THE PENNSYLVANIA STATE UNIVERISITY
ARCHITECTURAL ENGINEERING

PASEO CARIBE
CONDOMINIUM TOWER AND PARKING

Thesis Research

Coupled Shear Walls in High Seismic Zones

Lourdes Diaz
Structural Option
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April 10, 2006

Paseo Caribe Condominium Tower

PROJECT INFORMATION

LOCATION	OWNER
San Juan, Puerto Rico	San Geronimo Development

SIZE	BUILDING CODE
800,000 square feet	UBC 1997

CONSTRUCTION	COST
Design • Bid • Build	$170 Million

CONTRACTOR	ARCHITECT
F&R Contractors	Beame Architectural

ARCHITECTURE

•Fourteen levels of luxury apartments

•Ten levels of public and private parking – 1,700 spaces

•Total of 46 apartments – 3,200 sqft each

•Building envelop consists of reinforced concrete with colored stucco and panel glassing

STRUCTURE

•Primary lateral resisting system is comprised of reinforced concrete shear walls and post tensioned reinforced concrete slab system

•Reinforced concrete columns are used in lower levels of parking garage as transfer points to foundation piles

MECHANICAL

•Four in-line centrifugal exhaust fans (27,000 cfm each) provide ventilation to underground parking facility

• Each apartment is equipped with two individual fan coil units supplied with cold water by a chiller-cooling tower system

ELECTRICAL

•13.2KVΔ -120/240V Distribution transformer

•800KW-120/208V Diesel driven generator

•Feed four main switchboards, each serving four type A panel boards per floor

Lourdes F. Diaz

Structural Option

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Executive Summary

This paper contained the steps into the development of a lateral system for Paseo Caribe, a 22 story multi-use high end apartment complex and parking structure located in San Juan, Puerto Rico. The building is located in a High Seismic Zone. The current building is a bearing wall system out of cast-in-place concrete. The large weight of the building, coupled with the multiple lateral irregularities resulting from changes in stiffness, mass, and the location of lateral resisting elements, and the high seismic zone requirements have resulted in an over design of the current lateral structure.

Therefore, a study was conducted to evaluate the feasibility of a new system that would allow for a reduced more efficient number of lateral elements. A good lateral system will behave in a ductile manner and its behavior should be predictable. The location of plastic hinges or failures should be dictated by the designer to minimize the impact on the structure. This paper attempts at modeling such a lateral behavior by first, implementing a frame gravity system that will reduce the weight of the building and increase the R value allowed by code for the calculations of Vase Shear from the current 4.5 value. Second, higher strength concrete is used for the lateral elements with an f'c value of 5ksi from the existing 4ksi value. To be able to limit the amount of lateral discontinuities per story and still allow for the existing use of spaces, drive paths, and corridors, thicker walls (24" from 12") are used and coupled over the corridor with diagonal reinforcement. Finally, the walls are removed from the core, where they would experience larger torsional shear force and placed further out, where they can also be use as partitions between spaces that require large noise transmission losses such as kitchen to dinning room and bathroom to bedroom areas.

A large part of this project was devoted to the placement of the walls in order to first, make them fit with the architecture and second to minimize the redundancy factor, ρ by distributing the shear in the walls efficiently. The second part of the research was devoted to designing a lateral system for a base shear that was too high for its capacity by using a concrete frame with a large self-weight. Finally, a liter system was designed and the walls were sized and detailed accordingly for flexural strength, shear capacity, boundary zone tie detailing requirements, and diagonal reinforcement in the coupled beams. A analysis based on virtual work was performed on a typical wall in order to predict the plastic hinge development. A recommendation is also detailed for the design of the shear walls based on a magnified shear demand that will ensure flexural hinging at the wall base prior to shear failure.

Finally, the design is compared to the previous design in multiple aspects ranging from the architectural advantages of the new open design, acoustical consideration between spaces, a vibrations study and a cost comparison for the design in Puerto Rico and the US.

Acknowledgements

First of all I would like to thank my father Francisco Diaz and my mother Lourdes Diaz. Without you, there would be no me. I thank you for all your support and good guidance now and always!

I would also like to thank the entire faculty of the Architectural Engineering Department at The Pennsylvania State University for all the help throughout these five years in shaping my skills as a future structural engineer. Special recognition to my consultant Thomas Boothby for his immense help on this project and for always believed in my potential. I would like to thank Andres Lapage for his guidance and positive input on the achievement of the final design.

Thanks to the owner, Geronimo, and Javier Fullana for allowing me to obtain the permit for this building and providing me with a full set of drawings. I would like to thank Brian Quinn and Frank Burke for always helping me and answering my questions. I would like to thank all those people that supported through this project in the many hours that were involved. Finally, I would like to thank my friends and Luis Agosto for their love and support.

I. Introduction

I.1 Building History and Architecture

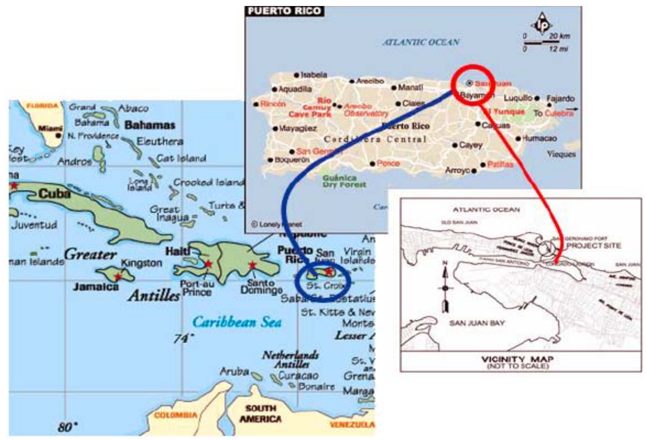
Paseo Caribe is a 240 million dollar mix–use mega project in Condado, San Juan's prime tourism sector. The project first developed when the Caribe Hilton was for sell in 1998. As part of the deal with Hilton International to buy the Hilton, the government stipulated a requirement for the development of the former federal seven-acre lot adjacent to the site. The development had to include more restaurants, retail and parking spaces, and 300 plus more hotel rooms to transform the Hilton into an important primary supporting element for the new Convention Center. The center being built was schedule to finish construction by October 2004. Together, the projects would further continue making Puerto Rico the number one tourist attraction in the Caribbean.

Paseo Caribe will consists of IV Phases to make it a world-class entertainment destination center: the Condado Lagoon Villas 88 condo-hotel villas (Phase I), the Caribe Plaza Condominium (Phase II), a multi-use Parking Lot (Phase II) , and the 185,000 sqft Entertainment Center that will host a 22,000 sqft casino, restaurants, retail and 7 big screen cinemas(Phase IV). The whole complex takes advantage of the central city location and ocean views.

This thesis project focuses on Paseo Caribe Condominium and Parking Garage, Phases II and III of IV. The parking lot is eight levels above grade with two below grade and serves 1,700 vehicles. The design is meant to alleviate traffic jams that tend to occur in the busy area of the Hilton downtown Condado. It will have five separate exit gates to allow reasonable traffic, including direct access to the Hilton and the main Avenue Ponce de Leon.

The Condominium Tower is an additional 14 story tower placed on top of the parking garage consisting of 40 luxury apartment each with 3,500 ft^2 and 6 penthouses each with 5,200 ft^2. The condominium and the garage work together to accommodate for a lobby, mezzanine, and a common area with a gym, swimming pool and garden. The building is to be in architectural harmony with Phase I. The building envelope is reinforced concrete with colored stucco as the exterior finish. Glass panels cover the majority of the exterior surface along with vertical pre cast concrete fins that serve as visual separators and block excess daylight sun from entering the apartments.

I. 2. General Information



Location: The site is located in Puerto Plata in the Municipality of San Juan, Puerto Rico. The site consisting of 215,470 ^{ft2} and was previously owned by the U.S Navy Coast Guard Parcel. It is situated between the Caribe Hilton Hotel (a national landmark) in old San Juan on the north and the entrance to the islet of San Juan on the east overlooking Condado Lagoon. It is enclosed by Ponce de Leon Avenue on the south and Luis Munoz Rivera Avenue on the west.

Building Occupancy: The project is owned by San Geronimo Development, Inc. The Paseo Caribe Condominium Tower is residential use. It has 46-luxury apartment to be sold to private parties. The 8 story parking garage below it is Open Parking and serves as private, valet, and public parking for the apartments, the neighboring Caribe Hilton Hotel, the Paseo Caribe Entertainment Center (Phase IV) and the new Convention Center.

Number of Stories: The Parking Garage has 10 stories, 2 below grade and 8 above grade. Each story height is 10' for a total of 75.1' above grade. The Condominium rises on top of the West corner of the Parking Garage and has additional 14 levels. The typical story height is 9'10" for a total height of 230' above grade.

Size: The footprint for the Condominium Tower and Parking Garage is approximately 270' by 240' giving an area of 65,240 sqft. The Condominium Tower has a total area of 284,480 ft^2 including 40 luxury apartment units with 3,500 ft^2 each and 4 penthouses with 5,200 ft^2 each. The parking garage serves 1,283 parking spaces and has a total area of 514,893 ft^2. The total area for both Condo and Parking is approximately 800,000 ft^2.

Zoning and History: The site belonged to the U.S Navy. It is in close proximity with the bay that serves as the islet to San Juan. The site has been classified as Commercial but debate on whether is should be classified as Marine Terrain has aided in the three year delay of the project. Setbacks are only implemented on the north boundary at 16.5'of the property line.

I. 3. Project Team

Owner & Developer: San Gerónimo Development Corp.
Arturo Madera Arboleda
Calle Bolivia 54 Suite 203
Hato Rey, Puerto Rico 00918

Contractor: F & R Contractors, Inc.
Jaime Sullana
P.O. Box 9932
San Juan, Puerto Rico, 00908-9932
Tel. (787)753 – 7010

Architects: Beame Architectural Partnership – Parking Garage
116 Alhambra Circle – Suite J
Coral Gables, Florida 33134
Tel. 305.444.7100
Website: http://www.bapdesign.com/portfolio/mixed-use.htm

Sierra, Cardona & Ferrer – Caribe Condominium
13 Street 2
Metro Office Park
Guaynabo, Puerto Rico 00968 – 1712
Tel. 787.781.9090
Website: http://www.scf-pr.com/

II .Existing Structural System

II.1. Codes and Requirements

The following is a summary of the applicable codes and requirements for the different components of the structure. Code requirements have been adopted by the city are still applicable to any changes in design.

Applicable Codes

Loads (includes wind):	ANSI/ASCE 7-95
Seismic:	UBC 1997
Reinforced Concrete:	ACI 318-95
Puerto Rico's current adopted code of practice:	UBC 1997
Post-Tensioned Concrete two way slab system:	ACI-ASE 423
Steel:	AISC
Welding:	AWS

Load Combinations

1. 1.4D + 1.7L
2. 0.75(1.4D + 1.7L + 1.7W)
3. 0.9D + 1.3W
4. 1.1(1.2D + f_1L + f_2S + 1.1E), f1 = 0.5 for live loads < 100psf, S = 0
5. 1.1(0.9D + 1.0E)

Minimum Required Reinforcement

Reinforced Concrete Walls	
- 6" Thick	#4@12 E.W
- 8"	#4@10 E.W
- 10"	#4@8 E.W
- 12"	#4@12 E.W
Masonry Walls (Vertical Reinforcement)	
- 6" Thick	#3@16" or #4 @ 32"
- 8"	#5@32" or #6@48"

Steel Cover Requirements

Footings	
- Side	3"
- Bottom	2"
Slab on Grade/Mat Foundation	1"
Wall	
- Pour	3"

- Exposed, up to #5	1-1/2”
- Exposed, #6 or larger	2”
- Not Exposed, up to #11	¾”
Slab/Joist	
- Up to #11	¾”
- #14 or larger	1-1/2”
Beams/Columns	1-1/2”

Post-Tensioning

Concrete	
- Compressive strength at transfer	3,000psi
Steel	
- Yield strength	270,000psi
- Effective stress after losses	171,000psi
- Preliminary long term losses	15,000psi

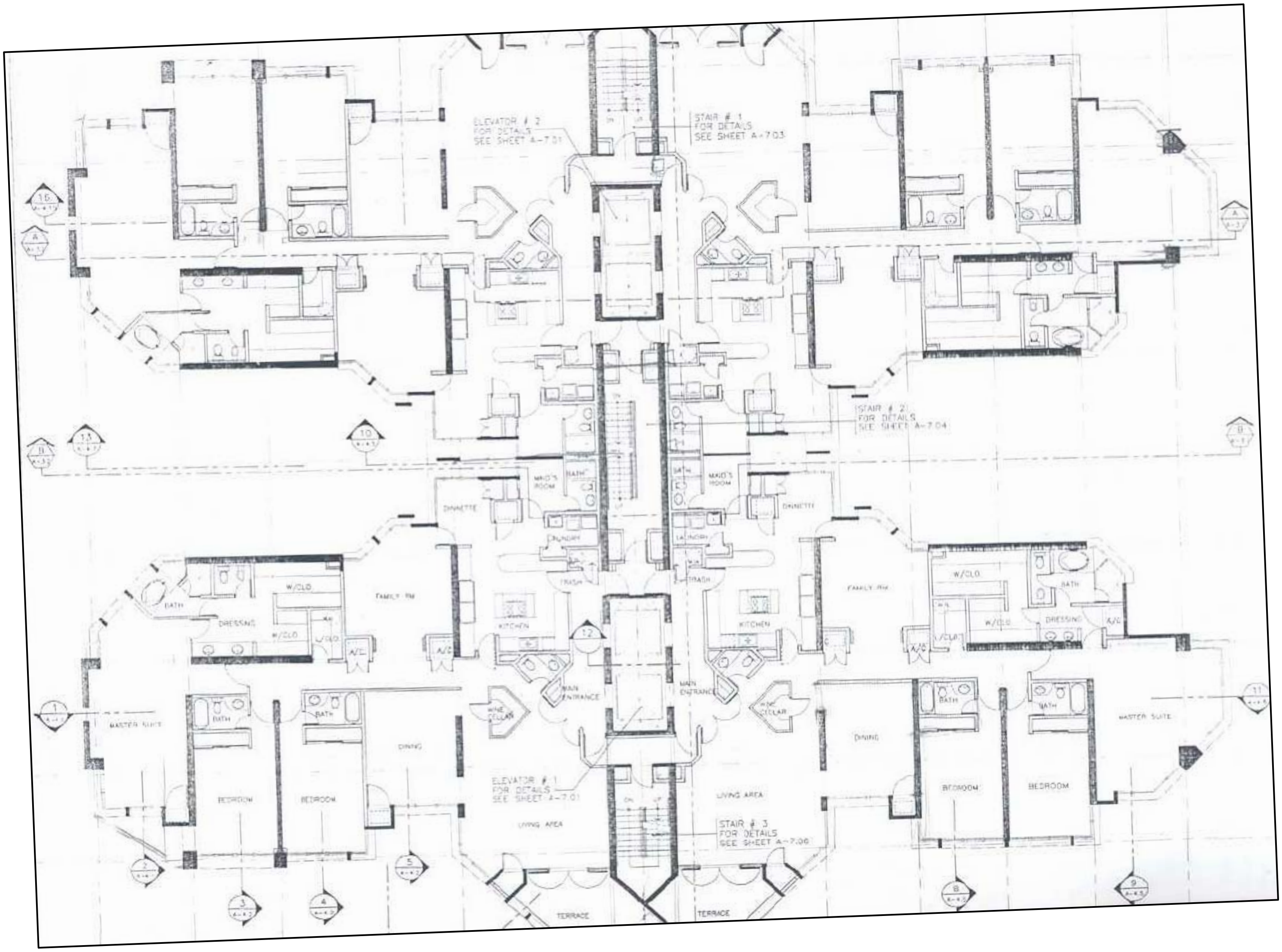
Strength Requirements

Concrete (28 day strength)	
- Structural Slabs:	4,500psi
- Beams:	4,500psi
- Columns:	5,000psi
- Walls:	4,000psi

II. 2. Gravity System

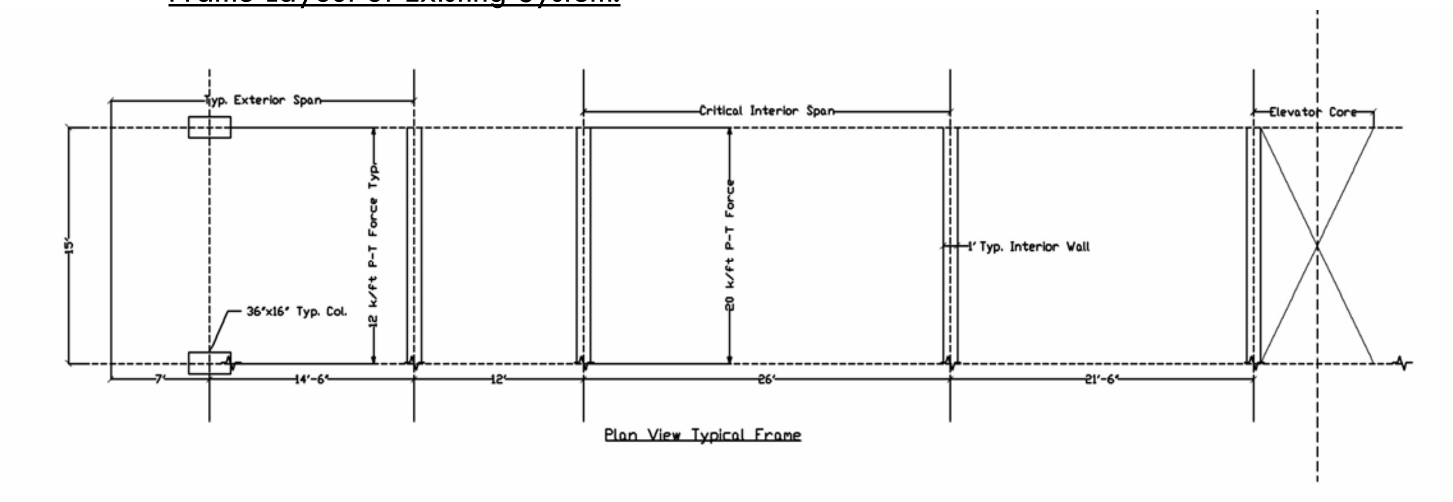
II. 2.i. Introduction

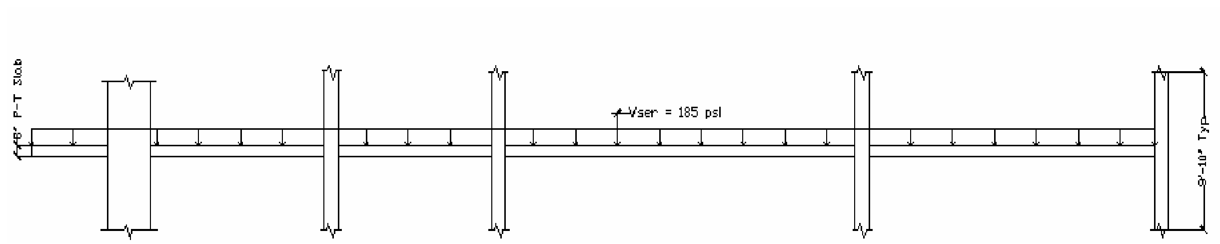
The structure of the Condominium and Parking Garage is reinforced cast in place concrete. There are four apartments per floor, two at each side of a 10’ wide core that contain the four elevator units and 3 sets of stairs. Each apartment is approximately a square with dimensions of 80’ east to west and 60’ north to south. Since the building is symmetrical about both axes the analysis of the structural floor system is based on a typical apartment span frame.



The current floor system consists of a one way cast in place post tensioned 8" concrete slab on each floor. The floor slab in supported in the interior bays by 12" wide interior shear walls spanning north to south and by 16 columns around the perimeter. There are 2 columns and 4 shear walls per apartment. The slab spans east to west between shear wall supports. The typical column size is 16" x 36". The shear walls run parallel to each other. The largest interior span in between shear walls is 26'; other interior spans are 22' and 14'. The largest exterior span between column and shear wall is 14.5'.

Frame Layout of Existing System:





Drawing specifications shows that the slab is designed for a post tensioned effective compressive stress of 12k/ft in both directions. This design value is increased to 20 k/ft at the location of the largest 26' span. Post-tensioning tendons for this slab are 7 wire. There is post-tensioning of the concrete on both directions, N-S and E-W. However, the primary action of this one way slab is from East to West, which coincides with the short direction between shear wall supports. There is also regular reinforcement in this directions further suggesting the one way action of the slab. In the transverse N-S direction, the tendons are located directly over the shear walls and are used for deflection and crack control.

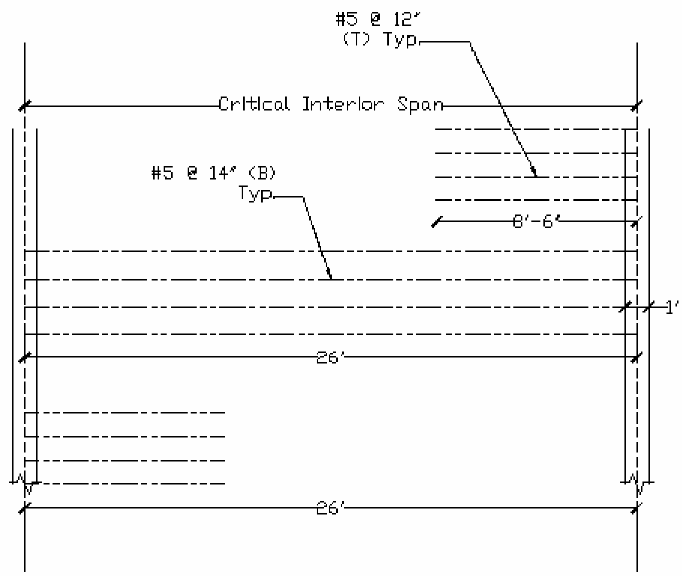
The slab is reinforced in the east to west direction with regular reinforcing bars. The typical bottom reinforcement is #5 bars. Typically:

- Spans < 15' #5@18"
- 15'-22' Spans #5@16"
- Spans > 22' #5@14"
- Middle core #5@12"
- North-South core perimeter #5@10"

Top positive reinforcement is provided over the shear wall supports. Reinforcement extends 1/3 times the span on each side of the span from the centerline of the support.

For the largest span, L_{max} = 26', the typical layout of the reinforcements is: negative reinforcement extends 8.5' from the centerline of the shear wall support. Typical reinforcement is #5 bars. For spans < 17', use #5@18". Larger spans use #5 @ 12.

Critical Reinforcement Layout:



Existing Typical Reinforcement Layout

II.2.ii. Loads

The resultant service dead loads and live loads on each member are obtained following UBC 1997 code references. Live and dead loads used are listed below. There were live loads reductions allowed for members carrying more than 150 ft^2. The reduction factor for members carrying only one floor is to be limited at 40% while the members carrying more floor loads can de reduced up to 60%. However, there is a note included that does not allow the reduction factor for parking garages to exceed 40% and lobbies and public spaces with live loads greater than 100 psf are not to be reduced at all. As a result live loads were reduced by 60% down to the 9th level (first apartment floor), below of which lays the parking garage, reduced by 40% with the exception of level 8, 7 and 1 that are common areas for the condominium, this were not reduced at all

Live Loads

Roof	40psf
Floor	40psf
Stairs	100psf

Corridors	100psf
Terrace	60psf
Parking	50psf
Storage	125psf
Pool Deck	100psf

<u>Dead Loads</u>

Slab – 8" thick	100psf
Non – Bearing Concrete Block Walls	20 psf
Superimposed MEP	25 psf
Shear walls - 9' 2" High (per longitudinal area of wall)	1375 psf

II. 2.iii. Design Check

An analysis of this system was performed by hand. The calculations are based on one foot strip. Calculation includes:

Three permissible stress checks:

1. Stresses at transfer due to self weight
   - Extreme fiber compression: fc < 0.6fci'
   - Extreme fiber in tension: ft < 6√fci'
2. Stresses at service unfactored loads
   - Sustained loads (Dead loads only)
     - Extreme fiber compression: fc<0.45fc'
     - Extreme fiber tension for Class U – assumes un-cracked under full service loads: ft<0.75fc'
   - Total Loads (Dead loads and live loads)
3. Flexural Strength check
   - Extreme fiber compression: fc<0.6fc'
   - Extreme fiber tension < 0.75fc'

A summary is provided here, detailed calculations can be found in Appendix B

<u>1. Permissible Stresses at Transfer</u>

D_p = 6.75"
L_{max} = 26'
S= 12*8^2/6 = 128 in^3
P_o= 12 k/ft

$A = 12*8 = 96in^2$
$e = 3"$
$f_{ci}' = 2500$ psi
Assume 5% initial losses
Initial Stress:

$M_d = 25\text{^}2(100)/11 = 6.15'\text{-k}$
$M_d/S = 576$ psi tension top
-576 psi compression bottom

Prestress Effect: $P_o/A \pm P_o(e)/S$
= -406.25 top compression
156.25 bottom tension

Net Stresses at transfer:

Top: 576 – 406.25 = **169.75 psi < 6√fci' = 300psi** Good
Bottom: -576 + 156.25 = **-419,75 < 0.6*f'ci = -1500psi** Good

2. Service Stress Check Summary

fc = 4500 psi	**Exterior Span**	**1st Int. Span**	**2nd Int. Span**	**3rd Int. Span**
Length	14.500	12.000	26.000	21.500
P (kip/ft)	12.000	12.000	20.000	12.000
A (in2)	96.000	96.000	96.000	96.000
S(in3)	128.000	128.000	128.000	128.000
P/A (psi)	125.000	125.000	208.333	125.000
e(in)	3.000	3.000	3.000	3.000
P(e)/S	281.250	281.250	468.750	281.250
Sustained Check	fc-allow (psi)	-2025.000	ft-allow (psi)	402.492
Wsus (psf)	125.000	125.000	125.000	125.000
Msus ('k)	2.628	1.636	7.682	5.253
Msus/S	246.387	153.409	720.170	492.454
fc-actual (psi)	**-90.137**	**2.841**	**-459.754**	**-336.204**
ft(psi)	**-159.863**	**-252.841**	**43.087**	**86.204**
Service Check	fc-allow (psi)	-2700	ft-allow (psi)	402.492
Wser (psf)	185.000	185.000	185.000	185.000
Mser ('k)	3.890	2.422	11.369	7.774
Mser/S	364.652	227.045	1065.852	728.832
fc-actual (psi)	**-208.402**	**-70.795**	**-805.436**	**-572.582**
ft(psi)	**-41.598**	**-179.205**	**388.769**	**322.582**

- compression + tension

3. Flexural Strength – Factor Loads

Without Rebar

According to UBC 97 and given live and dead loads:

$W_u = 1.4(W_{dl}) + 1.7(W_l l)$

W_{dl}= 150pcf*(8/12) + 25psf superimposed

W_{ll}= 40psf typ floor + 20psf partitions

W_u = 277 psf

Capacity for unbonded tendons

$f_{su} = f_{se} + 1.0f'_c/100p + 10ksi$

$p = A_{ps}/b_{dp} = (12/24.8)(.153)/(12*6.75") = .000914$

f_{se} = 171 ksi

f_{su} = 230 ksi

F_{ult} = (230/171)*12 = 16.14 k/ft

M_u = 0.9(16.14 k/ft)*(6.57"/12"/ft) = **8.1'k < 11.5 'k** → Rebar is needed

Strength Calculations including Rebar

A_s provided at L_{max} = #5 @ 14" = 0.265 in^2/ft

$F_{u\text{-}reb}$ = 0.265 * 60 = 15.94 k

a = (15.94 + 16.14)/(3.83*12) = 0.7"

$j_{d\text{-}p}$ = 8" -0.35"-1.25" = 6.4"

$j_{d\text{-}r}$ = 8" – 0.35" – 1.0" = 6.65"

M_u = (.9)(16.14'k(6.4"/12) + 15.94'k(6.65"/12)) = 15.84'k

By limit design: $W_u(l^{\wedge 2})/8$ = 15.84 ft-k + 8.1 ft-k = 23.94 ft-k

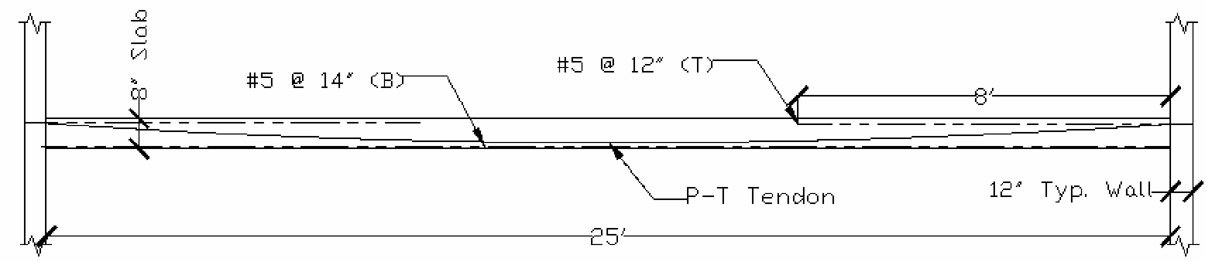
For L_{max} = 26' → Wu = 8(23.94 ft-k)/26$^{\wedge 2}$ = **283 psf > 277 psf**
Good

Check minimum reinforcement:

$A_{s,min}$ = 0.0015 * 8 * 12 = **0.144 in2/ft < 5 @ 18" = 0.2 in^2/ft** **Good**

Reinforcement was found adequate. The regular reinforcement was found necessary for strength requirements.

Reinforcement Layout Cross Section:



II.3. Lateral System

II. 3.i Introduction

Lateral forces due to wind and seismic on the building are designed to be sustained by shear walls in both north-south and east-west directions. The walls act as a cantilever, resisting the applied lateral loads at each level through deflection. In the north-south direction there are a total of 28 walls. In this direction the shear walls are 12" thick and they cover a total distance of approximately 629 linear feet per floor. In the east-west direction, there are 8 resisting lateral walls, also 12" thick. They are located in the center of the building cover approximately 145 linear feet.

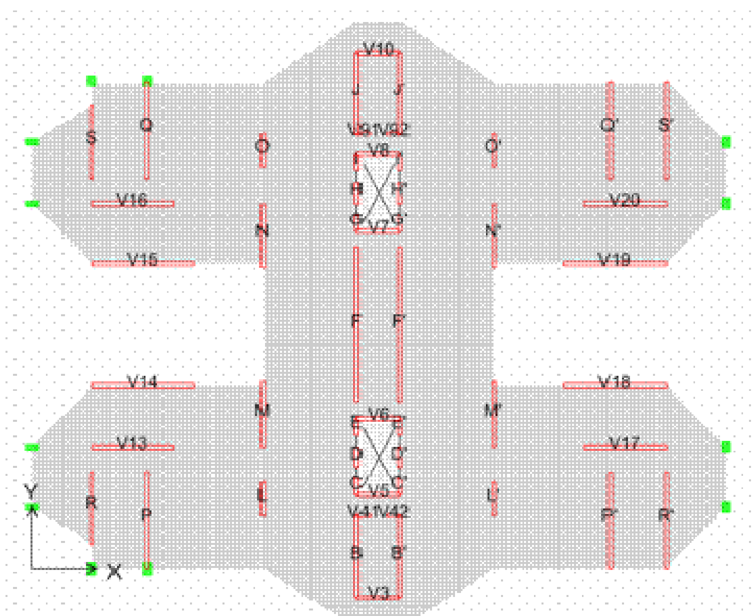


Figure 1: Typical Apartment Floor shear and bearing wall system and model labeling

All shear walls extend from the foundation and parking garage directly through the apartment building. There are some slight changes and modifications that were done to add stiffness while accommodating for the apartment's layout. Brief mentions of this for a typical apartment floor are (Refer to Figure 1for labeling).

- The 2 stair enclosures that extended through the 8 levels of parking lots and form part of the core are shifted at the lobby level 30' each inward toward the center of the building. A 3rd set of stairs was added along the core line and covers the space in-between the two elevator shafts. These changes allowed for better use of the middle core space and increased stiffness at the core.
- Shear walls L, O are extended 8' south over the original wall.
- Shear walls M, N are extended 13' south over the original wall.
- Shear wall V14-V18 extended 8' inward over original wall.

II. 3.ii Wind Loads

Preliminary calculations were performed using a spreadsheet for wind lateral and shear forces on Paseo Caribe following ANSI/ASCE 7-95 per drawing recommendations. Located in the Caribbean Sea and in a very hurricane prone region with five Category IV Hurricanes (wind speeds > 125 mph) directly hitting the island in the last 25 years and personally experiencing a couple of them, I was very concerned about lateral wind forces in my design. Paseo Caribe is not a typical square building. It has plenty of discontinuities in its "flower" shape arrangement. For my preliminary calculations I decided to conservatively make the building a square box with boundaries representing the largest dimensions of the building, 190' x 162'. This is conservative because the width represented by this dimensions (190') only occurs in about 20% the length of the building. The rest is much narrower, about 60' to 140' wide. The parameters used for the analysis were provided by the structural drawings:

Basic Wind Velocity	**100mph**
Building Classification	**II**
Importance Factor	**1.05**
Pressure Coefficient-Method 2	**1.4**

Table 1: Wind Load Story Force Calculations

Wind Loads - ASCE 7-95

				N-S	E-W
V	110	mph			
kd	0.85		Cp Windward	0.8	0.8
Importance I	1.05		Cp Leeward	-0.5	-0.4
Exposure Category	D		Gust, G	0.866	0.869
Surface Roughness	D		Dimensions (ft)	120	162
Kzt	1		Shear Wall Acting/Floor (ft)	600	250
GCpi	0.18		L of Shear Wall (ft)	23	23
Number of Stories, r	22				

					Resultant Pressure (psi)		Story Forces (K)	
Story Level	z (ft)	Kz	qz	qh	N-S	E-W	N-S	E-W
Roof	222.62	1.65	45.62	45.62	51.35	47.57	41	28
21	212.79	1.63	45.06	45.62	50.97	47.18	81	56
20	202.96	1.61	44.51	45.62	50.59	46.80	81	55
19	193.13	1.61	44.51	45.62	50.59	46.80	81	55
18	183.30	1.59	43.96	45.62	50.21	46.42	80	55
17	173.47	1.57	43.40	45.62	49.82	46.03	79	54
16	163.64	1.56	43.13	45.62	49.63	45.84	79	54
15	153.81	1.54	42.57	45.62	49.25	45.45	78	54
14	143.98	1.53	42.30	45.62	49.06	45.26	78	53
13	134.15	1.51	41.75	45.62	48.67	44.88	78	53
12	124.32	1.49	41.19	45.62	48.29	44.49	77	52
11	114.49	1.46	40.36	45.62	47.72	43.92	76	52
10	104.66	1.44	39.81	45.62	47.33	43.53	75	51
9	94.83	1.42	39.26	45.62	46.95	43.15	75	51
8	85.00	1.4	38.70	45.62	46.57	42.76	94	64
7	70.00	1.35	37.32	45.62	45.61	41.80	92	63
6	60.00	1.32	36.49	45.62	45.03	41.23	73	49
5	50.00	1.28	35.39	45.62	44.27	40.46	72	49
4	40.00	1.23	34.00	45.62	43.31	39.50	70	47
3	30.00	1.17	32.35	45.62	42.16	38.34	68	46
2	20.00	1.09	30.13	45.62	40.63	36.81	66	44
1	10.00	1.03	28.48	45.62	39.48	35.65	64	43
0	0.00	1.03	28.48	45.62	39.48	35.65	32	21

II. 3.iii Seismic Loads

Seismic forces were calculated based on UBC 1997 provisions. The building and soil classification parameters obtained from the structural drawing specify:

Seismic Zone 3, Z = 0.3

Seismic Type B

Soil Profile Sc

Period T = 1.35 (Method A)

R = 4.5 Bearing/ Shear Walls System

An important parameter in the determination of the seismic force acting on the building is the self weight. Therefore, it is important to make a good approximation on this value. The total dead weight of the building was calculated to be 95132 kips:

Table 2: Dead Weight Calculation for Existing All Concrete Structure

Floor Description	# Stories	Floor Area (ft^2)	Story Height (ft)	10" Wall (lf)	12" Wall (lf)	Column Area (ft^2)	Slab Load	Wall/Col Load (k)	Total Load(k)
Penthouse	2	10200	9.83	168	217.5	30	2040.00	1061.64	3101.64
Typical Apartments	12	15870	9.83	336	435	60	19044.00	12739.68	31783.68
Common Area	1	63084	15	404	502	199	6308.40	1924.31	8232.71
Parking Garage	7	63084	10	384	392	434	44158.80	7855.75	52014.55
Total Weight							71551.20	23581.38	**95132.58**

Table 3: Seismic Design Parameters and Loads

Calculated Parameters - UBC 1997	
W	95132.58
Cv	0.54
Ca	0.36
R	4.50
T	1.35
I	1.00
V = 2.5CaIW/R	19026.52
V = CvIW/RT	8456.23
V = 0.11CaIW	3767.25

Earthquake Design Loads - UBC 1997							
	V =	8456.23 kips			Ft = 0.7TV	799.11 kips	
	Level	**Story Weight, wx (k)**	**Height, hx(ft)**	**wxhx**	**Lateral Force, Fx* (k)**	**Story Shear, Vx (k)**	**Moments (FT-K)**
PENT4	**22**	1550.82	222.62	345244	1122	**1122**	0
PENT3	**21**	1550.82	212.79	329999	309	**1431**	11029
PENT2	**20**	2648.64	202.96	537568	503	**1933**	25092
PENT1	**19**	2648.64	193.13	511532	478	**2412**	44096
10TH	**18**	2648.64	183.30	485496	454	**2866**	67802
9TH	**17**	2648.64	173.47	459460	430	**3295**	95972
8TH	**16**	2648.64	163.64	433423	405	**3701**	128365
7TH	**15**	2648.64	153.81	407387	381	**4082**	164742
6TH	**14**	2648.64	143.98	381351	357	**4438**	204865
5TH	**13**	2648.64	134.15	355315	332	**4771**	248493
4TH	**12**	2648.64	124.32	329279	308	**5078**	295387
3RD	**11**	2648.64	114.49	303243	284	**5362**	345308
2ND	**10**	2648.64	104.66	277207	259	**5621**	398017
1ST	**9**	2648.64	94.83	251171	235	**5856**	453274
P8	**8**	8232.71	85.00	699781	654	**6511**	510840
P7	**7**	7430.65	70.00	520146	486	**6997**	608498
P6	**6**	7430.65	60.00	445839	417	**7414**	678468
P5	**5**	7430.65	50.00	371533	347	**7761**	752607
P4	**4**	7430.65	40.00	297226	278	**8039**	830220
P3	**3**	7430.65	30.00	222920	208	**8248**	910613
P2	**2**	7430.65	20.00	148613	139	**8387**	993091
LOBBY	**1**	7430.65	10.00	74307	69	**8456**	1076958
	VALUES	**95132.58**		**8188036**		**8456**	**1076958**

By comparing these results with the wind forces, it is clear that seismic forces control de lateral system design. The primary concern of high wind forces being inappropriately modeled as too conservative because of the larger area used was not a concern once the seismic results were obtained. The maximum story shears due to seismic is about 5 times larger than that due to wind

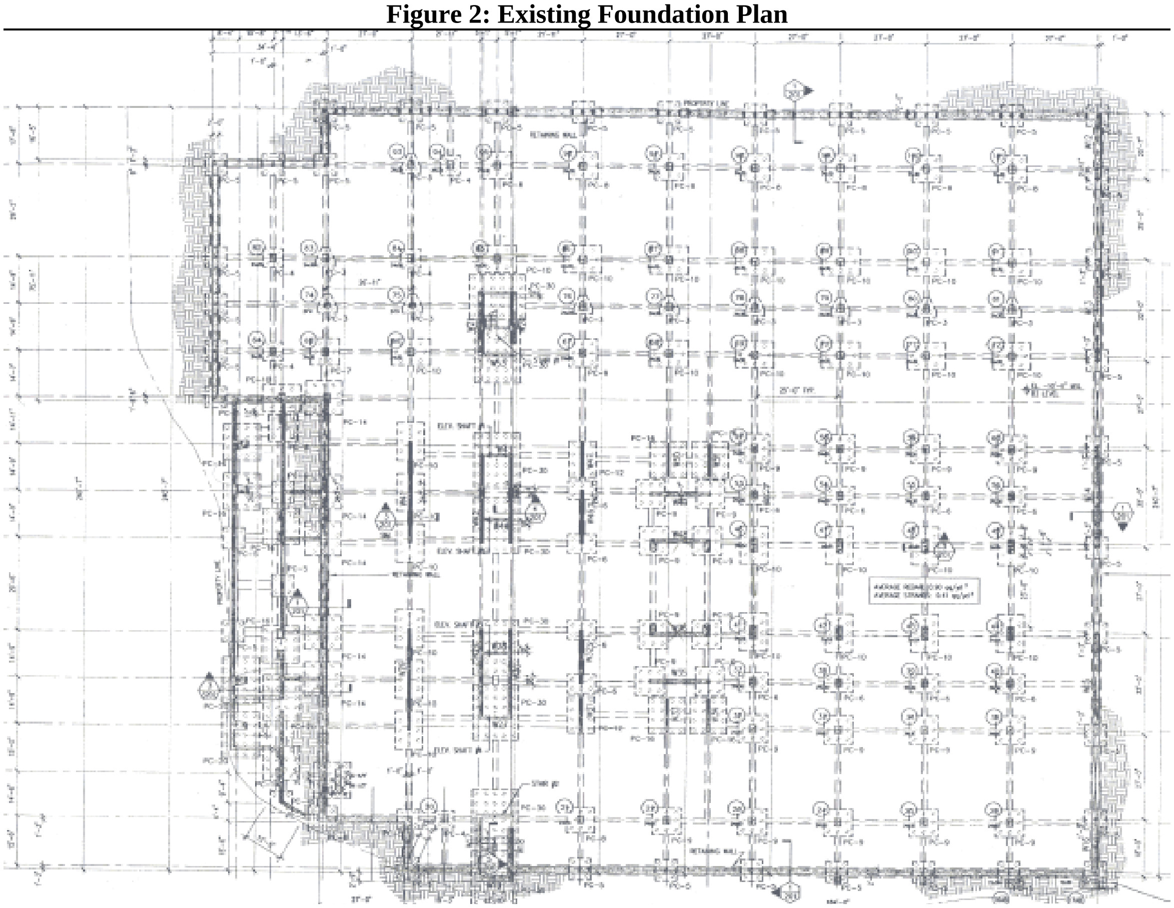
Seismic: 8456 kips
Wind: 1678 kips

This can be explained by many factors including location close to a fault line, bad soil characteristics, and a very large building weight!

II. 4 Foundation

The parking garage was designed first with the notion that a condominium was to be built a later time on top of it. This is evident in the layout of the foundation system. The foundation design consists of 40 to 50 inch deep pile caps. The typical pile cap consists of 10 piles placed 3' c/c. The layout of the foundation system is in a grid following that explained for the column layout of the parking garage. Typical spacing is 15' c/c north-south and 27' c/c east-west. The building is enclosed below grade by a 2' wide L retaining slurry wall around the perimeter that goes to a maximum depth of 22' with a 2' hydrostatic slab on grade. The location of the tower is evident by replacement of columns with shear walls in the west half of the parking garage foundation layout. This foundation shear walls extend from one pile cap mat to the next. As a result of this increased load that the shear walls will be experiencing, the pile cap sizes are increased from 10 piles/ pile cap to 30 piles/ pile cap side of the building were the elevators, stairs and the tower rises there is an increased mat size to 30 piles per cap with 50" deep caps.

Figure 2: Existing Foundation Plan

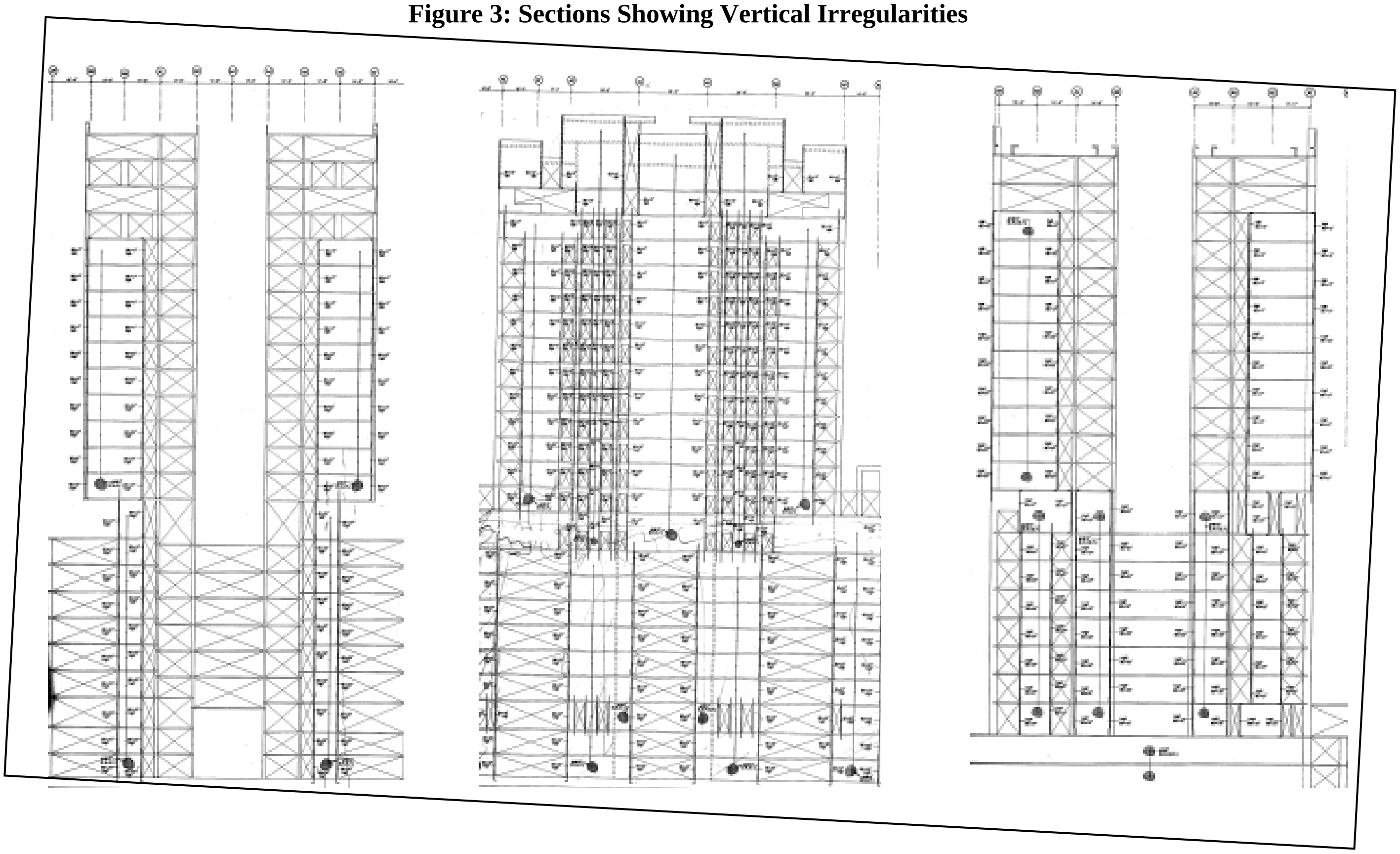


III. Proposal

III.1. Problem Statement

An aspect of the structure of Paseo Caribe that can not be over looked is the large number of irregularities that form the structure. It is a very complex building. First, plan irregularities are present with the large change in plan from the parking garage (240'x270') to the apartment floors (180'x162). There is a reduction in area of almost half. However of most concern to a structural engineer are the vertical irregularities of the lateral system. The current system consists of 36 walls, yet not one those walls are continuously or uninterrupted through the structure. In order to optimize the available space in the apartment floors, many of the walls from the parking structure had to be shifted at the apartment level. A number of examples are shown below. All the irregularities occur at the 8th level where the transition of occupancies occurs.

Figure 3: Sections Showing Vertical Irregularities



For example, let look at the core which is the main lateral resisting component because of its large stiffness compared to the other walls. At the 8th level the two outer sets of walls that comprise the stairs in the parking garage are removed and three set of interior walls

are introduced. This is necessary to allow for the location of vehicle circulation in the parking and reduce the amount of non-rentable square feet area in the apartments. In order to transfer the large seismic forces, a transfer girder had to be design at this level. The transfer girders are made of heavily reinforced concrete and span the whole length of the building, 180 ft, and have a depth of 15ft! There are two of these members at this floor level. This building is in a high seismic zone, Zone 3, and the lateral system is a very important part of its design. The lateral system is further hurt by the large weight of the building. Every 9" slab and 36 - 12" wall is cast in place reinforced concrete. The total weight of the building is up to 90,000 kips!

There are many disadvantages to this system:

1. Large weight associated with the excessive number of walls used for design
2. Small R value of 4.5 allowed by Code for a wall bearing gravity system in the determination of the Vase Shear.
3. There is no clear predictable failure mechanism to this system
4. There are many cost associated with the amount of concrete and reinforcement in the detailing of the walls at the transition levels and boundary zones near openings.
5. Longer construction time associated with the forming and placing of different wall section, their heavy reinforcement requirements, boundary zone detailing for each of the 36 walls and the two 15ft transfer girders at the 8th level.

III. 2. Proposed Solution

The goal of this research is to develop a design for the lateral system that will provide a ductile behavior and clean failure mechanism by using a considerable less number of walls. It is also a goal of the designer to allow for a more open space in the architecture of the building by substituting the unnecessary shear walls with a frame system for the gravity loads.

The design will attempt to reduce the number of shear walls by half to 4 shear walls in each direction. The strength of the concrete is 5 ksi and the walls will be no more than 24" thick. The walls shall not alter the architecture of the building in any way. Spaces allocated as living areas in the apartments and drive paths in the parking garage can not be interrupted. Also, the lateral system is to provide a continuous pathway to the foundation with no major irregularities that could qualify and as any of the irregularities listed in Table 16-L of the UBC.

IV. Structural Redesign

IV.1. Gravity System: Steel Frame

If the goal of the new lateral system is achieve, the fewer number of shear walls needed to resist the seismic forces will allow for a more open use of space in the apartment units. Therefore, a liter frame structure that follows closely to the already existing grid structure of the parking garage below the apartment units can be designed to support gravity loads. *Such a design is beneficial over the current bearing wall system because it allows using a reduced R factor of 5.5 instead of 4.5 specified by code.*

An attempt was made to use a concrete column frame with a flat plate. The process for design was followed and can be found in APPENDIX A. The results were discarded because the 27' x 30' bay on columns required substantial capitals around columns or transverse beams for punching shear. This design lead to an increase in the current weight of the structure and the proposed reduced coupled wall design was not attained efficiently as the maximum allowed shear capacity of the walls and coupling beams were exceeded. Various solutions included increasing the thickness of the walls to 30", increase the number of walls, or reduce the weight of the structure. I decided to explore what the advantages of a steel structure, which is littler in weight and faster to erect, would have on my lateral system and perhaps allow me to make the lateral system more efficient.

IV.1.i Methodology

The layout of the frame system in the apartment units had to achieve certain goals keeping in mind that the main objective of the new design is the efficiency of the lateral system:

1. <u>Limited floor-floor height:</u> By using a 9" P/T slab, the current system allowed for a floor-ceiling height of 9'-10". This is not a conceivable depth for a steel building. The height restriction imposed by UBC for a concrete shear wall system is 240'. Therefore, a larger floor-floor height of up to 10'-10" is a allowed for design. However, the offset is that the taller the building, the higher the shear walls need to be resulting in an increase of both the shear and moment forces at the base. As a result the design was limited to W10's within the apartment units and W14's around the perimeter and the communal corridors. By using a 2.5" slab on a 1.5" 20 gage deck, a floor-ceiling depth of 15" was achieved in the apartments and 20" in the corridor and around the perimeter. The beams around the perimeter will be encased and become part of the architecture.

2. Allocation of Dead Weight: From previous analysis of the structure, it was understood that it is in the advantage of the engineer to allocate as much of the floor weight into the shear walls to help increase the flexural capacity of the reinforced wall and counter act the amount of tension reinforcement need for the large overturning moments. For this reason, the beams are selected to span East-West bearing on the N-S walls.

3. Transfer Girder: One main advantage of the grid system is that it is already laid out in the parking structure below. An attempt was made to maintain this grid for the columns in the apartment units. However, a column was need for support in the bedroom end corners. This location lies directly above the vehicular circulation path in the parking garage. Therefore, these two columns could not be extended below the apartment units at the 9^{th} floor. The solution was to take advantage of the 15' story depth on this level because of the location on the west side of the apartments common area housing the pool and fitness area, and run transfer girders from this column to two adjacent columns in the lower floors.

The design was tested using the software program RAM for a composite steel deck with the parameters outlined before. Both the column supported by the transfer girder and the transfer girder were checked. Finally, the base plate and stiffeners designed for the column-girder connection.

IV. 1. ii. Results

Beams

The typical apartment unit is depicted below showing the beam sizes and shear studs needed to support a super imposed dead load of 58 psf that accounts for a 2.5" slab on 1.5" Lok deck and MEP loads. The spacing is 7'6" and a 20 gage deck was used to allow for un-shored construction. A live load of 60 psf is specified which includes 20 psf of partitions. The design also accounts for perimeter load of 30psf.

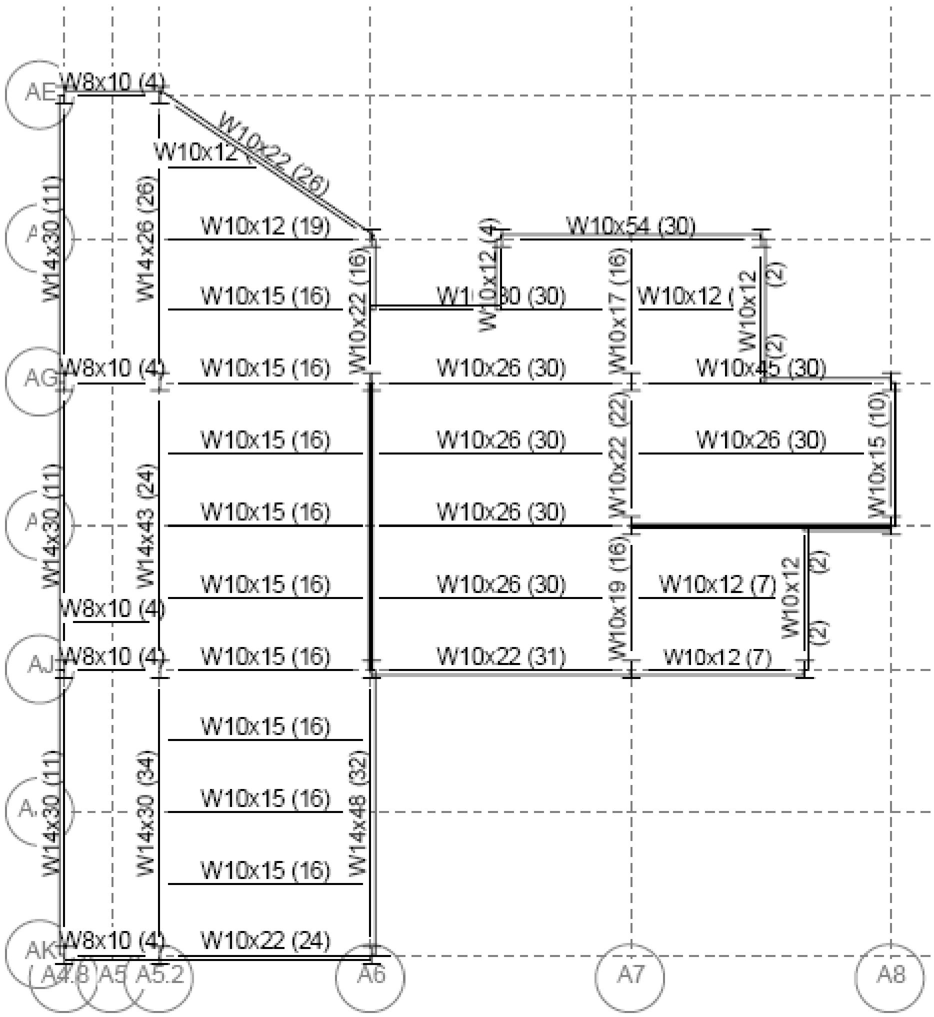


Figure 4: Typical Beam Layout for a Apartment Unit

The largest beam within the apartment space is W10x26. This size allows for a 10’4” story height while maintaining a 9’ clearance. In the corridor, the W14’s can be hidden inside of the plenum and the clear height will be 8’-6”.

Floor Type: Parking Structure

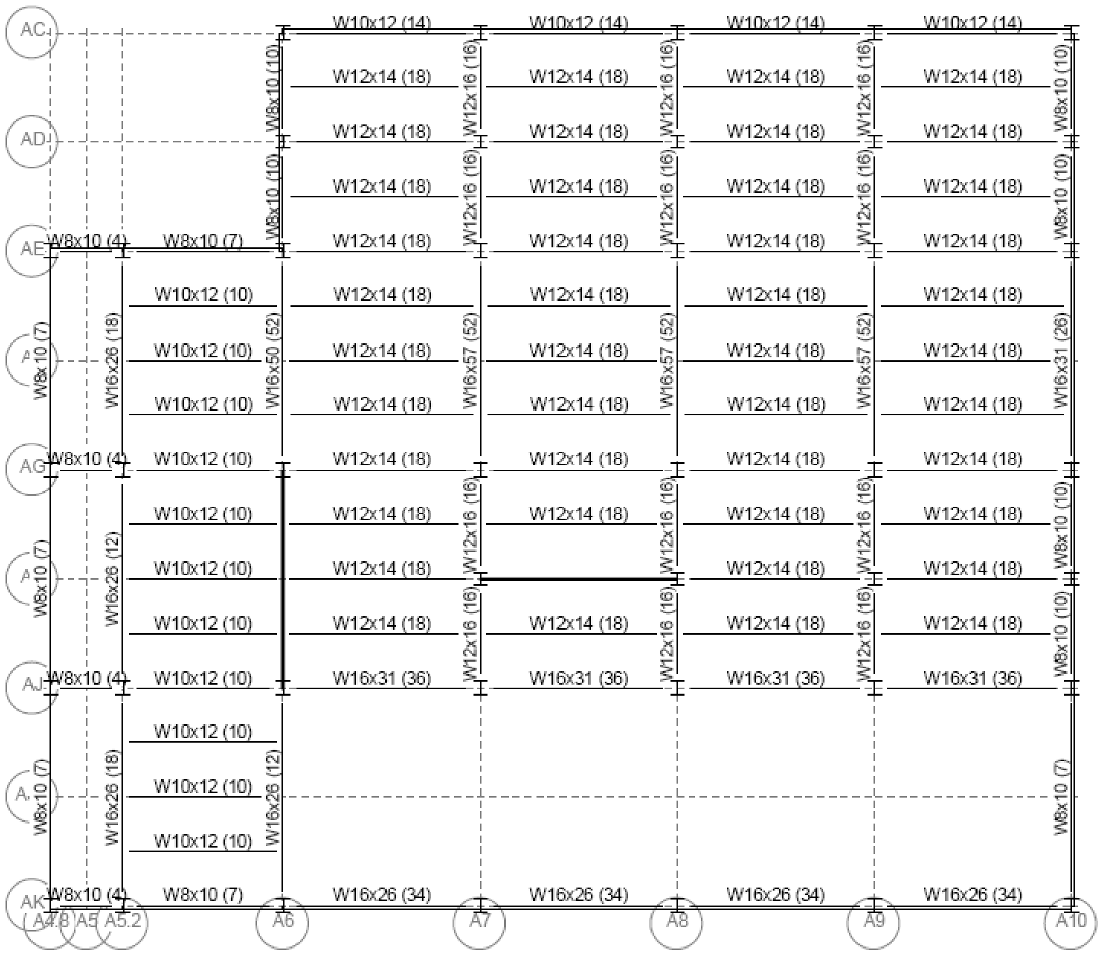


Figure 5: Typical Beam Layout for the Parking Garage

The parking garage is design to support 50 psf live load, irreducible. It has a super imposed dead load of 75 psf to account for the 6" slab. The deepest beam size is a W16. For the existing 10' story height, this beam size will maintain the clear height requirements of 8' to allow for van passage.

Floor Type: Parking Structure 9

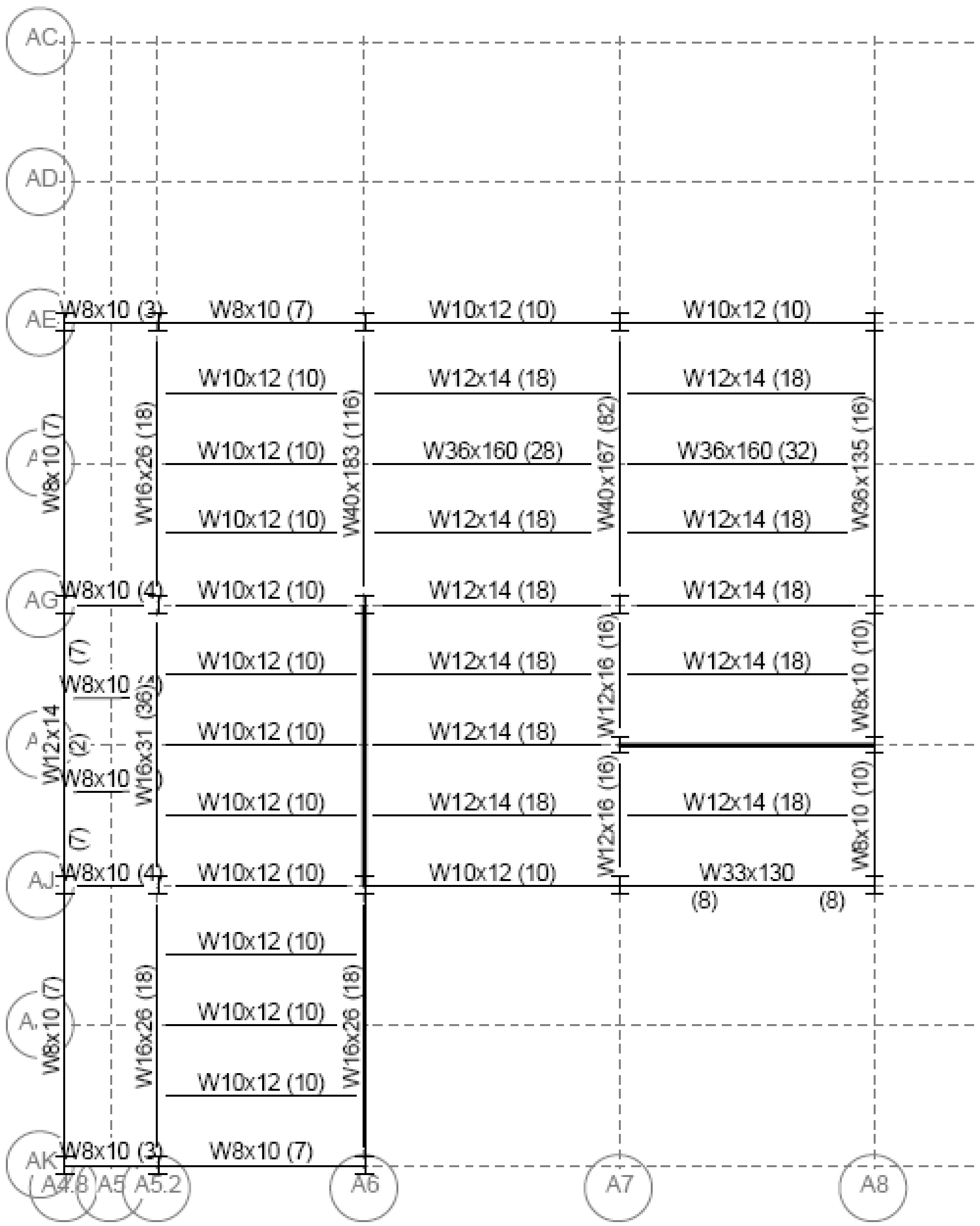


Figure 6: Transition Level: Framing Layout at 9th Story

At the 9th story level, where the transition from the apartment levels to the parking garage occurs, special considerations had to be taken. At this level, two columns located in grid point (AF-A6) and (AF-A7) that were necessary to support the apartment structure had to be removed in order to allow for the circulation of vehicles in the parking.

The solution was to provide a transfer girder at this level that will transfer the load to the two adjacent columns across from the drive path. An advantage is that this story is already designed to be 15' high because it houses the common area for the apartments. On the left side of the elevator core (Grid A5) are the pool and fitness center. This increased in height allowed for an ease in placing the two W40 x 183 transfer girders on the parking area.

Columns

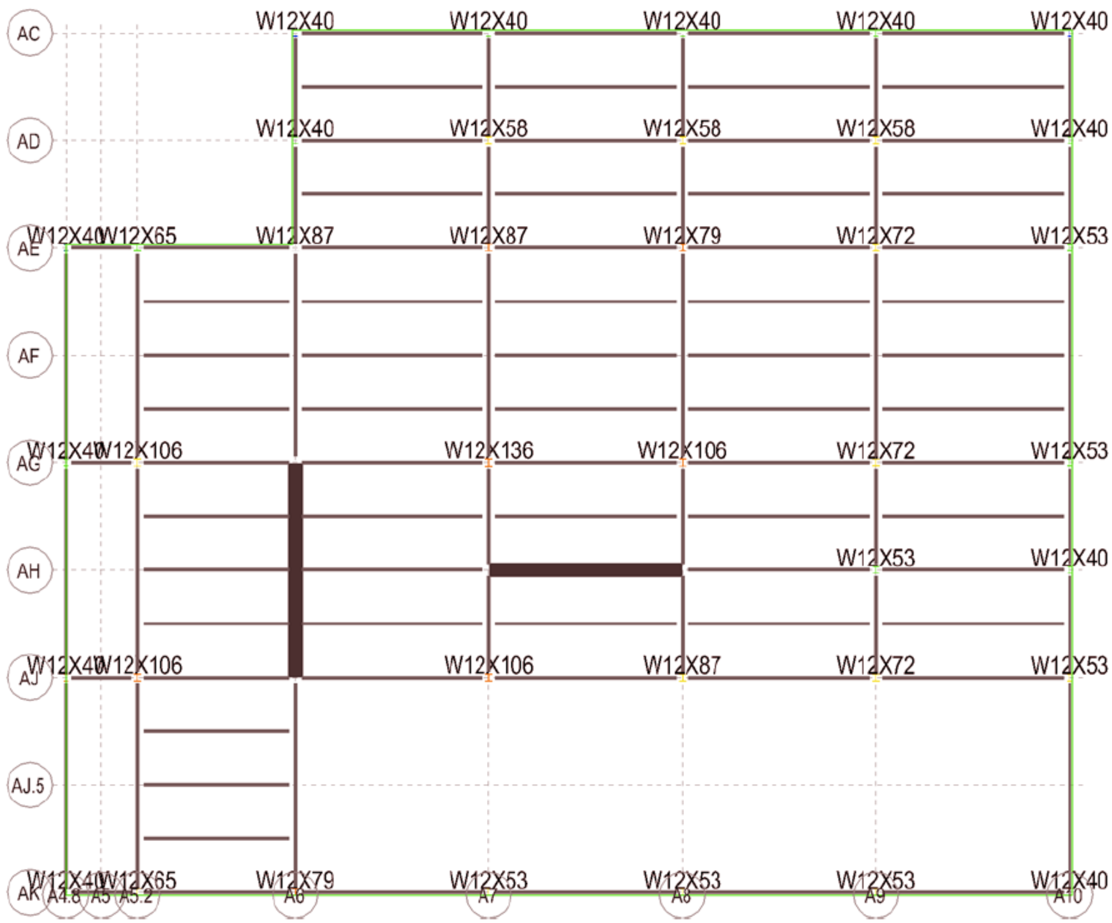


Figure 7: Typical Column sizes at Parking Garage Level

The most heavily loaded columns are those located in grids A4.8 through A7, because they are continuous from the 22nd story. The largest size is a ***W12x136*** for a tributary area of 15' x 30'. The columns that just support the parking structure see less load. The typical column for a tributary area of 18.5' x 27' is ***W12x72***. For the smaller tributary area of 7.5' x 27', the typical member size is ***W12x58***.

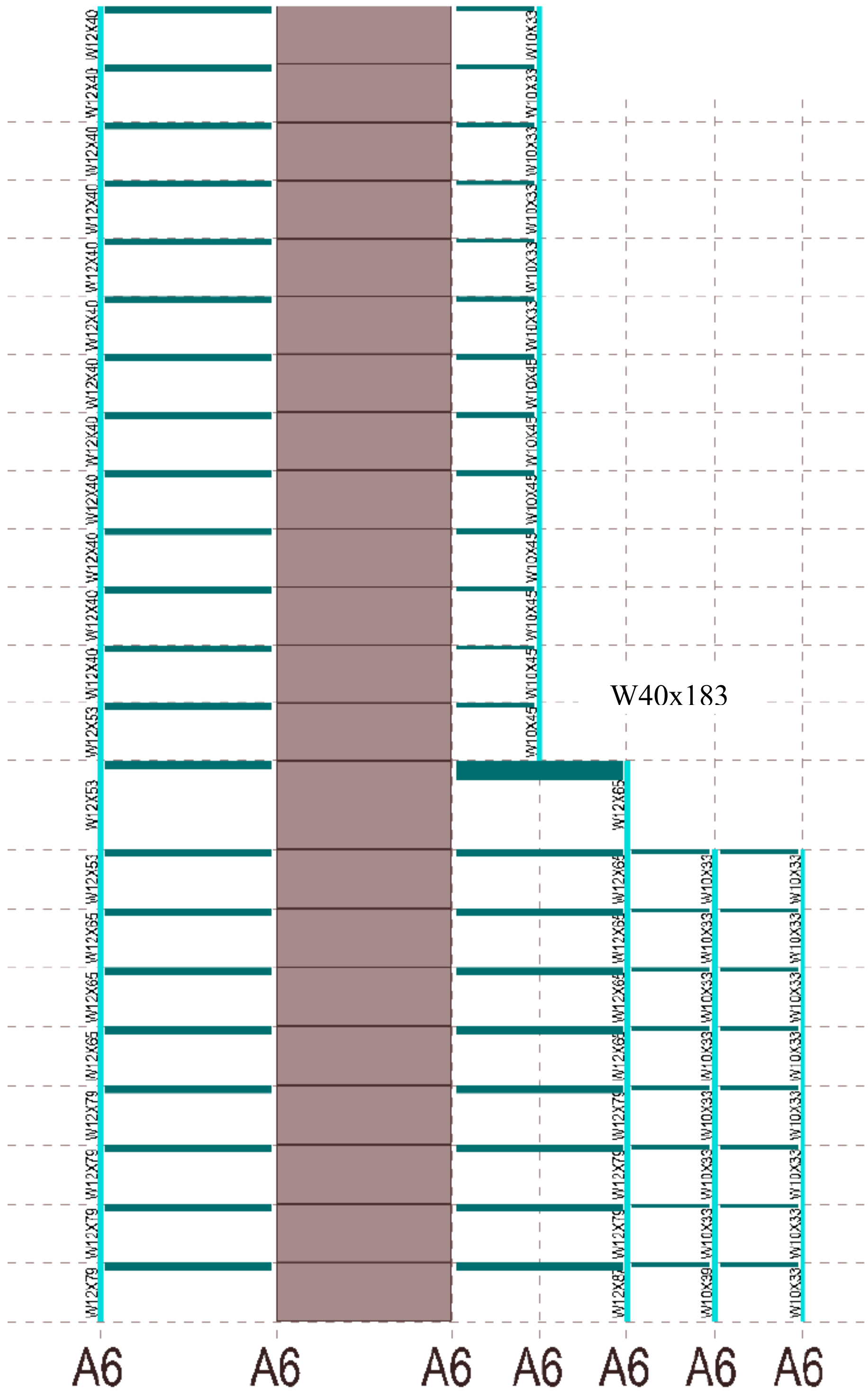


Figure 8: North-South Section through Grid A6 showing Shear Wall, Typical Column Sizes and the Transfer Girder at the 9th Story Level

IV. 1. iii. Structural Checks and Connections

The following is a summary of the checks performed for the key items:

1. **Column framing into girder**

Axial Loads:
P_{dead} = floor weight + column weight + perimeter load
=85psf*11ft*15ft*13floors + 45plf*10.25ft*13floors + 88kips = 290 kips
≈ 265 kips (RAM)
P_{live} = 60psf*11ft*15ft*13stories*0.5 = 64.5 kips
≈ 61 kips (RAM)
M_{yecc} = P*0.5d_{col} = 15 ft-kips

Load Combination = 1.2D + 1.6L P_u = 415 kips
Sidesway inhibited by shearwall, K=1.0
Selection: W10x49
$K_xL_x = K_yL_y$ = 10’
Φ_cP_n = 520 kips

$$\frac{Pu}{\phi cPn} = 0.798 > 0.2$$

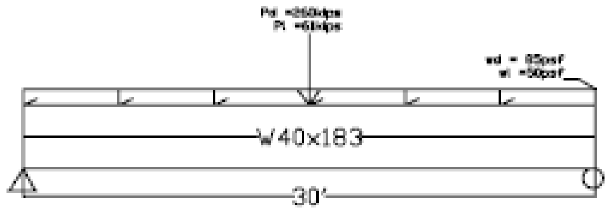
C_m = 1.0
P_{e1} = 5406 kips
B_1 = 1.08
L_p = 8.97ft Φ_pM_r = 164 ft-kips
L_r = 28.3ft Φ_bM_p = 250 ft-kips
Φ_bM_n = 245 ft-kips

$$\frac{Pu}{\phi cPn} + \frac{8}{9}\left(\frac{Mux}{\phi bMnx} + \frac{Muy}{\phi bMny}\right) = \frac{415}{520} + \frac{8}{9}\left(\frac{15}{245}\right) = 0.85 < 1.0$$

2. **Transfer girder**

f’c = 4 ksi
γ = 145pcf
Deck = 2”
Concrete, t = 6”
Shear Studs = ¾”ø
Stud Length = 3.5”
Strength = 24.6 kips

Stud Reduction Factor = 0.45
Stud Reduced Strength = 11.0 kips
Spacing = 7.5' each side
Effective slab width, b_{eff} = 90"



Constructions live load = 20 psf
Beam self-weight = 0.2klf
Slab = 0.543 klf

Pu = 415 kips
Wu = 1.0klf
Mu = 3113'kips + 118'kips = 3230'kips

$\sum Qn = 673$
a = 673 / (0.85*4*90) = 2.2
Y2 = 2.9"
ΦbMn = 3590 ft-kips > 3230 ft-kips

Use:

$$\frac{673kips}{11.0kip/stud} = 60stud * 2 = 120studs$$, 4.5" minimum spacing: use two rows

3. Connection between column and girder: base plate and stiffeners

Base Plate:
Column: W10x46
  d = 9.98"
  b_f = 10"
Girder: W40x183
  d = 39"
  b_f = 11.8"
Pu: 415 kips
A 36 Plate
Try plate with 4" larger on each side: 11" x 18" A_p = 198in2

fp = 2.1 psi

$$tp = l\sqrt{\frac{2fp}{\phi Fy}} = 2.5*\sqrt{\frac{2*2.1}{0.9*36}} = 0.9"$$ **<u>Use: 1” A36 Plate</u>**

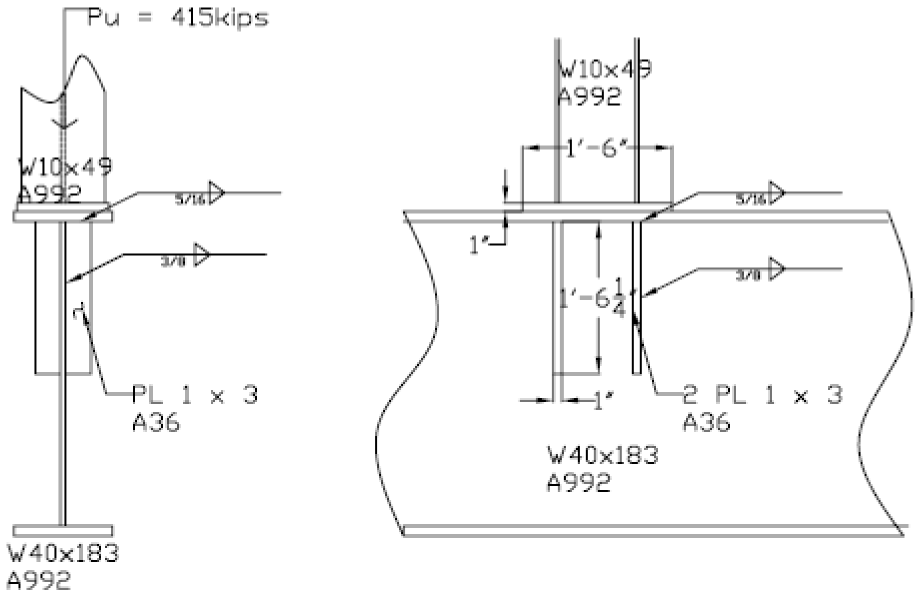
<u>Stiffeners:</u>

The girder was checked for local flange bending, local web yielding, local web crippling, and web sidesway buckling. The summary is contained below. The results show that the girder does not need stiffeners or doubler plate. However, because the allowable values are so close to the factored load, half-depth stiffeners where detailed for the connection at each side of the flange. This will allow a conservative design of such an important connection. The weld at the flange correspond to 5/16th which is the minimum weld allowed for a 1.22” flange thickness. The weld at the web corresponds to the development of the full strength of the stiffener.

Given:
W40x183
A = 53.8in2
D = 39
b_f = 11.8”
t_f = 1.22”
t_W = 0.650”
K=2.40”

Check	Rn, kips	Φ	ΦRn, kips	Ru, kips
Lateral Flange Bending	465	0.9	419	415
Local Web Yielding	519	1	519	415
Local Web Crippling	724	0.75	543	415
Web Sidesway Buckling	642	0.85	545	415

Figure 9: Column-Girder Connection Detail



4. Long term and short deflections for composite beam in apartment units.

Deflections are limited to L/360. For the 27' span, this is less than 0.9". Output from RAM shows the chosen design to be within this range. A quick check was also performed and summarized below:

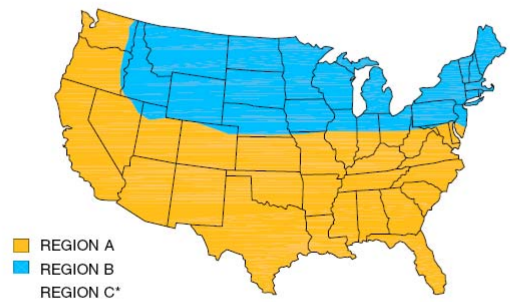
Table 4: Deflection Calculations for Composite Beam in Apartments

E_c	3492	ksi
E_s	29000	ksi
n	8.3	
b_{eff}	10.84	in
Transfer Slab Area, A_t	27.1	in^2
Neutral Axis	2.97	in
Transformed Moment of Inertia, I_t	524.14	in^4
Effective Moment of Inertia, I_{eff}	496	in^4
Short Term Deflection, $D_{short\text{-}Dead}$	**0.41**	in
Long Term Moment Inertia, $I_{eff\text{-}long}$	253.2	in^4
Long Term Deflection, $D_{long\text{-}live}$	**0.374**	in
Total Deflection	**0.78**	in
RAM Output	0.80	in
Maximum Allowable	0.9	in

IV. 1. iv. Other Consideration

Other considerations related to the parking structure include:

1. Durability: According to design guidelines by AISC for Open Deck Parking Structures, the following preventions are to be considered depending on the region. Puerto Rico is in Region A. The recommendation for the maintenance of such structures is to treat it with a sealer. It is also recommended that the underside of the deck be painted because of the close proximity to the body of water.



*Region C is defined as any site within ½ mile of a salt water body

Fig. 2-1. Map of Durability Regions

Table 2-1 Deck System Performance by Region

Region A	Mild conditions where few freeze-thaw cycles occur and/or deicing salts are not typically used on roadways
Region B	Areas where freeze-thaw cycle is typical and deicing salts are used on roadways
	Costal zones within .5 miles of body of salt water

System Type			
Cast-in-place conventionally reinforced on metal deck	A serviceable deck suitable for the climate. It should be treated with a sealer.	With a membrane coating, this deck system is susceptible to cracking. Not the system to be used in most cases for a stand-alone garage.	With a membrane coating, this deck system is also susceptible to cracking. Underside of galvanized metal deck should be painted.
Cast-in-Place Post tensioned slab	Sealed slab not required - with a sealed slab, more durable than climate requires	With a sealed slab, historically the most durable deck for this climate zone	With a sealed slab, historically the most durable deck for this climate zone

Figure 10: Recommended Parking Structures per Region

2. Fire Rating: According to NFPA, if the parking structure is less than 75ft in height and at least 1.4 sq ft of openings are provided for each linear foot of the perimeter on each side, the structure is consider Open Sides an no fire resistance requirements are enforced.

Table 1-2 NFPA Building Code and International Building Code Guidelines for Height and Tier Area Perimaters

	NFPA 88A Type II (000)		IBC Type IIB	
Fire Resistive Requirement	None		None	
Definition of Open Side	1.4 sq ft of each linear foot distributed along 40% of perimeter		50% of interior wall area of exterior wall	
	sq ft/tier	# of tiers	sq ft/tier	# of tiers
2 sides open	unlimited[1]	height<=75 ft	50,000	8
3 sides open	unlimited[1]	Height<=75 ft	62,500	9
4 sides open	unlimited[1]	Height<=75ft	75,000	9
Exception[1]			unlimited	height<=75 ft

[1]the distance from any point on the deck may not be greater than 200 feet from an open side

Figure 11: Definition of Open Parking Structure

In Puerto Rico, the Fire Department has increased this requirement from 1.4% to 2%. Documentation provided by the owner shows that if the same architecture is maintained, the building fits the requirements for fire resistance.

OPEN PARKING GARAGE AREA TABULATION OF EXTERIOR WALL OPENINGS

LEVEL/ TIER	EXTERIOR WALL PERIMETER	OPEN AREA REQUIRED IN EXTERIOR WALLS	OPEN AREA PROVIDED IN EXTERIOR WALLS	REQUIRED LENGTH OF OPENINGS	PROVIDED LENGTH OF OPENINGS
B2	*	*	*	*	*
B1	*	*	*	*	*
P1	917'-6" L.F.	2110.25 S.F.	3567.68 S.F.	367.00 L.F.	384.09 L.F.
P2	897'-0" L.F.	1794 S.F.	2047.54 S.F.	358.80 L.F.	592.25 L.F.
P3	1040'-0" L.F.	2080 S.F.	2107.37 S.F.	416.00 L.F.	457.43 L.F.
P4	1033'-0" L.F.	2066 S.F.	2329.51 S.F.	413.20 L.F.	434.17 L.F.
P5	1033'-0" L.F.	2066 S.F.	2071.98 S.F.	413.20 L.F.	438.19 L.F.
P6	1033'-0" L.F.	2066 S.F.	2277.37 S.F.	413.20 L.F.	456.65 L.F.
P7	1046'-0" L.F.	2092 S.F.	2482.60 S.F.	418.40 L.F.	426.66 L.F.
P8	**	**	**	**	**

* LEVELS B1 AND B2 BASEMENT LEVELS ARE NOT PART OF THIS CALCULATION AS THEY WILL NOT BE USED AS EGRESS FOR PHASE IV (RETAIL) AND SHALL BE MECHANICALLY VENTILATED AND FULLY SPRINKLERED

** LEVEL P8 IS AN OPEN AIR LEVEL OR TOP LEVEL OF PARKING GARAGE

Figure 12: Tabulation of Exterior Wall Openings for Parking Garage

IV. 1. v. Impact on Lateral System

By designing the gravity system using a steel frame there are two main structural impacts on the lateral system:

1. The weight of the structure is largely reduced. The net weight saving are approximately $^{60212kips}/_{79273kips} \approx 0.75$, or 25%, (see Table 5). Consequently, the Total Base Shear used for design decreases. This will reduce the internal stresses in the walls and increases the feasibility of a reduction in the number of walls.

Table 5: Story and Building Weights for Steel Frame Gravity System

Total Story Weights, psf

Story	# Stories	Area	Beam Weight, lbs	Beam Load, psf	Column Weight	Column Load, psf	Wall Load, psf	Super D, psf	Total Load,psf
22nd - 15th	8	15880.00	75272.00	**4.74**	29822.10	**1.88**	**24.67**	77.33	**108.62**
14th - 8th	7	15880.00	75272.00	**4.74**	36039.85	**2.27**	**24.67**	77.33	**109.01**
7th	1	48600.00	243601.00	**5.01**	154660.00	**3.18**	**8.00**	85.00	**101.19**
6th - 3rd	4	48600.00	241424.00	**4.97**	129550.00	**2.67**	**8.00**	85.00	**100.63**
2nd-1st	2	48600.00	241424.00	**4.97**	145800.00	**3.00**	**8.00**	85.00	**100.97**
								Total, kips	**60212.33**

2. On the other hand, the steel frame requires a deeper floor sandwich depth. For a typical member size of W10 x 22, with a depth of 10.2 in, in order to maintain a clear height of 9 ft, the total floor to floor height will increase to 10 ft 3in. This is an increase of 5 in per floor, for an overall building height increase of 5 ft 10 in. The taller walls will increase the weight experience higher overturning moments and deflections.

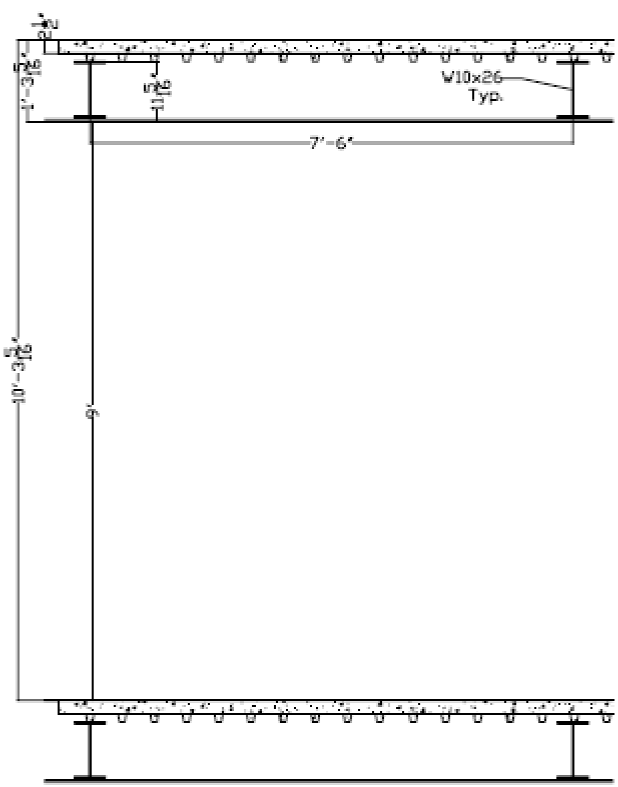


Figure 13: Required Floor Depth and Story Height

IV. 2. Lateral System

IV. 2. i Shear Wall Layout and Selection

The main objective is to minimize the number of walls that act as the lateral force resisting system. This will open up the floor area and provide more flexibility in the design for future occupants. Paseo Caribe is located in Condado, a prime tourist area surrounded by first class hotels, convention centers, international banks and offices. An open design will allow for a smoother transition of a commercial occupant to the building in the future, adding to its value.

The current design consisted of a total of 22 – 10in thick and approximately 17ft long walls in the North-South Direction and 4 – 12in thick walls in the East-West directions with a f'c = 4ksi. The goal is to reduce the number of walls by at least half. I will use 4 walls in each direction. To reduce the number of walls, the thickness of the wall will be increased and an f'c = 5ksi will be used from the previous 4ksi.

A preliminary thickness was obtained by limiting the amount of shear in the wall to half of the maximum allowed by ACI. $V = (\frac{1}{2})4\sqrt{f'c}$. Using the actual Vase Shear = 4030 kips and adding 25% for torsion, V = 5037.5 kips. Assuming that the most heavily loaded wall will take 1/3 of the load, the required thickness of the wall is:

Vmax = 5037.5 kips / 3 = 1680 kips

$$t > \frac{1680}{30*12*4*\sqrt{5000}} \geq 17in.$$

Later in the design it was recognize that to limit the amount of displacement and drift and increase the flexure capacity of the walls from the large overturning moments, the wall thickness was increased to 24 in. and coupling beams are introduced.

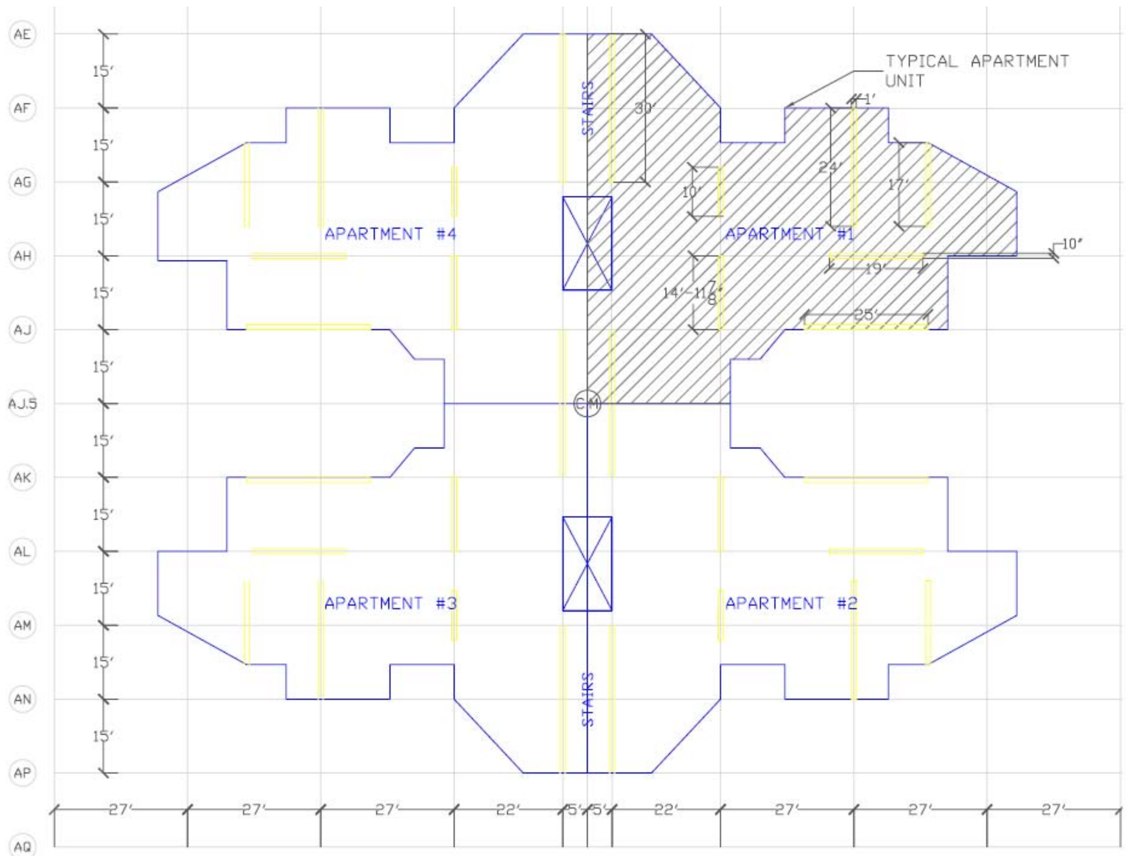


Figure 14: Typical Apartment Floor Showing Existing Walls and Dimension

An important decision is the new location of these walls in plan. Noting the discontinuities in floor plan from the apartments in the 8th level to the parking garage in the 7th level from the figures above, the location of the walls had to allow for two main structural goals:

1. First, no vertical structural irregularity of Type 4 according to UBC Table 16-L: "In plane discontinuity in vertical lateral-force-resisting element resulting from an in-plane offset of the lateral-load-resisting elements greater than the length of those elements".

For this reason the current locations of the wall around the core was not selected for the new design. The current lateral system is discontinuous at this section of the building in the 8th level, where the transition from the apartments to the parking occurs (See Figure below). Above this level, there are 5 sets of walls consisting of the 2 elevator units and 3 sets of stairs case located in between them. As can be seen from the elevation of the current structure in the figure below, there is a vertical discontinuity in the transmitting of the lateral forces from the 8th level and below. The wall layout at this location could not be modified to provide continuity because the openings below this level are required for vehicular circulation.

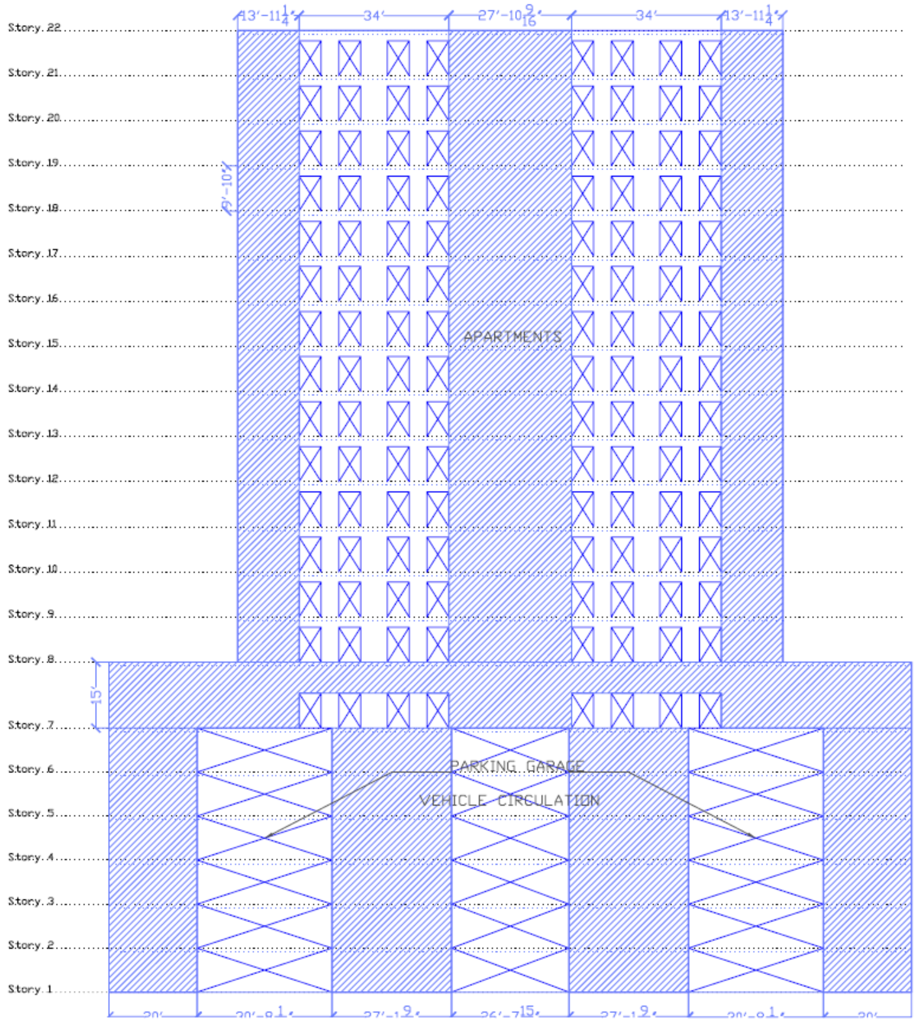


Figure 15 Section of Existing Shear Wall System

2. The second structural goal is to minimize the impact of the large torsion acting on the wall from the change in the floor's center of mass.

There is a large torsion force at and below the 8th level due to the plan irregularity from the apartment units to the parking garage (See Figure below). The resulting shear due to torsion was calculated to be very significant with e_y as large as 54ft. This provided further reasoning not to place the shear walls around the core area because of their close proximity to the center of rigidity. Placing the walls farther away from the center of rigidity will minimize the amount of torsional shear in the wall.

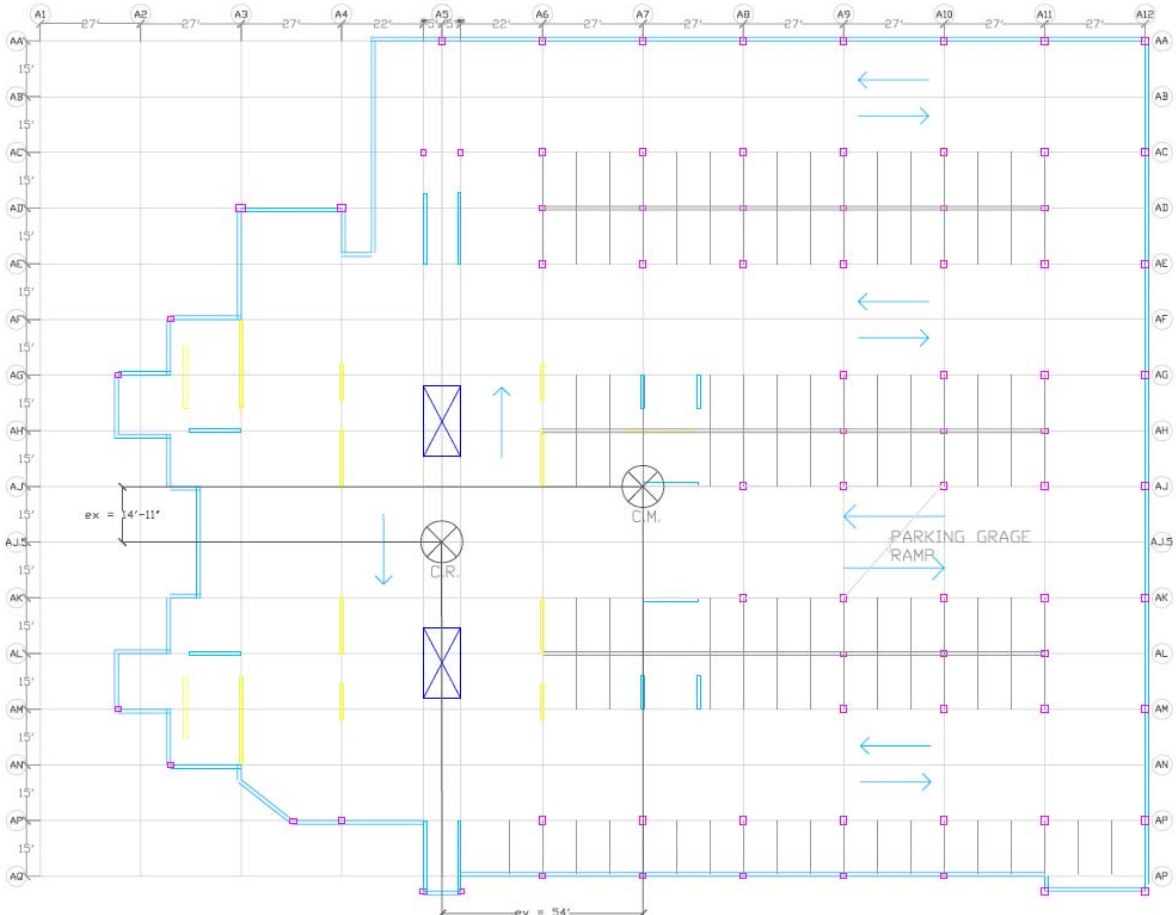


Figure 16: Typical Parking Level Floor Plan showing Center of Mass, Center of Rigidity and Torsional Eccentricities

Finally, the walls should not impact the architectural features of the apartment units. This limited the length and location of the walls considerably. A continuous wall could not be more than 20' long inside the apartment units without seriously affecting vehicular circulation in the 10 story parking garage. Because of this limitation and the large deflection that such narrow walls impart (for h/l>16), the decision was made to consider the used of coupled walls. After much consideration, trial and error, the following layout was selected.

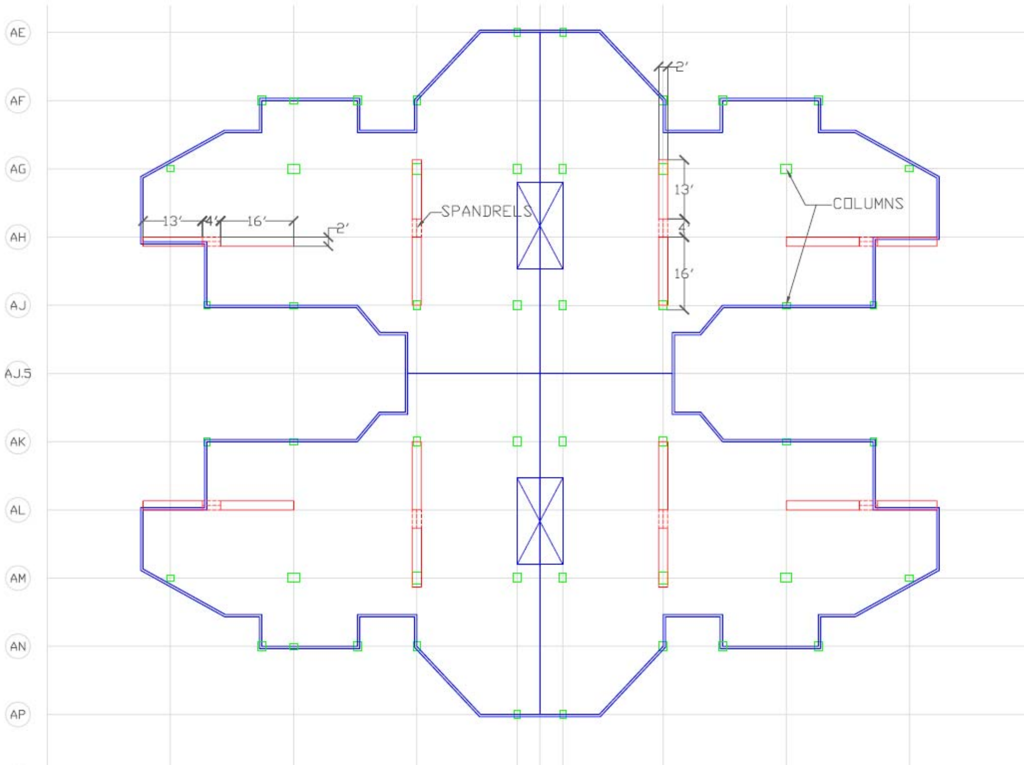


Figure 17: Coupled Shear Wall Location and Pier Label

The spandrel beams also have an f'c = 5ksi. Because of the larger floor height requirements of 10'-3" for the steel frame system, the spandrel depth was increased from 30" to 36" while still allowing for a clear opening of 7ft-2in. The final dimensions of a typical wall are:

13ft of wall (24in thick) - 4ft spandrel (24in thick, 36in deep) - 16ft of wall (24in thick)

In section,

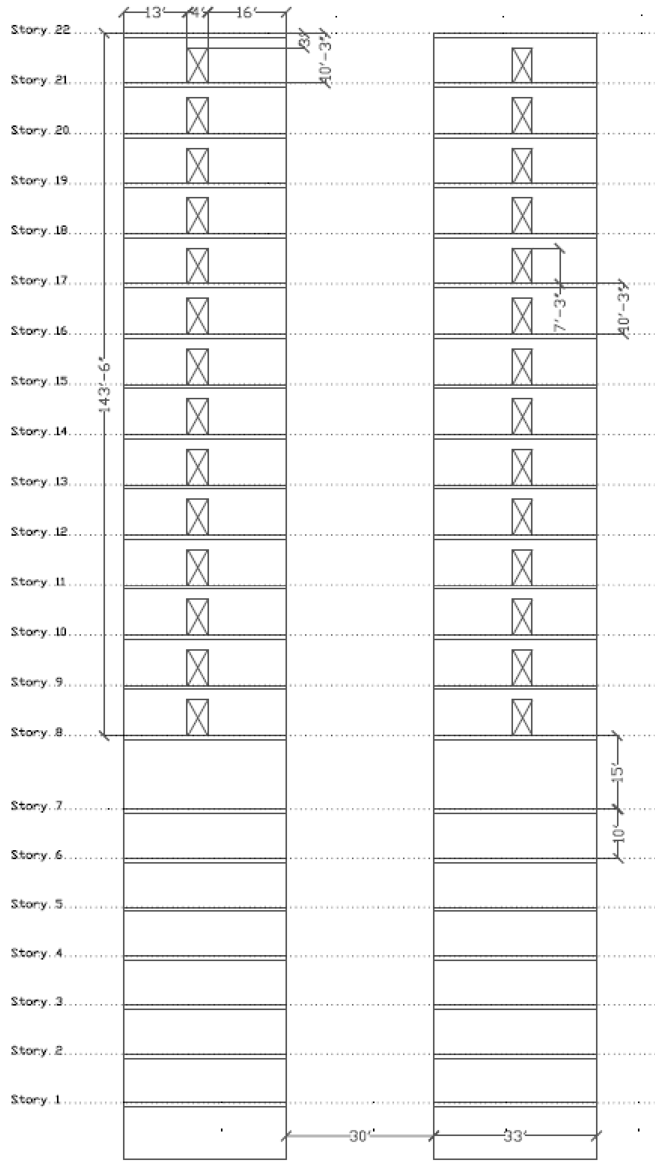


Figure 18: Overall Elevation showing two sets of coupled walls forming part of the lateral resisting system

IV. 2. ii Shear Wall Analysis

Method for Analysis

The un-factored lateral forces used for design are outputs form the finite element modeling software, ETABS. This outputs were modeled and obtained used the weight of the steel frame system once it was designed and analyzed using the computer program RAM. The axial forces due to axial forces were obtained independently from outputs for the steel frame gravity system and then used with the appropriate load combinations. The additional shear due to torsion was also computed independently and later applied at each level in the direction that produced the worst load on the wall. It is important to note that modified cross-sectional properties were used in the model to account for concrete cracking. The effective moment of inertia of $0.5I_g$ was used for modeling the piers and $0.25I_g$ for the coupled beams.

Code Requirements and Building Information

The following information is applicable to the design of the lateral system for a building in Puerto Rico. The parameters are based on UBC 1997 and the modified weight of the steel gravity system.

General Information	
Ct	0.02
UserT, Mode 1	4.69
TopStory	STORY22
BotStory	BASE
R	5.50
SoilType	SC
Z	0.30
Ca	0.33
Cv	0.45
SourceType	B
SourceDist	0.00
Na	1.30
Nv	1.60
I	1.00
TUsed	1.65
WeightUsed, kips	66808
BaseShear, kips	**3306**
FtUsed, kips	382.63

Figure 19: General Information and UBC Requirements for Seismic Loads

The 3-D representation of the building modeled in E-Tabs is depicted below:

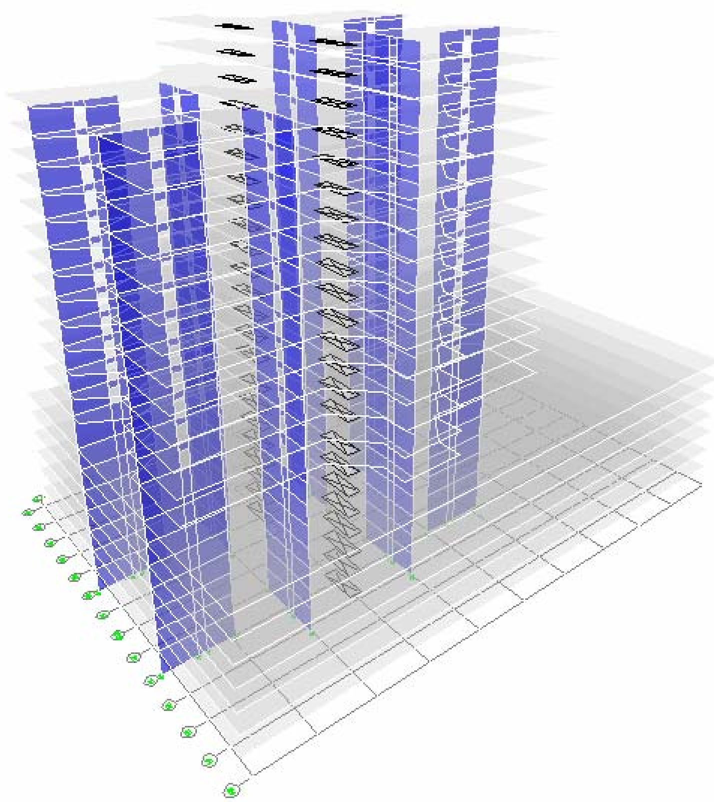


Figure 20: 3-D Model showing Story Floors and Later Walls

The plan drawing below (Figure 21) will serve as a reference for the location of the pier walls. The walls are labeled according to where they are located on the grid. The grid used for all modeling follows the grid of the original set of drawing obtained from the owner. There are 2 sets of coupled walls per grid in each of the following grids: A4 and A6 for EQ Y, AH and AL for EQ X. The piers are labeled 1 through 4 in each grid from bottom to top and left to right, accordingly.

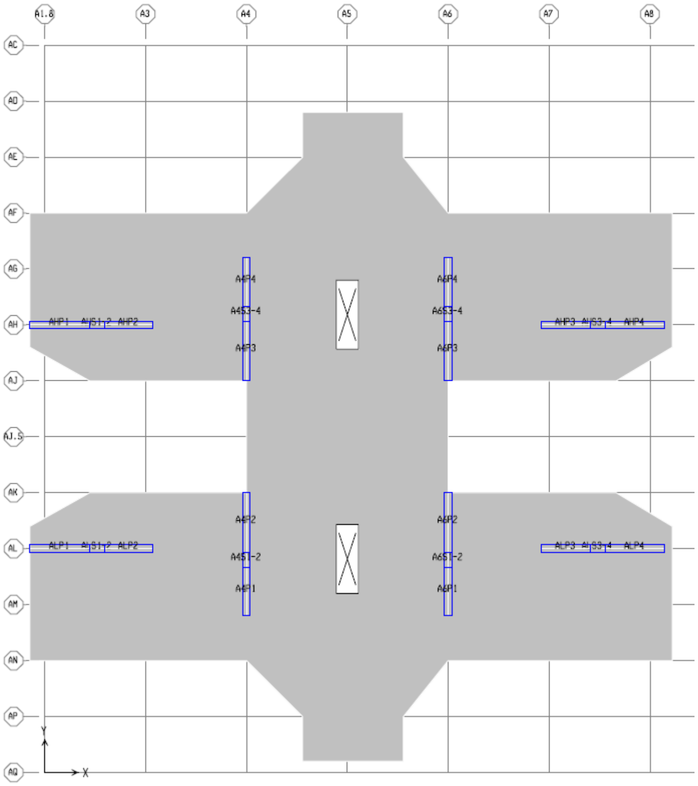


Figure 21: Model showing Pier and Spandrel Labels

The direct lateral story forces resulting from these changes obtained from E-TABS are:

Story	Fx ,kips
STORY22	502
STORY21	127
STORY20	201
STORY19	212
STORY18	201
STORY17	190
STORY16	179
STORY15	168
STORY14	157
STORY13	147
STORY12	136
STORY11	125
STORY10	114
STORY9	103
STORY8	99
STORY7	163

STORY6	135
STORY5	113
STORY4	91
STORY3	69
STORY2	47
STORY1	26
SUM (V)	3306

See Appendix B for Detailed Results Output of the Software.

Load Combinations for Design

Load combinations for seismic design of concrete are given in UBC Code 1612.2.1. Equations (12-5) and (12-6) of Chapter 16 are used along taking into consideration exception 2 of 1612.2.1 that states: "Factored load combinations of this section multiplied by 1.1 for concrete and masonry where load combinations include seismic forces." Thus the load combinations can be written:

$1.32D + 1.1E + 1.1(f_1L + f_2S)$ Equation 12-5

$0.99D \pm 1.1E$ Equation 12-6

The factors f_1 and f_2 are defined in UBC Section 1612.2.1. f_1 is 0.5 for living areas nad1.0 for parking garages. The factor f_2 is 0 for snow loads in Puerto Rico. The tern E refers to horizontal and vertical components according to Equation (30-1):

$E = \rho E_h \pm 0.5C_aID$

Substituting into the seismic load combinations gives:

$$1.48D + 1.1\rho E + 0.55f1$$

$$\mathbf{0.80D \pm 1.1\rho E}$$

The load combination used for a particular design depends on the largest negative effect it has on the structure. For walls with dead axial loads below the balance point of the moment caused by the seismic lateral forces, the second of the above combinations gives the lower bound axial load, and therefore is used for design.

The ρ factor is defined by Equation (30-3) . It is the redundancy factor and depends on the ratio of the maximum shear carried by one lateral element to the total shear carried by the story. It is defined as:

$$\rho = 2 - \frac{20}{r_{\max}\sqrt{A_x}}$$

Where,

$$r_{\max} = \frac{Vx_{\max}}{V_{story}} * \frac{10}{Lw}$$

The r_{max} and ρ factors were calculated at every story from the E-TABS output for story shears:
Note that only the bottom two-thirds height level of the building need to be considered.

Table 6: Determination of ρmax factor for Load Combinations

Story	Vstory,k	Vmax,k	Lw,ft	rmax	Ax,ft²	ρmax
Story 22	511.75	128.51	13	0.1932	11583.00	**1.0**
Story 21	648.46	160.73	13	0.1907	11583.00	**1.0**
Story 20	850.79	183.25	13	0.1657	18441.00	**1.1**
Story 19	1058.67	156.65	13	0.1138	18441.00	**0.7**
Story 18	1255.85	186.79	13	0.1144	18441.00	**0.7**
Story 17	1442.33	213.90	13	0.1141	18441.00	**0.7**
Story 16	1618.13	238.59	13	0.1134	18441.00	**0.7**
Story 15	1783.22	261.40	13	0.1128	18441.00	**0.7**
Story 14	1937.63	282.63	13	0.1122	18441.00	**0.7**
Story 13	2081.34	302.56	13	0.1118	18441.00	**0.7**
Story 12	2214.32	320.66	13	0.1114	18441.00	**0.7**
Story 11	2336.64	337.24	13	0.1110	18441.00	**0.7**
Story 10	2448.27	354.17	13	0.1113	18441.00	**0.7**
Story 9	2549.21	382.95	13	0.1156	18441.00	**0.7**
Story 8	2646.32	553.17	13	0.1608	18441.00	**1.1**
Story 7	2820.03	935.21	33	0.1005	52845.00	**1.1**
Story 6	2965.44	934.90	33	0.0955	52845.00	**1.1**
Story 5	3087.21	987.32	33	0.0969	52845.00	**1.1**
Story 4	3185.33	1021.75	33	0.0972	52845.00	**1.1**
Story 3	3259.81	1050.46	33	0.0977	52845.00	**1.1**
Story 2	3310.64	1078.71	33	0.0987	52845.00	**1.1**
Story 1	3338.14	1040.37	33	0.0944	52845.00	**1.1**

The larger ρ factor of 1.1 is used for design. The governing load combination is:

$$0.8D + 1.2E \qquad \text{(Load Case 1)}$$

$$1.48D + 1.2E + 0.55L \qquad \text{(Load Case 2)}$$

There are three main forces acting on the wall that will be considered in the use of these equations:

1. Axial Dead Load carried directly by the wall.
2. Direct Shear and Moment resulting from Seismic Loads
3. Torsion from eccentric loading of Seismic Loads on Diaphragm.

Axial Dead Loads

The dead load acting as compression on the shear walls come from two sources. The first is from the beam members that frame into the wall dispersing the dead load from the floor system and any superimposed load into the walls by tributary area. The second source is the wall's self weight. A 24" – 33' long wall with a tributary width of 25 ft and a floor load of 84 psf will see around 150 kip dead load per floor. The detailed un-factored dead loads affecting a typical wall in each direction are as calculated below.

Table 7: Example of Dead Loads Affecting Critical Piers in each Direction

Direction	N-S (EQ Y)	
Pier Label	**A6**	
Tributary Width	24.5	ft
Length:	33	ft
Spandrel:	4	ft
Thickness:	24	in
Unsupported Height:	9.91	ft
Floor Dead Load:	85	psf
Load due to Floor Dead:	**2082.5**	plf / story
Wall Density:	150	pcf
Load due to Self Weight:	**2973**	plf / story
Total Dead Load:	**5055.5**	plf / story
Total Point Load :		
Apartments	**146.61**	kips / story
Parking	**166.83**	kips / story

Story	P_D, k
22	146.61
21	293.22

Direction	E-W (EQ X)	
Pier Label	**AH**	
Tributary Width	9	ft
Length:	29	ft
Spandrel:	4	ft
Thickness:	24	in
Unsupported Height:	9.91	ft
Floor Dead Load:	85	psf
Load due to Floor Dead:	**765**	plf / story
Wall Density:	150	pcf
Load due to Self Weight:	**2973**	plf / story
Total Dead Load:	**3738**	plf / story
Total Point Load:		
Apartments	**93.45**	kips / story
Parking	**108.40**	kips / story

Story	P_D, k
22	93.45
21	186.90

20	439.83
19	586.44
18	733.05
17	879.66
16	1026.27
15	1172.88
14	1319.49
13	1466.10
12	1612.70
11	1759.31
10	1905.92
9	2052.53
8	2219.36
7	2386.20
6	2553.03
5	2719.86
4	2886.69
3	3053.52
2	3220.35
1	3387.19

20	280.35
19	373.80
18	467.25
17	560.70
16	654.15
15	747.60
14	841.05
13	934.50
12	1027.95
11	1121.40
10	1214.85
9	1308.30
8	1416.70
7	1525.10
6	1633.51
5	1741.91
4	1850.31
3	1958.71
2	2067.11
1	2175.52

Because the dead load will help to counter act the tensile forces from the large overturning moments, a major consideration in the arrangement of the floor framing layout was to *maximize* the amount of dead load that the wall carries.

Direct Seismic Loads

The un-factored story shears obtained following UBC 1997 requirements and the ETABS model depicted in the previous sections are the following.

Table 8: Story Shears

Story	V , k	M, ft- k
STORY22	-502	5140
STORY21	-629	11587
STORY20	-830	20091
STORY19	-1042	30767
STORY18	-1243	43504
STORY17	-1433	58190
STORY16	-1612	74713
STORY15	-1780	92961
STORY14	-1938	112822
STORY13	-2084	134185
STORY12	-2220	156938

STORY11	-2344	180969
STORY10	-2458	206167
STORY9	-2561	232419
STORY8	-2661	272328
STORY7	-2824	300563
STORY6	-2959	330151
STORY5	-3072	360871
STORY4	-3163	392504
STORY3	-3233	424829
STORY2	-3280	457627
STORY1	-3305	495639

Torsion

Due to the large change in plan geometry from the apartment building to the parking garage, seismic forces affecting the parking garage at its center of mass will produce a torque on the floor that will have to be resisted by the shear walls. The apartment building sits in one half of the footprint of the parking structure. While the center of rigidity lays in the center of mass in the levels containing the apartment units, the distance between the center of mass to the center of rigidity at the parking floor levels is **46 ft** in the X (E-W) direction! (See Figure 16 and Figure 15 above) This is a large eccentricity and the shear resulting form the moment created by this eccentricity as the load hits the parking needs to be considered for design. The N-S direction experiences a smaller eccentricity of 9 ft. Therefore, seismic forces in the Y Direction will be the controlling case for torsion.

First, the torsional rigidities and distribution factors must be computed for each wall below the 9th level so that the shear forces can be distributed according to their stiffness and distance from the center of rigidity. Because the walls have the same Modulus of Elasticity and the same thickness, their rigidities can be compared by l^3.

Torsion
Parking Garage: 8th - 1st Level
Pier Distribution Factors

Pier Label	L, ft	k	d	kd	kd²	kd/∑kd²
A4P1-2	33	35937	27	970299	26198073	**0.0048**
A4P3-4	33	35937	27	970299	26198073	**0.0048**
A6P1-2	33	35937	-27	-970299	26198073	**-0.0048**
A6P3-4	33	35937	-27	-970299	26198073	**-0.0048**
AHP1-2	30	27000	-30	-810000	24300000	**-0.0040**

AHP3-4	30	27000	-30	-810000	24300000	**-0.0040**
ALP1-2	30	27000	30	810000	24300000	**0.0040**
ALP3-4	30	27000	30	810000	24300000	**0.0040**
					201992292	

Then the torque was distributed according the each piers distribution factor.

Another important consideration is the determination of *torsion irregularity* according to Section 9.5.5.5.2. This section specifies that if the maximum displacement, Δ_{max}, at corner if larger than 1.2* the average displacement, Δ_{avg}, of two opposite corners, the shear due to accidental torsion needs to be multiplied by a amplification factor, A_x.

$$Ax = \left(\frac{\Delta_{\max}}{1.2 * \Delta_{avg}} \right)^2$$

Extreme torsion irregularity applies when the ratio is greater than 1.4.

Once the ETABS model was built and run, the displacements at extreme corner of the 19th and 7th floor where obtained. This allowed for the determination of the Ax where it was applicable. On the top floors, the apartments are symmetrical about both Center of Mass and Center of Rigidity. At these levels the displacements where almost identical. The values changed only at the 3rd decimal place. However, at the parking level where there is a large shift in the C.M. and C.R. location, the displacement in the East face due to EQY Seismic Loading was considerable larger than that in the West face (4.35in:0.73in). The conclusion was attained that an amplification factor, Ax, of 2.0 was necessary to in addition to the effect of the accidental torsion. The following is a summary of the findings:

Table 9: Determination of Amplification Factor, Ax, for Accidental Torsion

Story	Loading	Face	Point #	Δ, in	Δmax, in	Δavg, in	Δmax/ Δavg	Ax
19th	EQX	North	138	10.11	10.11	10.11	1	1
		South	2468	10.11				
	EQY	East	138	16.47	16.47	16.47	1	1
		West	144	16.47				
7th	EQX	North	20	1.8495	1.8495	1.84	1	1
		South	2474	1.8493				
	EQY	**East**	**2474**	**4.3501**	**4.35**	**2.54**	**1.7**	**2**
		West	**2480**	**0.7237**				

For the critical North – South Direction where the eccentricity is 46 ft plus an accidental torsion of 2*270*0.05 = 27ft, the total considered eccentricity if **73ft**! The torsion in the first floor is up to 217 kips, which is about 25% the direct shear component.

Table 10: Torsion Resulting from Eccentric Loading at Parking Structure

Torsion
Parking Garage: 8th - 1st Level

Load Case:	EQY Direction
Center of Mass, C.M. =	127 ft
Center of Rigidity. C.R. =	81 ft
eccentricity, e =	73 ft
Pier Label:	AHP3-4
kd/∑kd2	0.0040

Story Level	$P_{E\ Story}$, k	T, kip-ft	$V_{T,kips}$
STORY8	99	7256	**29**
STORY7	262	19151	**77**
STORY6	398	29023	**116**
STORY5	511	37289	**149**
STORY4	602	43950	**176**
STORY3	671	49006	**196**
STORY2	719	52458	**210**
STORY1	744	54327	**217**

The shear due to torsion in the East – West Direction is not as large but because the walls in this direction span up to the 22nd story (unlike the North – South walls that only span to the 20th story) their direct shear and moment are larger. For these reason, the torsion in this direction will also be computer to compare the total shear on each wall for both directions (See Appendix C for other Calculations referring to Torsion).

On the other hand, the apartment floors do not experience this torsion because the walls are arranged so that the Center of Mass is in the same location as the Center of Rigidity. For this floor level a minimum eccentricity of 5% was considered for design.

Following the same procedure,

Table 11: Torsion in Apartment Units as a Result of Minimum Eccentricity: 0.05L

Torsion

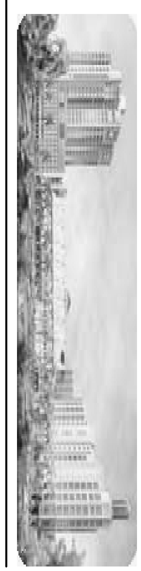
Apartments	22nd - 9th	
Load Case:	<u>EQX Direction</u>	
Center of Mass, C.M. =	81	ft
Center of Rigidity. C.R. =	81	ft
Length:	180	ft
eccentricity, e =	9	ft
Pier Label:	AHP3	

kd/∑kd2 0.0030

Story Level	$P_{E\ Story}$, k	T, kip-ft	V_{T} ,kips
STORY22	502	4514	**14**
STORY21	629	5660	**17**
STORY20	830	7467	**22**
STORY19	1042	9374	**28**
STORY18	1243	11184	**34**
STORY17	1433	12895	**39**
STORY16	1612	14508	**44**
STORY15	1780	16023	**48**
STORY14	1938	17439	**52**
STORY13	2084	18758	**56**
STORY12	2220	19978	**60**
STORY11	2344	21100	**63**
STORY10	2458	22125	**66**
STORY9	2561	23051	**69**

IV. 2. iii Summary

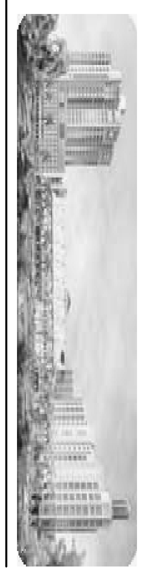
The following tables provide a summary of all the loads, including dead, seismic, and torsion affecting the critical pier in each direction. The results are a combination of outputs specified in the sections above. Since the walls change dimension and become coupled above the 9th story level (Refer to Figure 18), the 9th and 1st story level resultant forces are highlighted in blue because they are the critical sections for design. Note: Positive Axial Forces are in compression.



Pier Forces	A4											Units	ft-kip	
Load Case	EQ Y													
Load Combinations:	0.8*D + 1.2*E					Load Case 1								
	1.48*D + 1.2*E +1.1*L					Load Case 2					Factored Loads			

Story	Pier	Load	Loc	P_D	P_L	P_E (±)	$V_{E\ Direct}$	$V_{E\ Torsion}$	$V_{E\ Total}$	M_E	Pu_{COMP}, k	Pu_{TENS}, k	Vu, k	Mu, ft-k
STORY20	A4P4	EQY	Bottom	-70	-11	71	92	18	110	334	-201	30	132	401
STORY19	A4P4	EQY	Bottom	-139	-31	165	91	23	114	472	-438	87	137	567
STORY18	A4P4	EQY	Bottom	-209	-50	273	109	27	136	669	-692	161	164	803
STORY17	A4P4	EQY	Bottom	-278	-70	400	127	31	159	898	-968	257	190	1077
STORY16	A4P4	EQY	Bottom	-348	-89	544	145	35	180	1154	-1266	375	216	1385
STORY15	A4P4	EQY	Bottom	-417	-108	706	162	39	201	1433	-1584	514	241	1719
STORY14	A4P4	EQY	Bottom	-487	-128	886	178	42	221	1731	-1924	673	265	2077
STORY13	A4P4	EQY	Bottom	-556	-147	1081	194	46	240	2052	-2282	852	288	2463
STORY12	A4P4	EQY	Bottom	-625	-167	1291	208	48	257	2390	-2658	1049	308	2868
STORY11	A4P4	EQY	Bottom	-694	-186	1513	220	51	271	2745	-3047	1260	325	3294
STORY10	A4P4	EQY	Bottom	-763	-206	1741	230	54	283	3149	-3443	1479	340	3778
STORY9	**A4P4**	**EQY**	**Bottom**	-830	-225	1961	244	56	300	3758	**-3829**	**1689**	**360**	**4509**
STORY8	A4P3-4	EQY	Bottom	-2086	-244	0	588	78	666	55493	-3356	-1669	799	66591
STORY7	A4P3-4	EQY	Bottom	-2154	-264	0	582	114	696	61315	-3478	-1723	835	73578
STORY6	A4P3-4	EQY	Bottom	-2222	-283	0	591	144	735	67222	-3600	-1777	882	80666
STORY5	A4P3-4	EQY	Bottom	-2289	-364	0	601	169	770	73231	-3789	-1832	924	87877
STORY4	A4P3-4	EQY	Bottom	-2357	-388	0	608	189	797	79313	-3916	-1886	957	95176
STORY3	A4P3-4	EQY	Bottom	-2425	-412	0	612	204	816	85435	-4043	-1940	979	102522
STORY2	A4P3-4	EQY	Bottom	-2493	-437	0	618	215	833	91612	-4170	-1994	999	109934
STORY1	**A4P3-4**	**EQY**	**Bottom**	-2571	-461	0	614	220	834	98670	**-4312**	**-2057**	**1001**	**118404**

Table 12: Factored Forces on North - South Shear Walls



Pier Forces	AH												Units	ft-kip
Load Case:	EQ X													
Load Combinations:	0.8*D + 1.2*E				Load Case 1									
	1.48*D + 1.2*E + 1.1L				Load Case 2						Factored Loads			
Story	**Pier**	**Load**	**Loc**	**P_D**	**P_L**	**P_E (±)**	**$V_{E\ Direct}$**	**$V_{E\ Torsion}$**	**$V_{E\ Total}$**	**M_E**	**Pu_{CASE2}, k**	**Pu_{CASE1}, k**	**Vu, k**	**Mu, ft-k**
STORY22	AHP3	EQY	Bottom	-96	-4	87	141	14	154	393	-251	27	185	472
STORY21	AHP3	EQY	Bottom	-144	-10	192	176	17	193	901	-456	115	232	1081
STORY20	AHP3	EQY	Bottom	-193	-16	280	36	22	58	467	-639	182	70	560
STORY19	AHP3	EQY	Bottom	-241	-22	368	114	28	142	678	-822	249	171	813
STORY18	AHP3	EQY	Bottom	-289	-26	484	156	34	189	946	-1038	349	227	1135
STORY17	AHP3	EQY	Bottom	-338	-30	627	189	39	228	1240	-1286	483	273	1488
STORY16	AHP3	EQY	Bottom	-386	-33	795	218	44	261	1556	-1563	646	313	1867
STORY15	AHP3	EQY	Bottom	-434	-36	986	244	48	292	1892	-1866	836	350	2270
STORY14	AHP3	EQY	Bottom	-483	-39	1197	267	52	319	2246	-2194	1050	383	2695
STORY13	AHP3	EQY	Bottom	-531	-42	1425	289	56	345	2620	-2543	1285	414	3144
STORY12	AHP3	EQY	Bottom	-580	-45	1668	309	60	369	3016	-2910	1538	443	3619
STORY11	AHP3	EQY	Bottom	-628	-48	1921	329	63	392	3450	-3288	1803	471	4140
STORY10	AHP3	EQY	Bottom	-677	-51	2167	350	66	416	4000	-3658	2059	500	4800
STORY9	**AHP3**	**EQY**	**Bottom**	-725	**-53**	2366	371	69	440	5008	**-3971**	**2258**	**528**	**6009**
STORY8	AHP3-4	EQY	Bottom	-1769	-113	0	665	98	763	67872	-2742	-1415	916	81446
STORY7	AHP3-4	EQY	Bottom	-1874	-143	0	699	146	844	74861	-2930	-1499	1013	89833
STORY6	AHP3-4	EQY	Bottom	-1979	-152	0	738	185	923	82238	-3095	-1583	1107	98686
STORY5	AHP3-4	EQY	Bottom	-2083	-161	0	765	218	983	89887	-3261	-1667	1180	107864
STORY4	AHP3-4	EQY	Bottom	-2188	-170	0	788	245	1033	97764	-3426	-1751	1239	117317
STORY3	AHP3-4	EQY	Bottom	-2293	-179	0	806	265	1071	105820	-3591	-1834	1285	126984
STORY2	AHP3-4	EQY	Bottom	-2398	-188	0	815	279	1094	113969	-3756	-1918	1313	136763
STORY1	**AHP3-4**	**EQY**	**Bottom**	-2518	**-197**	0	828	286	1115	120194	**-3944**	**-2015**	**1337**	**144233**

Table 13: Factored Forces on East - West Shear Walls

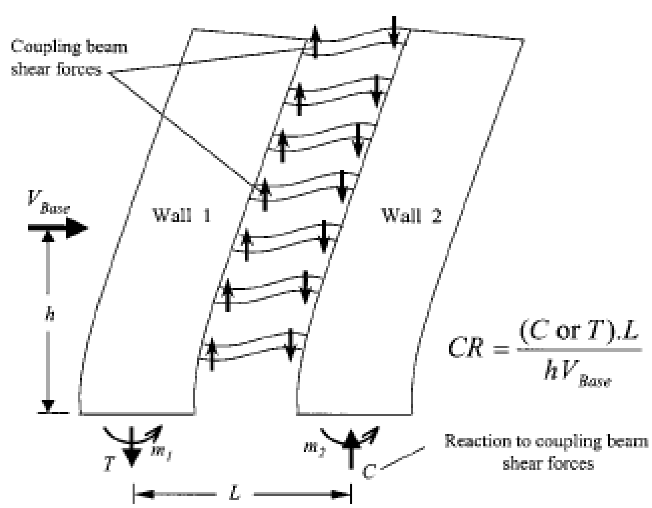
IV. 2. iv Coupled Beams

Reinforced concrete coupled wall systems use a reinforced concrete beam to tie two or more reinforced concrete walls together. Coupling beams provide transfer of vertical forces between adjacent coupled walls. This created an action that resists a portion of the total overturning moment induced by the seismic action. This coupling action has two main benefits. First, it reduced the moments that must be resisted by the individual walls and therefore results in a more efficient structural system. Second, it provides a means by which seismic energy is dissipated over the entire height of the wall system as the coupling beams undergo inelastic deformations.

The efficiency of this system can be assessed by degree in which it achieves a composite cantilever action. A shear wall by itself will deform like a cantilever, allocating the maximum tensile and compressive stresses on opposite edges of each wall (L_1 and L_2). On the other hand, is the wall is connected by a rigid beam, the two walls act as a single composite units, much like a frame. The bending stress will then be distributed linearly across the unit, the maximum compressive and tensile units occurring at the opposite extreme ends ($L_1 + L_2$).

One method to determine the impact of the coupling beam as part of the lateral resisting system is by computing the Coupling Ratio, CR (See Figure 22 below). A CR of zero means that there is no coupling action. A CR of 50% means that the coupling walls are resisting half of the overturning moment. Much research has been done to determine what a good CR ratio is. Findings by El-Tawil and Kuenzli(2002b) shows that a CR of 30% provides the most structural efficiency.

Figure 22: Coupled Shear Wall Action



Coupling Ratios (CR) obtain for the lateral design of the coupled system are in the order of 26%. More efficient coupling beams with coupling ratios of 30% were obtained when longer walls of 20' were used. However, the longer walls could not be used due to the architectural demands of the apartment layout.

Table 14: Coupling Ratios

Spandrel Coupling Ratios

Direction:	EQ X	
Pier:	AH	
V Shear:	3306	kips
L_{eff}:	18.5	ft

Story	h, ft	V_E, kips	P_E, kip	CR
22	230	251.01	87	0
16	168.5	388.43	795	12.00
15	158.25	434.96	986	16.49
14	148	477.17	1197	19.17
13	137.75	515.63	1425	20.81
12	127.5	550.52	1668	21.85
11	117.25	582.48	1921	22.68
10	107	609.3	2167	23.58
9	**96.75**	**636.15**	**2366**	**26.24**

Coupled Beam Design

Coupled beams are designed to develop full shear capacity to ensure plastic hinge formation due to flexural failure.

Requirements:

If the factored shear exceeds $4\sqrt{f'c}b_w d$, diagonal reinforcement is required.

Clear covering requirements are: 1" Coupling beam
1-3/8" Wall Pier

Spacing, s, between bars shall have a minimum core dimension of $b_w/2$ or 4". Because of the limited depth of the beam and to increase the angle the bar makes with the horizontal axis, α, the minimum spacing of 4" between diagonal bars was used. This allowed for a maximum α = 30° once the covering requirements where met.

For beams requiring diagonal reinforcement:

$$\Phi V_n = 2\Phi f_y \sin\alpha A_d < 10\Phi\sqrt{f'_c} b_w d$$

Where,

Φ=0.85

A_d = Area of reinforcement in each group of diagonal bars, in^2

Layout:
The dimensions of the coupled beam used for design are:

Length, l = 4' — to match corridor openings
b_w = 24" — to match wall thickness
Depth, d = 36" — to allow for a 7'-2" door way clearance
f'c = 5 ksi
fy = 60 ksi
α = 30 degrees

Resultant Forces and Required Reinforcement

Diagonal Reinforcement:
From Table 15: Shear Demand and Required Diagonal and Vertical Reinforcing for Coupling Beams below, beams at and below the 16th level required diagonal reinforcement. The code allows for re-distribution of forces as long as the sum of the strength of the coupled beams is larger than the sum of the shear forces acting on the beams. This is an advantage because the beams can be grouped by floors and a common design for a given group of floors can be maintained. This improves the performance of the structure and also reduces time and cost during detailing and erection.

The final design uses vertical reinforcement of #6 @ 9". This reinforcement corresponds to the required at the 17th story and it is kept throughout the top six stories to maintain uniformity. The diagonal bars consist of two set of 4-#10 spaced 4" on center in first four stories (9th to 12th) and 4 -#9 also spaced at 4" on center on the following four floors (13th-16th).

Table 15: Shear Demand and Required Diagonal and Vertical Reinforcing for Coupling Beams

Spandrel Reinforcement

f'c =	5 ksi
b_w, in =	24 in
Φ =	0.85
Vc =	97.75 kips
fy =	60 ksi
$ΦVn_{max}$ =	391 kips

Story	Spandrel	Load	V_E, kips	Vu, kips	h ,in	d,in	Vu/bwd√f'c	α, deg	$A_{s\text{-}v\ req}$, in²	Ad_{used}, in²	Diagonal Bars	Vertical Bars	ΦVn. Kips	ΦVn/Vu
STORY22	ALS3-4	EQX	85.34	**102.408**	36	28.8	2.1	0	0.19	0.59	0	**#6 @ 9"**	143.27	**1.40**
STORY21	ALS3-4	EQX	108.46	**130.152**	36	28.8	2.7	0	0.46	0.59	0	**#6 @ 9"**	143.27	**1.10**
STORY20	ALS3-4	EQX	86.85	**104.22**	36	28.8	2.1	0	0.21	0.59	0	**#6 @ 9"**	143.27	**1.37**
STORY19	ALS3-4	EQX	87.65	**105.18**	36	28.8	2.2	0	0.22	0.59	0	**#6 @ 9"**	143.27	**1.36**
STORY18	ALS3-4	EQX	115.8	**138.96**	36	28.8	2.8	0	0.55	0.88	0	**#6 @ 6"**	172.85	**1.24**
STORY17	ALS3-4	EQX	143.31	**171.972**	36	28.8	3.5	0	0.87	0.88	0	**#6 @ 6"**	172.85	**1.01**
STORY16	ALS3-4	EQX	168.18	**201.816**	36	28.8	**4.1**	30	3.96	5.08	**4-#9**	0	**259.08**	1.28
STORY15	ALS3-4	EQX	190.54	**228.648**	36	28.8	**4.7**	30	4.48	5.08	**4-#9**	0	**259.08**	1.13
STORY14	ALS3-4	EQX	210.62	**252.744**	36	28.8	**5.2**	30	4.96	5.08	**4-#9**	0	**259.08**	1.03
STORY13	ALS3-4	EQX	228.46	**274.152**	36	28.8	**5.6**	30	5.38	5.08	**4-#9**	0	**259.08**	0.95
STORY12	ALS3-4	EQX	243.29	**291.948**	36	28.8	**6.0**	30	5.72	6.24	**4-#10**	0	**318.24**	1.09
STORY11	ALS3-4	EQX	252.77	**303.324**	36	28.8	**6.2**	30	5.95	6.24	**4-#10**	0	**318.24**	1.05
STORY10	ALS3-4	EQX	245.7	**294.84**	36	28.8	**6.0**	30	5.78	6.24	**4-#10**	0	**318.24**	1.08
STORY9	ALS3-4	EQX	198.68	**238.416**	36	28.8	**4.9**	30	4.67	6.24	**4-#10**	0	**318.24**	1.33
													Avg =	1.12

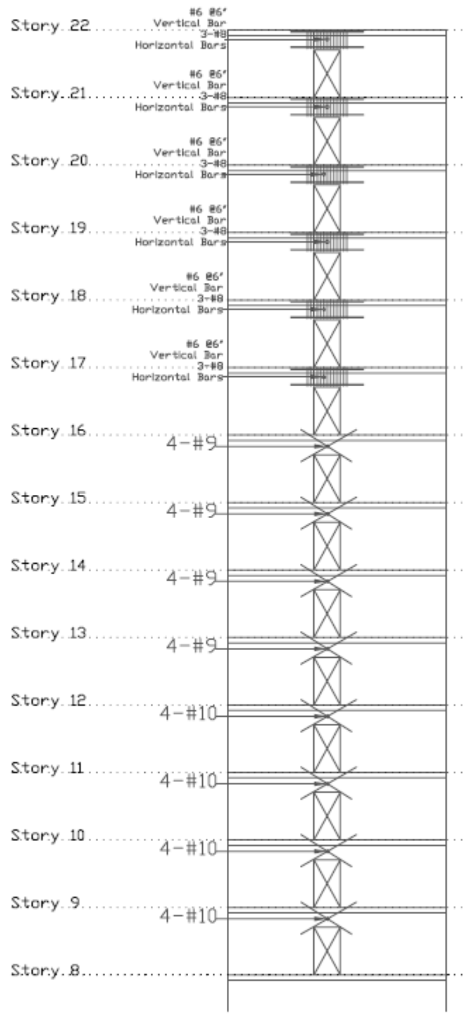


Figure 23: Required Reinforcement at Coupling Beam per Story

Detailing

Coupling Beam Ties:

Section 1921.4.4 specifies that the maximum ties spacing is the minimum of 4" or $b_w/4$. For a b_w = 24", 4" spacing governs. Because the design strength of the member core exceeds the load combination, Equation (21-4) governs and the amount of horizontal ties, A_{sh}=

$$Ash = 0.09 * s * hc * f'c / fy$$ for each direction

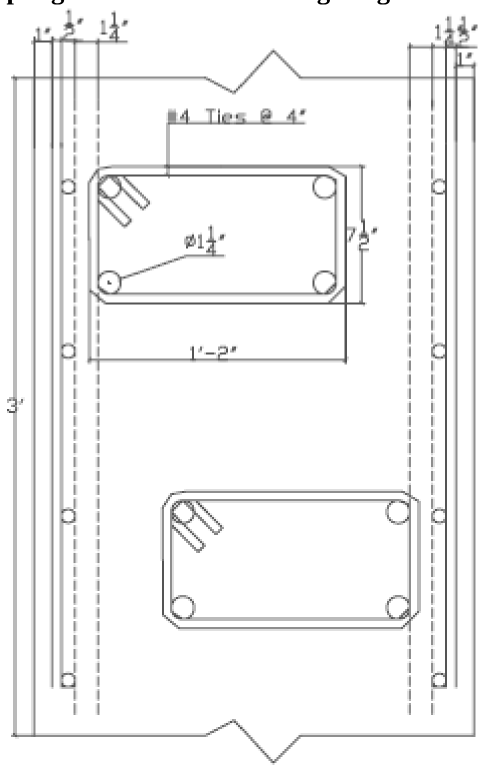
Where,

hc = tie-tie direction in the plane being considered.

s = spacing

A tie-to-ties spacing of 13.5" was selected for multiple reasons. First, it provides for the minimum core dimension of bw/2 = 12" specified in Section 1921.6.10.2. Second, it allows for cover requirements assuming #4 ties. And, it permits allowable spacing for the extension of the diagonal bars for development length within the wall pier's reinforcement. See Figure 24.

Figure 24: Coupling Beam Section Showing Diagonal Reinforcement and Ties



Using Equation 21-4: Ash = 0.40in2 / 2 legs = 0.24 in^2. Use, **#4 ties @ 4"**

Core spacing around ties: 6.125" x 14"

Parallel and Transverse Reinforcement:

Provide minimum reinforcement required by Section 1921.6.10.4.

Vertical Bars, Av > $0.0015b_{ws}s$ > $0.216in^2$ for s = 6" < d/5 = 7.68"

→ **Use:#4@6" Vertical Bars**

Minimum *Horizontal Bars*, Ah > 0.0025bws > 0.54in2 for s = 9" < d/3 < 9.6"

This allows for **#6 @ 9" Horizontal Bars**.

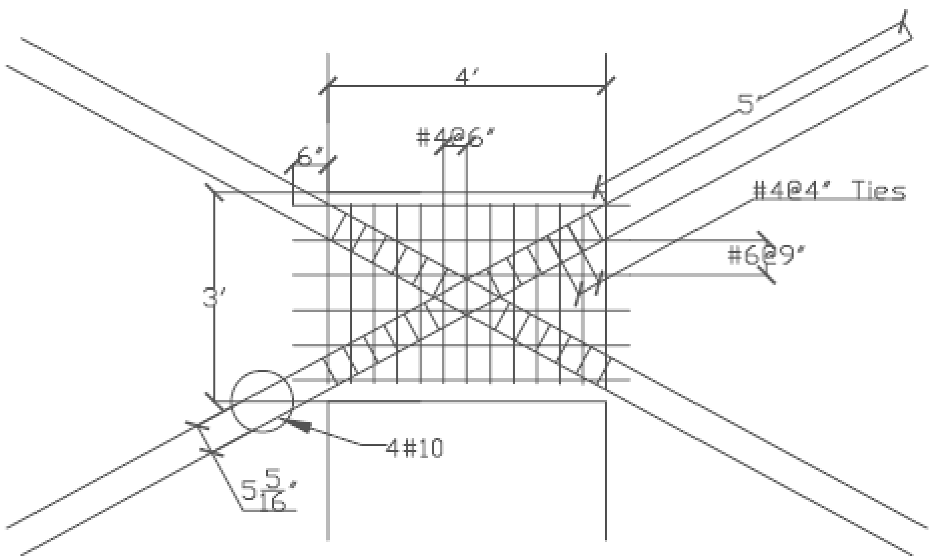
Development Length:

Diagonal Bars must extend a distance equal to 1.5*ld to ensure development of the shear capacity of the beam can be developed. According to Section 1912.2.3:

$$l_d/d_d = \frac{3 * fy * \alpha * \beta * \gamma * \lambda}{40 * \sqrt{f'c}(c/d_b)} = 28d_b$$

For normal weight concrete, all factors are zero but β=1.3
For 6" spacing and 3" cover, c = 3"
For a #10 bar ld = 38" → 1.5*ld = 60". Therefore: **Diagonal Bars must extend 5' into the wall piers on each side of the coupling beam.**

Figure 25: Coupled Beams Diagonal, Vertical and Transverse Reinforcement



IV. 2. v. Shear Wall Design

Method of Design

There are two critical floors that will be thoroughly designed using the results obtained from Table 12 and Table 13. The 1st floor level experiences the largest shear and moment. Above the 9th floor, the design will account for the reduced wall length and will include the axial seismic loads induced by the spandrels. The direction considered is the East-West (EQ X) Seismic Loading. This direction was found to be critical (See Table 13). Even when this wall experiences the least torsion shear force as compared to the North-South Direction (EQ Y), it has the largest direct shear and moment forces because it extends to the 22nd floor.

The method for determining the required reinforcement and detailing for the critical pier AH is the following:

1. The flexural strengths at the 1st and 9th floor are determined. The required reinforcement for the other levels is determined from graphs representing the bar cut-off requirements for flexural strength. A preliminary design for vertical reinforcement and boundary end zones is obtained.
2. The minimum required reinforcement for shear in the piers is determined at each floor level.
3. A ductility check is performed by the method of virtual work to locate probable plastic hinge region by calculating the required nominal shear strength necessary to ensure flexural failure. Changes are made to the flexural strength of the wall piers to ensure a plastic hinge development at the coupling beams and 1st floor level. A final design for vertical reinforcement is drawn.
4. Piers are redesigned for a nominal strength higher than the nominal shear strength capacity associated with flexural failure. This value was determined from the virtual work analysis in Step 3.
5. Boundary Zones at end of the walls and around openings are determined and confinement reinforcement is detailed for compression according to UBC simplified approach section 1921.6.6.4 and checked by a more detailed strain procedure.

Flexural Strength, ΦMn

1st Floor Level

The moment strength of the wall section under consideration will consider axial load contributions from Load Case 1, 0.8D + 1.2E, and vertical reinforcement of the wall web. It is understood that earlier version of the UBC required the wall boundaries to carry all of the moment and gravity forces. However, this practice has resulted in over strength in flexure of the wall, making it more likely to fail in shear.

The preliminary required reinforcement of the wall and boundary zones are approximated by hand assuming the conservative approach that the wall boundaries to carry all of the moment and gravity forces. Following, I developed a spreadsheet to calculate the moment strength of a wall section depending for a given factored axial force, assuming that all vertical reinforcement both in the end zones and the web yield. The spreadsheet calculates the flexural strength for a given axial load and reinforcement layout. It will also display the neutral axis depth, c (in), and the nominal axial strength under no eccentricity, Po. It can also be used to calculate the probable flexural strength; M_{pr}. All these values will become important later in the design for the determination of plastic hinge regions, flexural shear strengths, and boundary zone detailing. The spreadsheet assumes that the strain in the concrete is 0.003 and the strain in the steel varies linearly from the neutral axis depth to a value not greater than 0.002. The Φ factor for a tension controlled members is 0.9. For compression controlled members, Φ is calculated to vary linearly from 0.7 to 0.9 as Pu approaches 0.1*f'c*Acv, according to section 1909.3.2.2, but not less than 0.7.For the current design, f'c = 5 ksi, and fy = 60 ksi (See Figure 27 below). Finally, the combined axial and flexural strengths obtained from the spreadsheet are verified using PCA Col for the designed wall section.

<u>Reinforcement Requirements:</u>

1. Maximum bar size limited to #11 for ease of lap splices.
2. Maximum spacing at boundary zones < 12"
3. Maximum spacing in zones other than boundary zones, follow CRSI recommendations for spacing s, > 6*d_{bar}. Therefore: for #11, s_{min} = 9 in. For # 9,

Table 16: Minimum Bar Spacing in Wall Web

Bar Size	**Smin, in**
#11	9
#10	8
#9	7
#8	6

4. Minimum reinforcement ratio for both horizontal and vertical reinforcement is ρ_{min} = 0.0025. For a 24" wall, As_{min} = 0.0025*(12*24) = 0.72in^2 = **#8 @12" each face (E.F.)**
5. Maximum reinforcement ratio ρ_{max} = 0.08.

Summary of the Factored Loads:

From Table 13, the worst case factored loads at the first level are from loading in the East-West Direction.

Table 17: Summary of Factored Loads affecting Shear Walls at 1st Level East-West Direction

Factored Load		0.8D + 1.2E
Pu, k	**Mu, ft-kips**	**Vu, kips**
2015	**144233**	**1331**

Preliminary Reinforcement and Boundary End Zones:

$L = 33\ ft$
$d = 0.8*L = 26.4\ ft$
$P_{eff} = \frac{Mu}{d} = \frac{144233}{26.4} = -5463.37 kips$
$P_{net} = -5463.37 + 2057 = -3406.37 kips = Asfy$
$As > 56.77\ in^2$
Assuming $\rho_{max} = 0.08$,
Boundary End Zone, $> \frac{24"}{56.77*.08} > 5.28 ft$ on each side of the wall
Using a Boundary End Zone of 60 in. would require:

3 layers of #11 @ 6” → As = 46.8 in^2

In the web, start with minimum reinforcement, #8 @ 12” each face.

Minimum reinforcement of #8 @ 12” on the web did not provide sufficient flexural strength. Final iteration resulted in the following requirement of reinforcement:
Boundary End Zone: B.E = 6 ft with 3 layers of #11 @ 6 in
Web reinforcement: #10 @ 9 in o.c.

Graphically,

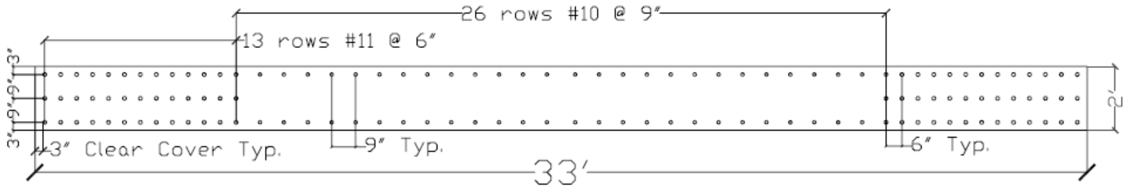


Figure 26: Wall Cross Section at 1st Floor Showing Required Reinforcement for Flexural Strength

The copy of the spreadsheet created to compute ΦMn as a function of the axial force is presented below. Following is a PCA Column output for verification of the designed wall section showing the factored load in the interaction diagram. The results match proving that the design meets the required flexural strength requirements.

Figure 27: Flexural Strength of Shear Wall at 1st Level using Strain Compatibility

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job:	EQX (E-W)	Date:	Feb. 06
Location:	1st Level		
Pier:	AH	Engineer:	Lourdes Diaz
Length:	33'		

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	396.00 in	
b =	24.00 in	
Ag =	9504.00 in^2	
x =	75.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	198.00 in	from extreme compression fiber
Strain =	-0.0030	Ultimate concrete strain (- compression)
c =	83.50 in	
β =	0.80	
ρ total =	0.0195	
Po=	43127 kips	

Governing Equation: 0.8D + 1.2E
Pu, kips = 2015.00
Mu, ft-kips = 144233.00
phi = 0.82

Reinforcement		cover	3.00			#/row	Area / Bar	X
Boundary rows	13	spacing	6.00	Bar Size	11	3	1.56	75.00
Internal rows	26	spacing	9.00	Bar Size	10	2	1.27	196.50
Total Rows	52							393.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	83.50	-0.00300	5.00		0.00	6813.60	0.00	1121518.56
As 1	3.00	-0.00289	60.00	4.68	0.00	280.80	0.00	54756.00
As 2	9.00	-0.00268	60.00	4.68	0.00	280.80	0.00	53071.20
As 3	15.00	-0.00246	60.00	4.68	0.00	280.80	0.00	51386.40
As 4	21.00	-0.00225	60.00	4.68	0.00	280.80	0.00	49701.60
As 5	27.00	-0.00203	60.00	4.68	0.00	280.80	0.00	48016.80
As 6	33.00	-0.00181	52.62	4.68	0.00	246.25	0.00	40630.67
As 7	39.00	-0.00160	46.37	4.68	0.00	216.99	0.00	34501.32
As 8	45.00	-0.00138	40.11	4.68	0.00	187.73	0.00	28723.07
As 9	51.00	-0.00117	33.86	4.68	0.00	158.48	0.00	23295.89
As 10	57.00	-0.00095	27.61	4.68	0.00	129.22	0.00	18219.80
As 11	63.00	-0.00074	21.36	4.68	0.00	99.96	0.00	13494.79
As 12	69.00	-0.00052	15.11	4.68	0.00	70.70	0.00	9120.87
As 13	75.00	-0.00031	8.86	4.68	0.00	41.45	0.00	5098.03
As 14	84.00	0.00002	0.52	2.54	1.32	0.00	-150.85	0.00
As 15	93.00	0.00034	9.90	2.54	25.14	0.00	-2639.85	0.00
As 16	102.00	0.00066	19.28	2.54	48.96	0.00	-4700.13	0.00
As 17	111.00	0.00099	28.65	2.54	72.78	0.00	-6331.67	0.00
As 18	120.00	0.00131	38.03	2.54	96.60	0.00	-7534.49	0.00
As 19	129.00	0.00163	47.41	2.54	120.41	0.00	-8308.58	0.00
As 20	138.00	0.00196	56.78	2.54	144.23	0.00	-8653.95	0.00
As 21	147.00	0.00228	60.00	2.54	152.40	0.00	-7772.40	0.00
As 22	156.00	0.00260	60.00	2.54	152.40	0.00	-6400.80	0.00
As 23	165.00	0.00293	60.00	2.54	152.40	0.00	-5029.20	0.00
As 24	174.00	0.00325	60.00	2.54	152.40	0.00	-3657.60	0.00
As 25	183.00	0.00357	60.00	2.54	152.40	0.00	-2286.00	0.00
As 26	192.00	0.00390	60.00	2.54	152.40	0.00	-914.40	0.00
As 27	201.00	0.00422	60.00	2.54	152.40	0.00	457.20	0.00
As 28	210.00	0.00454	60.00	2.54	152.40	0.00	1828.80	0.00
As 29	219.00	0.00487	60.00	2.54	152.40	0.00	3200.40	0.00
As 30	228.00	0.00519	60.00	2.54	152.40	0.00	4572.00	0.00
As 31	237.00	0.00551	60.00	2.54	152.40	0.00	5943.60	0.00
As 32	246.00	0.00584	60.00	2.54	152.40	0.00	7315.20	0.00
As 33	255.00	0.00616	60.00	2.54	152.40	0.00	8686.80	0.00
As 34	264.00	0.00649	60.00	2.54	152.40	0.00	10058.40	0.00
As 35	273.00	0.00681	60.00	2.54	152.40	0.00	11430.00	0.00
As 36	282.00	0.00713	60.00	2.54	152.40	0.00	12801.60	0.00
As 37	291.00	0.00746	60.00	2.54	152.40	0.00	14173.20	0.00
As 38	300.00	0.00778	60.00	2.54	152.40	0.00	15544.80	0.00
As 39	309.00	0.00810	60.00	2.54	152.40	0.00	16916.40	0.00
As 40	318.00	0.00843	60.00	2.54	152.40	0.00	18288.00	0.00
As 41	324.00	0.00864	60.00	4.68	280.80	0.00	35380.80	0.00
As 42	330.00	0.00886	60.00	4.68	280.80	0.00	37065.60	0.00
As 43	336.00	0.00907	60.00	4.68	280.80	0.00	38750.40	0.00
As 44	342.00	0.00929	60.00	4.68	280.80	0.00	40435.20	0.00
As 45	348.00	0.00950	60.00	4.68	280.80	0.00	42120.00	0.00
As 46	354.00	0.00972	60.00	4.68	280.80	0.00	43804.80	0.00
As 47	360.00	0.00993	60.00	4.68	280.80	0.00	45489.60	0.00
As 48	366.00	0.01015	60.00	4.68	280.80	0.00	47174.40	0.00
As 49	372.00	0.01037	60.00	4.68	280.80	0.00	48859.20	0.00
As 50	378.00	0.01058	60.00	4.68	280.80	0.00	50544.00	0.00
As 51	384.00	0.01080	60.00	4.68	280.80	0.00	52228.80	0.00
As 52	390.00	0.01101	60.00	4.68	280.80	0.00	53913.60	0.00
					6927.04	9368.38	602602.88	1551535.01
				Pn =	2471.81			
					9398.85	9368.38		

Mn = 179511.49 ft-kip

phi M =	146336.62 ft-kips	>	144233.00
phi P =	2015.00 kips	>	2015.00

PCA Column Output for the same wall and reinforcement shows that the factored load is within the interaction curve and therefore acceptable for design:

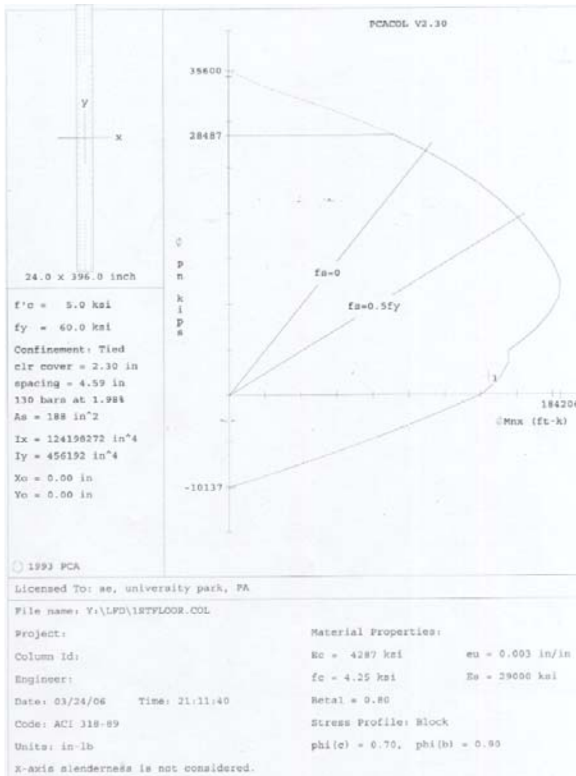


Figure 28: PCA Column Output Flexural Strength at 1st Level

<u>Lower Levels, 2nd – 9th</u>

<u>Bar and Cutt-Offs Requirements:</u>

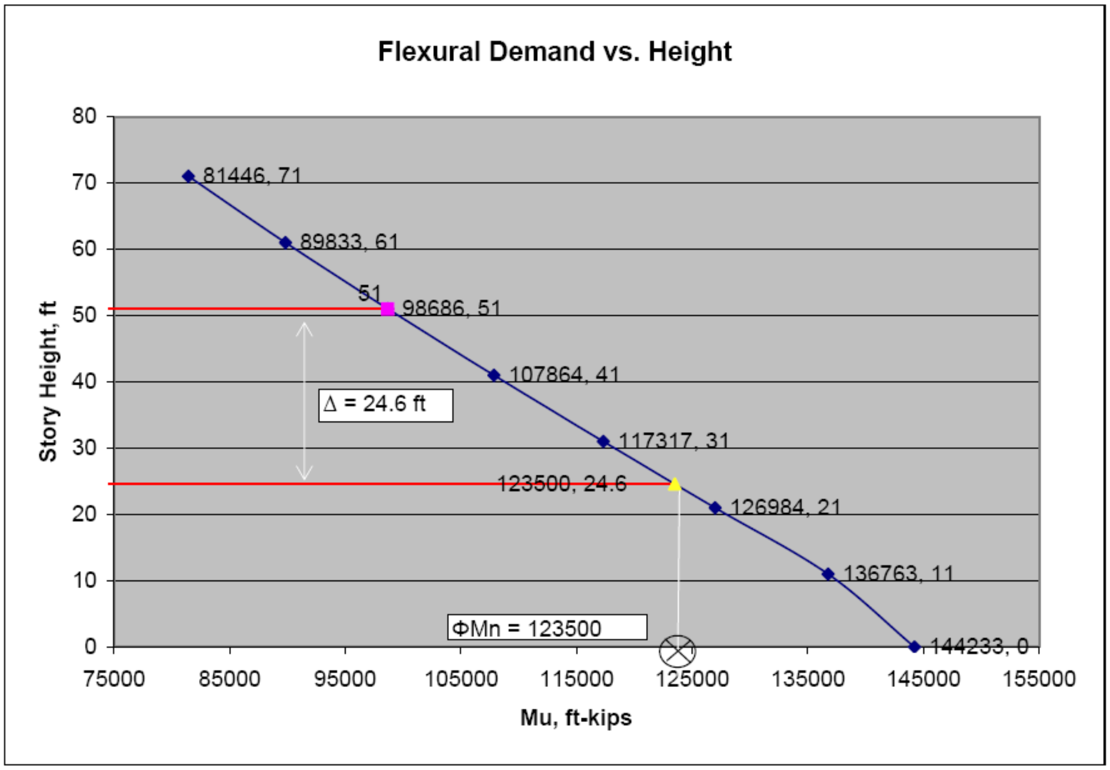
The design for flexural strength above the first level is governed by the requirements set forth by Code Section 1912.10.3 which states that "reinforcement shall extend beyond the point at which it is no longer required to resist flexure for a distance equal to the *effective depth* of the member or 12*d_b, whichever is greater.

Applying the bar cut-off requirements, the flexural strength of the wall will be reduced once before the 9th level, where the transition to the apartment floors occurs. (Recall that above the 9th level the design of the walls includes consideration of the coupling beams and change is stiffness; these walls will be design in the following section.)

Since there is an approximate linear variation in the moment demand of the wall, the required moment strength at the cut-off point can be represented graphically.

For a length, L = 33 ft, the required extension of flexural reinforcement is the effective depth, which can be approximated as 0.8*L = 26.4 ft.

Figure 29: Flexure Demand Requirements for Bar Cut-Off, Below 9th Level

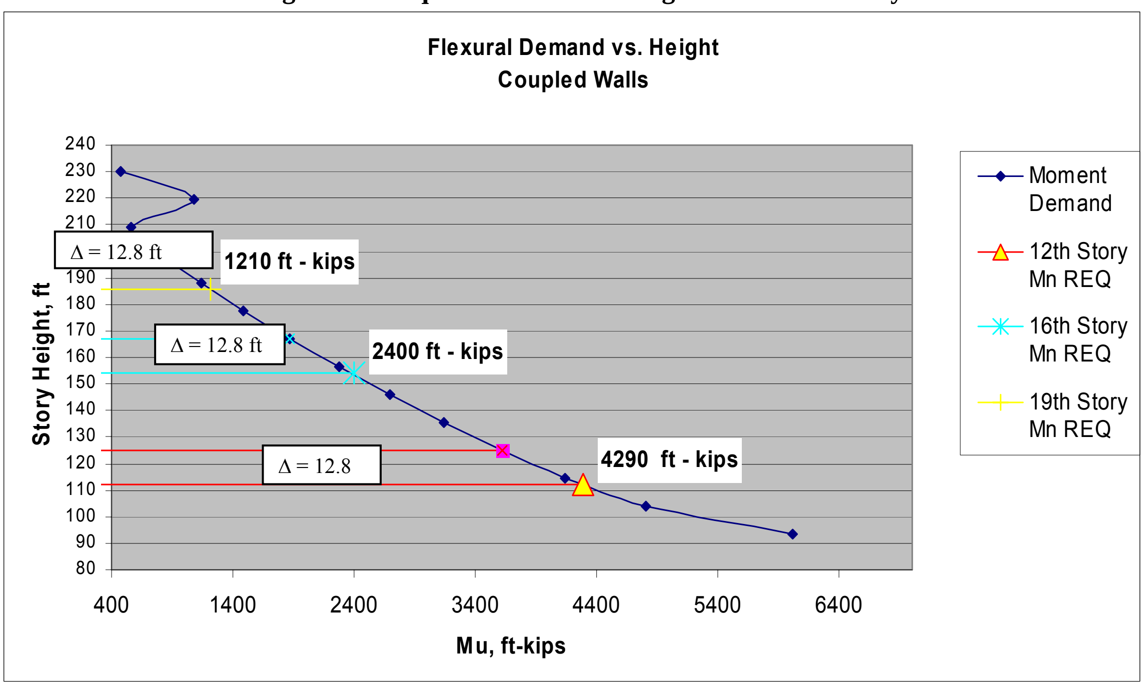


From Figure 29 above, the required ΦMn at the 6th level = 123500 ft-kips. The required reinforcement is selected to provide the required flexural strength and the layout is selected to reduce the boundary end zones length for a smoother transition into the reduced length of the coupled wall in and at above the 9th level. For these walls, 13 and 16 ft in length, the maximum practical boundary zone is 3 ft (0.15*L_w) on each side. The boundary zone in the first 5 levels is at 6 ft. At the 6th level, maintaining the same spacing and bar size a boundary end zone of 3' provides a flexural strength ΦMn = 124251 ft – kips > Mu = 1234500 ft-kips required by ACI for cut-off requirements (See **Error! Reference source not found.**).

Upper Levels, 9th – 22nd

Similar bar cut-off requirement apply to the coupled walls above the 9th level. For the 16 ft walls, the length that the reinforcement must extend past the point where it is required for strength is 0.8*L or 12.8 ft. It was decided that for ease of constructions and sequencing, the cut-off will be designed at the same levels where the spandrel beams are change diagonal reinforcement requirements. Following, the most efficient design leads to grouping the walls at every 4 levels. This results in changes in reinforcement above levels 12th, 16th, and 19th.

Figure 30: Required Flexural Strengths at Cut-Off Story Levels



The piers forming the coupled walls will experience both tension and compression as a result of the seismic axial load, P_E, acting thought the coupling beam to resist overturning moment. Therefore, these walls need to be checked both as tension and compression controlled sections. Load case 1, 0.8D + 1.2E results in a tension controlled section. Load case 2, 1.42D + 1.1L + 1.2E, leads to compression controlled sections. The following is an example of the design performed for the 12th story pier following the cut-off required flexural strength and the combined axial force due to both load cases (Figure 31).

Figure 31: Example of 12th Story Pier Combined Flexural and Axial Strength of Coupled Wall for Tension and Compression

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: 12th Level **Tension Controlled**
Pier: AH Engineer: Lourdes Diaz
Length: 16'

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in²	
x =	21.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in	from extreme compression fiber
Strain =	-0.0030	Ultimate concrete strain (- compression)
c =	5.32 in	
β =	0.80	
ρ total =	0.0094	

Governing Equation: 0.8D + 1.2E
Pu, kips = -1538.00
Mu, ft-kips = 4290.00
phi = 0.90

Reinforcement						#/row	Area / Bar	X
		cover	3.00					
Boundary rows	4	spacing	6.00	Bar Size	9	2	1.00	21.00
Internal rows	11	spacing	12.00	Bar Size	9	2	1.00	93.00
Total Rows	19							186.00

Layer	y,in	Strain	c or fs, ksi	As, in²	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	5.32	-0.00300	5.00		0.00	434.11	0.00	40750.96
As 1	3.00	-0.00131	37.94	4.68	0.00	177.56	0.00	16512.94
As 2	9.00	0.00208	60.00	2.00	120.00	0.00	-10440.00	0.00
As 3	15.00	0.00546	60.00	2.00	120.00	0.00	-9720.00	0.00
As 4	21.00	0.00884	60.00	2.00	120.00	0.00	-9000.00	0.00
As 5	33.00	0.01561	60.00	2.00	120.00	0.00	-7560.00	0.00
As 6	45.00	0.02238	60.00	2.00	120.00	0.00	-6120.00	0.00
As 7	57.00	0.02914	60.00	2.00	120.00	0.00	-4680.00	0.00
As 8	69.00	0.03591	60.00	2.00	120.00	0.00	-3240.00	0.00
As 9	81.00	0.04268	60.00	2.00	120.00	0.00	-1800.00	0.00
As 10	93.00	0.04944	60.00	2.00	120.00	0.00	-360.00	0.00
As 11	105.00	0.05621	60.00	2.00	120.00	0.00	1080.00	0.00
As 12	117.00	0.06298	60.00	2.00	120.00	0.00	2520.00	0.00
As 13	129.00	0.06974	60.00	2.00	120.00	0.00	3960.00	0.00
As 14	141.00	0.07651	60.00	2.00	120.00	0.00	5400.00	0.00
As 15	153.00	0.08328	60.00	2.00	120.00	0.00	6840.00	0.00
As 16	165.00	0.09005	60.00	2.00	120.00	0.00	8280.00	0.00
As 17	171.00	0.09343	60.00	2.00	120.00	0.00	9000.00	0.00
As 18	177.00	0.09681	60.00	2.00	120.00	0.00	9720.00	0.00
As 19	183.00	0.10020	60.00	4.68	280.80	0.00	24429.60	0.00
					2320.80	611.67	18309.60	57263.90
				Pn =	-1708.89			
					611.91	611.67		

Mn = 6297.79 ft-kip
phi M = 5668.01 ft-kips > 4290.00
phi P = -1538.00 kips > -1538.00

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: 12th Level **Compression Controlled**
Pier: AH Engineer: Lourdes Diaz
Length: 16'

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in²	
x =	21.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in	from extreme compression fiber
Strain =	-0.0030	Ultimate concrete stra (- compression)
c =	59.20 in	
β =	0.80	
ρ total =	0.0094	

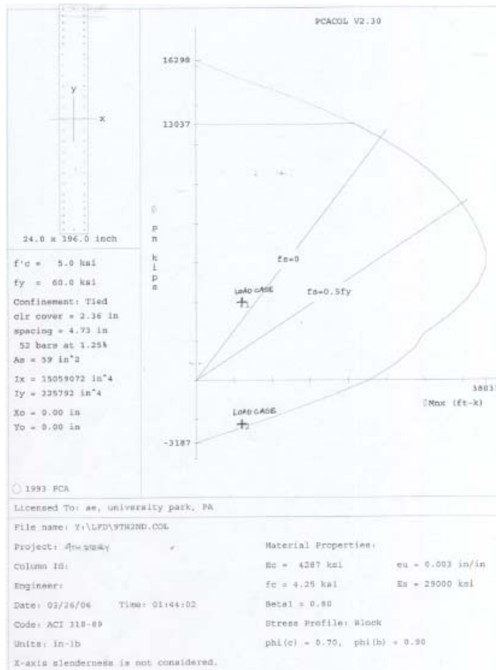
Governing Equation: 0.8D + 1.2E
Pu, kips = 2910.00
Mu, ft-kips 4290.00
phi = 0.70

Reinforcement						#/row	Area / Bar	X
		cover	3.00					
Boundary rows	4	spacing	6.00	Bar Size	9	2	1.00	21.00
Internal rows	11	spacing	12.00	Bar Size	9	2	1.00	93.00
Total Rows	19							186.00

Layer	y,in	Strain	c or fs, ksi	As, in²	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	59.20	-0.00300	5.00		0.00	4830.72	0.00	349357.67
As 1	3.00	-0.00131	37.94	4.68	0.00	177.56	0.00	16512.94
As 2	9.00	0.00208	60.00	2.00	120.00	0.00	-10440.00	0.00
As 3	15.00	0.00546	60.00	2.00	120.00	0.00	-9720.00	0.00
As 4	21.00	0.00884	60.00	2.00	120.00	0.00	-9000.00	0.00
As 5	33.00	0.01561	60.00	2.00	120.00	0.00	-7560.00	0.00
As 6	45.00	0.02238	60.00	2.00	120.00	0.00	-6120.00	0.00
As 7	57.00	0.02914	60.00	2.00	120.00	0.00	-4680.00	0.00
As 8	69.00	0.03591	60.00	2.00	120.00	0.00	-3240.00	0.00
As 9	81.00	0.04268	60.00	2.00	120.00	0.00	-1800.00	0.00
As 10	93.00	0.04944	60.00	2.00	120.00	0.00	-360.00	0.00
As 11	105.00	0.05621	60.00	2.00	120.00	0.00	1080.00	0.00
As 12	117.00	0.06298	60.00	2.00	120.00	0.00	2520.00	0.00
As 13	129.00	0.06974	60.00	2.00	120.00	0.00	3960.00	0.00
As 14	141.00	0.07651	60.00	2.00	120.00	0.00	5400.00	0.00
As 15	153.00	0.08328	60.00	2.00	120.00	0.00	6840.00	0.00
As 16	165.00	0.09005	60.00	2.00	120.00	0.00	8280.00	0.00
As 17	171.00	0.09343	60.00	2.00	120.00	0.00	9000.00	0.00
As 18	177.00	0.09681	60.00	2.00	120.00	0.00	9720.00	0.00
As 19	183.00	0.10020	60.00	4.68	280.80	0.00	24429.60	0.00
					2320.80	5008.28	18309.60	365870.61
				Pn =	4157.14			
					6477.94	5008.28		

Mn = 32015.02 ft-kip
phi M = 22410.51 ft-kips > 4290.00
phi P = 2910.00 kips > 2910.00

The results where verified with PCA Col and found to be accurate:



Preliminary Design:

Using the results obtained from RAM, ETABS and torsion analysis along with the spreadsheet, cut-ff requirements and using PCA Column for verification as explained above, the following preliminary design is drawn:

Table 18: Preliminary Bar Sizes for Wall Piers

Summary Flexural Strenght: Preliminary Design

					Boundary End Zone				Wall Web						
Story Level	L, ft	Pu	$Mu_{,SEISMIC}$	$Mu_{,CUTOFF}$	# Layers	# Rows	Bar Size	Spacing	# Layers	# Rows	Bar Size	Spacing	ΦMn	Mpr	P_o
1st - 4th	33	-2015	144233	144233	3	13	11	6	2	26	10	9	146337	224387	43127
5th - 8th	33	-1583	98686	123500	3	7	11	6	2	34	9	9	132047	195107	41429
9th - 12th	16	2258	6009	8056	2	6	10	6	2	13	9	9	7320.87	48408	19500
13th - 16th	16	1285	3144	4290	2	4	9	6	2	11	9	9	5668	38695	18701
17th - 22th	16	483	1488	2400	2	0	8	6	2	16	8	12	6578	24746	17919

Minimum Shear Capacity, ΦVn

UBC Section 1921.6.5 specifies the shear strength for building subjected to seismic forces. For an hw/le > 2.0,

$$\Phi Vn = \Phi A_{cv}(2\sqrt{f'c} + \rho_n f_y)$$

Where, Φ = 0.6 for nominal shear strength less than the shear strength corresponding to the development of the nominal flexural strength.

For the 33' wall $\Phi Vn = 806.44 + 342144\rho_n$ (kips)

For the 16' walls above the 9th level, $\Phi Vn = 391.00 + 165888\rho_n$ (kips)

Other requirements include:

Section 1921.6.2.1, $\rho_{min} > 0.0025$

Section 1921.6.5.6, Vn < $8A_{cv}\sqrt{f'c}$

The results are tabulated in Table 19 below. All walls are required to have a minimum reinforcement of #6 @ 12" on each face.

Table 19: Minimum Reinforcement for Shear Strength

Minimum Reinforcement for Shear Strenght

Units:	Kips-ft	
Direction:	EQ X	
Pier Label:	AH	
ρmin =	0.0025	
hw =	230	ft
lw =	33	ft
hw/lw =	6.97	
Φ =	0.6	
Vn <	5376	(33' length)
Vn <	2606	(16' length)

Level	Vu, kips	Length, ft	ρ_{REQ}	ρ_{USED}	Horizontal Reinforcement	ΦVn, kips
22	185	16	-0.0012	0.00305	#6 @ 12" E.F.	897
21	232	16	-0.0010	0.00305	#6 @ 12" E.F.	897
20	70	16	-0.0019	0.00305	#6 @ 12" E.F.	897
19	171	16	-0.0013	0.00305	#6 @ 12" E.F.	897
18	227	16	-0.0010	0.00305	#6 @ 12" E.F.	897
17	273	16	-0.0007	0.00305	#6 @ 12" E.F.	897
16	313	16	-0.0005	0.00305	#6 @ 12" E.F.	897
15	350	16	-0.0002	0.00305	#6 @ 12" E.F.	897
14	383	16	0.0000	0.00305	#6 @ 12" E.F.	897
13	414	16	0.0001	0.00305	#6 @ 12" E.F.	897
12	443	16	0.0003	0.00305	#6 @ 12" E.F.	897
11	471	16	0.0005	0.00305	#6 @ 12" E.F.	897
10	500	16	0.0007	0.00305	#6 @ 12" E.F.	897
9	**528**	**16**	**0.0008**	**0.00305**	**#6 @ 12" E.F.**	**897**
8	916	33	0.0003	0.00305	#6 @ 12" E.F.	1850
7	1013	33	0.0006	0.00305	#6 @ 12" E.F.	1850
6	1107	33	0.0009	0.00305	#6 @ 12" E.F.	1850
5	1180	33	0.0011	0.00305	#6 @ 12" E.F.	1850
4	1239	33	0.0013	0.00305	#6 @ 12" E.F.	1850
3	1285	33	0.0014	0.00305	#6 @ 12" E.F.	1850
2	1313	33	0.0015	0.00305	#6 @ 12" E.F.	1850
1	**1337**	**33**	**0.0016**	**0.00305**	**#6 @ 12" E.F.**	**1850**

Ductility and Plastic Hinge Region

The preferred behavior of a wall occurs when the plastic hinges occur at the bas of the wall piers and in the coupling beams. This provides a mean by which seismic energy is dissipated over the entire height of the wall as the coupling beam undergoes inelastic deformations.

For the selected wall design, there are two possible failure mechanisms: one where the plastic hinge occurs at the first level and the other where it occurs and the 9th level where the walls are coupled. The former is the preferred mechanism. If the latter occurred, the wall could experience too much deformation as the rotation in that level, θ, increases to meet the design roof story displacement. To ensure that the plastic hinge occurs at the 1st story level and not in the 9th, a virtual work analysis was used to evaluate the required flexural strength, as function of M_{pr}, at the 1st and 9th floor levels.

The plastic hinge length, lp, is calculated according to Section 1921.6.6.5 as $0.5l_w$= 16.5'. The external work is calculated by assuming a linear increase in the plastic lateral story displacement and setting the total displacement at the roof level, $\Delta_{roof} = 1.0$.

The *external work* per story can be calculated to equal

$$f_i \Delta_i = \frac{V_i}{V_{BASE}} \Delta_i .$$

The *total external work*

$$= \sum f_i \Delta_i .$$

The *internal work* results from the plastic rotation of all coupling beams and the piers at the story being evaluated.
The rotation angles of the wall pier

$$\theta = \frac{\Delta_{roof}}{h - (l_p / 2)} .$$

The internal work associated with the yielding of the pier at the base
$= \theta * M_{pr}$
Where, Mpr is the maximum probable flexural strength defined by Section 1921.0. It is calculated assuming the worst factored axial compression occurring in the member and:
$f_s = 1.25 f_y$
$\Phi = 1.0$

The rotation of the coupling beam can be calculated as

$$\theta_{cb} = \theta \frac{l_{pier}}{l_{cb}}.$$

Where,

l_{pier} = clear distance between the centroids of the pier section.
l_{cb} = clear distance of the coupling beam.

It follows, the internal work associated with the plastic yielding of the coupling beams

$$= \sum \theta_{cb} M_{pr-cb} * 2$$
$$= \sum \theta_{cb} (1.25 M_n) * 2$$
$$= \sum \theta \frac{l_{pier}}{l_{cb}} 1.25 (V_n \frac{l_{cb}}{2}) * 2$$
$$= \sum \theta * 1.25 * V_n * l_{pier}$$

A copy of the results for the virtual work analysis is provided in Figure 32 below. The first round of results concluded that shear strength corresponding to flexural yielding at the 9th level was lower than that at the 1st level. This would result in a plastic hinge formation at the 9th story and not at the 1st story. The iteration was repeated for different flexural strengths of the wall section at the 9th level until the shear value in this level was higher than at the lower levels.

Figure 32: Final Virtual Work Formulation to Determine Plastic Hinge Location

Required Shear Strenght to Develop Plastic Hinge at 1st Story Level
Ip = 16.5 ft
Pier: AH
V = 1026 kips

Story Level	hi	hi - lp/2	Δi, ft	ƒi / V_{BASE}	Work / V
22	230	221.75	1	0.271	0.2710
21	219.75	211.5	0.954	0.067	0.0641
20	209.5	201.25	0.908	-0.230	-0.2088
19	199.25	191	0.861	0.149	0.1284
18	189	180.75	0.815	0.079	0.0644
17	178.75	170.5	0.769	0.068	0.0525
16	168.5	160.25	0.723	0.058	0.0423
15	158.25	150	0.676	0.056	0.0376
14	148	139.75	0.630	0.048	0.0301
13	137.75	129.5	0.584	0.046	0.0268
12	127.5	119.25	0.538	0.042	0.0225
11	117.25	109	0.492	0.037	0.0182
10	107	98.75	0.445	0.067	0.0299
9	96.75	88.5	0.399	0.032	0.0128
8	86.5	78.25	0.353	0.031	0.0110
7	71.5	63.25	0.285	0.038	0.0108
6	61.5	53.25	0.240	0.042	0.0101
5	51.5	43.25	0.195	0.029	0.0057
4	41.5	33.25	0.150	0.025	0.0038
3	31.5	23.25	0.105	0.019	0.0020
2	21.5	13.25	0.060	0.010	0.0006
1	11.5	3.25	0.015	0.015	0.0002
				1	**0.6361**

Required Shear Strenght to Develop Plastic Hinge at 9 th Story Level
Ip = 8 ft
Pier: AH
V = 811 kips

Story Leve	hi	hi - lp/2	Δi, ft	ƒi / VBASE	Work / V
22	143.45	139.45	1.0000	0.3428	0.3428
21	133.2	129.2	0.9265	0.0851	0.0788
20	122.95	118.95	0.8530	-0.2910	-0.2482
19	112.7	108.7	0.7795	0.1887	0.1471
18	102.45	98.45	0.7060	0.0999	0.0705
17	92.2	88.2	0.6325	0.0863	0.0546
16	81.95	77.95	0.5590	0.0740	0.0414
15	71.7	67.7	0.4855	0.0703	0.0341
14	61.45	57.45	0.4120	0.0604	0.0249
13	51.2	47.2	0.3385	0.0580	0.0196
12	40.95	36.95	0.2650	0.0530	0.0140
11	30.7	26.7	0.1915	0.0469	0.0090
10	20.45	16.45	0.1180	0.0851	0.0100
9	10.2	6.2	0.0445	0.0407	0.0018
				1.0000	**0.6004**

θ = 1.0 / 221.75 = 0.00450958

Internal Work , Coupling Beams			
Story	$1.25V_n$	l_c, ft	Work (ft-kip)
22	210.69	18.5	17.58
21	210.69	18.5	17.58
20	210.69	18.5	17.58
19	210.69	18.5	17.58
18	254.19	18.5	21.21
17	254.19	18.5	21.21
16	381	18.5	31.79
15	381	18.5	31.79
14	381	18.5	31.79
13	381	18.5	31.79
12	468	18.5	39.04
11	468	18.5	39.04
10	468	18.5	39.04
9	468	18.5	39.04
			396.04

θ = 1.0 / 139.45 = 0.00717103

Internal Work , Coupling Beams			
Story	1.25Vn	Ic, ft	Work (ft-kip)
22	210.69	18.5	27.95
21	210.69	18.5	27.95
20	210.69	18.5	27.95
19	210.69	18.5	27.95
18	254.19	18.5	33.72
17	254.19	18.5	33.72
16	381.00	18.5	50.54
15	381.00	18.5	50.54
14	381.00	18.5	50.54
13	381.00	18.5	50.54
12	468.00	18.5	62.09
11	468.00	18.5	62.09
10	468.00	18.5	62.09
9	468.00	18.5	62.09
			629.77

Internal Work, Piers			
Base	Mpr, (k -ft)		Work,(k -ft)
AH	224387		**1011.89**

Internal Work, Piers			
Level	Mpr, (k -ft)		Work,(k -ft)
9th A1	56800		407.31
9th A2	40780		292.43
			699.75

V = (1066.90 + 396.04)/.6361 = **2213** kips < V = (585.94 + 629.77)/.6004 = **2214** kips

To ensure the formation of the plastic hinge at the 1st story level the M_{pr} of the 9th story level had to be increased considerably. The final design called for the 9th story wall section to have an M_{pr} of 56800 k-ft. The preliminary design provided an M_{pr} of 48408 ft-kip at this story level. To increase the flexural strength of the wall, the original cut-off of 2 rows of #10 @ 6" (B.E = 32") with #9@9" in the web is no longer permitted. The wall

section at the 9th floor is now required to maintain the internal row of reinforcement and #11 bar size in the boundary zone.

<u>3 rows #11 @ 6" (B.E = 38") with #9 @ 9" in the web.</u>

The validity of the design is shown by Figure 33 below.

Figure 33: Probable Flexural Strength at 9th Level

<u>Strain Compatibility for Shear Walls</u>
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: 9th Level **Probable Flexural Strength**
Pier: AH Engineer: Lourdes Diaz
Length: 16'

General Information
f'c = 5.00 ksi
fy = 75.00 ksi
l = 192.00 in
b = 24.00 in
Ag = 4608.00 in^2
x = 32.00 in Compression Zone Boundary Element Length
y = 24.00 in Compression Zone Boundary Element Width
cg = 96.00 in from extreme compression fiber
Strain = -0.0030 Ultimate concrete strain (- compression)
c = 63.00 in
β = 0.80
ρ total = 0.0172

Governing Equation: 1.48D +1.2E + 1.1L
Pu, kips = 3971.00
Mu, ft-kips = 6009.00
phi = 1.00

Reinforcement		cover	2.00			#/row	Area / Bar	X
Boundary rows	6	spacing	6.00	Bar Size	11	3	1.56	32.00
Internal rows	13	spacing	9.00	Bar Size	9	2	1.00	95.00
Total Rows	25							190.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	63.00	-0.00300	5.00		0.00	5140.80	0.00	363968.64
As 1	2.00	-0.00290	60.00	4.68	0.00	280.80	0.00	26395.20
As 2	8.00	-0.00262	60.00	4.68	0.00	280.80	0.00	24710.40
As 3	14.00	-0.00233	75.00	4.68	0.00	351.00	0.00	28782.00
As 4	20.00	-0.00205	75.00	4.68	0.00	351.00	0.00	26676.00
As 5	26.00	-0.00176	51.10	4.68	0.00	239.13	0.00	16738.80
As 6	32.00	-0.00148	42.81	4.68	0.00	200.35	0.00	12822.31
As 7	41.00	-0.00105	30.38	2.00	0.00	60.76	0.00	3341.90
As 8	50.00	-0.00062	17.95	2.00	0.00	35.90	0.00	1651.62
As 9	59.00	-0.00019	5.52	2.00	0.00	11.05	0.00	408.76
As 10	68.00	0.00024	6.90	2.00	13.81	0.00	-386.67	0.00
As 11	77.00	0.00067	19.33	2.00	38.67	0.00	-734.67	0.00
As 12	86.00	0.00110	31.76	2.00	63.52	0.00	-635.24	0.00
As 13	95.00	0.00152	44.19	2.00	88.38	0.00	-88.38	0.00
As 14	104.00	0.00195	56.62	2.00	113.24	0.00	905.90	0.00
As 15	113.00	0.00238	75.00	2.00	150.00	0.00	2550.00	0.00
As 16	122.00	0.00281	75.00	2.00	150.00	0.00	3900.00	0.00
As 17	131.00	0.00324	75.00	2.00	150.00	0.00	5250.00	0.00
As 18	140.00	0.00367	75.00	2.00	150.00	0.00	6600.00	0.00
As 19	149.00	0.00410	75.00	2.00	150.00	0.00	7950.00	0.00
As 20	158.00	0.00452	75.00	2.00	150.00	0.00	9300.00	0.00
As 21	164.00	0.00481	75.00	4.68	351.00	0.00	23868.00	0.00
As 22	170.00	0.00510	75.00	4.68	351.00	0.00	25974.00	0.00
As 23	176.00	0.00538	75.00	4.68	351.00	0.00	28080.00	0.00
As 24	182.00	0.00567	75.00	4.68	351.00	0.00	30186.00	0.00
As 25	188.00	0.00595	75.00	4.68	351.00	0.00	32292.00	0.00
					2972.62	6951.59	175010.95	505495.63
				Pn =	3971.00			
					6943.62	6951.59		

Mn = 56708.88 ft-kip
phi M = 56708.88 ft-kips > 6009.00
phi P = 3971.00 kips > 3971.00

<u>Strain Compatibility for Shear Walls</u>
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: 9th Level **Probable Flexural Strength**
Pier: AH Engineer: Lourdes Diaz
Length: 13.00

General Information
f'c = 5.00 ksi
fy = 75.00 ksi
l = 156.00 in
b = 24.00 in
Ag = 3744.00 in^2
x = 32.00 in Compression Zone Boundary Element Length
y = 24.00 in Compression Zone Boundary Element Width
cg = 78.00 in from extreme compression fiber
Strain = -0.0030 Ultimate concrete strai (- compression)
c = 58.50 in
β = 0.80
ρ total = 0.0162

Governing Equation: 1.48D +1.2E + 1.1L
Pu, kips = 3971.00
Mu, ft-kips 6009.00
phi = 1.00

Reinforcement		cover	2.00			#/row	Area / Bar	X
Boundary rows	6	spacing	6.00	Bar Size	11	3	1.56	32.00
Internal rows	9	spacing	9.00	Bar Size	9	2	1.00	77.00
Total Rows	21							154.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	58.50	-0.00300	5.00		0.00	4773.60	0.00	260638.56
As 1	2.00	-0.00290	60.00	4.68	0.00	280.80	0.00	26395.20
As 2	8.00	-0.00262	60.00	4.68	0.00	280.80	0.00	24710.40
As 3	14.00	-0.00233	75.00	4.68	0.00	351.00	0.00	28782.00
As 4	20.00	-0.00205	75.00	4.68	0.00	351.00	0.00	26676.00
As 5	26.00	-0.00176	51.10	4.68	0.00	239.13	0.00	16738.80
As 6	32.00	-0.00148	42.81	4.68	0.00	200.35	0.00	12822.31
As 7	41.00	-0.00105	30.38	2.00	0.00	60.76	0.00	3341.90
As 8	50.00	-0.00062	17.95	2.00	0.00	35.90	0.00	1651.62
As 9	59.00	-0.00019	5.52	2.00	0.00	11.05	0.00	408.76
As 10	68.00	0.00024	6.90	2.00	13.81	0.00	-386.67	0.00
As 11	77.00	0.00067	19.33	2.00	38.67	0.00	-734.67	0.00
As 12	86.00	0.00110	31.76	2.00	63.52	0.00	-635.24	0.00
As 13	95.00	0.00152	44.19	2.00	88.38	0.00	-88.38	0.00
As 14	104.00	0.00195	56.62	2.00	113.24	0.00	905.90	0.00
As 15	113.00	0.00238	75.00	2.00	150.00	0.00	2550.00	0.00
As 16	122.00	0.00281	75.00	2.00	150.00	0.00	3900.00	0.00
As 17	131.00	0.00324	75.00	2.00	150.00	0.00	5250.00	0.00
As 18	140.00	0.00367	75.00	2.00	150.00	0.00	6600.00	0.00
As 19	149.00	0.00410	75.00	2.00	150.00	0.00	7950.00	0.00
As 20	158.00	0.00452	75.00	2.00	150.00	0.00	9300.00	0.00
As 21	164.00	0.00481	75.00	4.68	351.00	0.00	23868.00	0.00
					1568.62	6584.39	58478.95	402165.55
				Pn =	3971.00			
					5539.62	6584.39		

Mn = 38387.04 ft-kip
phi M = 38387.04 ft-kips > 6009.00
phi P = 3971.00 kips > 3971.00

Magnified Shear Demand and Amplification Factor

From the previous section, the virtual work analysis required a design shear of 2213 kips to ensure flexural yielding of the wall prior to shear failure (See Figure 32). The actual factored shear was calculated to be 1104 kips (Table 13). The increase in shear demand is a result of modeling the structure for a ductile behavior. The impact of this larger shear requirement at the base can be assumed to vary linearly. Therefore, the magnified shear demand, V_u*, at each story level can be calculated by multiplying the factored shear by a dynamic amplification factor, w_v.

$$V_u * = \varpi_v (M_{pr} / M_u) * V_u$$

Where for building higher than 6 stories, w_v can be approximated as:

$$\varpi_v = 0.85(1.3 + \frac{n}{30}),$$ where n is the number of stories

Following, the amplified shear demand in each story is

$$V_u * = 3.45 * V_u$$

The resulting required horizontal reinforcement at each pier is summarized below. Note that the maximum allowable V_n is not reached. Going back to Section IV.2.v, the design for minimum shear was #6 @ 12" E.F. Ductile behavior will require #8 @ 12: E.F. up to the 14th story.
Note: This design is not required by code. It is recommended in regions of high seismic demand and for those reasons the design has been done and detailed. However, this is only a recommendation.

Table 20: Shear Demand with Dynamic Amplification Factors

Minimum Reinforcement for Shear Strenght

Units:	Kips-ft	
Direction:	EQ X	
Pier Label:	AH	
ρmin =	0.0025	
hw =	230	ft
lw =	33	ft
hw/lw =	6.97	
Φ =	0.85	
Vn <	5376	(33' length)
Vn <	2606	(16' length)

Dynamic Amplification Factor 3.45

Level	Vu, kips	Length, ft	ρ_{REQ}	ρ_{USED}	Horizontal Reinforcement	ΦVn, kips	Ductile Behavior: Vu, kips	Vu*, kips	ρ_n	ρ_{USED}	Horizontal Reinforcemetn	ΦVn, kips	ΦVn/Vu
22	185	16	-0.0012	0.00305	#6 @ 12" E.F.	897	185	638	0.0004	0.00305	#6 @ 12" E.F.	1271	1.99
21	232	16	-0.0010	0.00305	#6 @ 12" E.F.	897	232	799	0.0010	0.00305	#6 @ 12" E.F.	1271	1.59
20	70	16	-0.0019	0.00305	#6 @ 12" E.F.	897	70	242	-0.0013	0.00305	#6 @ 12" E.F.	1271	5.26
19	171	16	-0.0013	0.00305	#6 @ 12" E.F.	897	171	588	0.0001	0.00305	#6 @ 12" E.F.	1271	2.16
18	227	16	-0.0010	0.00305	#6 @ 12" E.F.	897	227	783	0.0010	0.00305	#6 @ 12" E.F.	1271	1.62
17	273	16	-0.0007	0.00305	#6 @ 12" E.F.	897	273	943	0.0017	0.00305	#6 @ 12" E.F.	1271	1.35
16	313	16	-0.0005	0.00305	#6 @ 12" E.F.	897	313	1081	0.0022	0.00305	#6 @ 12" E.F.	1271	1.18
15	350	16	-0.0002	0.00305	#6 @ 12" E.F.	897	350	1207	0.0028	0.00305	#6 @ 12" E.F.	1271	1.05
14	383	16	0.0000	0.00305	#6 @ 12" E.F.	897	383	1322	0.0033	0.00550	**#8 @ 12" E.F.**	**1847**	**1.40**
13	414	16	0.0001	0.00305	#6 @ 12" E.F.	897	414	1429	0.0037	0.00550	**#8 @ 12" E.F.**	**1847**	**1.29**
12	443	16	0.0003	0.00305	#6 @ 12" E.F.	897	443	1527	0.0041	0.00550	**#8 @ 12" E.F.**	**1847**	**1.21**
11	471	16	0.0005	0.00305	#6 @ 12" E.F.	897	471	1625	0.0046	0.00550	**#8 @ 12" E.F.**	**1847**	**1.14**
10	500	16	0.0007	0.00305	#6 @ 12" E.F.	897	500	1724	0.0050	0.00550	**#8 @ 12" E.F.**	**1847**	**1.07**
9	**528**	**16**	**0.0008**	**0.00305**	**#6 @ 12" E.F.**	**897**	528	1822	0.0054	0.00550	**#8 @ 12" E.F.**	**1847**	**1.01**
8	885	33	0.0002	0.00305	#6 @ 12" E.F.	1850	885	3053	0.0039	0.00550	**#8 @ 12" E.F.**	**3808**	**1.25**
7	931	33	0.0004	0.00305	#6 @ 12" E.F.	1850	931	3212	0.0043	0.00550	**#8 @ 12" E.F.**	**3808**	**1.19**
6	983	33	0.0005	0.00305	#6 @ 12" E.F.	1850	983	3390	0.0046	0.00550	**#8 @ 12" E.F.**	**3808**	**1.12**
5	1019	33	0.0006	0.00305	#6 @ 12" E.F.	1850	1019	3516	0.0049	0.00550	**#8 @ 12" E.F.**	**3808**	**1.08**
4	1050	33	0.0007	0.00305	#6 @ 12" E.F.	1850	1050	3622	0.0051	0.00550	**#8 @ 12" E.F.**	**3808**	**1.05**
3	1074	33	0.0008	0.00305	#6 @ 12" E.F.	1850	1074	3705	0.0053	0.00550	**#8 @ 12" E.F.**	**3808**	**1.03**
2	1087	33	0.0008	0.00305	#6 @ 12" E.F.	1850	1087	3749	0.0054	0.00550	**#8 @ 12" E.F.**	**3808**	**1.02**
1	**1104**	**33**	**0.0009**	**0.00305**	**#6 @ 12" E.F.**	**1850**	1104	3808	0.0055	0.00550	**#8 @ 12" E.F.**	**3808**	**1.00**

Boundary Zones

Shear walls that also serve as bearing walls can experience large compressive forces. Because concrete is a brittle material, it will tend to spall and crush under large loads. To ensure the integrity of the concrete, Section 1921.6.6.4 has requirements for confinement of concrete at the ends of walls. Boundary Zones need not be provided if:

1. $P_u < 0.10 A_g f'c$, and
2. $\dfrac{M_u}{l_u V_u} < 1.0$, or
3. $V_u < 3A_{cv}\sqrt{f'_c}$ and $\dfrac{M_u}{l_u V_u} < 3$

<u>Requirements:</u>

Where the conditions below are not met, boundary zone detailing should extend a distance:

$$\frac{B.Z.(ft)}{l_w} = \frac{0.1P_u}{0.2P_o} > 0.15$$

Table 21: Required Boundary Lengths at selected Story Levels

Required Bondary Zones Accordng to UBC Simplied Procedure
f'c = 5 ksi
t = 24 in
B.Z. $_{min}$ = 0.15Lw

Story	Lw, ft	P_{uCASE2}	>0.10f'cAg	Vu, k	Vu > 3Acv√f'c	Mu, ft-kip	Mu/(luVu) > 3	Po	0.35Po	0.15Po	B.Z./Lw	B.Z., in
1st	33	3944	4752	1104	2016	144233	3.96	43127	15094	6469	0.046	60
5th	33	3261	4752	1019	2016	107864	3.21	41429	14500	6214	0.039	60
9th	16	3971	2304	528	978	6009	0.71	203413	71195	30512	0.010	30
12th	16	2910	2304	443	978	3619	0.51	18701	6545	2805	0.078	30
16th	16	1866	2304	313	978	2270	0.45	17919	6272	2688	N.R	N.R.

Other considerations:
The minimum thickness = lu/16 = 15*12/16 = 12" < 24"

The area defined as pertaining to the boundary zone should be confined with hoops or crossties having a minimum area:

$$A_{sh} = 0.09sh_c f'_c / f_y$$

Where,

s = spacing, taken as not less than the greater of 6" or 6*d_b = 8.25" for #11's or 6.75" for #9's. Use 6" spacing.

h_c = tie-tie spacing for the direction under consideration. For the controlling direction = 19.5" across the wall's thickness.

Therefore,

$$A_{sh} = \frac{0.87in^2}{3legs/tie} = 0.29in^2 \qquad \text{Use } \textbf{\#5 ties @ 6"}$$

Above the 16th story level, the boundary zones are not required by Section 1921. However, these were detailed in accordance with column provisions in order to maintain a space frame that allowed for a increased R factor of 5.5. A minimum boundary zone dimension of 0.1Lw was detailed with a maximum tie spacing for an ordinary frame of 8*db.

Development Lengths:
All horizontal reinforcement need to extend a distance l_d past the point where it is needed or if it is to be lap spliced.

$$\frac{l_d}{d_b} = \frac{3f_y\alpha\beta\gamma}{40\sqrt{f'c}\ \dfrac{(c+k_{tr})}{d_b}} < \frac{3f_y\alpha\beta\gamma}{100\sqrt{f'_c}}$$

Where,

c = smallest dimension of cover and half the spacing between the bars. For a 2" cover, 6" spacing, c = 2"

α,β,γ = depend on the type of bar and concrete placement

For Horizontal Bars, $\beta = 1.3$, $\alpha=\gamma=1.0$

$$l_d = 41.5{d_b}^2$$

#11 bar: **l_d = 6'-6"**

#9 bar: **l_d = 4'-4"**

For Vertical Bars, $\beta = \alpha=\gamma=1.0$

$$l_d = 32{d_b}^2$$

#7 bar: **ld = 2'-2"**

#6 bar: **ld = 1'–6"**

Since l_d < boundary zone length, a standard hook for vertical bars is not needed.

Final Design and Detailing

The final requirements summarized from the section above are represented graphically by Figure 35 and Figure 34 below.

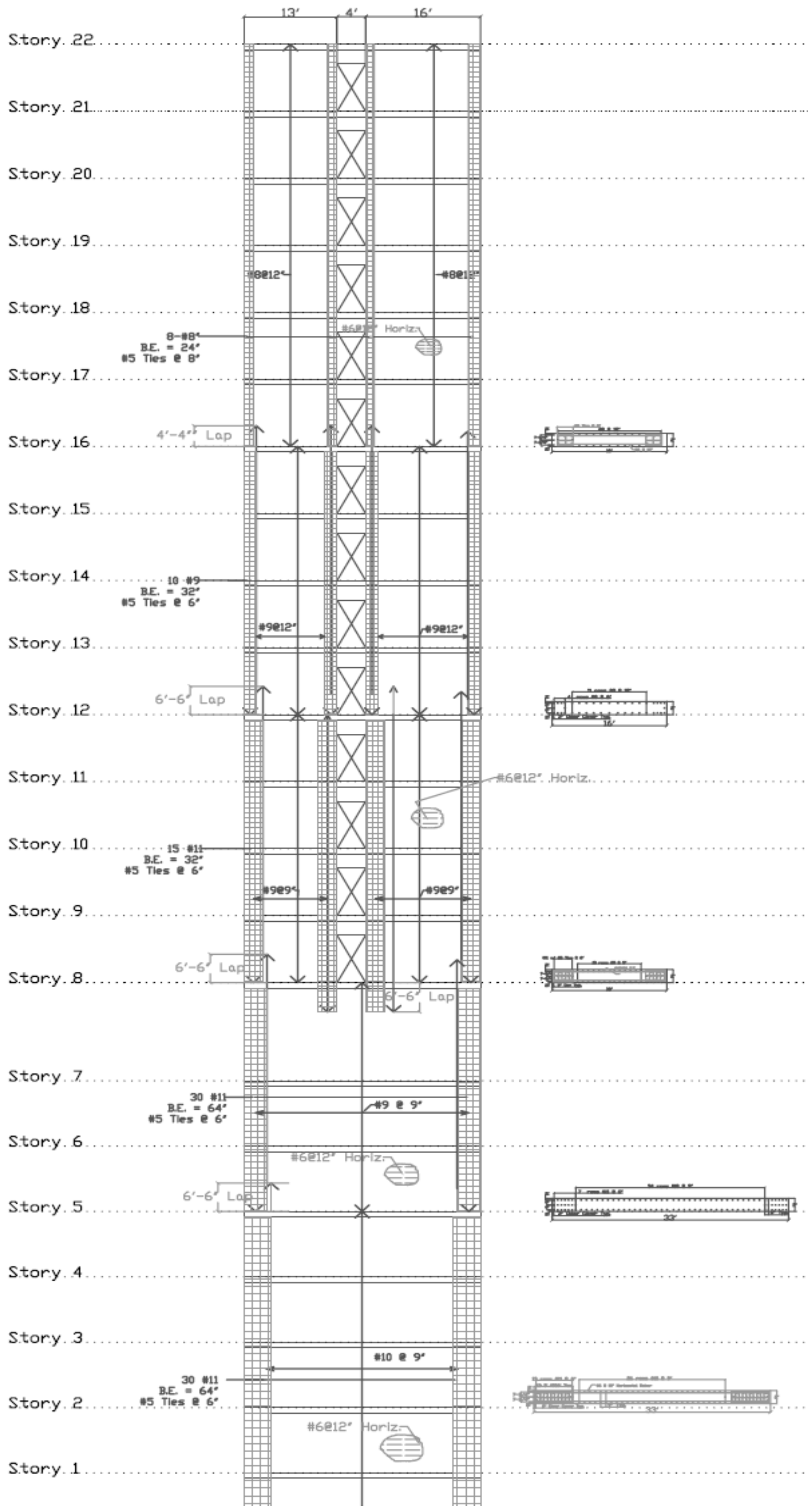


Figure 34: Final Detail showing Vertical, Horizontal, and Lap Slice Reinforcement

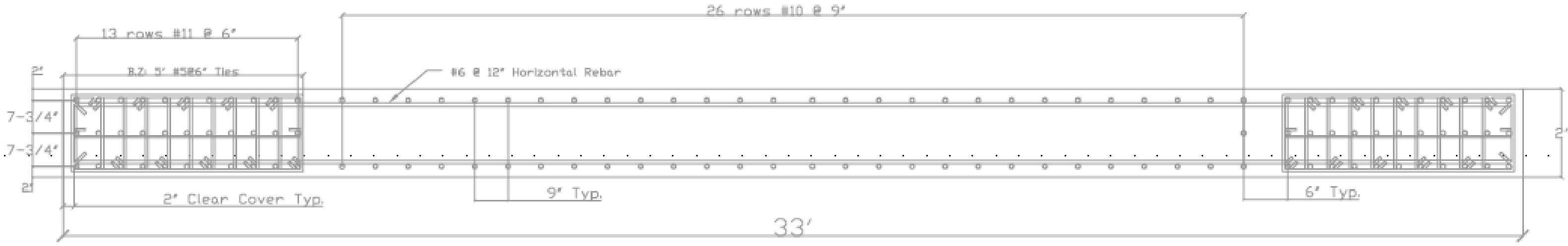


Figure 35: Section of Wall Detail at 1st Floor Level including Vertical, Horizontal, Boundary Zone and Tie Reinforcement

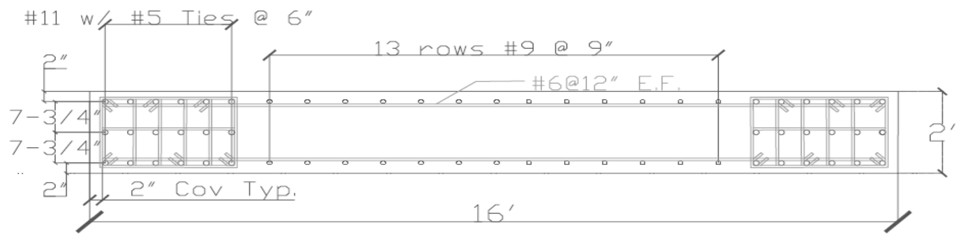


Figure 36: Section of Wall Detail at 9th Floor Level

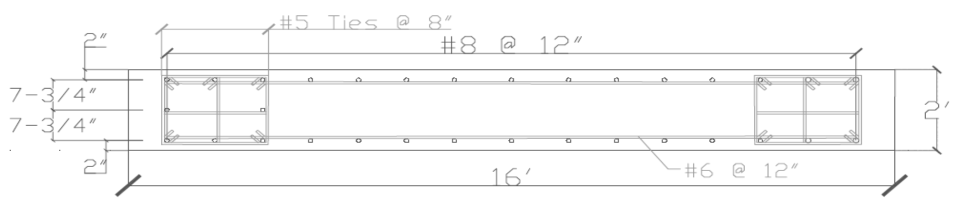


Figure 37: Section of Wall Detail at 16th Floor Level

IV. 2. vi. Displacements and Drifts

A final check involves the displacement response of the structure under loading. The UBC does not limit the total lateral displacement of any story due to lateral loads. However, recommended values for the maximum inelastic response displacement, Δ_m are based on the ratio $\frac{H}{\Delta_m} \leq 180$ to ensure occupants comfort and cladding and components integrity. For an overall height of 230 ft, the maximum displacement at the top should be limited to $\frac{230'*12}{180} \leq 15.33''$. The largest displacement occurs in the E.Q.Y Direction. The maximum displacement are calculated using a Response Spectrum Function in ETABS for the parameter corresponding to Ca = 0.33 and Cv = 0.45. The predominant natural period for mode 1 is calculated to be 4.388. The Maximum Inelastic Response Displacement Δ_m is 21.5466 in. This number is allowed to be reduced by a factor of 0.7 as specified in Section 1630.9.2 Equation (30-17). This results in a maximum displacement Δ_m = *15.05 in*. This result was compared to the maximum displacement from the ETABS output for a static lateral load case of Δ_m = 15.35 in. This confirms that the displacement is correct and is in close proximity to the maximum desired of 15.33 in.

UBC Section 1630.10.2 sets a limit on the drift per story. For structures having a fundamental period of 0.7 seconds or greater, the maximum story drift is required to be less than $0.020*h_{story}$. The maximum story drift calculated is 0.01 in/in, which is less than the maximum allowable 0.020 in/in. Both maximum displacement and drift requirements are satisfied for the chosen wall configurations. The two figures below summarize the story displacements and drift for both EQX and EQY Directions.

Load Case	Earthquake	Etabs Model:	Shear Walls for Steel Gravity Frame
Direction	X		
Period	4.38		

Story Level	Max. CM UX, in	Max. CM UY, in	Max Drift	Location (ft) X	Y	Z
22	12.377	0.0002	0.006173	114	90	230
21	11.6186	0.0003	0.006161	116	90	220
20	10.8609	0	0.006097	85	90	210
19	10.1113	0	0.006077	82	90	199
18	9.3638	0	0.006047	82	90	189
17	8.6201	0	0.005998	82	90	179
16	7.8824	0	0.005928	82	90	169
15	7.1533	0	0.005833	82	90	158
14	6.4358	0	0.005712	82	90	148
13	5.7332	0	0.005562	82	90	138
12	5.0492	0	0.005379	82	90	128
11	4.3875	0	0.005161	82	90	117
10	3.7528	0	0.004899	82	90	107
9	3.1502	0	0.004569	82	90	97
8	2.5882	0	0.004105	82	90	87
7	1.8494	0.0001	0.003647	123	90	72
6	1.4117	0	0.003237	124	90	62
5	1.0233	0	0.002784	124	90	52
4	0.6893	0	0.002288	124	90	42
3	0.4148	0	0.001747	124	90	32
2	0.2051	0	0.001162	124	90	22
1	0.0657	0	0.000476	124	90	12

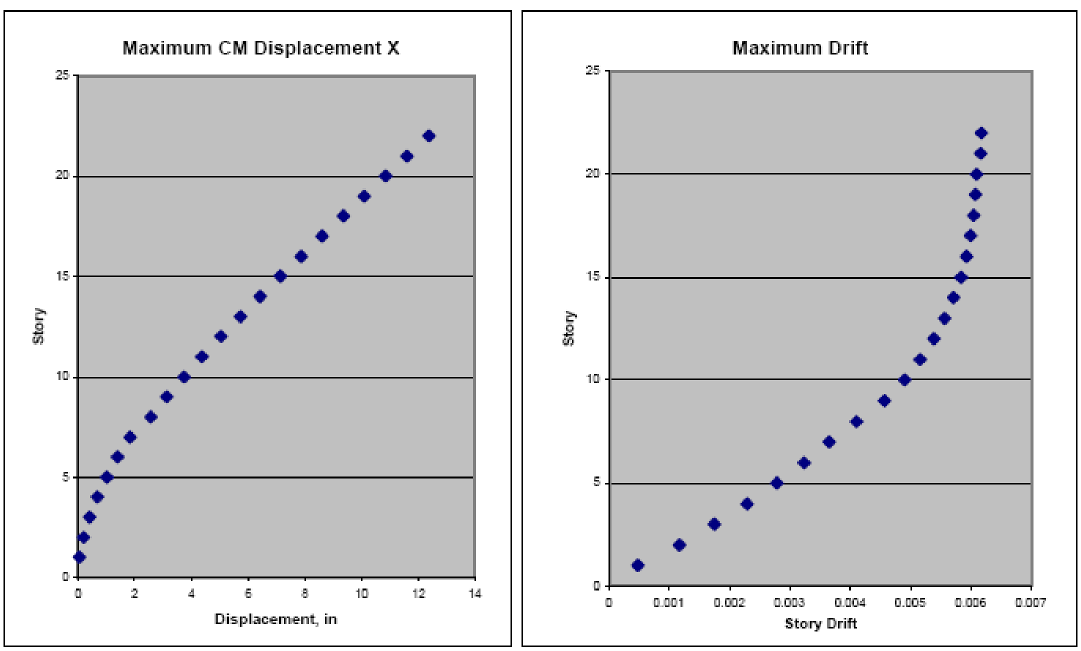


Figure 38: Maximum Displacements and Drift for Seismic Loads in EQX Direction

As seen from the displacement graph, the introduction of the coupling beam results in a relatively linear displacement above the 9th level instead of an independent cantilever behavior.

Load Case Earthquake
Direction Y
Period 4.38

Etabs Model: Shear Walls for Steel Gravity Frame

				Location (ft)		
Story Level	Max. CM UX, in	Max. CM UY, in	Max Drift	X	Y	Z
22	0	15.3578	0.01024	114	90	230
21	0.0013	14.6055	0.010226	116	90	220
20	0	11.213	0.010143	85	90	210
19	0	10.1706	0.010081	82	90	199
18	0	9.4196	0.009995	82	90	189
17	0	8.6724	0.009876	82	90	179
16	0	7.9314	0.009722	82	90	169
15	0	7.199	0.00953	82	90	158
14	0	6.4783	0.009296	82	90	148
13	0	5.7726	0.009018	82	90	138
12	0.0003	5.0851	0.008693	82	90	128
11	0	4.4207	0.008316	82	90	117
10	0	3.7827	0.007884	82	90	107
9	0	3.1762	0.007377	82	90	97
8	0	2.6064	0.006589	82	90	87
7	0	2.404	0.008479	123	90	72
6	0	1.8528	0.007551	124	90	62
5	0	1.3456	0.006531	124	90	52
4	0	0.9083	0.005403	124	90	42
3	0	0.5479	0.004159	124	90	32
2	0	0.2718	0.002796	124	90	22
1	0	0.087	0.000783	124	90	12

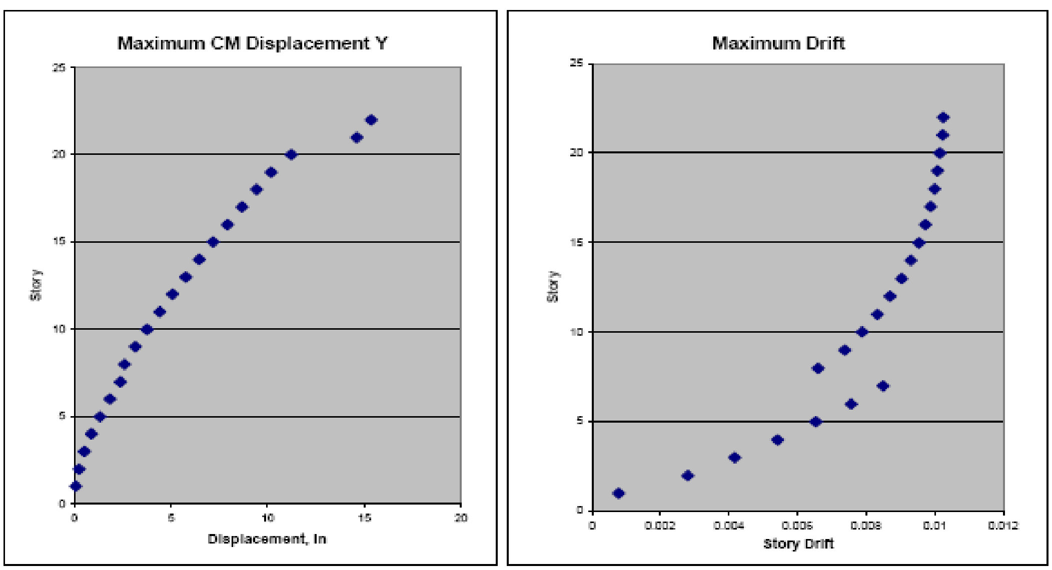


Figure 39: Maximum Displacement and Story Drift for EQY Direction

However looking at Figure 39 above, there is a major concern with the story drift resulting from lateral displacements in the EQY direction. While the maximum displacements and drifts requirements are met. The large jump (reduction) in the drift at the 7th story level demonstrates that there is a discontinuity at this floor level.

While the design of the lateral wall system attempted to minimize the impact of vertical discontinuities encountered in the previous system by maintaining a relatively constant stiffness at each story and limiting the vertical shifts in lateral elements, the fact is that the plan irregularity due to size and mass still play a large effect in the behavior of the structure. These irregularities can not be altered due to design requirements for space and use. The solution will be to perform a more detailed modal superposition or spectral response analysis of the lateral system. For matters of this report and time constraints, I will use this design as an estimate of what the wall sizes and detailing requirements are for Seismic Design. Time permitting, I recommend a more detailed dynamic analysis for verification.

V. Multidisciplinary Studies

V.1. Commodities: Architecture, Acoustics, and Vibrations

Major advantages in the design of the frame system are *weight savings* and *freedom in the architecture* of the apartment units for all three key parties: the architect, the current owner, and future tenants. The original bearing wall design consisted of 7 – 12” thick walls per apartment. This results in a total of 38 -12” walls at each floor for a total of 624 linear feet of wall. Each wall range from 16 to 23 ft in length and limits any open space to no more than 17 feet in the East-West Direction (See Figure 40 below).

By removing the bearing wall system and replacing them with fewer, thicker, and stronger coupled walls and a frame gravity system the total number of square feet of wall per apartment was reduced and larger usable spaces are now available for the architect, owner, and future occupants to work with.

	Bearing Wall System	**Frame System with Shear Wall**
Wall thickness, in	12	24
# Walls/ Apartment	7	4
Linear Feet Wall/ Apartment	128	58
Total Square Feet Wall/ Floor	624	464
Reduction, ft^2 Wall (Plan)	**160**	
Typical Open Area/Apartment	17’ x 23’	30’ x 27’

Knowing that you will be able to arrange the spaces as necessary is a major commodity for a location where construction practices provide very little leeway. Also, the location of this building in the middle of the city is a major attraction for business and offices. The proposed layout would offer an opportunity for different types of occupancies without any major renovation of the structure.

Figure 40: Existing Bearing/Shear Wall Arrangement

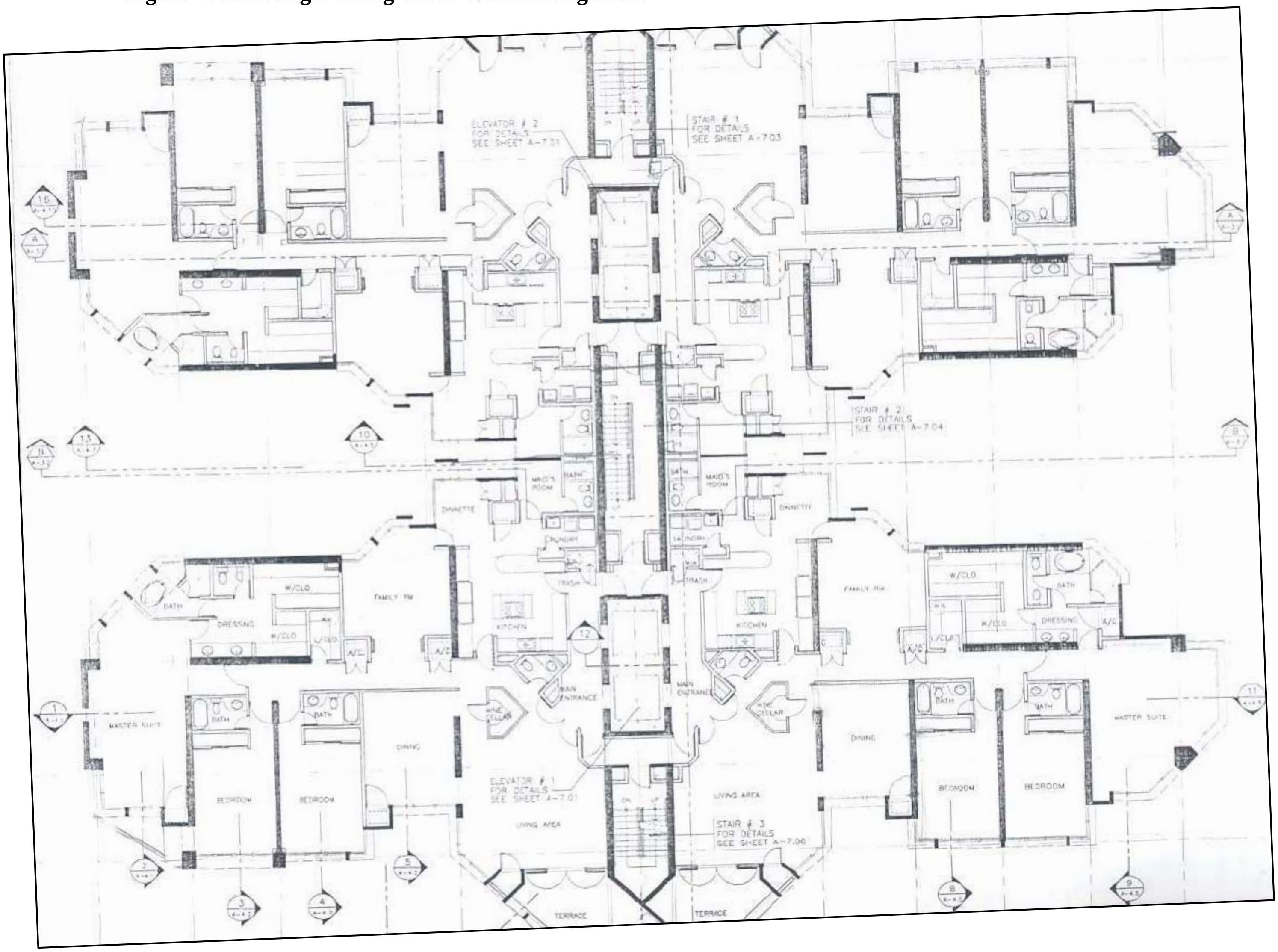
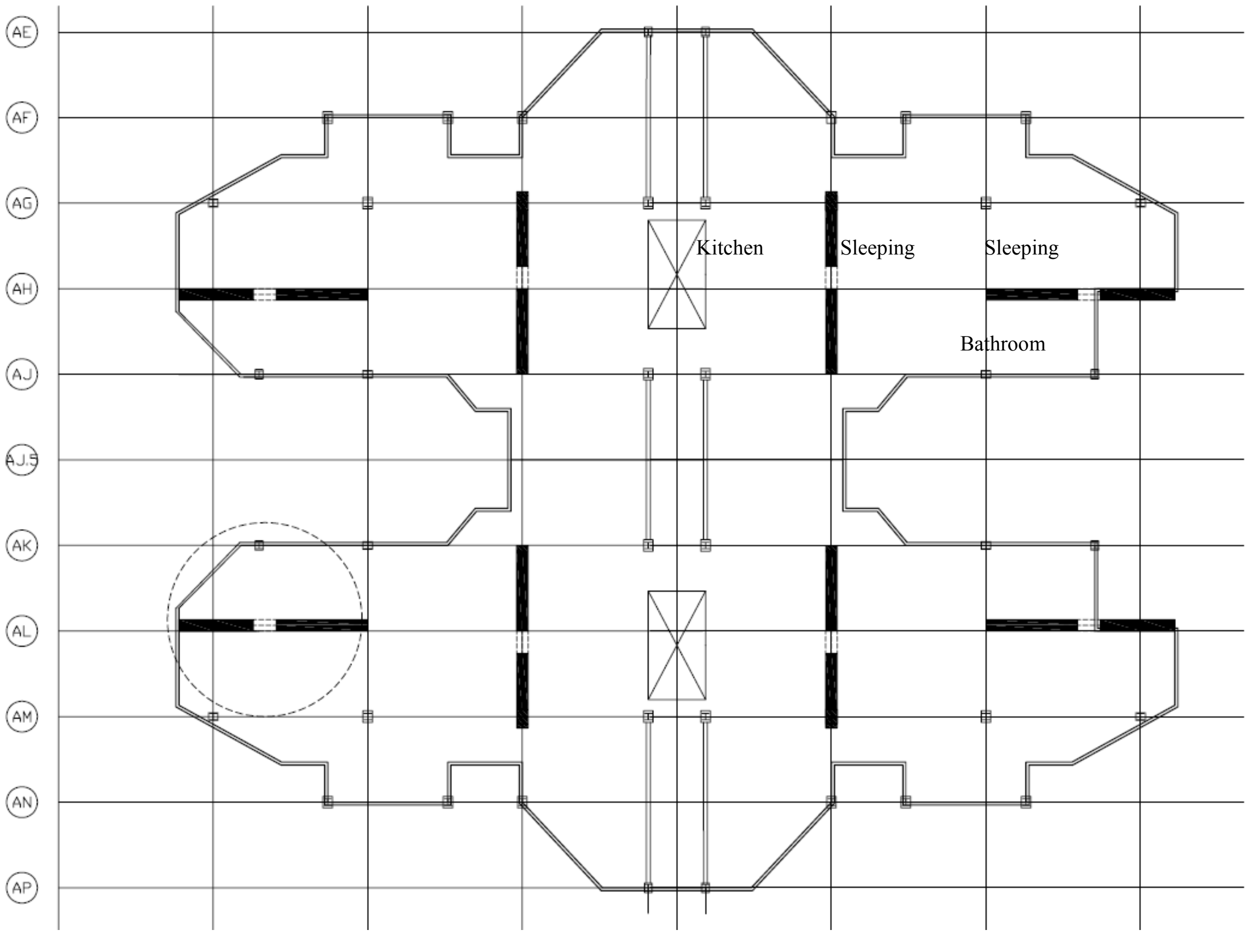


Figure 41: Proposed Open Plan Layout



By removing the concrete bearing walls the weight of the structure was greatly reduced, a more efficient lateral system design with a larger R value, and an open flexible design was achieve. However, the concrete system allowed for many advantages. Amount them are good resistance to vibrations and noise control. Therefore, a quick check on the impact of comfort conditions of the new apartment was performed

Acoustics

The original partition walls between the spaces in the apartment units and within the apartments were 12" painted cast in place concrete. Even when concrete does not posses good sound absorption properties within a space (NRC=0), it is very effective at providing more than adequate sound-transmission losses between spaces (STC=59). Sound isolation between apartments and between living spaces in an apartment is an important architectural appeal to the units that are being sold. Therefore, a study was performed to see whether the lighter acoustical partitions where acceptable for an NC-30.

An advantage clearly noticeable in the current configuration of the apartments units is the location of the maid's bedroom, which tends to be a quite space, between them. An apartment unit is connected by just 7 ft of a wall with the other. Another advantage is the location of the shear walls within the apartments. The 24" shear walls are located separating spaces that generally generate the greatest noise levels:

1. Between the kitchen and living room
2. Between the bathroom and sleeping areas

The partitions between apartments are made out of 3-5/8" channel studs 24in oc with two layers of 5/8in gypsum board on both sides and a sound attenuating blanket. The partitions within the apartment will only have one layer of gypsum board on each side. The following Figure shows how the partitions provide adequate transmission loss through the common space to maintain a Noise Criteria below the acceptable of NC-30. The study includes noise level data from a stereo being played in adjacent rooms.

Figure 42: Typical Apartment Acoustical Partition

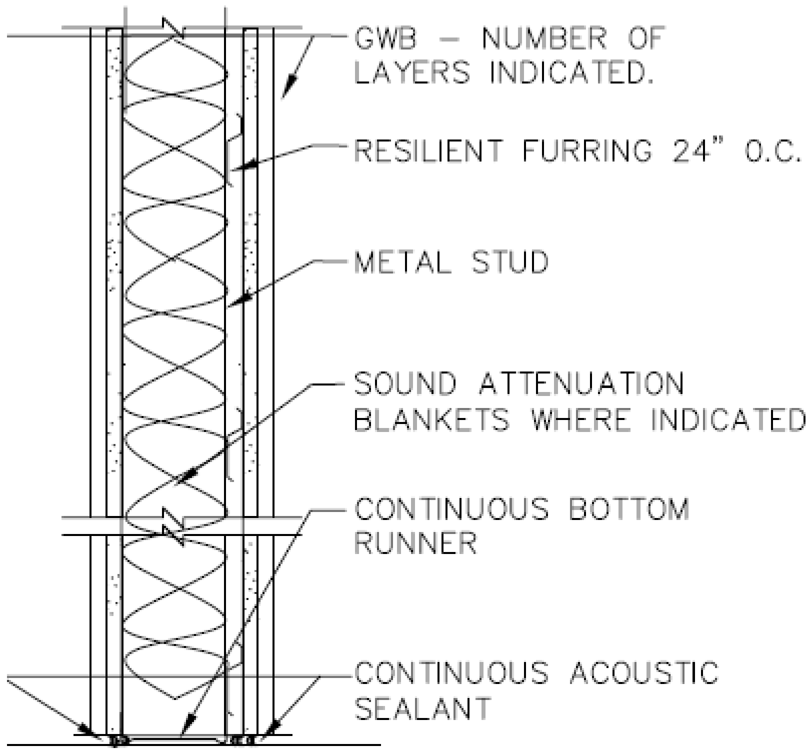


Figure 43: Sound Absorption and Transmission Properties for Existing Concrete Finishes

Current Finishe: Absorption Coefficients									
Room	**Bedroom**								
Location	Specification	Area,ft^2	125	250	500	1000	200	4000	NRC
Exterior Wall	Glass	81	0.35	0.25	0.18	0.12	0.07	0.04	0.15
	Concrete-Painted	54	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Interior Walls	Concrete-Painted	468	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Ceiling	Concrete	390	0.01	0.01	0.02	0.02	0.02	0.02	0
Floor	Glazed Tile	390	0.01	0.01	0.01	0.01	0.02	0.02	0
	Total a , sabins		88.35	54.15	57.60	57.96	68.25	60.60	
Room	**Dining Room**								
Location	Specification	Area,ft^2	125	250	500	1000	200	4000	NRC
Exterior Wall	Glass	50.4	0.35	0.25	0.18	0.12	0.07	0.04	0.15
	Concrete-Painted	150.6	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Interior Walls	Concrete-Painted	234	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Ceiling	Concrete	238	0.01	0.01	0.02	0.02	0.02	0.02	0
Floor	Glazed Tile	238	0.01	0.01	0.01	0.01	0.02	0.02	0
	Total		60.86	36.59	39.29	40.11	47.66	42.30	
Common Partion - Transmittion Loss									
Construction:	Concrete Wall - Painted 12" thick	117	44	48	57	64	72	77	48
Noise Reduction, NR									
	To Dining Room		**41**	**43**	**52**	**59**	**68**	**73** dB	
	To Bedroom		**43**	**45**	**54**	**61**	**70**	**74** dB	

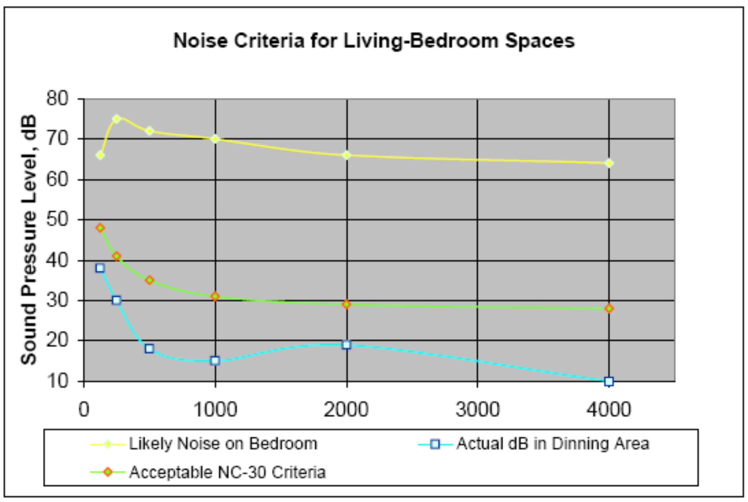
Figure 44: Sound Absorption and Transmission Properties for Lighter Acoustical Partitions

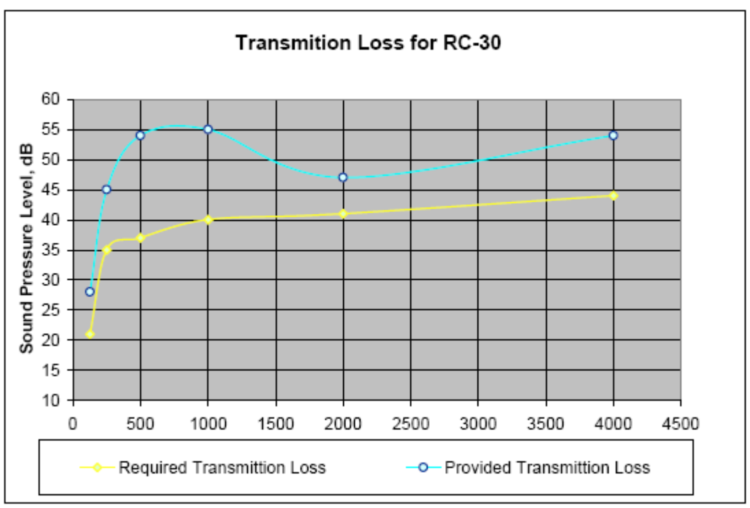
New Finishes: Absorption Coefficients									
Room	**Bedroom**								
Location	Specification	Area,ft^2	125	250	500	1000	200	4000	NRC
Exterior Wall	Glass	81	0.35	0.25	0.18	0.12	0.07	0.04	0.15
	Concrete-Painted	54	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Interior Walls	5/8" GWB w/ Insulation	468	0.55	0.14	0.08	0.04	0.12	0.11	0.1
Ceiling	Suspended Acoustical Tile	390	0.15	0.1	0.05	0.04	0.07	0.09	0.05
Floor	Glazed Tile	390	0.01	0.01	0.01	0.01	0.02	0.02	0
	Total a , sabins		353.55	131.37	78.66	51.72	101.79	101.94	
Room	**Dining Room**								
Location	Specification	Area,ft^2	125	250	500	1000	2000	4000	NRC
Exterior Wall	Glass	50.4	0.35	0.25	0.18	0.12	0.07	0.04	0.15
	Concrete-Painted	150.6	0.1	0.05	0.06	0.07	0.09	0.08	0.05
Interior Walls	5/8" GWB w/ Insulation	234	0.55	0.14	0.08	0.04	0.12	0.11	0.1
Ceiling	Suspended Acoustical Tile	238	0.15	0.1	0.05	0.04	0.07	0.09	0.05
Floor	Glazed Tile	238	0.01	0.01	0.01	0.01	0.02	0.02	0
	Total		199.48	79.07	51.11	37.85	66.58	65.98	
Common Partion - Transmittion Loss									
Construction:	3 5/8" steel channel studs 24in oc with one layer of 5/8" gypsum board on each side and sound attenuating blanket	117	28	45	54	55	47	54	48
Noise Reduction, NR									
	To Dining Room		**30**	**43**	**50**	**50**	**45**	**52** dB	
	To Bedroom		**33**	**46**	**52**	**51**	**46**	**53** dB	

The Noise Reduction Provided by the new partitions is well below the Noise Reduction Provided by concrete walls. However, further studies show that the NR provided by the partition walls is still acceptable to maintain a sound level below NC 30.

Figure 45: Required vs. Provided Sound Transmission and Noise Criteria Comparison

Is this Still Acceptable?								
For: Stereo in the Bedroom while Dinner								
Sound Pressure Level, Stereo		66	75	72	70	66	64	
Background Level Noise								
RC-30		-45	-40	-35	-30	-25	-20	
Required NR		21	35	37	40	41	44	dB
Required Transmittion Loss, TL	117	**19**	**37**	**41**	**45**	**43**	**46**	dB
TL Provided:								
3 5/8" steel channel studs 24in oc with one layer of 5/8" gypsum board on each side and sound attenuating blanket		**28**	**45**	**54**	**55**	**47**	**54**	dB
Sound Pressure Level in Dinning Room after TL		38	30	18	15	19	10	dB
Acceptable Noise Criteria								
NC30		48	41	35	31	29	28	





Floor Vibrations

A dynamic floor vibration study was performed on the typical framing members for the apartment units. Priority was given to those members framing the living area, such a living room, dinning room, and kitchen. The criteria used follow the work of Allen and Murray and has been adopted by the AISC in their Guidelines for Floor Vibrations. It is based on calculating the required damping coefficient that will provide adequate mitigation of the vibrations imposed by the floor member's deflections. If the damping required is larger than the damping provided by the system (usually 5% for floor with finishes) then the floor is considered to be unacceptable and larger members or shorter spans should be considered. The governing equation is:

$$\frac{a_p}{g} = \frac{P_o \exp(-0.35 f_n)}{\beta W}$$

Where,

$\frac{a_p}{g} =$ Predicted actual damping ratio, 5%

P_o, β = Load and Reduction Factor Criteria for the given floor configuration equal to 65lbs and 0.3, respectively.

$f_n =$ Combined effective stiffness of floor system. Simplified to:

$$= 0.18\sqrt{\frac{g}{(\Delta_{beam} + \Delta_{girder})}}$$

W = Combined effective load carried by the tributary floor width.

$$= \left(\frac{\Delta_{beam}}{\Delta_{beam} + \Delta_{girder}}\right) W_{beam} + \left(\frac{\Delta_{girder}}{\Delta_{beam} + \Delta_{girder}}\right) W_{girder}$$

Because of the composite floor system, transformed properties were used for the calculation of the beam and girder's stiffness. An increase of 35% was allowed for the Modulus of Elasticity of concrete because of its improved resistance to strain under dynamic loads.

The first study involved the determination of the damping necessary for the W10x15 members in the living areas. The damping required was 0.56%, which is greater than the probable actual damping of 0.5%. Therefore, a second study was performed using the larger member, W10x26, already present in the larger spans of 27ft on the bedroom areas. This member was found satisfactory for both locations. The critical spans are depicted below. The necessary damping ratio was found to be 0.48% which is just below the 0.5% mark. Therefore, it is recommended that the beams spanning the living room, dinning, and kitchen areas be increase in size from W10x15 to W10x26 for adequate vibration control.

Figure 46: Summary of Study Performed for Vibration of Steel Floor System

Floor Vibration Summary			
Slab			
	f'c, ksi	5	
	t, in	2.5	
	deck, in	1.5	
	n	6.15	
Beam			
	Size	W10x26	
	w, klf	0.59	
	Area, in^2	7.61	
	d, in	10.2	
	I, in^4	144	
	L, ft	27	
	Tributary Width, ft	7.5	
	d_{eff},in	90	
	$I_{transformed}$, in4	551	
	Δ, in	0.442	
	f_{beam}, Hz	5.32	
	B_b, ft	28.35	Effective Width
	W_b, kips	90.32	Effective Weight
Girder			
	Size	W10x22	
	w, klf	2.15	
	Area, in^2	6.49	
	d, in	10.2	
	I, in^4	188	
	L, ft	15	
	Tributary Width, ft	27	
	d_{eff},in	45	
	$I_{transformed}$, in4	597	
	Δ, in	0.141	
	f_{beam}, Hz	9.42	
	B_g, ft	19.58	Effective Width
	W_g, kips	23.39	Effective Weight
Combined Floor System			
	W, kips	81.1	
	f_n, Hz	4.92	
	a_p/g, %	0.48	> 0.5
			Acceptable

V.2 Multi-Disciplinary Study: Construction Quantities and Costs

The initial goal of this research was to improve the behavior of the structure and make it more efficient in design. It was believed that by designing a liter system and reducing the amount of shear walls, the cost would be impacted and possibly reduced specially when you take into account the cost associated in formwork and rebar placement for 624 linear feet of wall per floor for 22 level! Such would be the case in America, where labor is valued and paid accordingly. I was very surprise about what I discovered in my second disciplinary study. By changing a building from all concrete to steel you would save $1.0 million dollars in cost associated with the structure of this building. This is mainly because of the high labor and material cost associated with form work of concrete. However, in Puerto Rico it is not that concrete is much more inexpensive. According to RS Means is it actually 94.9% of the national average cost. However, the labor costs associated with construction practices in Puerto Rico are unacceptable to me and a personal reflection of the multiple problems that face this country. **The labor for concrete and formwork is at 19.5% of the national average and only at 12% for placing reinforcement!** Therefore, it is of no surprise to me that they build a concrete block while they have all the people to carve it out rather than to do a design that is not only less material and labor intensive but demonstrate the progress of society in the understanding of materials, science, engineering and even natural phenomena's.

Another problem is the cost associated with partitions, gypsum boards and acoustical finished that would need to replace the bearing walls and concrete floors. According to RS Means, the city cost index in Puerto Rico for these goods is at 258% for plaster and gypsum board and 334% for ceiling and acoustical treatment. This is not an option for a building in this location.

Table 22: Cost Comparison, US and Puerto Rico

											Adjusted Values	
Existing Building												
Lateral System											**Adjusted Values**	
Walls				Quantity	Daily Output	Material	Labor	Equipm.	Total	Total	**City Index - PR**	
	Concrete	4,000 psi	cy	4661		91			91	**$424,151.00**	0.761	$322,778.91
	Form	4 uses	sfca	281740	235	2.42	7.05		9.47	**$2,668,077.80**	0.299	$797,755.26
	Placing	w/ Crane and Bucket	cy	4661	95		22.5	10.65	33.15	**$154,512.15**	0.761	$117,583.75
	Finishes	Break ties/ patch voids	sfca	126297	540	0.03	0.51	0.54	0.78	**$136,400.76**	0.761	$103,800.98
	Rebar	#8-18 over 100 tons	tons	530	3.2	855	418.5	7.05	1280.55	**$678,691.50**	1.01	$685,478.42
	Unloading and Sorting		tons	530	100		21.5	6.5	28	**$14,840.00**	1.01	$14,988.40
									Total	**$4,076,673.21**		**$2,042,385.71**
Floor System/ Floor Cost												
Number Floors		14		Quantity	Daily Output	Material	Labor	Equipm.	Total	Total		
Floor Slab												
	Concrete	4,500psi	cy	467		93			93	**$608,034.00**	0.761	$462,713.87
	Form	Flat Plate - 4 Uses	sf	16813	560	1.4	2.96	0	4.36	**$1,026,265.52**	0.299	$306,853.39
	Placing	9" thick elvated	cy	467	160		11.9	4.65	16.55	**$108,203.90**	0.761	$82,343.17
	Finishing	Screen, Float, Hand Tr	sf	16813	600		0.46	0.46	0.68		0.761	$0.00
	Rebar	Elevated Slab #4-#7	tons	8.8	2.9	905	435		1340	**$165,088.00**	1.01	$166,738.88
	P/T Tendons	Ungrouted 100' span	lbs	11200	1500	0.47	0.85	0.02	1.34	**$210,112.00**	1.01	$212,213.12
	Unloading and Sorting		tons	8.8	100		21.5	6.5	28	**$246.40**	1.01	$248.86
									Total	**$2,117,703.42**		**$1,231,111.30**
									Total	$6,194,376.63		$3,273,497.01
Steel Frame & Coupled Walls												
				Quantity	Daily Output	Material	Labor	Equipm.	Total	Total	**Adjusted Values**	
Lateral System											**City Index**	
Walls	Concrete	5,000 psi	cy	3960		96			96	**$380,160.00**	0.761	$289,301.76
	Form	4 uses	sfca	81760	235	2.42	7.05		9.47	**$774,267.20**	0.299	$231,505.89
	Placing	w/ Crane and Bucket	cy	3960	95		22.5	10.65	33.15	**$131,274.00**	0.761	$99,899.51
	Finishes	Break ties/ patch voids	sfca	496	540	0.03	0.51	0.54	0.78	**$386.88**	0.761	$294.42
	Rebar	#8-18 over 100 tons	tons	477	3.2	855	418.5	7.05	1280.55	**$610,822.35**	1.01	$616,930.57
	Unloading and Sorting		tons	477	100		21.5	6.5	28	**$13,356.00**	1.01	$13,489.56
	Coupling Beams			23.85	3.2				5% Rebar	**$31,208.92**	1.01	$31,521.01
								Total		**$1,941,475.35**		**$1,282,942.72**
Frame System/ Floor Cost												
Number of Floors				14								
	Steel	Apartments>15 story	tons	735	13.9	2050	365	118	2533	**$1,861,755.00**	0.878	$1,634,620.89
	Deck	20 gage, 1.5"	sf	16813	4300	1.24	0.3	0.02	1.56	**$367,195.92**	0.878	$322,398.02
	Concrete	4500psi	cy	182		93			93	**$236,964.00**	0.761	$180,329.60
	Placing	<6" Pumped	cy	182	140		13.55	5.3	18.85	**$48,029.80**	0.761	$36,550.68
	Finishes	Screen, Float, Hand Tr	sf	16813	600		0.46	0.46	0.68	**$160,059.76**	0.761	$121,805.48
	Studs	3/4" dia 3-3/8" long	ea	2700	950	0.43	0.69	0.28	1.4	**$52,920.00**	0.878	$46,463.76
									Total	**$2,726,924.48**		**$2,342,168.43**
Additional Cost												
	Exterior Shell	6" thick, 4000psi, with	sf	4225					12.2	**$51,545.00**	0.868	**$44,741.06**
									Total	$4,719,944.83		$3,669,852.21
								Difference		**$1,474,431.80**		**-$396,355.20**

Leaving cost aside, I believe there are many construction related advantages to this design. First, it requires less than half the amount of materials, concrete, formwork, rebar. This is a major advantage when we think about sustainability and efficiency in the structures we inhibit. Another advantage is that it requires less than half the mount of man hours and provides a much faster and efficient construction time and sequencing: the 8 coupled walls are built; the lower steel is erected while the top walls are formed and placed. The floors then can be poured once the steel is placed.

VI.2 Conclusion

In conclusion, I believe that the new design is efficient at supporting the building. It also makes good use of materials, labor and space. It was satisfying to know that the structure could be designed completely different while still keeping the architecture of the building and the different uses for the spaces undisturbed. The new design actually increased the square feet area of usable space in the apartments by replacing the entire bearing wall system with a frame structure. Also, there was no need to remove any parking spaces or alter the vehicular circulation.

There are many advantages to the new gravity system. The frame system allowed for a larger R factor to be used in the lateral design. It also increased the typical uninterrupted usable area from 17’ to 30’ in one direction. This provides more flexibility for multiple parties: the architect, the owner, and future tenants.

Sadly, due to current practices in Puerto Rico and their situation on the labor market this design is not monetarily feasible. Even when it would be faster to build, it would be considerably more expensive because of the high cost associated with the materials of steel, interior partitions, gypsum board and finishes. The only practicable solution is to build the walls like they always do: all cast-in-place concrete or concrete block. This posse a difficult situation because the concrete system has very large dead loads associated with it making any lateral system in a Seismic Zone 3 very heavy stressed.

I believe that it might have been too much of a challenge to be able to make this hybrid structure work efficiently when materials and solutions are so limited by the location. I also believe that the engineers in the project designed the cheapest and workable structure for this location and with the requirements that the parking structure had to be incorporated below the apartment units.

Appendix A: Proposal 1 - Concrete Frame

The initial proposed layout for the system was to redesign the bearing wall gravity system as a column frame system in order to take advantage of the large R value, open space, and reduced weight. However, in order to allow for a column-slab concrete frame that will follow the 27'x30' grid in the already existing parking garage, the thickness of the slab would need to be increased to 11", capitals or transverse beams would have to be introduced in order to resist the punching shear around the columns while maintaining them at a reasonable size not larger than . it was later in the design that it is evident that such a system would not permit for the reduced number of shear coupled walls that were initially proposed. The results summarized below show that the required coupled beams are over stressed and the required reinforcement in the walls to resist the over turning moment is very dense making the design very congested.

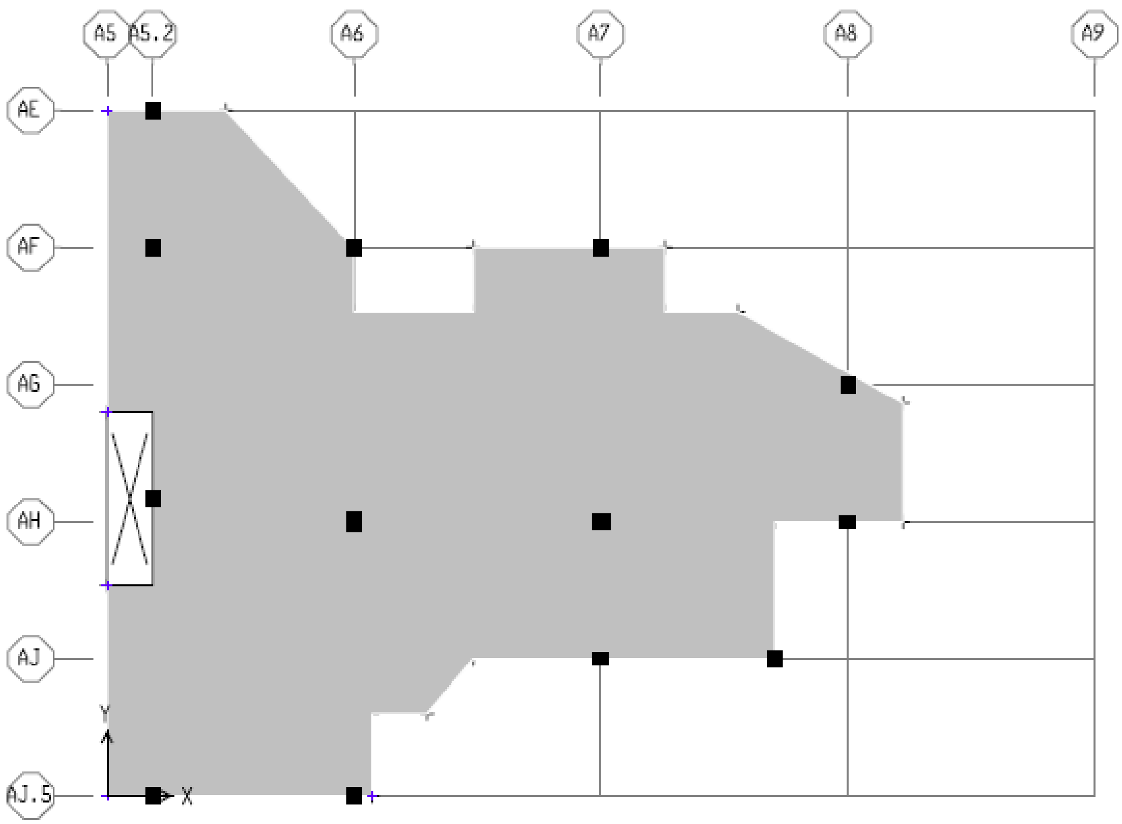


Figure 47: Concrete Column Frame Layout

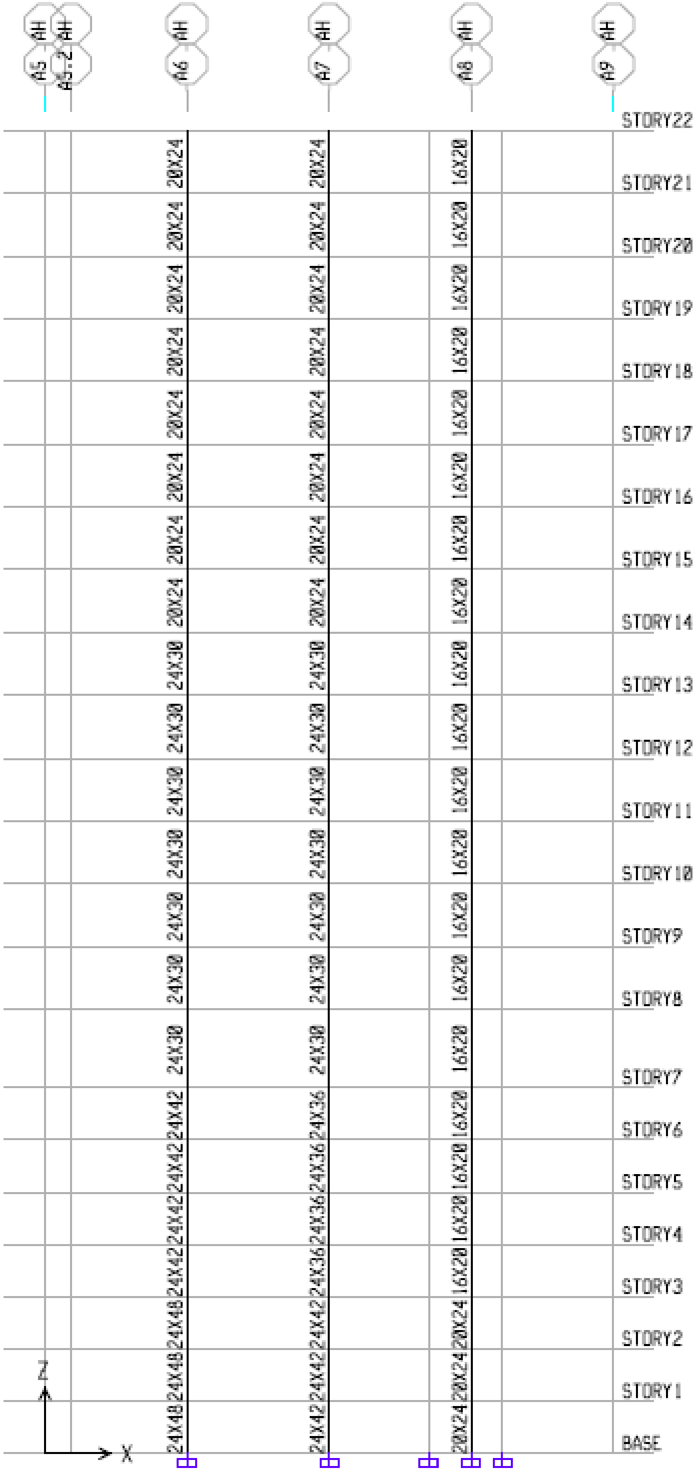


Figure 48: Column Selection for Critical Section AH

Table 23: Required Flexural Reinforcement is excessive at Lower Levels

Tension Reinforcement

Story	Length (ft)	Mu (kip-ft)	Pu(quake),k	Pc-dead,k	P eff (k)	Pnet (k)	As,req (in2)	BZ, ft	ρn
STORY22	13.0	585.4	142.2	93.6	198.4	104.9	1.9	**0.2**	**0.000**
STORY21	13.0	1227.8	328.3	187.1	446.3	259.2	4.8	**0.6**	**0.001**
STORY20	13.0	1258.1	527.3	280.7	648.2	367.6	6.8	**0.9**	**0.001**
STORY19	13.0	1755.0	752.0	374.2	920.7	546.5	10.1	**1.3**	**0.001**
STORY18	13.0	2349.9	1018.0	467.8	1243.9	776.2	14.4	**1.8**	**0.002**
STORY17	13.0	3005.3	1324.4	561.3	1613.4	1052.1	19.5	**2.4**	**0.002**
STORY16	13.0	3712.0	1668.8	654.9	2025.7	1370.8	25.4	**3.2**	**0.003**
STORY15	13.0	4465.5	2048.4	748.4	2477.8	1729.4	32.0	**4.0**	**0.004**
STORY14	13.0	5262.9	2460.8	842.0	2966.8	2124.9	39.3	**4.9**	**0.005**
STORY13	13.0	6103.8	2903.3	935.5	3490.2	2554.7	47.3	**5.9**	**0.005**
STORY12	13.0	6990.0	3372.7	1029.1	4044.8	3015.7	55.8	**7.0**	**0.006**
STORY11	13.0	7945.1	3864.6	1122.6	4628.6	3506.0	64.9	**8.2**	**0.008**
STORY10	13.0	8991.7	4369.0	1216.2	5233.6	4017.5	74.4	**9.4**	**0.009**
STORY9	13.0	10418.8	4865.9	1309.7	5867.7	4558.0	84.4	**10.6**	**0.010**
STORY8	13.0	14400.8	5369.1	1403.3	6753.7	5350.5	99.1	**12.5**	**0.011**
STORY7	13.0	5810.3	5291.6	1496.8	5850.3	4353.5	80.6	**10.1**	**0.009**
STORY6	13.0	6493.9	5095.5	1590.4	5719.9	4129.6	76.5	**9.6**	**0.009**
STORY5	13.0	7446.3	5082.0	1683.9	5798.0	4114.1	76.2	**9.6**	**0.009**
STORY4	13.0	8406.9	5207.7	1777.5	6016.1	4238.6	78.5	**9.9**	**0.009**
STORY3	13.0	9400.7	5427.0	1871.0	6330.9	4459.9	82.6	**10.4**	**0.010**
STORY2	13.0	10619.6	5692.9	1964.6	6714.0	4749.4	88.0	**11.1**	**0.010**
STORY1	13.0	13167.3	5920.7	2058.1	7186.7	5128.6	95.0	**11.9**	**0.011**

Table 24: Shear Capacity in Coupled Beams is exceeded

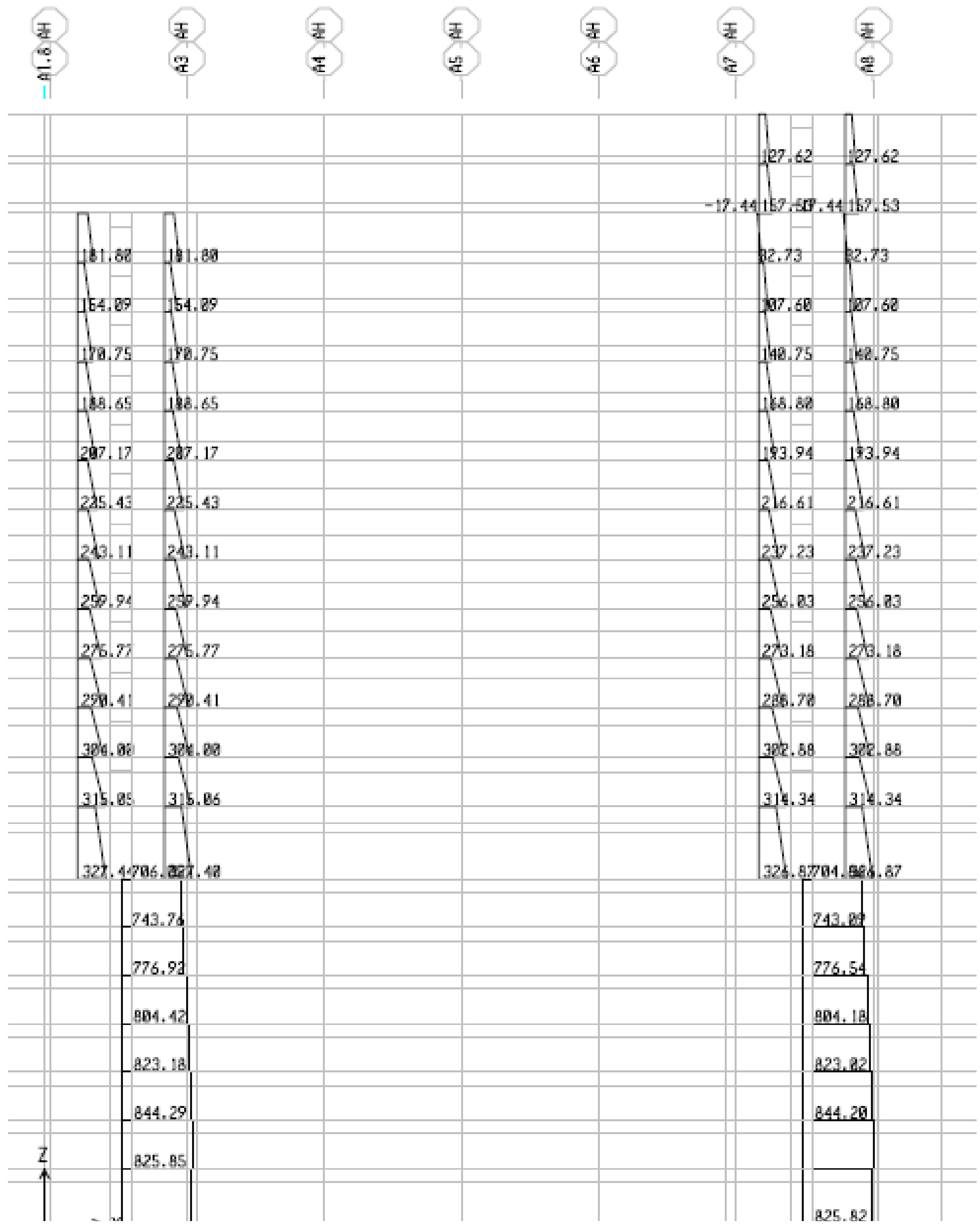
Spandrel Design	
Vc, kips =	3.22
Φ =	0.60
L/d =	1.60

Story	Vu, kips	Av, in	Av	Ah	Vu/Φ <	8.00 √f'c*b*d
STORY 22	110.75	0.10	0.036	0.06	184.58	407.29
STORY 21	132.90	0.12	0.036	0.06	221.50	407.29
STORY 20	117.89	0.11	0.036	0.06	196.48	407.29
STORY 19	119.69	0.11	0.036	0.06	199.48	407.29
STORY 18	148.24	0.14	0.036	0.06	247.07	407.29
STORY 17	179.63	0.16	0.036	0.06	299.38	407.29
STORY 16	208.88	0.19	0.036	0.06	348.13	407.29
STORY 15	235.54	0.22	0.036	0.06	392.57	407.29
STORY 14	259.47	0.24	0.036	0.06	**432.45**	**407.29**
STORY 13	280.13	0.26	0.036	0.06	**466.88**	**407.29**

STORY 12	295.66	0.27	0.036	0.06	**492.77**	**407.29**
STORY 11	301.47	0.28	0.036	0.06	**502.45**	**407.29**
STORY 10	283.57	0.26	0.036	0.06	**472.62**	**407.29**
STORY 9	216.31	0.20	0.036	0.06	360.52	407.29

Appendix B: Example of ETABS Output

Figure 49: Example Output of Etabs Results for Critical Section



Appendix C: Example of Torsion Calculations

Torsion

Parking Garage: 8th - 1st Level

Load Case:	EQ X Direction
Center of Mass, C.M. =	98 ft
Center of Rigidity. C.R. =	90 ft
eccentricity, e =	8 ft
Pier Label:	AHP3-4
kd/∑kd2	0.0040

Story Level	$P_{E\ Story}$, k	T, kip-ft	V_{T} ,kips
STORY8	99	746	**3**
STORY7	262	1968	**8**
STORY6	398	2982	**12**
STORY5	511	3831	**15**
STORY4	602	4515	**18**
STORY3	671	5035	**20**
STORY2	719	5390	**22**
STORY1	744	5582	**22**

Torsion

Apartments 22nd - 9th

Pier Distribution Factors

Pier Label	L, ft	k	d	kd	kd^2	$kd/\sum kd^2$
A4P1	13	2197	27	59319	1601613	**0.0014**
A4P2	16	4096	27	110592	2985984	**0.0027**
A4P3	16	4096	27	110592	2985984	**0.0027**
A4P4	13	2197	27	59319	1601613	**0.0014**
A6P1	13	2197	-27	-59319	1601613	**-0.0014**
A6P2	16	4096	-27	-110592	2985984	**-0.0027**
A6P3	16	4096	-27	-110592	2985984	**-0.0027**
A6P4	13	2197	-27	-59319	1601613	**-0.0014**
AHP1	13	2197	-30	-65910	1977300	**-0.0016**
AHP2	16	4096	-30	-122880	3686400	**-0.0030**
AHP3	16	4096	-30	-122880	3686400	**-0.0030**
AHP4	13	2197	-30	-65910	1977300	**-0.0016**
ALP1	13	2197	30	65910	1977300	**0.0016**
ALP2	16	4096	30	122880	3686400	**0.0030**
ALP3	16	4096	30	122880	3686400	**0.0030**
ALP4	13	2197	30	65910	1977300	**0.0016**
					41005188	

Appendix D: Wall Reinforcements and Flexural Strengths

Figure 50: Wall Combined Flexural and Axial Strength at 6th Floor Level

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W)
Location: 6th Level
Pier: AH
Length: 33'

Date: Feb, 06
Engineer: Lourdes Diaz

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	396.00 in	
b =	24.00 in	
Ag =	9504.00 in²	
x =	57.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	198.00 in from extreme compression fiber	
Strain =	-0.0030 Ultimate concrete strain	(- compression)
c =	69.10 in	
β =	0.80	
ρ total =	0.0159	

Governing Equation: 0.8D + 1.2E
Pu, kips = 1583.00
Mu, ft-kips = 123500.00
phi = 0.83

Reinforcement						#/row	Area / Bar	X
		cover	3.00					
Boundary rows	10	spacing	6.00	Bar Size	11	3	1.56	57.00
Internal rows	30	spacing	9.00	Bar Size	9	2	1.00	196.50
Total Rows	50							393.00

Layer	y,in	Strain	c or fs, ksi	As, in²	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	69.10	-0.00300	5.00		0.00	5638.56	0.00	960585.08
As 1	3.00	-0.00287	60.00	4.68	0.00	280.80	0.00	54756.00
As 2	9.00	-0.00261	60.00	4.68	0.00	280.80	0.00	53071.20
As 3	15.00	-0.00235	60.00	4.68	0.00	280.80	0.00	51386.40
As 4	21.00	-0.00209	60.00	4.68	0.00	280.80	0.00	49701.60
As 5	27.00	-0.00183	53.01	4.68	0.00	248.07	0.00	42419.47
As 6	33.00	-0.00157	45.45	4.68	0.00	212.71	0.00	35097.66
As 7	39.00	-0.00131	37.90	4.68	0.00	177.36	0.00	28200.10
As 8	45.00	-0.00105	30.34	4.68	0.00	142.01	0.00	21726.79
As 9	51.00	-0.00079	22.79	4.68	0.00	106.65	0.00	15677.72
As 10	57.00	-0.00053	15.23	4.68	0.00	71.30	0.00	10052.90
As 11	66.00	-0.00013	3.90	2.00	0.00	7.81	0.00	1030.40
As 12	75.00	0.00026	7.43	2.00	14.86	0.00	-1827.38	0.00
As 13	84.00	0.00065	18.76	2.00	37.52	0.00	-4277.23	0.00
As 14	93.00	0.00104	30.09	2.00	60.18	0.00	-6319.15	0.00
As 15	102.00	0.00143	41.42	2.00	82.85	0.00	-7953.13	0.00
As 16	111.00	0.00182	52.75	2.00	105.51	0.00	-9179.19	0.00
As 17	120.00	0.00221	60.00	2.00	120.00	0.00	-9360.00	0.00
As 18	129.00	0.00260	60.00	2.00	120.00	0.00	-8280.00	0.00
As 19	138.00	0.00299	60.00	2.00	120.00	0.00	-7200.00	0.00
As 20	147.00	0.00338	60.00	2.00	120.00	0.00	-6120.00	0.00
As 21	156.00	0.00377	60.00	2.00	120.00	0.00	-5040.00	0.00
As 22	165.00	0.00416	60.00	2.00	120.00	0.00	-3960.00	0.00
As 23	174.00	0.00455	60.00	2.00	120.00	0.00	-2880.00	0.00
As 24	183.00	0.00495	60.00	2.00	120.00	0.00	-1800.00	0.00
As 25	192.00	0.00534	60.00	2.00	120.00	0.00	-720.00	0.00
As 26	201.00	0.00573	60.00	2.00	120.00	0.00	360.00	0.00
As 27	210.00	0.00612	60.00	2.00	120.00	0.00	1440.00	0.00
As 28	219.00	0.00651	60.00	2.00	120.00	0.00	2520.00	0.00
As 29	228.00	0.00690	60.00	2.00	120.00	0.00	3600.00	0.00
As 30	237.00	0.00729	60.00	2.00	120.00	0.00	4680.00	0.00
As 31	246.00	0.00768	60.00	2.00	120.00	0.00	5760.00	0.00
As 32	255.00	0.00807	60.00	2.00	120.00	0.00	6840.00	0.00
As 33	264.00	0.00846	60.00	2.00	120.00	0.00	7920.00	0.00
As 34	273.00	0.00885	60.00	2.00	120.00	0.00	9000.00	0.00
As 35	282.00	0.00924	60.00	2.00	120.00	0.00	10080.00	0.00
As 36	291.00	0.00963	60.00	2.00	120.00	0.00	11160.00	0.00
As 37	300.00	0.01002	60.00	2.00	120.00	0.00	12240.00	0.00
As 38	309.00	0.01042	60.00	2.00	120.00	0.00	13320.00	0.00
As 39	318.00	0.01081	60.00	2.00	120.00	0.00	14400.00	0.00
As 40	327.00	0.01120	60.00	2.00	120.00	0.00	15480.00	0.00
As 41	336.00	0.01159	60.00	2.00	120.00	0.00	16560.00	0.00
As 42	342.00	0.01185	60.00	4.68	280.80	0.00	40435.20	0.00
As 43	348.00	0.01211	60.00	4.68	280.80	0.00	42120.00	0.00
As 44	354.00	0.01237	60.00	4.68	280.80	0.00	43804.80	0.00
As 45	360.00	0.01263	60.00	4.68	280.80	0.00	45489.60	0.00
As 46	366.00	0.01289	60.00	4.68	280.80	0.00	47174.40	0.00
As 47	372.00	0.01315	60.00	4.68	280.80	0.00	48859.20	0.00
As 48	378.00	0.01341	60.00	4.68	280.80	0.00	50544.00	0.00
As 49	384.00	0.01367	60.00	4.68	280.80	0.00	52228.80	0.00
As 50	390.00	0.01393	60.00	4.68	280.80	0.00	53913.60	0.00
					5828.11	7727.66	485013.52	1323705.34
			Pn =		1899.50			
					7727.62	7727.66		

Mn = 150726.57 ft-kip

phi M =	125611.82 ft-kips	>	123500.00
phi P =	1583.00 kips	>	1583.00

Figure 51: Computed Wall Flexural Strength at 9th Level

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job:	EQX (E-W)	Date:	Feb, 06
Location:	9th Level		
Pier:	AH	Engineer:	Lourdes Diaz
Length:	16'		

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in^2	
x =	26.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in from extreme compression fiber	
Strain =	-0.0030 Ultimate concrete strain	(- compression)
c =	10.00 in	
β =	0.80	
ρ total =	0.0157	
Po=	20064.0 kip	

Governing Equation: 0.8D + 1.2E
Pu, kips = -2258.00
Mu, ft-kips = 8056.00
phi = 0.90

Reinforcement						#/row	Area / Bar	X
		cover	2.00					
Boundary rows	5	spacing	6.00	Bar Size	11	3	1.56	26.00
Internal rows	14	spacing	9.00	Bar Size	9	2	1.00	93.50
Total Rows	24							187.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	10.00	-0.00300	5.00		0.00	816.00	0.00	75072.00
As 1	2.00	-0.00240	60.00	4.68	0.00	280.80	0.00	26395.20
As 2	8.00	-0.00060	17.40	4.68	0.00	81.43	0.00	7166.02
As 3	14.00	0.00120	34.80	4.68	162.86	0.00	-13354.85	0.00
As 4	20.00	0.00300	60.00	4.68	280.80	0.00	-21340.80	0.00
As 5	26.00	0.00480	60.00	4.68	280.80	0.00	-19656.00	0.00
As 6	35.00	0.00750	60.00	2.00	120.00	0.00	-7320.00	0.00
As 7	44.00	0.01020	60.00	2.00	120.00	0.00	-6240.00	0.00
As 8	53.00	0.01290	60.00	2.00	120.00	0.00	-5160.00	0.00
As 9	62.00	0.01560	60.00	2.00	120.00	0.00	-4080.00	0.00
As 10	71.00	0.01830	60.00	2.00	120.00	0.00	-3000.00	0.00
As 11	80.00	0.02100	60.00	2.00	120.00	0.00	-1920.00	0.00
As 12	89.00	0.02370	60.00	2.00	120.00	0.00	-840.00	0.00
As 13	98.00	0.02640	60.00	2.00	120.00	0.00	240.00	0.00
As 14	107.00	0.02910	60.00	2.00	120.00	0.00	1320.00	0.00
As 15	116.00	0.03180	60.00	2.00	120.00	0.00	2400.00	0.00
As 16	125.00	0.03450	60.00	2.00	120.00	0.00	3480.00	0.00
As 17	134.00	0.03720	60.00	2.00	120.00	0.00	4560.00	0.00
As 18	143.00	0.03990	60.00	2.00	120.00	0.00	5640.00	0.00
As 19	152.00	0.04260	60.00	2.00	120.00	0.00	6720.00	0.00
As 20	161.00	0.04530	60.00	2.00	120.00	0.00	7800.00	0.00
As 21	167.00	0.04710	60.00	4.68	280.80	0.00	19936.80	0.00
As 22	173.00	0.04890	60.00	4.68	280.80	0.00	21621.60	0.00
As 23	179.00	0.05070	60.00	4.68	280.80	0.00	23306.40	0.00
As 24	185.00	0.05250	60.00	4.68	280.80	0.00	24991.20	0.00
					3647.66	1178.23	39104.35	108633.22
			Pn =		-2508.89			
					1138.78	1178.23		

Mn =	12311.46 ft-kip		
phi M =	11080.32 ft-kips	>	8056.00
phi P =	-2258.00 kips	>	-2258.00

Figure 52: Walls Flexural Stenght at 12th Level under Compression and Tension Controlled Regions

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: **12th Level** **Tension Controlled**
Pier: AH Engineer: Lourdes Diaz
Length: 16'

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in^2	
x =	21.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in	from extreme compression fiber
Strain =	-0.0030	Ultimate concrete strain (- compression)
c =	5.90 in	
β =	0.80	
ρ total =	0.0091	
Po=	18637 kips	

Governing Equation: 0.8D + 1.2E
Pu, kips = -1538.00
Mu, ft-kips = 4290.00
phi = 0.90

Reinforcement		cover	3.00			#/row	Area / Bar	X
Boundary rows	4	spacing	6.00	Bar Size	9	2	1.00	21.00
Internal rows	11	spacing	12.00	Bar Size	9	2	1.00	93.00
Total Rows	19							186.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	5.90	-0.00300	5.00		0.00	481.44	0.00	45082.04
As 1	3.00	-0.00147	42.76	3.00	0.00	128.29	0.00	11930.80
As 2	9.00	0.00159	45.71	2.00	91.42	0.00	-7953.86	0.00
As 3	15.00	0.00463	60.00	2.00	120.00	0.00	-9720.00	0.00
As 4	21.00	0.00768	60.00	2.00	120.00	0.00	-9000.00	0.00
As 5	33.00	0.01378	60.00	3.00	180.00	0.00	-11340.00	0.00
As 6	45.00	0.01988	60.00	2.00	120.00	0.00	-6120.00	0.00
As 7	57.00	0.02598	60.00	2.00	120.00	0.00	-4680.00	0.00
As 8	69.00	0.03208	60.00	2.00	120.00	0.00	-3240.00	0.00
As 9	81.00	0.03819	60.00	2.00	120.00	0.00	-1800.00	0.00
As 10	93.00	0.04429	60.00	2.00	120.00	0.00	-360.00	0.00
As 11	105.00	0.05039	60.00	2.00	120.00	0.00	1080.00	0.00
As 12	117.00	0.05649	60.00	2.00	120.00	0.00	2520.00	0.00
As 13	129.00	0.06259	60.00	2.00	120.00	0.00	3960.00	0.00
As 14	141.00	0.06869	60.00	2.00	120.00	0.00	5400.00	0.00
As 15	153.00	0.07480	60.00	3.00	180.00	0.00	10260.00	0.00
As 16	165.00	0.08090	60.00	2.00	120.00	0.00	8280.00	0.00
As 17	171.00	0.08395	60.00	2.00	120.00	0.00	9000.00	0.00
As 18	177.00	0.08700	60.00	2.00	120.00	0.00	9720.00	0.00
As 19	183.00	0.09005	60.00	3.00	180.00	0.00	15660.00	0.00
					2311.42	609.73	11686.14	57012.84
				Pn =	-1708.89			
					602.53	609.73		

Mn = 5723.25 ft-kip
phi M = 5150.92 ft-kips > 4290.00

Strain Compatibility for Shear Walls
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job: EQX (E-W) Date: Feb, 06
Location: **12th Level** **Compression Controlled**
Pier: AH Engineer: Lourdes Diaz
Length: 16'

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in^2	
x =	21.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in	from extreme compression fiber
Strain =	-0.0030	Ultimate concrete stra (- compression)
c =	78.00 in	
β =	0.80	
ρ total =	0.0091	
Po=	18637 kips	

Governing Equation: 0.8D + 1.2E
Pu, kips = 2910.00
Mu, ft-kips 4290.00
phi = 0.70

Reinforcement		cover	3.00			#/row	Area / Bar	X
Boundary rows	4	spacing	6.00	Bar Size	9	2	1.00	21.00
Internal rows	11	spacing	12.00	Bar Size	9	2	1.00	93.00
Total Rows	19							186.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	78.00	-0.00300	5.00		0.00	6364.80	0.00	412439.04
As 1	3.00	-0.00147	42.76	3.00	0.00	128.29	0.00	11930.80
As 2	9.00	0.00159	45.71	2.00	91.42	0.00	-7953.86	0.00
As 3	15.00	0.00463	60.00	2.00	120.00	0.00	-9720.00	0.00
As 4	21.00	0.00768	60.00	2.00	120.00	0.00	-9000.00	0.00
As 5	33.00	0.01378	60.00	3.00	180.00	0.00	-11340.00	0.00
As 6	45.00	0.01988	60.00	2.00	120.00	0.00	-6120.00	0.00
As 7	57.00	0.02598	60.00	2.00	120.00	0.00	-4680.00	0.00
As 8	69.00	0.03208	60.00	2.00	120.00	0.00	-3240.00	0.00
As 9	81.00	0.03819	60.00	2.00	120.00	0.00	-1800.00	0.00
As 10	93.00	0.04429	60.00	2.00	120.00	0.00	-360.00	0.00
As 11	105.00	0.05039	60.00	2.00	120.00	0.00	1080.00	0.00
As 12	117.00	0.05649	60.00	2.00	120.00	0.00	2520.00	0.00
As 13	129.00	0.06259	60.00	2.00	120.00	0.00	3960.00	0.00
As 14	141.00	0.06869	60.00	2.00	120.00	0.00	5400.00	0.00
As 15	153.00	0.07480	60.00	3.00	180.00	0.00	10260.00	0.00
As 16	165.00	0.08090	60.00	2.00	120.00	0.00	8280.00	0.00
As 17	171.00	0.08395	60.00	2.00	120.00	0.00	9000.00	0.00
As 18	177.00	0.08700	60.00	2.00	120.00	0.00	9720.00	0.00
As 19	183.00	0.09005	60.00	3.00	180.00	0.00	15660.00	0.00
					2311.42	6493.09	11686.14	424369.84
				Pn =	4157.14			
					6468.57	6493.09		

Mn = 36336.33 ft-kip
phi M = 25435.43 ft-kips > 4290.00

Figure 53: Walls Combined Flexural and Axial Strength at 16th Level

<u>**Strain Compatibility for Shear Walls**</u>
Limiting Concrete Strain
Rectangular Stress Block (ultimate)

Job:	EQX (E-W)	Date:	Feb, 06
Location:	**16th Level (Tension Controlled)**		
Pier:	AH	Engineer:	Lourdes Diaz
Length:	16'		

General Information

f'c =	5.00 ksi	
fy =	60.00 ksi	
l =	192.00 in	
b =	24.00 in	
Ag =	4608.00 in^2	
x =	15.00 in	Compression Zone Boundary Element Length
y =	24.00 in	Compression Zone Boundary Element Width
cg =	96.00 in from extreme compression fiber	
Strain =	-0.0030 Ultimate concrete strain	(- compression)
c =	8.70 in	
β =	0.80	
ρ total =	0.0058	
Po=	17919 kips	

Governing Equation	0.8D + 1.2E
Pu, kips =	-646.00
Mu, ft-kips =	2400.00
phi =	0.90

Reinforcement		cover	3.00			#/row	Area / Bar	X
Boundary rows	2	spacing	12.00	Bar Size	8	2	0.79	15.00
Internal rows	12	spacing	12.00	Bar Size	8	2	0.79	93.00
Total Rows	16							186.00

Layer	y,in	Strain	c or fs, ksi	As, in^2	Ts, kips	Cc or Cs, kips	Moment Calculations	
Compression Zone	8.70	-0.00300	5.00		0.00	709.92	0.00	65681.80
As 1	3.00	-0.00197	57.00	1.58	0.00	90.06	0.00	8375.58
As 2	15.00	0.00217	60.00	1.58	94.80	0.00	-7678.80	0.00
As 3	27.00	0.00631	60.00	1.58	94.80	0.00	-6541.20	0.00
As 4	39.00	0.01045	60.00	1.58	94.80	0.00	-5403.60	0.00
As 5	51.00	0.01459	60.00	1.58	94.80	0.00	-4266.00	0.00
As 6	63.00	0.01872	60.00	1.58	94.80	0.00	-3128.40	0.00
As 7	75.00	0.02286	60.00	1.58	94.80	0.00	-1990.80	0.00
As 8	87.00	0.02700	60.00	1.58	94.80	0.00	-853.20	0.00
As 9	99.00	0.03114	60.00	1.58	94.80	0.00	284.40	0.00
As 10	111.00	0.03528	60.00	1.58	94.80	0.00	1422.00	0.00
As 11	123.00	0.03941	60.00	1.58	94.80	0.00	2559.60	0.00
As 12	135.00	0.04355	60.00	1.58	94.80	0.00	3697.20	0.00
As 13	147.00	0.04769	60.00	1.58	94.80	0.00	4834.80	0.00
As 14	159.00	0.05183	60.00	1.58	94.80	0.00	5972.40	0.00
As 15	171.00	0.05597	60.00	1.58	94.80	0.00	7110.00	0.00
As 16	183.00	0.06010	60.00	1.58	94.80	0.00	8247.60	0.00
As X	195.00	0.06424	60.00	1.58	94.80	0.00	9385.20	0.00
					1516.80	799.98	13651.20	74057.38
			Pn =		-717.78			
					799.02	799.98		

Mn =	7309.05 ft-kip		
phi M =	**6578.14 ft-kips**	>	**2400.00**
phi P =	**-646.00 kips**	>	**-646.00**