous Edge (c2 side) (in.)	0	
b_o (in.)	79	
Ac (in^2)	395	
c_3 (in.)	1.177532	
c_4 (in.)	29.32247	
c_5 (in.)	1.177532	
c_6 (in.)	29.32247	
c_7 (in.)	15.32247	
c_8 (in.)	15.32247	
γ_{v1}	0.4	
γ_{v2}	0.4	
J_{c1} (in^4)	42551.33	
J_{c2} (in^4)	42551.33	
M_{u1} (Orthagonal to Frame considederd) (ft-k)	89.4	
M_{u2} (In Direction of Frame considederd) (ft-k)	62.64	
v_{u1} (psi)	89.43256	
v_{u2} (psi)	160.2845	
v_{u3} (psi)	245.2445	
v_{uMAX} (psi)	245.2445	
Can Prestress Shear Strenth be Used?	NO	
α_s	20	
β	1	
v_n (psi)	230.9285	
ϕv_n (psi)	173.1964	NG, NEED STUDRAILS

PUNCHING SHEAR CHECK AT EXTERIOR EDGE COLUMN		
c_1 (in.)	20	
c_2 (in.)	20	
d (in.)	5	
Distance to Discontinuous Edge (in.)	0	
b_o (in.)	70	
Ac (in^2)	350	
c_3 (in.)	7.23214	
c_4 (in.)	15.2679	
c_5 (in.)	5.26786	
γ_v	0.38743	
J_c (in^4)	20131.1	
M_u (Orthagonal to Frame considederd) (ft-k)	89.64	
V_u (kips)	46	
v_{u1} (psi)	247.418	
v_{u2} (psi)	376.296	
v_{uMAX} (psi)	376.296	
Can Prestress Shear Strength be Used?	NO	
α_s	30	
β	1	
v_n (psi)	282.843	
ϕv_n (psi)	212.132	NG, NEED STUDRAILS

DEFLECTION		
$\Delta_{allowable}$ (in.)	0.68889	
Δ_{actual} (in.)	0.13276	OK

Column Load Take Downs

COLUMN: 24 Axial Force	f'c (psi) =	5000

FLOOR	Story Height	Area (sq. ft.)	Influence Area	Cladding width	Col Width	Col Depth	Slab Thick	Slab DL (psf)	SDL (psf)	Cladding (psf)	Floor LL (psf)	MISC. DL (kips)	MISC. LL (kips)	Relative Floor	Col. Weight	Cladding (kips)	Floor DL (kips)	Total DL (kips)	LL Reductio	Total LL (kips)	Design DL (kips)	Design LL (kips)	Total Design
Bulkhead	9													Main Roof									
Main Roof	10													16									
16	10	167	582	26	28	28	12	150	15	70	40			15	8.16667	13.2	27.555	53.9217	0.37177	5.82342	64.706	9.31748	74.0235
15	9	167	1140	26	28	28	6	75	15	70	40			14	7.35	16.38	15.03	38.76	0.59426	4.63767	46.512	7.42027	127.956
14	9	167	1698	26	28	28	6	75	15	70	40			13	7.35	16.38	15.03	38.76	0.51402	4.10164	46.512	6.56262	181.03
13	9	167	2256	26	28	28	6	75	15	70	40			12	7.35	16.38	15.03	38.76	0.56581	3.77959	46.512	6.04734	233.59
12	9	167	2814	26	28	28	6	75	15	70	40			11	7.35	16.38	15.03	38.76	0.53277	3.55889	46.512	5.69422	285.796
11	9	167	3372	26	28	28	6	75	15	70	40			10	7.35	16.38	15.03	38.76	0.50831	3.39554	46.512	5.43286	337.741
10	9	167	3930	26	28	28	6	75	15	70	40			9	7.35	16.38	15.03	38.76	0.48927	3.26835	46.512	5.22936	389.482
9	9	167	4488	26	28	28	6	75	15	70	40			8	7.35	16.38	15.03	38.76	0.47391	3.16569	46.512	5.0651	441.059
8	9	167	5046	26	28	28	6	75	15	70	40			7	7.35	16.38	15.03	38.76	0.46116	3.08057	46.512	4.92891	492.5
7	9	167	5604	26	28	28	6	75	15	70	40			6	7.35	16.38	15.03	38.76	0.45037	3.0085	46.512	4.8136	543.826
6	9	167	6162	26	28	28	6	75	15	70	40			5	7.35	16.38	15.03	38.76	0.44109	2.94646	46.512	4.71433	595.052
5	9	167	6720	26	28	28	6	75	15	70	40			4	7.35	16.38	15.03	38.76	0.43298	2.89231	46.512	4.6277	646.192
4	9	167	7278	26	28	28	6	75	15	70	40			3	7.35	16.38	15.03	38.76	0.42583	2.84452	46.512	4.55124	697.255
3	9	167	7836	26	28	28	6	75	15	70	40			2	7.35	16.38	15.03	38.76	0.41945	2.80193	46.512	4.48309	748.25
2	13.75	167	8394	26	28	28	6	75	15	70	40			1	11.2292	25.025	15.03	51.2842	0.41372	2.76366	61.541	4.42186	814.213
Ground	13.33	167	8952	26	28	28	12	150	15	70	100			0	10.8862	24.2606	27.555	62.7018	0.40854	6.82257	75.2421	10.9161	900.371

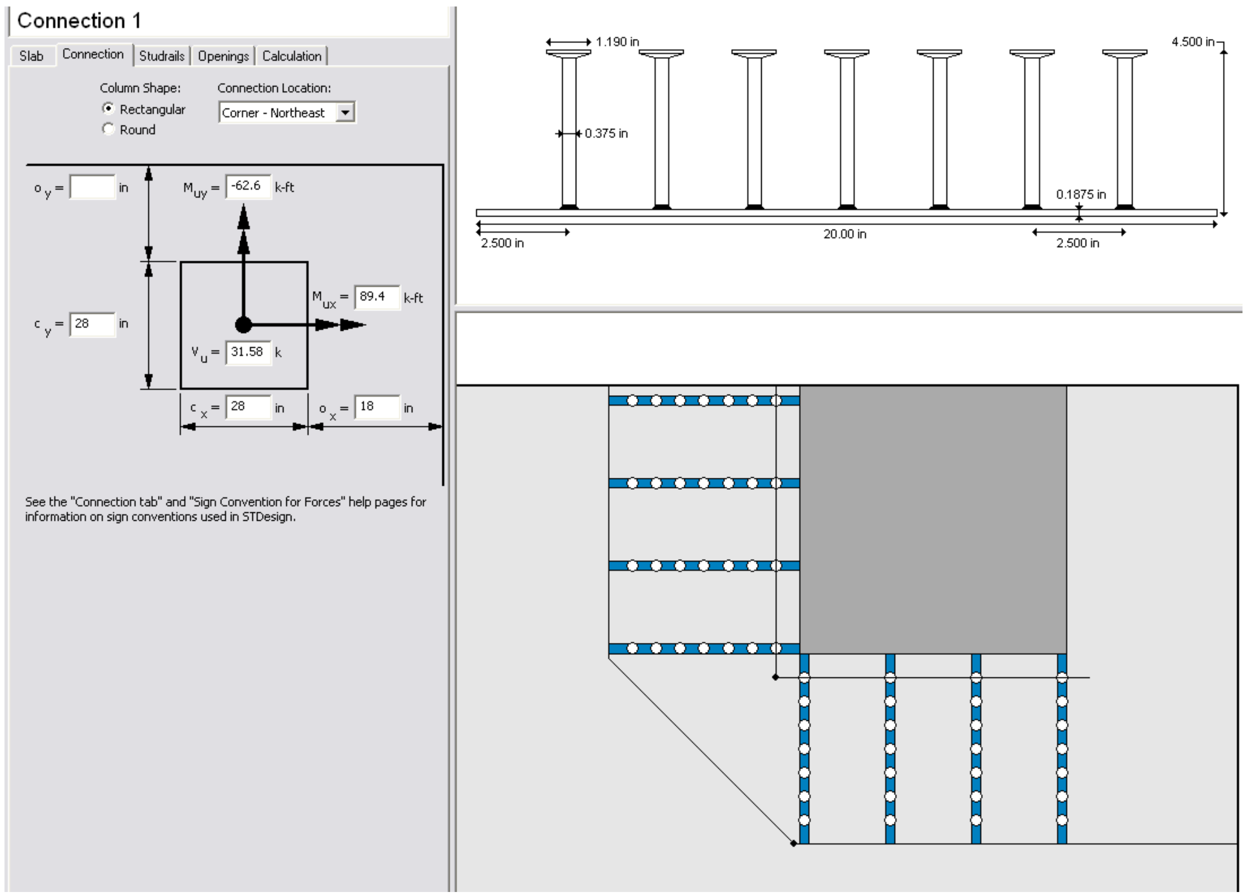
COLUMN: 24 w-w Direction Moment

FLOOR	Column Height (l_u)	K	r_w (in.)	Col Width (in.)	kl_u/r_w	kl/r_w limit	Consider Slenderness	M2 (ft-k)	M1 (ft-k)	$M_{2,MIN}$ (ft-k)	M1/M2 (w-w)	E_c (psi)	I_g (in^4)	EI (lb-in^2)	P_c (kips)	C_m	δns	Design M_{w-w} (ft-k)
Bulkhead	8																	
Main Roof	9																	
16	9	1	8.4	28	12.8571	25.7377	NO	61	42	8.88282	0.68352	4030509	51221.3	5.2E+07	43672	0.87541	1	61
15	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	15.3547	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
14	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	21.7236	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
13	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	28.0308	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
12	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	34.2955	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
11	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	40.5289	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
10	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	46.7379	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
9	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	52.9271	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
8	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	59.1	0.68352	4030509	51221.3	5.2E+07	48961	0.87541	1	61
7	8.5	1	8.4	28	12.1429	26.2769	NO	61	42	65.2591	0.64359	4030509	51221.3	5.2E+07	48961	0.85744	1	65.2591
6	8.5	1	8.4	28	12.1429	26.9418	NO	61	42	71.4063	0.58818	4030509	51221.3	5.2E+07	48961	0.83527	1	71.4063
5	8.5	1	8.4	28	12.1429	27.5004	NO	61	42	77.543	0.54163	4030509	51221.3	5.2E+07	48961	0.81665	1	77.543
4	8.5	1	8.4	28	12.1429	27.9764	NO	61	42	83.6706	0.50197	4030509	51221.3	5.2E+07	48961	0.80079	1	83.6706
3	8.5	1	8.4	28	12.1429	28.3869	NO	61	42	89.79	0.46776	4030509	51221.3	5.2E+07	48961	0.7871	1	89.79
2	13.25	1	8.4	28	18.9286	28.8416	NO	61	42	97.7056	0.42986	4030509	51221.3	5.2E+07	20149.1	0.77195	1	97.7056
Ground	12.33	1	8.4	28	17.6143	29.3353	NO	61	42	108.045	0.38373	4030509	51221.3	5.2E+07	23268.1	0.75549	1	108.045

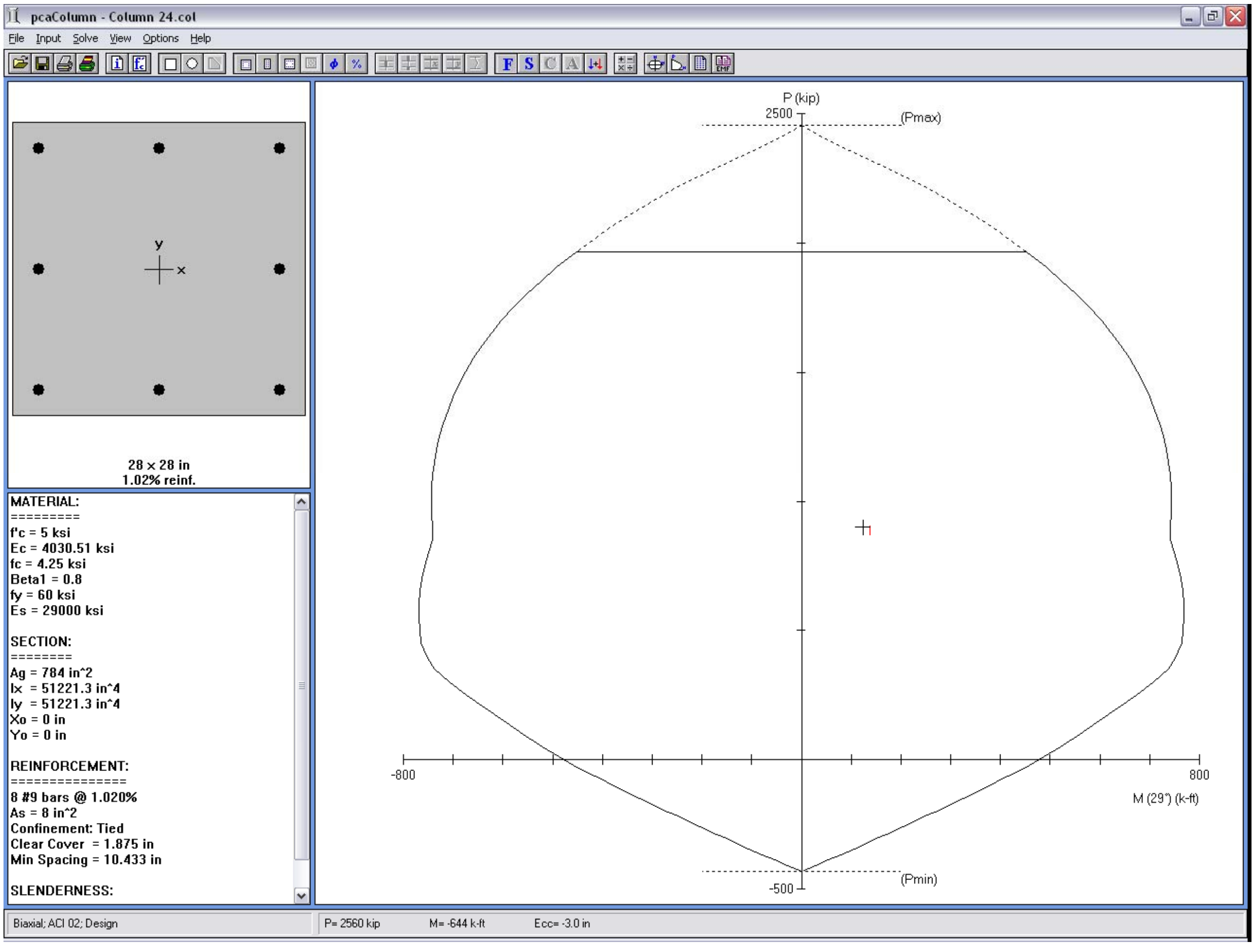
COLUMN: 24 d-d
Direction Moment

FLOOR	Column Height (l_u)	K	r_d (in.)	Col Width (in.)	kl_u/r_d	kl/r_d limit	Consider Slenderness	M2 (ft-k)	M1 (ft-k)	$M_{2,MIN}$ (ft-k)	M1/M2 (w-w)	E_c (psi)	I_g (in^4)	EI ($lb\text{-}in^2$)	P_c (kips)	C_m	δ_{ns}	Design $M_{d\text{-}d}$ (ft-k)
Bulkhead	8																	
Main Roof	9																	
16	9	1	8.4	28	12.8571	25.7377	NO	61	42	8.88282	0.68852	4030509	51221.3	5.2E+07	43672	0.87541	1	61
15	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	15.3547	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
14	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	21.7236	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
13	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	28.0308	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
12	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	34.2955	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
11	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	40.5289	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
10	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	46.7379	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
9	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	52.9271	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
8	8.5	1	8.4	28	12.1429	25.7377	NO	61	42	59.1	0.68852	4030509	51221.3	5.2E+07	48961	0.87541	1	61
7	8.5	1	8.4	28	12.1429	26.2769	NO	61	42	65.2591	0.64359	4030509	51221.3	5.2E+07	48961	0.85744	1	65.2591
6	8.5	1	8.4	28	12.1429	26.9418	NO	61	42	71.4063	0.58818	4030509	51221.3	5.2E+07	48961	0.83527	1	71.4063
5	8.5	1	8.4	28	12.1429	27.5004	NO	61	42	77.543	0.54163	4030509	51221.3	5.2E+07	48961	0.81665	1	77.543
4	8.5	1	8.4	28	12.1429	27.9764	NO	61	42	83.6706	0.50197	4030509	51221.3	5.2E+07	48961	0.80079	1	83.6706
3	8.5	1	8.4	28	12.1429	28.3869	NO	61	42	89.79	0.46776	4030509	51221.3	5.2E+07	48961	0.7871	1	89.79
2	13.25	1	8.4	28	18.9286	28.8416	NO	61	42	97.7056	0.42986	4030509	51221.3	5.2E+07	20149.1	0.77195	1	97.7056
Ground	12.33	1	8.4	28	17.6143	29.3353	NO	61	42	108.045	0.38873	4030509	51221.3	5.2E+07	23268.1	0.75549	1	108.045

Studrail Calculations (corner column)



Column Design (corner column)



Appendix – B

Lateral Calculations

MULKEY
ENGINEERS & CONSULTANTS

CALCULATION SHEET

PAGE 1 OF 2

CLIENT THESIS SUBJECT SEISMIC CALCS Prepared By DPD Date

PROJECT No. Reviewed By Date

REFERENCE ASCE 7-05

$S_s = 0.363$
$S_1 = 0.07$
} NEW YORK, NY (FROM USGS WEBSITE)

SITE CLASS : C PER GEO-TECH

	CODE REF.
$F_a = 1.2$	11.4.3
$F_v = 1.7$	11.4.3
$S_{MS} = F_a S_s = (1.2)(0.363) = 0.4356$	11.4.3
$S_{M1} = F_v S_1 = (1.7)(0.07) = 0.119$	11.4.3
$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.4356) = 0.2904$	11.4.4
$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.119) = 0.0793$	11.4.4
OCCUPANCY CATEGORY II	1.5.1
$I = 1.0$	11.5.1
SEISMIC DESIGN CATEGORY B	11.6
$R = 4$, $\Omega_0 = 2\frac{1}{2}$, $C_d = 4$ } ORDINARY REINF. CONC. SHEAR WALL	12.2.1
CHECK VERTICAL IRREGULARITY TYPE 2:	12.3.2.2

FLOOR	WEIGHT
2	$\left(\frac{12"}{12"/'}\right)(150\ PCF)(99' \times 99') = 1470^k$
3	$\left(\frac{6"}{12"/'}\right)(150\ PCF)(99' \times 68') = 505^k$

ME-01

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE 2 OF 2

CLIENT: THESIS — SUBJECT: SEISMIC CALCS — Prepared By: DPD — Date:

PROJECT No. — Reviewed By: — Date:

	CODE REF.
$505^k \times 1.5 = 758^k$	
$758^k < 1470^k$ ∴ VERTICALL IRREGULARITY TYPE II	12.3.2.2
HOWEVER NO ADDITIONAL MEASURES NEED TO BE TAKEN SINCE BLDG. NOT IN SDC D, E, OR F	TABLE 12.3-2
$\rho = 1.0$	12.3.4.1
USE ELF PROCEEDURE	TABLE 12.6-1
$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.2904}{\left(\frac{4}{1}\right)} = 0.0726 > 0.01$ ∴ ok	12.8.1.1
$T_L = 6$	FIG 22-15
$T_{ANALYSIS} = 2.1$ SEC (FROM ETABS)	
$T_a = C_t h_n^x = (0.02)(160)^{0.75} = 0.90$ SEC	12.8.2.1
$C_u = 1.7$	12.8.2
$T_{MAX} \leq (1.7)(0.9) = 1.53$ SEC ← GOVERNS	12.8.2
$C_s \leq \frac{S_{D1}}{T_a\left(\frac{R}{I}\right)} = \frac{0.0793}{(1.53)\left(\frac{4}{1}\right)} = \boxed{0.013}$ ← GOVERNS	12.8.1.1
$\boxed{V = (0.013)W}$	12.8.1

ME-01

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE 1 OF 1

CLIENT: THESIS — SUBJECT: SEISMIC WT. CALCS — Prepared By: DPD — Date:

PROJECT No.: — Reviewed By: — Date:

	CODE REF.
REFERENCE ASCE 7-05	
20% P_f INCLUDED IN SEISMIC WT. IF $P_f \geq 30$ PSF	12.7.2
P_f CALCULATION	
$P_g = 25$ PSF	FIG 7-1
$C_e = 0.9$	TABLE 7-2
$C_t = 1.0$	TABLE 7-3
$I = 1.0$	TABLE 7-4
$P_f = 0.7 C_e C_t I P_g = (0.7)(0.9)(1.0)(1.0)(25)$	7.3
$P_f = 15.75$ PSF	
HOWEVER $P_f \geq 20$ PSF ← GOVERNS	7.3
SINCE $P_f = 20$ PSF < 30 PSF	12.7.2
↳ NO SNOW LOAD INCLUDED	
WEIGHTS TO BE INCLUDED FOR SEISMIC:	12.7.2
• DEAD LOAD	
• SUPER IMPOSED DEAD LOAD	

ME-01

Building Weight Calculation

Floor	Story ht.	h_i	SLAB AREA	SLAB OPN'G	NET SLAB AREA	SLAB DL	SDL	Slab weight	Misc LL
	(ft)	(ft)	(ft^2)	(ft^2)	(ft^2)	[psf]	[psf]	[kips]	[psf]
Bulkhead		160.00	1040	0	1040	75	0	78	0
	9.00								
Roof		151.00	4722	200	4522	150	10	724	0
	10.17								
16		140.84	6732	200	6532	150	25	1143	0
	10.17								
15		130.67	6732	200	6532	75	25	653	0
	9.00								
14		121.67	6732	200	6532	75	25	653	0
	9.00								
13		112.67	6732	200	6532	75	25	653	0
	9.00								
12		103.67	6732	200	6532	75	25	653	0
	9.00								
11		94.67	6732	200	6532	75	25	653	0
	9.00								
10		85.67	6732	200	6532	75	25	653	0
	9.00								
9		76.67	6732	200	6532	75	25	653	0
	9.00								
8		67.67	6732	200	6532	75	25	653	0
	9.00								
7		58.67	6732	200	6532	75	25	653	0
	9.00								
6		49.67	6732	200	6532	75	25	653	0
	9.00								
5		40.67	6732	200	6532	75	25	653	0
	9.00								
4		31.67	6732	200	6532	75	25	653	0
	9.00								
3		22.67	6732	200	6532	75	25	653	0
	9.00								
2		13.67	9801	200	9601	116	25	1354	0
	13.67								
Base		0.00							

100	psf for brick & block
40	psf @ windows/precast

Floor	Brick & Block Façade [ft.]	Window/ Stone Façade [ft]	Façade DL [kips]	Cols [ft²]	Shear walls [ft²]	Cols [kips]	Shear walls [kips]	Eqmt [kips]	W (floor) [kips]
Bulkhead								0	185
	100	0	90	11	81	15	109		
Roof								50	1,099
	90	200	173	40	133	61	203		
16								0	1,617
	140	200	224	55	133	84	203		
15								0	1,135
	140	200	198	55	133	75	180		
14								0	1,105
	140	200	198	55	133	75	180		
13								0	1,105
	140	200	198	55	133	75	180		
12								0	1,105
	140	200	198	55	133	75	180		
11								0	1,105
	140	200	198	55	133	75	180		
10								0	1,105
	140	200	198	55	133	75	180		
9								0	1,105
	140	200	198	55	133	75	180		
8								0	1,105
	140	200	198	55	133	75	180		
7								0	1,105
	140	200	198	55	133	75	180		
6								0	1,105
	140	200	198	55	133	75	180		
5								0	1,105
	140	200	198	55	133	75	180		
4								0	1,105
	140	200	198	55	133	75	180		
3								0	1,105
	140	200	198	55	133	75	180		
2								0	2,012
	275	100	431	79	133	162	273		
Base									
									19,311

Seismic Force Calculation

C_s	W	V	Floor	k	h_x	w_x	$w_x h_x^k$	$\Sigma w_i h_i^k$	C_v	F_x (kips)	Over Turning Moment (ft-k)
#####	19,311	251	Bulkhead	1.02	160.00	185.17	31972.26		0.02	4.82	770.665
			Roof	1.02	151.00	1099.03	178929.96		0.11	26.96	4070.36
			16	1.02	140.84	1616.78	245244.11		0.15	36.95	5203.28
			15	1.02	130.67	1134.58	159497.30		0.10	24.03	3139.72
			14	1.02	121.67	1105.27	144520.56		0.09	21.77	2648.96
			13	1.02	112.67	1105.27	133676.11		0.08	20.14	2268.94
			12	1.02	103.67	1105.27	122844.66		0.07	18.51	1918.54
			11	1.02	94.67	1105.27	112027.32		0.07	16.88	1597.71
			10	1.02	85.67	1105.27	101225.42		0.06	15.25	1306.41
			9	1.02	76.67	1105.27	90440.56		0.05	13.62	1044.6
			8	1.02	67.67	1105.27	79674.71		0.05	12.00	812.228
			7	1.02	58.67	1105.27	68930.39		0.04	10.38	609.24
			6	1.02	49.67	1105.27	58210.85		0.03	8.77	435.572
			5	1.02	40.67	1105.27	47520.57		0.03	7.16	291.15
			4	1.02	31.67	1105.27	36866.01		0.02	5.55	175.888
			3	1.02	22.67	1105.27	26257.40		0.02	3.96	89.6736
			2	1.02	13.67	2012.43	28610.55	1666448.74	0.02	4.31	58.9191
										Total	**26441.8**

Wind Load Calculations

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET

PAGE 1 OF ____

CLIENT THESIS SUBJECT WIND LOAD CALCS Prepared By DPD Date ____

PROJECT No. N-S DIRECTION Reviewed By ____ Date ____

	CODE REF.
REFERENCE ASCE 7-05	
METHOD 2 – ANALYTICAL PROCEEDURE	
$V = 110$ MPH	6.5.4
$K_d = 0.85$ MWFRS	6.5.4.4
OCCUPANCY CATEGORY II	1.5.1
$I = 1.0$	6.5.5
SURFACE ROUGHNESS : B	6.5.6.2
EXPOSURE CATEGORY : B	6.5.6.3
$K_z \rightarrow$ FROM TABLE 6-3	6.5.6.6
$K_{zt} = 1.0$	6.5.7.2
NATURAL FREQUENCY $\eta_1 = \frac{1}{T} = \frac{1}{2.1} = 0.476$ Hz	
$\eta_1 = 0.476$ Hz < 1.0 Hz $\rightarrow$ FLEXIBLE STRUCTURE	6.5.8.2
GUST FACTOR CALC	6.5.8.2
$g_R = \sqrt{2\ln(3600\eta_1)} + \frac{0.577}{\sqrt{2\ln(3600\eta_1)}}$	
$= \sqrt{2\ln(3600 \times 0.476)} + \frac{0.577}{\sqrt{2\ln(3600\times 0.476)}}$	
$g_R = 4.01$	
$g_Q = g_v = 3.4$	
$\bar{z} = 0.6h = (0.6)(160) = 96'$	TABLE 6-2
$z_{MIN} = 30' < \bar{z}$ $\therefore$ ok	TABLE 6-2

ME-01

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE 2 OF ____

CLIENT ________ SUBJECT ________ Prepared By ____ Date ____
PROJECT No. ________ Reviewed By ____ Date ____

CODE REF.

$c = 0.3$; $\ell = 320$; $\bar{\epsilon} = 1/3$; $\bar{\alpha} = 1/4$; $\bar{b} = 0.45$ — TABLE 6-2

$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.3\left(\frac{33}{96}\right)^{1/6} = 0.251$

$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{96}{33}\right)^{1/3} = 457$

$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}} V\left(\frac{88}{60}\right) = (0.45)\left(\frac{96}{33}\right)^{1/4}(110)\left(\frac{88}{60}\right) = 94.8$

$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{(0.476)(457)}{94.8} = 2.29$

$B = 99'$
$h = 160'$
$L = 68'$
N-S DIRECTION

$R_n = \frac{7.47 N_1}{(1 + 10.3 N_1)^{5/3}} = \frac{(7.47)(2.29)}{[1 + (10.3)(2.29)]^{5/3}} = 0.0822$

$\eta_h = 4.6 n_1 h / \bar{V}_{\bar{z}} = [4.6 \times 0.476 \times 160]/94.8 = 3.70$

$R_h = \frac{1}{\eta_h} - \frac{1}{2\eta_h^2}(1 - e^{-2\eta_h}) = \frac{1}{3.7} - \frac{1}{2(3.7)^2}(1 - e^{-(2 \times 3.7)})$

$R_h = 0.234$

$\eta_B = 4.6 n_1 B / \bar{V}_{\bar{z}} = [4.6 \times 0.476 \times 99]/94.8 = 2.29$

$R_B = \frac{1}{2.29} - \frac{1}{2(2.29)^2}(1 - e^{-2 \times 2.29}) = 0.342$

$\eta_L = 15.4 n_1 L / \bar{V}_{\bar{z}} = (15.4 \times 0.476 \times 68)/94.8 = 5.26$

$R_L = \frac{1}{5.26} - \frac{1}{2(5.26)^2}(1 - e^{-2 \times 5.26}) = 0.172$

$\beta = 0.02$ — C6.5.8

ME-01

MULKEY ENGINEERS & CONSULTANTS — CALCULATION SHEET — PAGE 3 OF

CLIENT ___ SUBJECT ___ Prepared By ___ Date ___
PROJECT No. ___ Reviewed By ___ Date ___

CODE REF.

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)}$$ 6.5.8.2

$$= \sqrt{\frac{1}{0.02}(0.0822)(0.234)(0.342)(0.53 + [0.47][0.172])}$$

$$R = 0.448$$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = \sqrt{\frac{1}{1 + 0.63\left(\frac{99+160}{457}\right)^{0.63}}}$$ 6.5.8.1

$$Q = 0.833$$

$$G_f = 0.925\left(\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right)$$ 6.5.8.2

$$= 0.925 \; \frac{1 + (1.7 \times 0.251)\sqrt{(3.4^2)(0.833^2) + (4.01^2)(0.448^2)}}{1 + (1.7 \times 3.4 \times 0.251)}$$

$$G_f = 0.918$$

$C_p = 0.8$ WINDWARD WALLS 6.5.11.2

$L/B = 68/99 < 1.0 \rightarrow C_p = -0.5$ LEEWARD WALLS

$q_z = 0.00256 K_z K_{zt} K_d V^2 I$ 6.5.10

$K_z = 1.13$ @ 160' TABLE 6-3

$q_z = (0.00256)(1.13)(1.0)(0.85)(110)^2(1.0) = 29.75$ PSF

P_z @ 160' $= qGC_p = (29.75)(0.918)(0.8) = \underline{21.85 \text{ PSF}}$

ME-01

Gust Factor Calculation Per ASCE 7-05 6.5.8.2		
h (ft)	160	ASCE 7-05 Code Reference
L (ft)	68	Width Paralell to Wind Direction
B (ft)	99	Width Normal to Wind Direction
V	110	
g_Q	3.4	6.5.8.2
g_v	3.4	6.5.8.2
n_1	0.476	C6.5.8 EQ.(C6-16)
g_R	4.008623	6.5.8.2 EQ.(6-9)
z	96	Table 6-2
c	0.3	Table 6-2
I_z	0.251089	6.5.8.1 EQ. (6-5)
ℓ	320	Table 6-2
ϵ	0.333333	Table 6-2
L_z	456.8101	6.5.8.1 EQ. (6-7)
Q	0.833148	6.5.8.1 EQ. (6-6)
b	0.45	Table 6-2
α	0.25	Table 6-2
V_z	94.81476	6.5.8.2 EQ.(6-14)
N_1	2.293331	6.5.8.2 EQ.(6-12)
R_n	0.082212	6.5.8.2 EQ.(6-11)
η_h	3.694952	6.5.8.2 EQ.(6-13)
R_h	0.234039	6.5.8.2 EQ.(6-13)
η_B	2.286252	6.5.8.2 EQ.(6-13)
R_B	0.342727	6.5.8.2 EQ.(6-13)
η_L	5.257274	6.5.8.2 EQ.(6-13)
R_L	0.172123	6.5.8.2 EQ.(6-13)
β	0.02	C6.5.8
R	0.448803	6.5.8.2 EQ.(6-10)
G_f	0.917869	6.5.8.2 EQ.(6-8)

North/South Wind Load Calc. (Windward Face)

story	story height	height (ft)	k_z	k_{zt}	k_d	V	I	q_z	G_f	C_p	p_z
Bulkhead		160.00	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.80	**21.85**
	9.00										
Roof		151.00	1.11	1.00	0.85	110.00	1.00	29.23	0.92	0.80	**21.46**
	10.17										
16.00		140.84	1.09	1.00	0.85	110.00	1.00	28.70	0.92	0.80	**21.07**
	10.17										
15.00		130.67	1.07	1.00	0.85	110.00	1.00	28.17	0.92	0.80	**20.69**
	9.00										
14.00		121.67	1.04	1.00	0.85	110.00	1.00	27.38	0.92	0.80	**20.11**
	9.00										
13.00		112.67	1.02	1.00	0.85	110.00	1.00	26.86	0.92	0.80	**19.72**
	9.00										
12.00		103.67	1.00	1.00	0.85	110.00	1.00	26.33	0.92	0.80	**19.33**
	9.00										
11.00		94.67	0.98	1.00	0.85	110.00	1.00	25.80	0.92	0.80	**18.95**
	9.00										
10.00		85.67	0.95	1.00	0.85	110.00	1.00	25.01	0.92	0.80	**18.37**
	9.00										
9.00		76.67	0.91	1.00	0.85	110.00	1.00	23.96	0.92	0.80	**17.59**
	9.00										
8.00		67.67	0.88	1.00	0.85	110.00	1.00	23.17	0.92	0.80	**17.01**
	9.00										
7.00		58.67	0.85	1.00	0.85	110.00	1.00	22.38	0.92	0.80	**16.43**
	9.00										
6.00		49.67	0.81	1.00	0.85	110.00	1.00	21.33	0.92	0.80	**15.66**
	9.00										
5.00		40.67	0.76	1.00	0.85	110.00	1.00	20.01	0.92	0.80	**14.69**
	9.00										
4.00		31.67	0.71	1.00	0.85	110.00	1.00	18.69	0.92	0.80	**13.73**
	9.00										
3.00		22.67	0.64	1.00	0.85	110.00	1.00	16.85	0.92	0.80	**12.37**
	9.00										
2.00		13.67	0.57	1.00	0.85	110.00	1.00	15.01	0.92	0.80	**11.02**
	13.67										
BASE		0.00									

North/South Wind Load Calc. (Leeward Face)

story	story height	height (ft)	k_z	k_{zt}	k_d	V	I	q_z	G_f	C_p	p_z
Bulkhead		160.00	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
Roof		151.00	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	10.17										
16.00		140.84	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	10.17										
15.00		130.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
14.00		121.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
13.00		112.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
12.00		103.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
11.00		94.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
10.00		85.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
9.00		76.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
8.00		67.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
7.00		58.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
6.00		49.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
5.00		40.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
4.00		31.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
3.00		22.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	9.00										
2.00		13.67	1.13	1.00	0.85	110.00	1.00	29.75	0.92	0.50	13.65
	13.67										
BASE		0.00									

15 STORY TOWER N/S WIND FORCES									
story	story height	height (ft)	p_z (windward)	p_z (leeward)	Total Pressure (psf)	building length (ft)	Story Force (kips)	Diaphram Force (kips)	Over-Turning Moment (ft-k)
Bulkhead		160.00	21.85	13.65	35.50	50.00		7.99	1278.09
	9.00						15.98		
Roof		151.00	21.46	13.65	35.11	99.00		25.66	3874.75
	10.17						35.34		
16.00		140.84	21.07	13.65	34.73	99.00		35.15	4950.37
	10.17						34.96		
15.00		130.67	20.69	13.65	34.34	99.00		32.78	4282.92
	9.00						30.60		
14.00		121.67	20.11	13.65	33.76	99.00		30.34	3691.45
	9.00						30.08		
13.00		112.67	19.72	13.65	33.37	99.00		29.91	3369.87
	9.00						29.74		
12.00		103.67	19.33	13.65	32.99	99.00		29.56	3064.97
	9.00						29.39		
11.00		94.67	18.95	13.65	32.60	99.00		29.22	2766.27
	9.00						29.05		
10.00		85.67	18.37	13.65	32.02	99.00		28.79	2466.40
	9.00						28.53		
9.00		76.67	17.59	13.65	31.25	99.00		28.19	2161.06
	9.00						27.84		
8.00		67.67	17.01	13.65	30.67	99.00		27.58	1866.59
	9.00						27.33		
7.00		58.67	16.43	13.65	30.09	99.00		27.07	1588.01
	9.00						26.81		
6.00		49.67	15.66	13.65	29.31	99.00		26.46	1314.46
	9.00						26.12		
5.00		40.67	14.69	13.65	28.35	99.00		25.69	1044.76
	9.00						25.26		
4.00		31.67	13.73	13.65	27.38	99.00		24.83	786.28
	9.00						24.40		
3.00		22.67	12.37	13.65	26.03	99.00		23.79	539.41
	9.00						23.19		
2.00		13.67	11.02	13.65	24.67	99.00		28.29	386.75
	13.67						33.39		
BASE		0.00						TOTAL	39432.41

Drift Calculations

Story	Story Height	EQ Load Building Drift E/W	EQ Story Drift E/W	Amplified Story Drifts	Allowable Story Drift	
BULKHEAD	9	2.9144	0.2141	0.8564	2.7	OK
MAIN ROOF	10.167	2.7003	0.2231	0.8924	3.0501	OK
STORY16	10.167	2.4772	0.2273	0.9092	3.0501	OK
STORY15	9	2.2499	0.204	0.816	2.7	OK
STORY14	9	2.0459	0.2024	0.8096	2.7	OK
STORY13	9	1.8435	0.2	0.8	2.7	OK
STORY12	9	1.6435	0.1963	0.7852	2.7	OK
STORY11	9	1.4472	0.1913	0.7652	2.7	OK
STORY10	9	1.2559	0.1847	0.7388	2.7	OK
STORY9	9	1.0712	0.1765	0.706	2.7	OK
STORY8	9	0.8947	0.1666	0.6664	2.7	OK
STORY7	9	0.7281	0.1548	0.6192	2.7	OK
STORY6	9	0.5733	0.141	0.564	2.7	OK
STORY5	9	0.4323	0.1253	0.5012	2.7	OK
STORY4	9	0.307	0.1072	0.4288	2.7	OK
STORY3	9	0.1998	0.0872	0.3488	2.7	OK
STORY2	13.75	0.1126	0.1126	0.4504	4.125	OK

Story	Story Height	EQ Load Building Drift N/S	EQ Story Drift N/S	Amplified Story Drifts	Allowable Story Drift	
BULKHEAD	9	2.4291	0.1771	0.7084	2.7	OK
MAIN ROOF	10.167	2.252	0.1957	0.7828	3.0501	OK
STORY16	10.167	2.0563	0.1953	0.7812	3.0501	OK
STORY15	9	1.861	0.1751	0.7004	2.7	OK
STORY14	9	1.6859	0.1736	0.6944	2.7	OK
STORY13	9	1.5123	0.1712	0.6848	2.7	OK
STORY12	9	1.3411	0.1678	0.6712	2.7	OK
STORY11	9	1.1733	0.163	0.652	2.7	OK
STORY10	9	1.0103	0.1569	0.6276	2.7	OK
STORY9	9	0.8534	0.1492	0.5968	2.7	OK
STORY8	9	0.7042	0.14	0.56	2.7	OK
STORY7	9	0.5642	0.129	0.516	2.7	OK
STORY6	9	0.4352	0.1162	0.4648	2.7	OK
STORY5	9	0.319	0.1015	0.406	2.7	OK
STORY4	9	0.2175	0.085	0.34	2.7	OK
STORY3	9	0.1325	0.0665	0.266	2.7	OK
STORY2	13.75	0.066	0.066	0.264	4.125	OK

Story	Story Height	Wind Load Building Drift E/W	Wind Story Drift E/W	Allowable Story Drift	
BULKHEAD	9	2.4453	0.1687	0.27	OK
MAIN ROOF	10.167	2.2766	0.1836	0.30501	OK
STORY16	10.167	2.093	0.1848	0.30501	OK
STORY15	9	1.9082	0.1661	0.27	OK
STORY14	9	1.7421	0.1655	0.27	OK
STORY13	9	1.5766	0.1641	0.27	OK
STORY12	9	1.4125	0.1621	0.27	OK
STORY11	9	1.2504	0.1588	0.27	OK
STORY10	9	1.0916	0.1546	0.27	OK
STORY9	9	0.937	0.149	0.27	OK
STORY8	9	0.788	0.1419	0.27	OK
STORY7	9	0.6461	0.1332	0.27	OK
STORY6	9	0.5129	0.1227	0.27	OK
STORY5	9	0.3902	0.1103	0.27	OK
STORY4	9	0.2799	0.0958	0.27	OK
STORY3	9	0.1841	0.0799	0.27	OK
STORY2	13.75	0.1042	0.1042	0.4125	OK

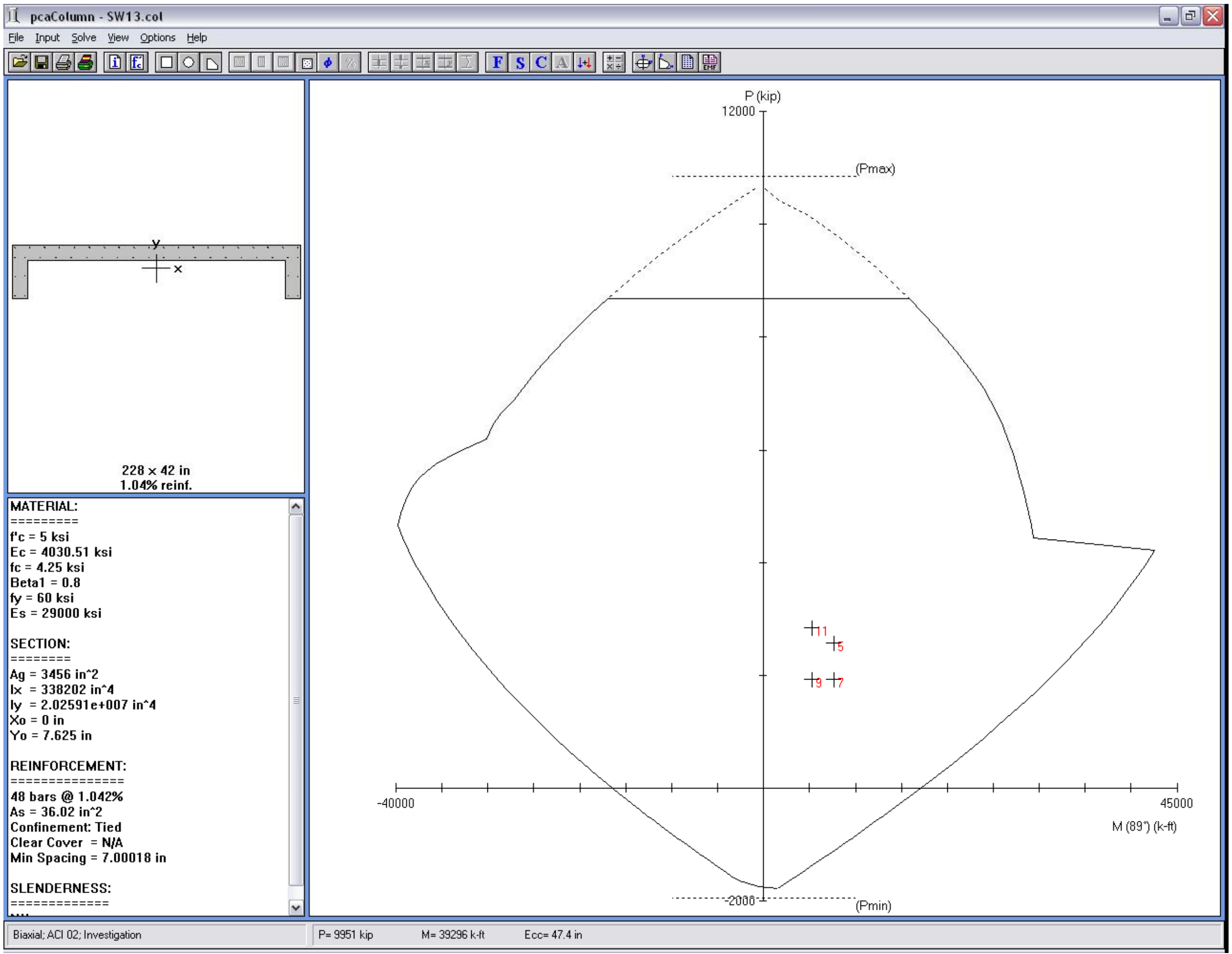
Story	Story Height	Wind Load Building Drift N/S	Wind Story Drift N/S	Allowable Story Drift	
BULKHEAD	9	3.377	0.2403	0.27	OK
MAIN ROOF	10.167	3.1367	0.2656	0.30501	OK
STORY16	10.167	2.8711	0.2649	0.30501	OK
STORY15	9	2.6062	0.2379	0.27	OK
STORY14	9	2.3683	0.2364	0.27	OK
STORY13	9	2.1319	0.2338	0.27	OK
STORY12	9	1.8981	0.2301	0.27	OK
STORY11	9	1.668	0.2247	0.27	OK
STORY10	9	1.4433	0.2175	0.27	OK
STORY9	9	1.2258	0.2084	0.27	OK
STORY8	9	1.0174	0.1969	0.27	OK
STORY7	9	0.8205	0.1831	0.27	OK
STORY6	9	0.6374	0.1665	0.27	OK
STORY5	9	0.4709	0.147	0.27	OK
STORY4	9	0.3239	0.1243	0.27	OK
STORY3	9	0.1996	0.0989	0.27	OK
STORY2	13.75	0.1007	0.1007	0.4125	OK

Shear Wall Design Calcs (SW 13)

	SW 7	SW 9	SW 11	SW 13
Max In-Plane Wind Moment (ft-k)	9863	4815	9863	4796
Max out-of-plane Wind Moment (ft-k)	144	758	144	123
Max Wind Shear (kips)	376	176	376	150
Max In-Plane EQ Moment (k-ft)	7262	5328	7262	5287
Max out-of-plane EQ Moment (k-ft)	124	506	124	82
Max EQ Shear (kips)	267	182	267	157

Fy (psi)	60000
f'c (psi)	5000
lw (in.)	228
hw (in.)	1920
h (in.)	12
N (kips)	2140
V (kips)	182
M (ft-k)	4815
Nu (lbs)	1926000
Vu (lbs)	291200
Mu (in-lb)	92448000

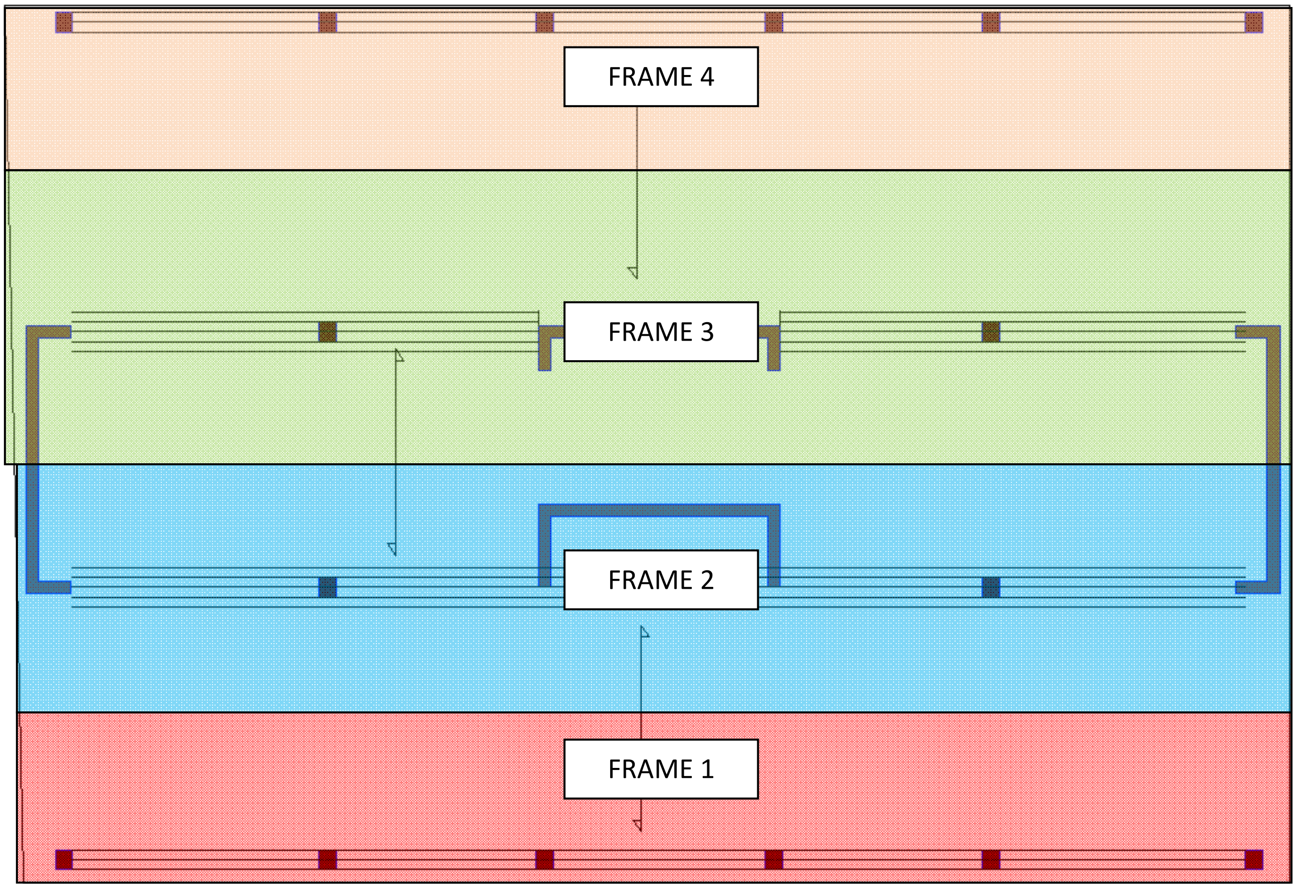
d (in.)	182.4	
Vc (kips)	654.9553	
1/2ΦVc (kips)	245.6083	< Vu, SHEAR REINF. PER 11.10.9.1
$S_{max\ horiz}$ (in.)	18	
$S_{max\ horiz}$ (in.)	18	
$\rho_{t,min}$	0.0025	
$\rho_{l,min}$	0.0025	
$A_{horiz,\ min}$ (in^2/S_{max})	0.54	
$A_{vert,\ min}$ (in^2/S_{max})	0.54	
$V_{s\ required}$ (kips)	-266.689	
$A_{horiz,\ required}$ (in^2/S_{max})	-0.43863	
$A_{horiz\ provided}$ (in^2/S_{max})	0.6	#4 bars @ 12" EACH FACE
ΦV_n (kips)	764.8165	


pcaColumn - SW13.col
File Input Solve View Options Help
228 x 42 in
1.04% reinf.
MATERIAL:
f'c = 5 ksi
Ec = 4030.51 ksi
fc = 4.25 ksi
Beta1 = 0.8
fy = 60 ksi
Es = 29000 ksi
SECTION:
Ag = 3456 in^2
Ix = 338202 in^4
Iy = 2.02591e+007 in^4
Xo = 0 in
Yo = 7.625 in
REINFORCEMENT:
48 bars @ 1.042%
As = 36.02 in^2
Confinement: Tied
Clear Cover = N/A
Min Spacing = 7.00018 in
SLENDERNESS:
P (kip)
12000
(Pmax)
-40000
45000
M (89°) (k-ft)
-2000
(Pmin)
Biaxial; ACI 02; Investigation
P= 9951 kip
M= 39296 k-ft
Ecc= 47.4 in

APPENDIX – C

Building layout

Banded Tendon Frames



Column and Shear Wall labels