Washington Park Condominiums

Mt. Lebanon, Pennsylvania

Final Thesis Report : Design of Reinforced Concrete Structures

Architectural Engineering
2008-2009 Senior Thesis

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Mt. Lebanon, Pennsylvania

General Building Data

Location: Bower Hill Rd, Mt. Lebanon, PA
Size: 148,000 Sq. Ft,
Height: 9 levels above ground, 2 levels below
Construction Dates: Fall 2008 to Fall 2010
Cost: $ 24 million
Delivery Method: Design-Bid-Build

Design Team

Owner: Zamagias Properties
Architect: Indovina Associates Architects
General Contractor: PJ Dick, Inc.
Structural Engineer: WBCM, LLC
MEP Engineers: CJL Engineering
Geotechnical Engineer: PSI, Inc.

Architectural

9 Story Mixed Use Building containing:
- 1st floor retail shops
- 2nd thru 8th floors are condos

Facade includes red and tan brick veneers, up to the 6th floor

7th and 8th Floor Penthouses have painted horizontail lapped fiber-cement plank siding

Structural

Pile and Spread Footing Foundation System, with Slab On Grade used in places to tie the system together.

Precast Concrete Plank system in Parking areas, as well as 1st and 2nd Floors.
Vescom Composite Joist Floor System used as Architectural element for ceiling and MEP installation.

Fully and Partially Restrained Moment Frames

MEP

208Y/120V, 3 phase 4 Wire system, with individual fixed mounted panels in each Condo.

High Efficiency Split System with Natural Gas Furnace and DX Cooling Units in each Conco.

2- Roof top Heat Recovery Units

Wet and Dry Pipe Automatic Sprinkler Systems throughout building.

Benjamin Follett | Structural Option
http://www.engr.psu.edu/ae/thesis/portfolios/2009/blf5000/

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Executive Summary:

This report is the result of a yearlong study of Washington Park Condominiums, which is a 9 story, 148,000 ft^2 multi-use retail and residential building located in Mt. Lebanon, Pennsylvania. The existing design consists of a precast concrete plank system on the first two floors and a composite steel joist system on the remaining seven floors. Both systems are used to resist the gravity loads on the structure. The composite steel joist system was also used as an architectural element in the building because of its ability to integrate the mechanical systems within the depth of the joists. This proved to be a valuable aspect of the design, which allowed for higher ceiling heights and a more upscale feel to the apartment units. To resist the lateral loads of the structure, steel moment resisting frames were designed. These frames begin on the second floor and continue up through the top of the building. Brace frames were also designed and placed in the basement of the building to help transfer the lateral loads due to wind, seismic and soil pressure to the foundations. Overall, this design effectively resists the gravity and lateral loads of the building.

The purpose of this study is to redesign the existing structural system of the building, while determining whether or not the existing design is the most efficient. The proposed gravity system of the structure consists of a two way flat plate slab supported by reinforced concrete columns. This system allows for little change in the buildings column grid and floor plans. The planned lateral system of the building originally consisted of reinforced concrete shear walls located around the stair and elevator shafts of the building. However, after initial design analysis the design was updated to include exterior concrete moment frames. These moment frames will add stiffness to the building along with reducing the torsional effects of the lateral forces.

The shear walls in the building were designed as ordinary reinforced concrete shear walls because of the seismic design category that the building falls in. In contrast, the concrete moment frames of the building are designed as intermediate moment frames so more in depth detailing, reinforcement and design could be explored as part of the study. Overall, the new lateral system is designed to comply with all code requirements as laid out in ASCE 7-05 and ACI 318-08.

Lastly, two breadth studies will be completed as a way to see how the change in structure impacts other systems within the building. The acoustics study to be performed was requested by the owner because of concerns stemming from the amount of sound transmission that would occur from noisy spaces into the apartment units. Finally, an architectural detailing study will be completed in order to determine the impact that changing the structure has on the placement of the mechanical systems of the building.

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Introduction:

Washington Park Condominiums is a multi-use retail and residential building located at the intersection of Bower Hill Road and Washington Road in Mt. Lebanon, Pennsylvania. Site work and excavation has begun at the site and construction should begin sometime before the end of the Fall 2008, with the project lasting until Fall 2010. Washington Park Condominiums is the first of two buildings proposed to be built on the site. Building One is a nine-story, 148,000 ft^2 structure which is owned by Zamagias Properties of Pittsburgh, PA. A site plan for the building can be seen in Figure 1 below. The building was architecturally designed by Indovina Associates Architects and is being constructed by PJ Dick, Inc. for a price of $23,418,000. The building's primary use is residential and it contains 7 stories of condominiums on the 2^{nd} through 8^{th} floors. The first floor of the building is used for retail space and as a location for extra amenities for the residents of Washington Park. The building also contains two below grade levels of parking. The enclosed parking garage contains 78 parking spaces that can be used by the residents. Two elevators and two stairs serve the parking areas that also contain resident storage, a wine room and trash collection along with mechanical and electrical rooms. The ground floor serves primarily as retail space with four separate areas available for possible tenants. Also contained on the floor is a resident exercise room and a private entrance and lobby for the residents.

As the building moves to the second floor, the function changes from primarily retail to one of solely residential with six upscale condominiums located on the floor. These condominiums each have different floor plans and layouts with overall areas ranging from 1523 ft^2 to 2288 ft^2. Each unit contains two or three bedrooms and bathrooms depending on size, along with a living room, dining room, kitchen, study, laundry, entry and in some cases a balcony. This floor layout continues throughout the next four floors, with a total of 30 units on floors 2 through 6. The 7^{th} and 8^{th} floors of the building are the penthouse level. This floor contains five condominiums that range from 1732 ft^2 to 2453 ft^2. These units contains the same amenities and spaces as the units on the below floors do. All of the condominiums floors are served by two elevators and two stairways that are connected by a hallway that runs through the center of the building in the long direction. Finally, the roof contains mechanical spaces that are accessed by using the northern most stairway or elevator.

The typical exterior wall system of the building consists mainly of 4" brick veneer backed by a 2" airspace and 2" of rigid XPS insulation, then containing another 2" layer of rigid spray-foam insulation that is followed by an airspace and then 5/8" gypsum board. This exterior wall system is typical for the first 6 floors of the building. The 7^{th} and 8^{th} floors of the building consist of a similar wall construction except for the exterior façade which is a 5/16" layer of painted fiber-cement siding.

Figure 1: Site Plan

Existing Composite Joist and Precast Concrete Plank System:

Foundations

The foundation system can be best described as a spread footing system with attached concrete piers. The sizes for the spread footings range from the smallest, a 4’-0” x 4’-0” x 2’-0” footing with #8 @ 12” each way, to a 14’-0” x 14-0” x 3’-6” footing with #8 @ 6” each way with the deepest of the footings will be 25’-0” below grade. In addition to the spread footings, interior and exterior wall footings were used and are either 2’-0” or 3’-0” wide by 1’-4” deep. The steel reinforcing in these wall footings are (3) #5 continuous bars and #5 x 1’-8” @ 16”.

The slab on grade in this system consist of either a 6” or 8” normal weight concrete slab reinforced with 6x6-W2.9xW2.9 welded wire fabric or 6x6-W4xW4 welded wire fabric. The slab on grade is also thickened to a minimum if 1’-0” at non-load bearing walls and (2) #4 bars are added for tensile strength. Connecting the columns to the slab on grade and the footings are column piers that range from 16” x 16” with (4) #7 of vertical reinforcement to 40” x 40” w/ (12) #7 of vertical reinforcement and f'_c = 4000 psi concrete is used for the entire system.

Floor Systems

Two separate floors systems are typical within the structure of Washington Park. The first is a precast concrete plank system that is used in the parking areas as well as the first and second floor framing. The precast concrete plank is 8” thick and also contains a 2” thick structural topping. The reinforcing in the structural topping is 6x6-W1.4xW1.4 welded wire fabric. The precast concrete plank system bears on W shapes which then carry the load to the columns. This system was used in the parking areas because of the systems diaphragm capacity (ability to transfer horizontal loading) and because of its durability and strength.

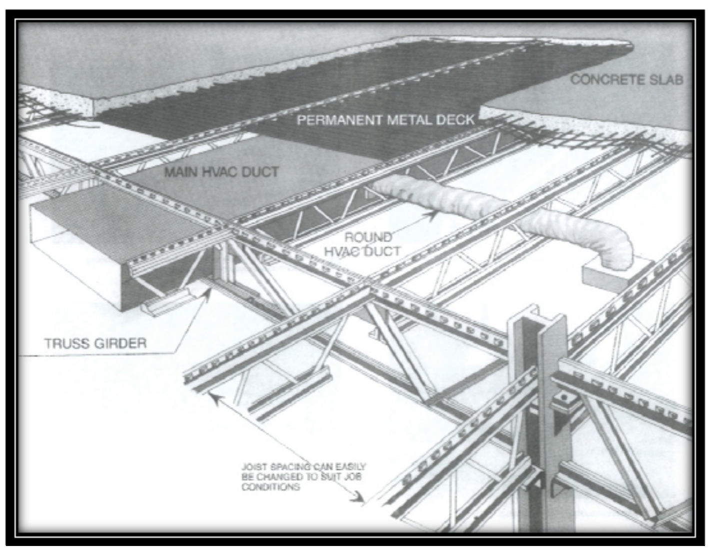


Figure 2: VESCOM Floor System

The second primary floor system in the building is the VESCOM composite joist floor system. The composite joist system interlocks the top chord of a joist with the concrete producing less deflection, less vibration and greater stiffness. The floor construction consists of a 2 11/16” reduced weight concrete slab that is poured on top of the 1 5/16", 22 Gage galvanized floor decking. The bottom chord acts as the main tension member, and in the composite stage the embedded top chord serves as a continuous shear connection. The concrete is also reinforced with welded wire fabric and compressive strength of the concrete is $f'_c = 3500$ psi. Finally, the system was used as an architectural element since the ceiling could be installed directly to the joist bottom chord and the mechanical systems (HVAC, plumbing, fire protection, electrical and telecommunications) could be installed with the joist system, saving space and allowing for higher ceilings and floor to floor height within the apartments.

Lateral System

The lateral resisting system within the building is mainly moment resisting steel frames made up of wide flange beams. These frames begin on the second floor and continue up through the top of the building. These frames run in the north-south direction and run along column lines A, B, C and D. Rigid connections also occur on these floors along column lines 1 through 9. Figure 1 below shows the four different types of moment frames that exist within the building. Since the VESCOM floor system is being used as a diaphragm to transfer shear loading the load path begins at the exterior beams and then continue on through the floor system to joist girders which are to be designed and manufactured by the joist manufacturer.

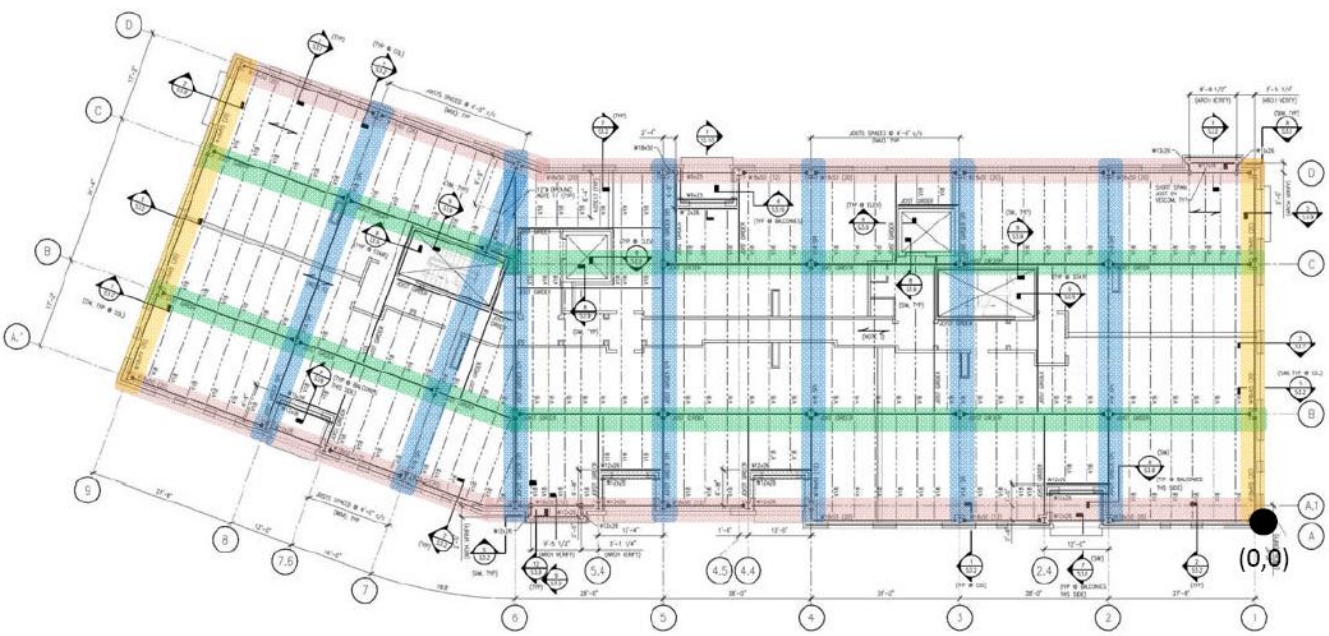


Figure 3: Moment Frame Diagram

North – South Frames (A & D)

East – West Frames (F thru L)

North – South Frames (B & C)

East – West Frames (E & M)

The load is then transferred into the large W14 columns, and finally to the brace frames and the foundations. There are a total of eleven braced frames located in the basement and subbasement levels running along column lines 1 through 11 from column lines A.1 to B. The brace frames are 17'-2" in length and they begin at the sub-basement level and connect into the framing for the ground floor. The bracing in the frames consists of HSS 8x8x1/2 up to the basement level, and HSS 6x6x3/8 from the basement level to the ground floor. These frames are shown in Figure 3 above. This plan detail and the detail of the brace frames can be found in Figures 4 and 5.

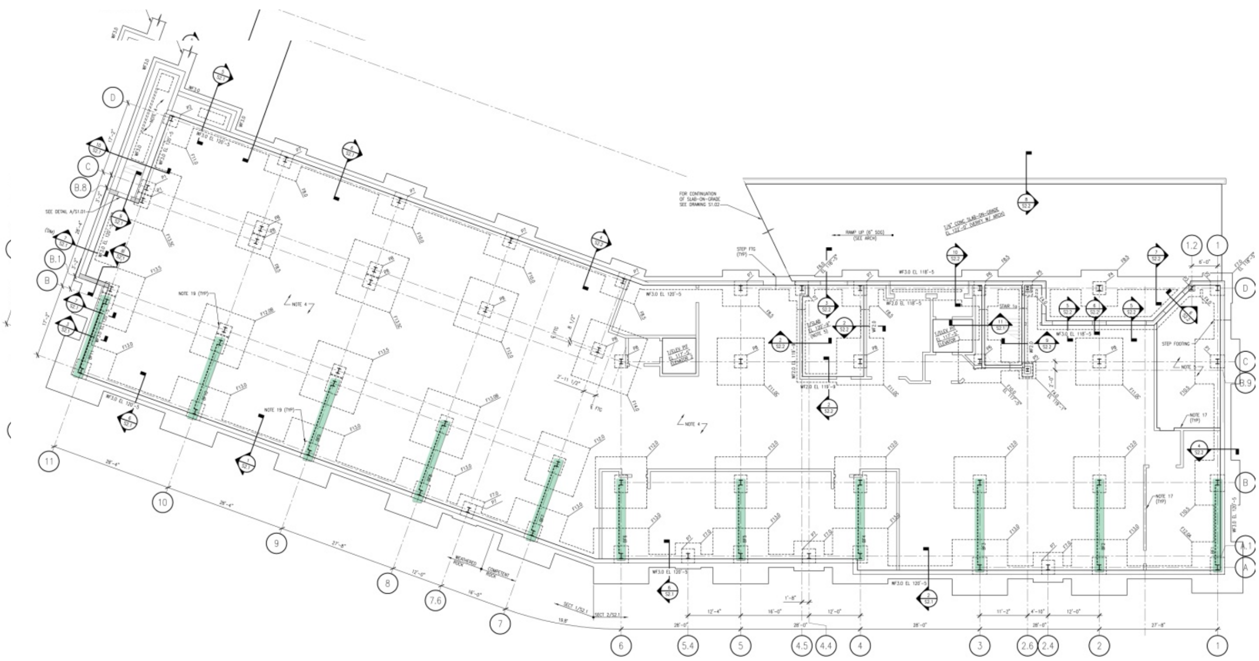


Figure 4: Braced Frame Plan

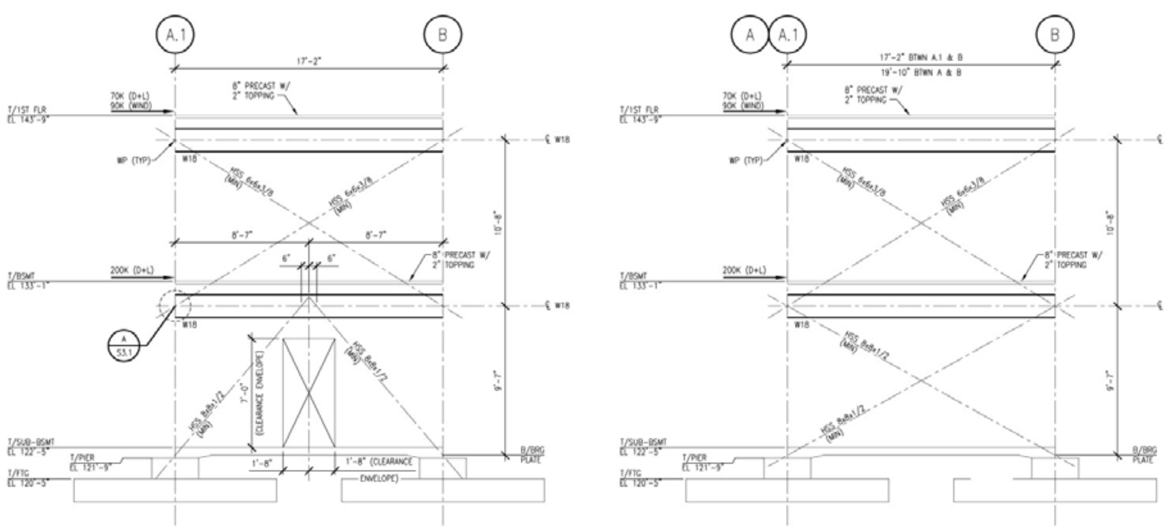


Figure 5: Braced Frame Elevations

Roof System

The flat roof system of the building is similar to floors 3 through 6 in that it also utilizes the VESCOM composite floor system. The only main difference in the system is that the concrete is 4 11/16" thick. The pitched roof is framed with a typical pre-engineered light gauge roof truss system spaced at 2'-0" on center. This roof is topped with asphalt shingles and ½" cement bonded particle board. Finally, the roof of the stair towers are framed with light gauge purlins, topped with roof insulation and shingles.

Columns

The columns in Washington Park Condominiums have all been designed using AISC 9^{th} Ed. ASD and are ASTM A992 Grade 50 wide flange columns. The columns are spaced at 27'-8" or 28'-0" in the north-south direction and 17'-2" or 28'-4" in the east-west direction. The columns at the base of the structure that run the entire height of the building range from W12x96 to W14x193 at the bottom to W12x40 and W14x74 at the top of the structure.

Building Systems:

Construction

The construction of Washington Park Condominiums is slated to begin sometime in the fall of 2008 with an estimated completion date of fall 2010. The general contractor on the project is PJ Dick Inc. and the building is being delivered as a design-bid-build construction project. Site work has begun and the site has been excavated up to 25 feet in some places to reach the lower garage finished floor elevation.

Fire Protection

The owner of Washington Park Condominiums chose to protect the building from fire with an automatic sprinkler system. The basement and sub-basement areas are protected by a dry pipe system with the first through penthouse floors protected by a wet pipe system. Smoke detectors are also located throughout the first floor lobby and retail spaces as well as in the residential units on floors 2 thru 8. There are also fire alarms located in the hallways and lobby on each floor, including the parking garages. These alarms are connected to a main fire alarm control box located on the first floor. Since the building is a multi-use structure there is some fire separation required. For most of the building a 2 hour fire separation between building functions is recommended by code.

Electrical

The electrical system in Washington Park consists of a 208Y/120V 3 phase, 4 wire system with individually fixed mounted panels in each unit and separate panels in the electrical rooms located on the sub basement, basement and first floors. The building's main electrical room as well as the emergency generator is located on the sub basement garage floor. The emergency generator provides emergency power to operate life safety items such as fire alarm, exit signs, emergency lighting and necessary mechanical systems.

Lighting

Primary hallway lighting on the 1st thru 8th floors is provided by fluorescent wall mounted scones. In the condominiums there are two types of fixtures that are primarily used. These include 4' fixed mounted T5 fluorescents in the closets and 75W recessed down lights with 6" aperture white baffled reflector trim throughout the rest of the spaces. The basement and garage spaces use 150W Quartz restrike surface mounted lights as well as 4' fixed mounted T8 fluorescents. Exterior lighting includes decorative pole mounted fluorescent fixtures and wall mounted metal halide fixtures. The building also utilizes natural lighting through large windows, 10' ceiling heights and fact that the long direction of the building is in the east to west direction.

Mechanical

The mechanical system of the building is mostly individualized to each residential unit. Each unit shall have an electric water heater located in the unit's mechanical room. Units on floors 2 through 6 shall have a 50 gallon tank and units on 7th and Penthouse shall have 80 gallon tanks. Each unit is also conditioned with a high efficiency gas fired furnace and air conditioning unit utilizing environmentally friendly refrigerant. The furnaces are located in the units mechanical room with the associated air conditioning unit located on the roof. These furnaces are individually controlled through thermostats in the each apartment for desire temperature and comfort. The building also has conditioned fresh outside air supplied to each retail and unit space. The outside air is processed by a roof mounted energy recovery unit and then delivered through insulated metal ductwork to each furnace system.

Transportation

The main entrances to the building are located on the east and west sides of the building. The southwest corner of the building contains the ground floor entrance and lobby for residents. This lobby serves floors 2 thru 8 by a main stair and elevator. The parking garage entrance is located on the northwest corner of the building. From the garage, either of the two stairways or elevators can be used depending on where the resident parks. The eastern side of the building contains the main entrances to the retail spaces. These retail spaces can also be accessed through the garage by using either of the stairs or elevators and then using the retail service corridor.

Other Systems

The building also utilizes a security system with key card access for the residents to the lobby which controls the electromagnetic locks on the lobby doors. Each residential unit also contains multiple land line telephone outlets, cable television outlets and data receptacles.

Applicable Codes, Design Requirements and Load Cases:

Design Standards

International Building Code 2003 w/ Amendments for Mt. Lebanon

ACI 318-08 (Reinforced Concrete Design)

AISC 13th Edition ASD (Structural Steel Design)

ACI 530-02 (Masonry Design)

ASCE 7-05 (Minimum Design Loads for Buildings and other structures)

Deflection Criteria

Floor Deflection Criteria

L/240 Total Load and L/360 Live Load Deflections

L/360 Gravity Loads on Exterior Stud Walls

L/600 Gravity Loads on Masonry/Veneer Backup Walls

Lateral Deflection Criteria

Δ = H/400 for Allowable Story and Building Drift due to Wind Loading

$\Delta = 0.020h_{sx}$ for Allowable Story and Building Drift due to Seismic Loading

Design Load Combinations

The following Load and Resistance Factor Design load combinations were considered for analysis, as noted in ASCE 7-05 Chapter 2:

1.4(Dead)
1.2(Dead) + 1.6(Live) + 0.5(Roof Live)
1.2(Dead) + 1.6(Roof Live) + 0.8(Wind)
1.2(Dead) + 1.6(Snow) + 0.8(Wind)
1.2(Dead) + 1.6(Snow) + 1.0(Live)
1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Snow)
1.2(Dead) + 1.6(Wind) + 1.0(Live) + 0.5(Roof Live)
1.2(Dead) + 1.0(Earthquake) + 1.0(Live) + 0.2(Snow)

Gravity Design Loads:

These loads were calculated by using the square foot weight found either in the specifications or the code and were multiplied by the square footage of the floor. The weight for each level was calculated using the superimposed dead load and dead load of the slab and columns. Live loads for the building were determined using the information provided by the design professionals and by using the live load tables in ASCE 7-05 Chapter 4.

Dead Load Table				
Floor Dead Load			Roof Dead Load	
Material/System	Load		Material/System	Load
10" Reinforced Concrete Slab	125 psf		10" Reinforced Concrete Slab	125 psf
Brick Veneer w/ studs	40 psf		MEP	6 psf
Normal Weight Concrete	150 pcf		Sprinklers	3 psf
MEP	6 psf		Ceiling	8 psf
Sprinklers	3 psf		Asphalt Shingles/Felts	4 psf
Ceiling	5 psf			
Floor Finishes	5 psf			
Partitions	20 psf			
Total Superimposed Dead Load	39 psf			

Table 1: Dead and Superimposed Dead Loads

Live Load Table				
Floor Live Load Table			Roof Live and Snow Load Table	
Occupancy	Load		Material/System	Load
Typ. Condominium Floor	40 psf		Roof Live Load	20 psf
Stairs	100 psf		Roof Live Load (Mechanical)	150 psf
First Level (Plaza and Traffic/Parking Areas	250 psf		Ground Snow Load (P_g)	25 psf
First Level (Non-Plaza Areas)	100 psf		Flat Roof Snow Load (P_f)	23.1 psf
Basement Level Parking Areas/Ramps	50 psf		Exposure Factor (C_e)	1.2
Slabs-on-Grade	150 psf		Thermal Factor (C_t)	1
Exercise Area at Ground Floor	150 psf		Importance Factor (I)	1.1
Corridors On 1st Floor	100 psf		Terrain Category	B
Corridors Above 1st Floor	80 psf			
Mech/Elec Spaces	150 psf			
Second Floor Terrace	100 psf			
Apartment Balconies	100 psf			

Flat Roof Snow Load Equation:
$P_f = 0.7C_eC_tIp_g$

Table 2: Live and Snow Loads

Lateral Design Loads:

Wind

The wind loads for Washington Park Condominiums were calculated using the design criteria found in ASCE 7-05, Chapter 6 and it was determined that it was permitted to use Method 2 – Analytical Procedure for the design. The table below lists the applicable wind design factors.

Basic Wind Speed (V)	90 mph
Wind Direction Factor (K_d)	0.85
Importance Factor (I)	1
Exposure Category	C
Velocity Pressure Coefficient (K_z)	Case 2
Topographic Factor (K_{zt})	1
Enclosure Class	Enclosed

Table 3: Wind Design Criteria

The Analytical Procedure for design was used to determine the wind loading in the North-South, East-West and North West-South East direction. The loads shown in the tables below were calculated by hand and were used solely for comparison purposes with the wind loading found using ETABS. The use of the loading in ETABS allowed for the consideration of accidental torsion caused by the difference in the location of the center of mass and center of rigidity. The wind loads found using ETABS are shown throughout the report for specific shear walls, coupling beams, moment frame beams and columns.

Wind (North - South Direction) B = 68'-8" & L = 216'-5 1/2"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	12.5	0.849	0.00	0.00	0.00	0.00	0.00	133.686	6487.847
Second	14.33	12.667	0.849	14.964	10.403	-5.758	16.161	20.158	133.686	6487.847
Third	25.33	11.000	0.948	16.709	11.616	-5.758	17.374	13.123	113.528	4905.356
Fourth	36.33	11.000	1.023	18.031	12.535	-5.758	18.293	13.817	100.405	3728.723
Fifth	47.33	11.000	1.081	19.053	13.246	-5.758	19.003	14.354	86.588	2076.624
Sixth	58.33	11.000	1.130	19.917	13.846	-5.758	19.604	14.807	72.235	1826.709
Seventh	69.33	13.167	1.172	20.657	14.361	-5.758	20.118	18.190	57.427	1113.563
Eighth	82.67	13.500	1.216	21.433	14.900	-5.758	20.658	19.150	39.237	536.123
Roof	96.33	13.833	1.256	22.138	15.390	-5.758	21.148	20.088	20.088	277.877

Table 4: ASCE 7-05 Wind Loading Direction #1

Wind (East - West Direction) B = 216'-5 1/2" & L = 68-8"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	12.5	0.849	0.00	0.00	0.00	0.00	0.000	493.889	23767.869
Second	14.33	12.667	0.849	14.964	10.140	-9.353	19.493	76.644	493.889	23767.869
Third	25.33	11.000	0.948	16.709	11.322	-9.353	20.675	49.228	417.245	17927.222
Fourth	36.33	11.000	1.023	18.031	12.218	-9.353	21.571	51.361	368.017	13608.280
Fifth	47.33	11.000	1.081	19.053	12.910	-9.353	22.263	53.010	316.656	9889.078
Sixth	58.33	11.000	1.130	19.917	13.496	-9.353	22.849	54.404	263.646	6650.892
Seventh	69.33	13.167	1.172	20.657	13.997	-9.353	23.350	66.551	209.242	4050.000
Eighth	82.67	13.500	1.216	21.433	14.523	-9.353	23.876	69.770	142.692	1947.879
Roof	96.33	13.833	1.256	22.138	15.001	-9.353	24.354	72.922	72.922	1008.730

Table 5: ASCE 7-05 Wind Loading Direction #2

Wind (Southeast - Northwest Direction) B = 73'-5 1/4" & L = 66-0"										
Floor	Height (ft)	Tributary Height (ft)	K_z	q_z (psf)	Windward (psf)	Leeward (psf)	Total (psf)	Story Force (kips)	Story Shear (kips)	Overturning Moment (ft-kips)
Ground	0.00	12.5	0.849	0.00	0.00	0.00	0.00	0.000	171.518	8251.916
Second	14.33	12.667	0.849	14.964	10.379	-9.574	19.953	26.617	171.518	8251.916
Third	25.33	11.000	0.948	16.709	11.589	-9.574	21.163	17.096	144.901	6225.748
Fourth	36.33	11.000	1.023	18.031	12.506	-9.574	22.080	17.837	127.805	4725.865
Fifth	47.33	11.000	1.081	19.053	13.215	-9.574	22.789	18.409	109.968	3418.113
Sixth	58.33	11.000	1.130	19.917	13.814	-9.574	23.388	18.893	91.559	2309.715
Seventh	69.33	13.167	1.172	20.657	14.328	-9.574	23.902	23.112	72.666	1414.554
Eighth	82.67	13.500	1.216	21.433	14.866	-9.574	24.440	24.230	49.554	676.454
Roof	96.33	13.833	1.256	22.138	15.355	-9.574	24.929	25.324	25.324	350.307

Table 6: ASCE 7-05 Wind Loading Direction #3

Seismic

The seismic loads for Washington Park Condominiums were calculated using ASCE 7-05, Chapter 12, as well as using the information provided by the structural engineer and the geotechnical engineer. From the geotechnical report, it was determined that the Site Class for construction would be Site Class C. The remainder of the information needed to calculate seismic loading and base shear was found in Chapter 12 of ASCE 7-05. For the building, the base shear was calculated to be approximately 339 kips in both directions. The table below lists the applicable seismic design factors.

Seismic Parameters for Washington Park Condominiums		
Occupancy Category		II
Seismic Use Group		I
Site Class		C
Seismic Design Category		B
Short Period Spectral Response	S_S	0.128
Spectral Response (1 sec)	S_1	0.058
Design Short Period Spectral Response	S_{DS}	0.102
Design Spectral Response (1 sec)	S_{D1}	0.0646
Importance Factor	I	1.0
Response Modification Factor	C_S	0.017
Seismic Response Coefficient	R	4
Coefficient for Upper Limit	C_u	1.7
Approximate Fundamental Period	T_a	0.615
Upper Limit of Period	T	1.046
Long Period Transition Period	T_L	12

Table 7: ASCE 7-05 Seismic Design Criteria

Seismic Base Shear							
Floor	Height (ft)	Tributary Height (ft)	Dead Load (kips)	$w_x h_x^k$	C_{vx}	Lateral Force (kips)	Story Shear (kips)
Roof	96.333	13.833	2539.41	851291.9155	0.281064355	95.433	95.433
8th	82.667	13.5	2211.15	610073.3682	0.201423125	68.392	163.825
7th	69.333	13.167	2204.15	486135.9702	0.160503689	54.498	218.323
6th	58.333	11	2158.65	382112.418	0.126159051	42.836	261.159
5th	47.333	11	2158.65	292864.0366	0.096692615	32.831	293.990
4th	36.333	11	2158.65	209144.1564	0.069051481	23.446	317.436
3rd	25.333	11	2158.65	132152.4757	0.043631743	14.815	332.251
2nd	14.333	12.667	2193.65	65040.58905	0.02147394	7.291	339.543
Ground	0	12.5	2190.15	0	0	0	339.543
Total	96.333		19973.09	3028814.93	1.0000	339.54	339.54

Table 8: Seismic Loads per ASCE 7-05

Problem Statement:

The current design of Washington Park Condominiums implements a composite joist floor system for the resistance of gravity loads and a steel moment frames for the resistance of the lateral loads found on the site. These systems are sufficient in carrying the loading on the structure and also accomplish the architectural requirements required by the architect. Although, the composite joist floor system is optimized for residential applications, it creates a few major problems for engineers. Since the system is used less than typical steel construction, many engineers and construction managers are not fully familiar with the system. This causes issues and delays in design and construction of the structure and ultimately costs the owner of the building precious time and money. These issues add to the notion that the current system used for the resistance of gravity loads within the structure is not the most efficient or cost effective solution.

In conjunction with the gravity load resisting system, steel moment frames are used for the resistance of the lateral loads. There are thirteen primary moment frames that can be found on floors 1 thru 8. Four of the frames run in the entire length of the building in the north-south direction while the other nine frames run the length of the building in the east-west direction. These thirteen frames use the majority of the columns, girders and beams within the frames to resist the lateral load on the building. Since most of the structure is moment frames, most of the connection between girders or beams and columns are moment connections. The connections between in the frames running in the north-south direction are primarily semi-rigid moment connections whereas the connections in the frames in the east-west direction are rigid moment connections. Both of these types of moment connections are more expensive than conventional gravity connections between beams and columns. Because of the need for so many moment frames, and therefore so many moment connections, it is likely that there is a better and more efficient structural solution available that can be used to resist the lateral loads found on the building.

Problem Solution:

In an effort to alleviate the shortcomings that are found in the current structural system, a complete redesign of the gravity and lateral systems are proposed for Washington Park Condominiums. The redesign follows the conditions and requirements set out in ASCE 7-05 (Minimum Design Loads for Buildings) and ACI 318-08 (Building Code Requirements for Structural Concrete).

The new structural design will be a two way flat plate concrete slab system with cast in place concrete columns. The design will take into account the fact that the slab thickness needs to be optimized within the design so the ceiling height within the apartments can be maximized. Moreover, this system will be able to use the same column grid as the steel structure with possibly a few changes near the elevator shafts and stairways. The foundations for the building will also be resized and redesigned where necessary to account for the additional weight of the building. The lateral system will consist of a shear walls designed to carry the lateral loads on the building. These shear walls can be placed around the elevator shafts and staircases located within the interior of the building causing minimal interference with the architectural aspects of the floor plan. The design and analysis of the building will be done using PCASlab for two way slab design, PCAColumn for reinforcement concrete column design and ETABS for the design of the reinforced concrete shear walls. The concrete structural system is a possible design alternative because one of the major advantages concrete construction for high-rise buildings is the material's inherent properties of heaviness and mass, which create lateral stiffness, or resistance to horizontal movement. Occupants of concrete towers are less able to perceive building motion than occupants of comparable tall buildings with non-concrete structural systems. The ability to perceive less building motion is also important in the case of vibration caused by lateral loads, mechanical equipment and elevators.

Design Goals:

The main goal of a new structural system for Washington Park Condominiums is to replace the current gravity resistance floor system consisting of composite steel joists and precast planks with a reinforced flat plate concrete slab. Moreover, the lateral force resisting steel moment frame system will be replaced with shear walls that are placed around the elevator and stairway shafts. This study is being conducted primarily to explore reinforced concrete as a structural system and to learn what benefits it has to offer over the current structural steel system. In addition to this general objective, other goals where determined before the beginning of the study in hopes of advancing knowledge and understanding of concrete structures. These goals are listed as follows:

- Study and compare the differences between a structural steel and structural concrete system
- Adhere to the current column layout within the building to reduce the impact a new structural system will have on the floor plans and architecture of the building.
- Design a two way flat plate concrete system that efficiently resists the gravity loads of the building.
- Use PCASlab to design the two way flat plate system and then verify the results using hand calculation learned in AE 431 (Design of Concrete Structures)
- Use ETABS to analyze a 3D structure and obtain the lateral loads caused by wind and seismic forces
- Use loads determined from ETABS to design reinforced concrete shear walls, coupling beams and columns.
- Maintain the allowable story and overall drift of the structure to be less than H/400 for wind and $\Delta = 0.007h_{sx}$ for seismic.
- Effectively design all structural systems so that IBC 2003, ASCE 7-05, and ACI 318-08 are satisfied.
- Use all design information to advance my knowledge and proficiency of concrete structures
- Conclude whether or not a concrete structural system is an efficient and capable redesign option

Structural Depth Study – Gravity Redesign:

The solutions and redesigns found in this section are in direct response to the problem statement given in the proposal and earlier in this report. The design of the new structural system has been completed so that it complies with all codes and specifications listed above. The overall design of the new concrete system will ultimately be compared to the existing steel structure with conclusions being drawn concerning constructability, cost, acoustics, and building system coordination.

Introduction

The proposed gravity system uses a two way reinforced concrete flat plate system, which transfers loads to all supporting columns. The fact that the existing system employs very typical interior square bays makes the two way flat plate system a viable option for analysis. This system was chosen based on the fact that the existing column layout could remain intact and a new lateral system could be designed to resist the lateral loads on the structure.

Design Process – Two Way Flat Plate

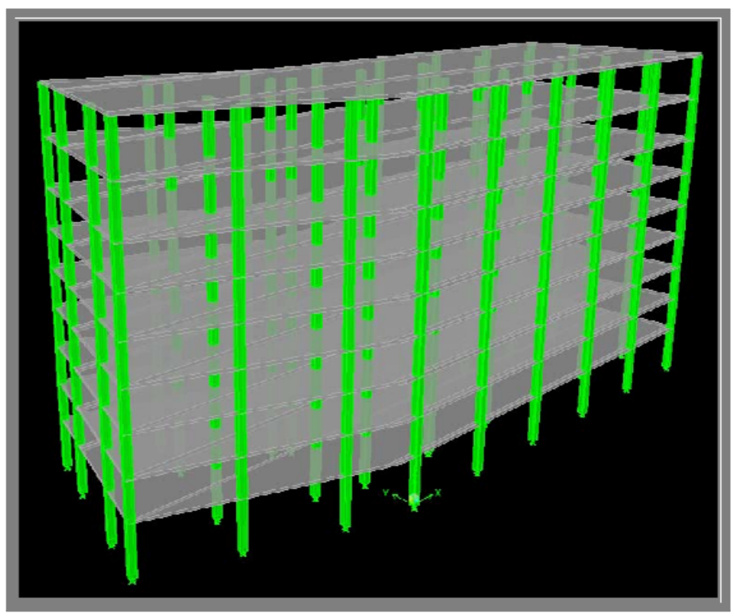


Figure 6: 3D ETABS Model Isometric

To begin the initial design process for the redesign of the gravity load resistance structure, it was necessary to determine the minimum slab thickness that could be used to span the longest bay distance within the structure. The minimum thickness was designed based on Table 9.5(c) of ACI 381-08. This table states that the miniumum thickness of a two way slab without interior beams and drop panels can be $l_n/33$. This led to a minimum slab thickness of 10". Once the minimum slab thickness was established a more indepth study of the slab was completed using both hand calculations and PCASlab to determine if the minimum thickness determined by code is sufficient to carry the given gravity loads and span the typical 28'-0" x 27'-8" bay. Both PCASlab and the hand calculations were also used to determine the amount of steel reinforcing required in the slab. First, hand calculations were completed using the Direct Design Method for two way reinforced concrete slab construction. All of the tables and calculations used to complete the Direct Design Method can be found in Appendix A. For simplicity purposes the design of only the typical interior bay, in both directions was analyzed. Figures 7 and 8 below show the interior bay and its corresponding column and middle strips.

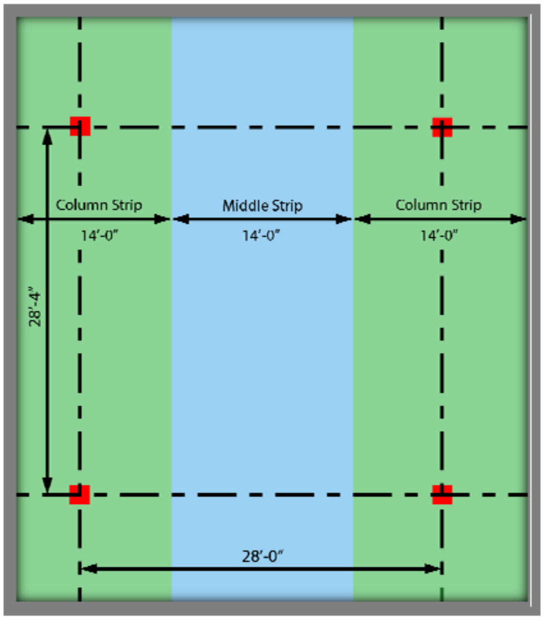


Figure 7: Two Way Slab – Frame A

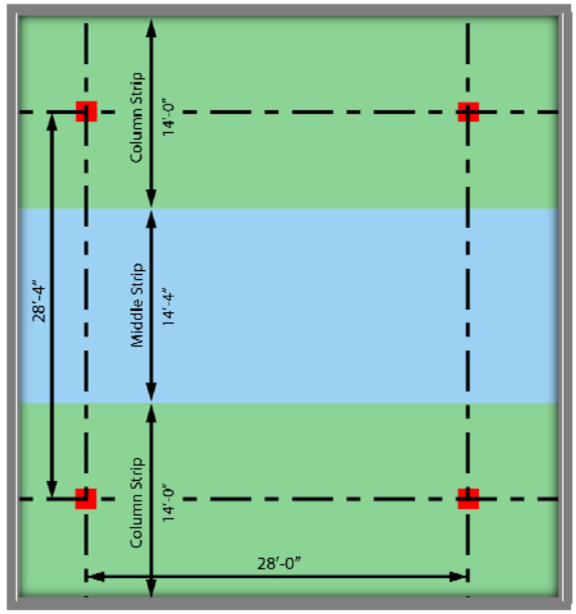


Figure 8: Two Way Slab – Frame B

The final designs using hand calculations yielded the following flexural and shear reinforcement in the two frames that were analyzed. The shear reinforcement is used because of punching shear around the columns. The addition of shear reinforcement could be neglected if drop panels were added around the columns. However, the addition of drop panels would create problems with the ceiling cavity in those places, causing the ductwork, piping and electrical wiring to be moved.

Frame A - Reinforcement		Frame B - Reinforcement	
M^- (Column Strip)	#5 @ 5.5" O/C	M^- (Column Strip)	#5 @ 6" O/C
M^+ (Column Strip)	#5 @ 14" O/C	M^+ (Column Strip)	#5 @ 14" O/C
M^- (Middle Strip)	#5 @ 15.25" O/C	M^- (Middle Strip)	#5 @ 15.75" O/C
M^+ (Middle Strip)	#5 @ 15.25" O/C	M^+ (Middle Strip)	#5 @ 15.75" O/C

Table 9: Flexural Reinforcement in Frames A & B

Shear Capacity in Slab			**Shear Reinforcement**	
V_u	52.41	OK	Bar/Wire Limit - V_c	244.19
ϕV_c	142.44		$V_u \le V_c$	USE BAR/WIRE
Punching Shear Capacity in Slab			V_s	188.68
V_u	222.9051	NO GOOD	s = d/2	4.5
ϕV_c	128.7158		A_v	1.57
			Use (15) #3 Stirrups @ 4.5"	

Table 10: Shear Reinforcement for Punching Shear

Upon completion of the hand calculations used to determine the necessary reinforcement in the two way flat plate slab, a more in depth analysis was performed using PCASlab. PCASlab uses Equivalent Frame Method for design which is slightly different than Direct Design Method in that it represents a three dimensional slab system using a series of two dimensional frames that are then analyzed for loads acting in the plane of the frames. Using PCASlab for design and analysis allowed for a more precise and exact design and placement of the reinforcement needed within the slab. Another benefit of using PCASlab for design was its efficiency in producing design for different types of bays found in the building. For simplicity purposes, only an interior bay was analyzed using hand calculation and therefore is more of an example rather than an exhaustive design. Using PCASlab allowed for an expanded design of four different bays or spans. They included; the interior span which was done by hand, an exterior span, and both an interior and exterior span in long direction of the building. These different spans which were evaluated in PCASlab can be seen in Figure 9 and 10 below.

The use of PCASlab also allowed for the investigation of concrete edge beams to be used for added stiffness and deflection resistance. As a preliminary design these beams were designed as 12" wide by 18" deep. This size proved to be adequate for the beams contribution to the resistance of the gravity loads on the structure. These edge beams will also be discussed in more depth later in the report because of their utilization within the concrete moment frames that were added to the design for the purpose of gaining the extra stiffness needed in the structure along with reducing the period of the building.

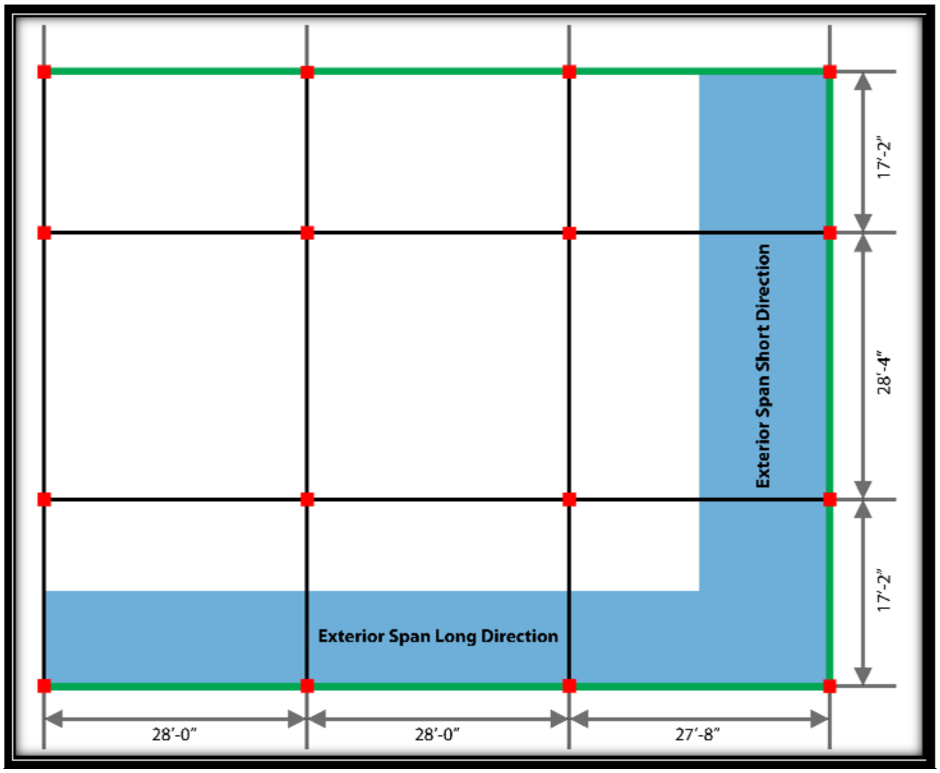


Figure 9: Exterior Spans analyzed and designed using PCASlab

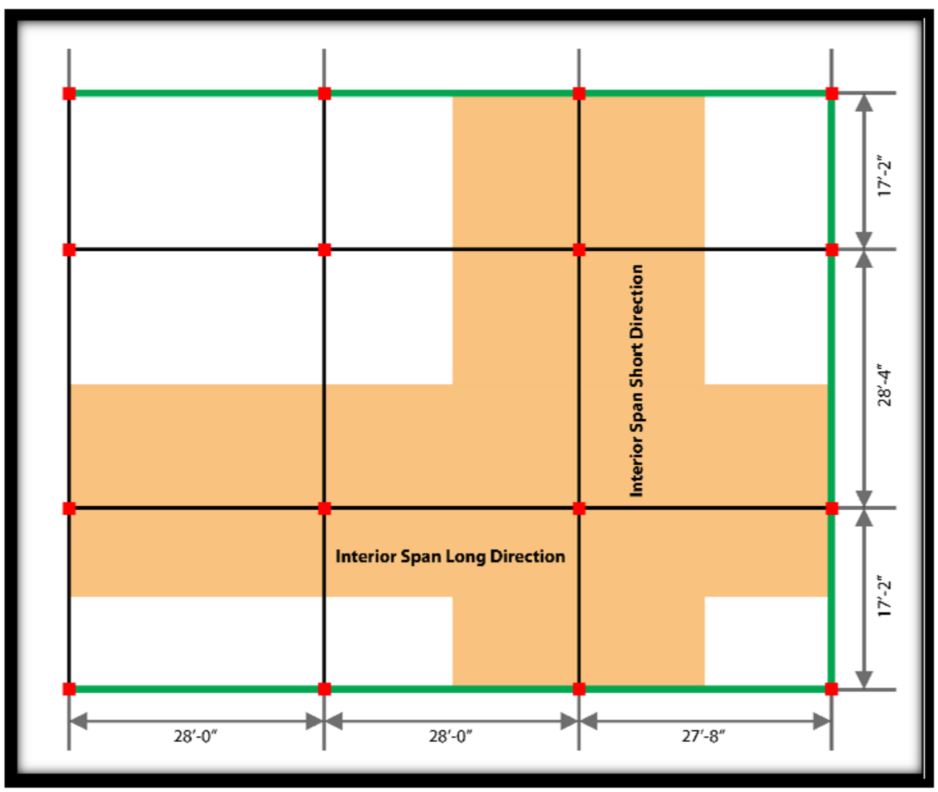


Figure 10: Interior Spans analyzed and designed using PCASlab

One of the main reasons for using PCASlab as a design aid was because of its ability to develop reinforcement sections through the various column, middle and beam strips of the slab. The reinforcement sections show all the reinforcement necessary to resist the gravity loading of the structure and both flexural and shear reinforcing is shown in the sections. However, the reinforcement in the sections is only shown in the longitudinal direction because the reinforcement that would be needed in the transverse direction would be shown on the section for the span that runs perpendicular to it. An example of the reinforcement sections that are produced by PCASlab can be seen in Figure 11 below. The rest of the sections for the other spans that were analyzed can be found in Appendix A. When compared to the Direct Design Method, it can be concluded that the reinforcement determined using PCASlab is more precise and exhaustive. However, it should also be noted that the Direct Design Method yielded the same rebar size for both the flexural reinforcement and shear reinforcement needed because of punching shear. Also the number of bars needed was roughly the same. This confirms that the hand calculations done were sufficient enough to verify the results of PCASlab and therefore the reinforcement can be fully designed using the PCASlab design program.

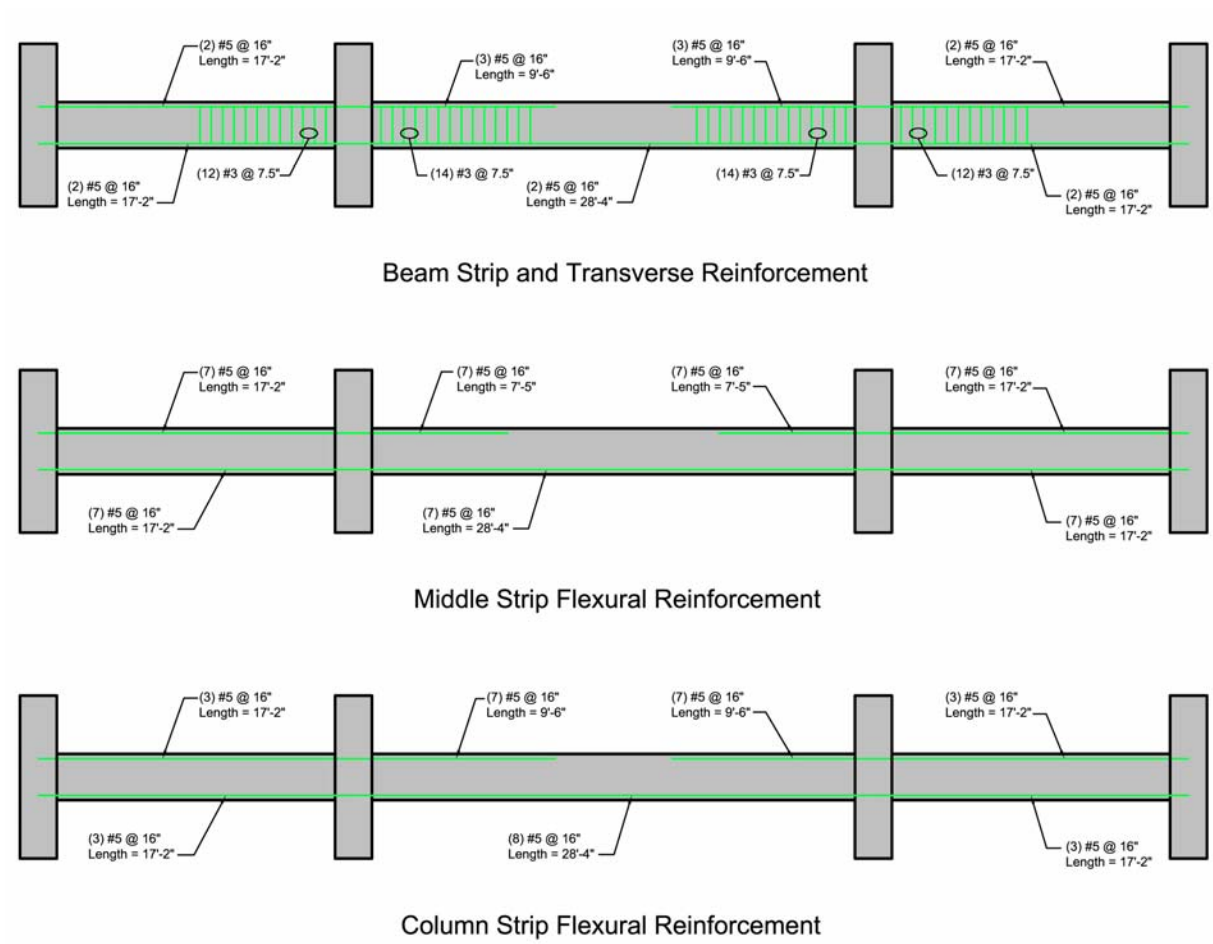


Figure 11: Exterior Span Reinforcement

Design Process – Slab Deflections

To complete the slab design process, the deflections in the two way slab were obtained using PCASlab and then were compared to the allowable live load deflection and total load deflection given in Table 9.5(b) of ACI 318-08. The limits for the allowable deflections along with the actual deflections found in each of the four spans analyzed can be found in Table 11 below. The results of the table show that deflections within the slab from the given gravity load were not an issue and fell well within the limits given by ACI 318-08. Since the slab exceeds the minimum thickness requirements set out by Chapter 9 of ACI 318-08, the span to depth ratio falls well within the acceptable range for a flat plate slab with exterior beams and therefore causes no real issues in terms of live or total load deflection.

Deflections for Two Way Slabs				
	Interior Span (Short Direction)	Exterior Span (Short Direction)	Interior Span (Long Direction)	Exterior Span (Long Direction)
Allowable Live Load Deflection	l/360 = 0.944 in	l/360 = 0.944 in	l/360 = 0.933 in	l/360 = 0.944 in
Actual Live Load Deflection	0.111 in	0.139 in	0.149 in	0.118 in
Allowable Total Load Deflection	l/240 = 1.417 in	l/240 = 1.417 in	l/240 = 1.417 in	l/240 = 1.417 in
Actual Total Load Deflection	0.326 in	0.412 in	0.421 in	0.353 in

Table 11: Deflection for Two Way Slabs

Design Process – Gravity Columns

After designing the two way slab to support the gravity loads for each floor, the vertical supporting elements of the structure needed to be designed to transfer the loading down through the building and ultimately to the foundations. The columns designed were 24" x 24" reinforced concrete columns with an $f'_c = 5000$ psi. For this design process only the interior columns shown in Figure 12 below will be discussed. These interior columns were design solely to support and transfer the gravity loads of the building. The exterior edge and corner columns will be discussed and designed during the lateral design process because they are part of the concrete moment frames.

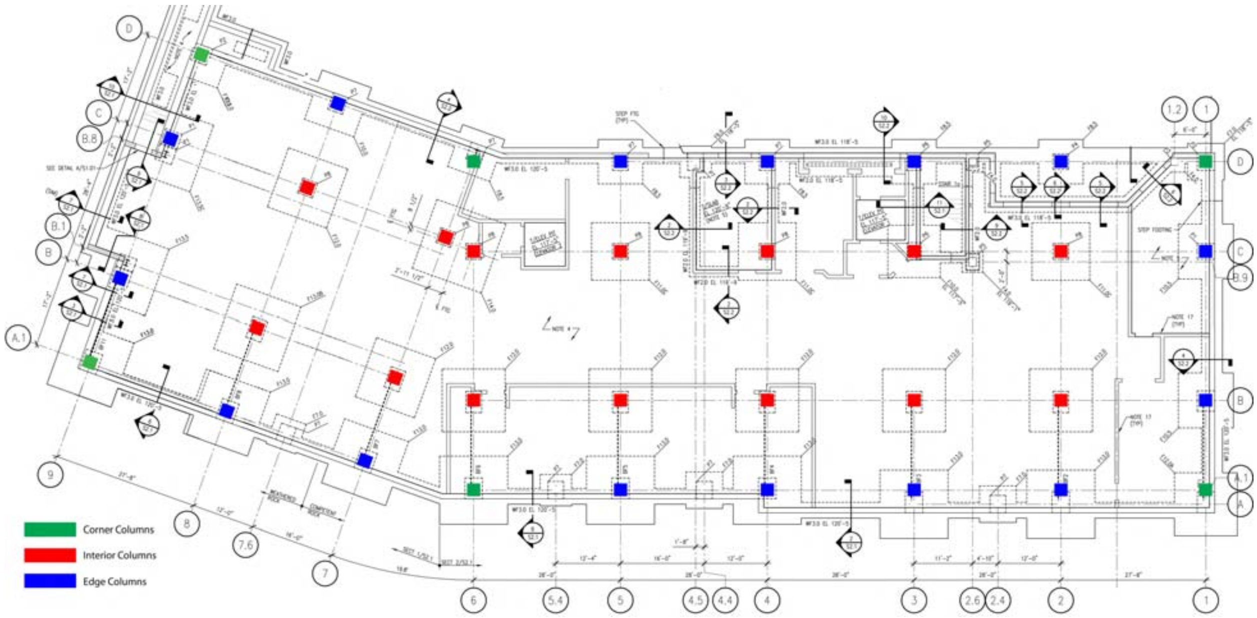


Figure 12: Concrete Column Layout

For simplicity and continuity in the design it was decided that all of the interior columns should be design to have the same size, dimensions and reinforcement. The design was based on the worst case scenario which turns out to be the columns on the bottom floor of the building because they support the most axial load. The column loading was based on the controlling load case found in ASCE 7-05. This load case was determined to be 1.2Dead + 1.6Live + 0.5Snow. After determining the appropriate load case the loading for the interior column was found and is displayed below in Table 12.

Concrete Column Loading (loads in kips)								
Type	**Floor Area**	**Self Wt.**	**Dead**	**Live**	**Quake**	**Wind**	**Snow**	**LC**
Interior	627.667	49.027	1029.374	282.450	0.000	0.000	14.436	1753.22

Table 12: Controlling Loads on Interior Concrete Column

To complete the design of the columns, PCAColumn was utilized so the appropriate size and reinforcement could be determined. Hand calculations were also utilized to determine the appropriate amount of transverse reinforcement required by ACI 318-08. In PCAColumn a trial column size of 24" x 24" was chosen and then the total axial load of 1753.22 kips was added. The analysis of the column yielded the following interaction diagram showing that the column falls within the acceptable curve.

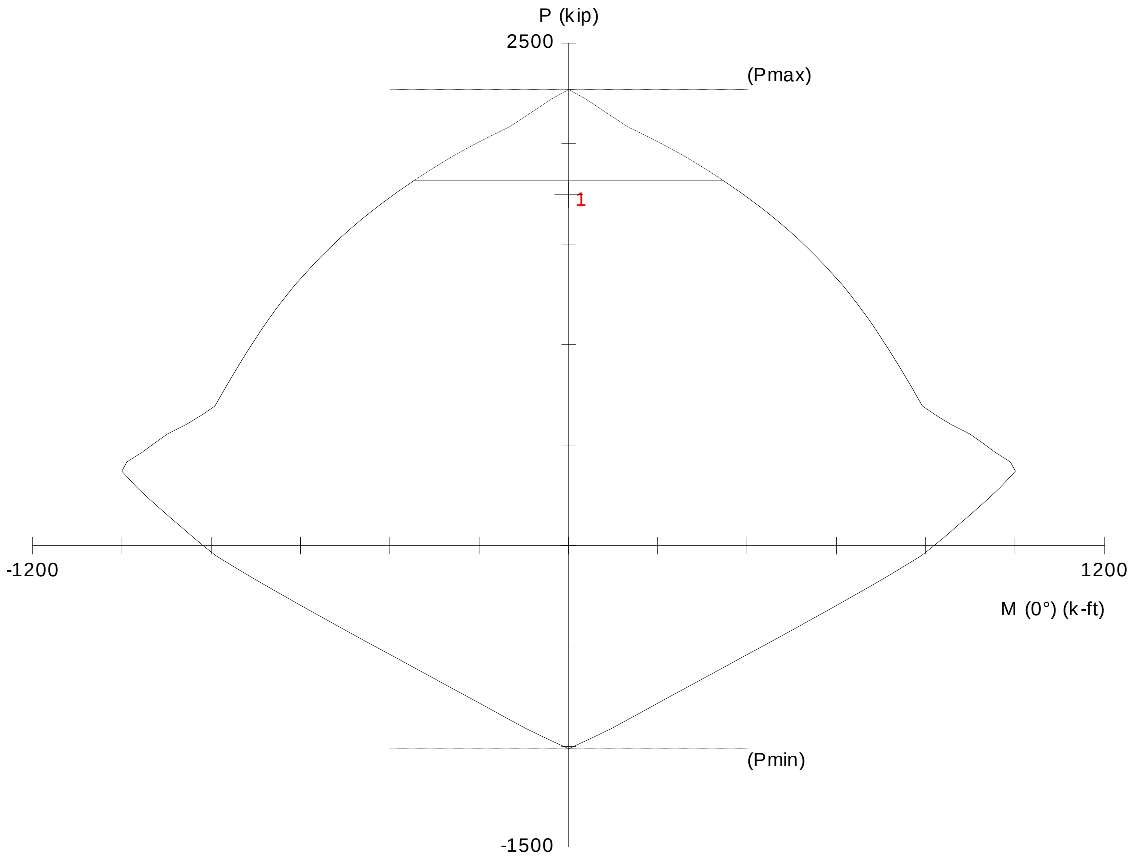


Figure 13: Interior Column Interaction Diagram

The flexural reinforcement and transverse reinforcement needed in column are shown in the Table 13 below. The transverse reinforcement was designed using minimum design requirements found in Chapter 7 of ACI 318-08.

Type	Flexural Reinf.	Shear Reinf.		Transverse Reinf.
		Av_{min}	0.240	
Interior	(12) #11 @ 7"	None		Use (3) #4 Ties @ 24" throughout

Table 13: Interior Column Required Reinforcement

With the design of the interior columns completed using both PCAColumn and hand calculations the details for the placement of the reinforcement can be completed. Again using Chapter 7 of ACI 318-08, it is determined that the minimum clear cover for gravity columns in a reinforced concrete system is 1.5". Also, the ties shown in the detail could be configured differently but was drawn to show one design option. The detail below shows all of the reinforcement necessary to resist the loading as well as the required spacing chosen for the design of the interior columns.

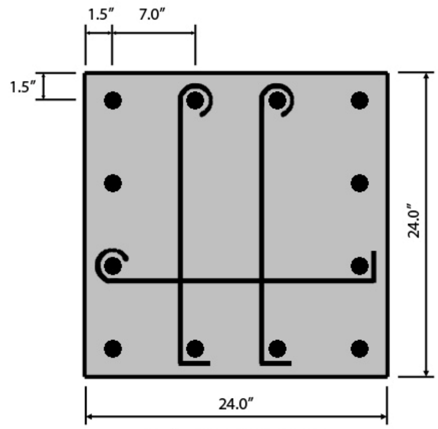


Figure 14: Interior Column Detail

More information on the design of the columns such as the loads and stresses within the concrete can be found in Appendix B.

Design Process – Foundation Considerations

The redesign of the structural system used to resist the gravity loads of Washington Park Condominiums allowed for a system that is comparable in efficiency and constructability to that of the existing steel system. However, the concrete system used provides some inherent differences in the support of the columns for the system.

One impact on the foundations that needs to be considered is caused by wind and seismic forces producing an overturning moment for the building. In turn, the overturning moment, may cause uplift within the foundations of the building because there is not enough weight on the columns and foundations to resist the overturning moment Since there was a change in systems from steel to concrete, there was also a significant increase to the overall weight of the structure. The new weight of the structure is 19973.1 kips which is 5400 kip increase over the original structure weight of 14555.97 kips. These figures can be used to check and see if overturning moments caused by the lateral loads on the building will be an issue.

To do this the shear walls that have the largest moment due to the wind and seismic forces were determined. Using ETABS, it was determined that shear walls ST2 and SL2 had the greatest overturning moment due to the lateral forces at its base. These two walls were chosen not only because they had the greatest overturning moments but also because they had differing wall lengths, therefore allowing for a more exhaustive uplift check. After finding the overturning moments at the base of each wall, the weight of the wall and axial load on each wall was calculated to determine the total resisting moment. Using the two moments calculated the factor of safety against overturning and uplift in each wall can be calculated. This was done by using the following equation for factor of safety:

$$Overturning\ Factor\ of\ Safety = \frac{Resisting\ Moment}{Overturning\ Moment}$$

The results for the factor of safety for both walls due to wind and seismic loading can be found below in Table 14. The results show that the factor of safety for both walls in above the recommended factor of 3.0 and therefore is no problem due to uplift or overturning moment.

Uplift Check - Shear Wall (Wind)								
	Overturning Moment (k-ft)	Wall Length (ft)	Wall Weight (kips)	Axial Load on Wall (kips)	Resisting Moment (k-ft)	Factor of Safety (Calculated)	Factor of Safety (Recommended)	Uplift Problem
ST2	3127.59	19.5	422.66	981.9	13694.0	4.38	3.0	No
SL2	1385.26	10	216.75	619.7	4182.1	3.02	3.0	No
Uplift Check - Shear Wall (Wind)								
	Overturning Moment (k-ft)	Wall Length (ft)	Wall Weight (kips)	Axial Load on Wall (kips)	Resisting Moment (k-ft)	Factor of Safety (Calculated)	Factor of Safety (Recommended)	Uplift Problem
ST2	2543.45	19.5	422.66	981.9	13694.0	5.38	3.0	No
SL2	715.11	10	216.75	619.7	4182.1	5.85	3.0	No

Table 14: Uplift/Overturning Moment Check

Another impact on the foundations that needed to be considered was whether or not they would need to be resized or changed because of the additional 5400 kips of weight from the concrete structure. To determine this, the foundations were redesigned using the structural analysis program EnerCalc. Although the overall weight of the building increased there were only minimal changes needed in terms of the spread footing sizes. Spread footings are recommended for the foundation because of their ease of construction and cost. The spread footing foundation design is also possible due in part to the fact that the entire building bears on siltstone, shale or sandstone bedrock at a depth of 9 to 25 feet, with an allowable bearing capacity of 9,000 ksf.

Spread Footing Sizes			
Type	**Existing Design**	**EnerCalc Design**	**Optimized Design**
Interior Col	12'-0" x 12'-0"	13'-0" x 13'-0"	13'-0" x 13'-0"
Corner Column (C55)	11'-0" x 11'-0"	7'-0" x 7'-0"	11'-0" x 11'-0"
Exterior Column (C65)	8'-0 x 8'-0"	9'-6" x 9'-6"	9'-6" x 9'-6"
Exterior Column (C80)	13'-0" x 13'-0"	8'-6" x 8'-6"	13'-0" x 13'-0"

Table 15: Spread Footing Sizes

For more in depth calculations for the foundation sizes and the output from the EnerCalc can be found in Appendix B.

Structural Depth Study – Lateral System Redesign:

Introduction

The proposed lateral force resistance system for Washington Park Condominiums uses interior shear walls placed around the stair and elevator shafts. This allows for minimum interruption with the existing architectural floor plan. For added stiffness, concrete moment frames will be added around the exterior of the building. This system was chosen because it was determined to be the most efficient at both resisting the lateral loads, while reducing the period and drift of the building.

Design Process – Reinforced Concrete Shear Walls

To begin the initial design process for the redesign of the lateral load resistance structure, it was necessary to determine the most appropriate placement for the shear walls. This was done by examining the existing floor plans and locating spaces that would cause the least interruption and therefore would eliminate any unessarsary reorganization of the interior architecture. The most efficient place for the shear walls was determined to be around the interior stair and elevator shafts because they were currently already being constructed with masonry block and were thicker than any other wall in the building. Figure 15 shows the placement of the four new "shaft" shear walls located around each of the two staircases and two elevator shafts.

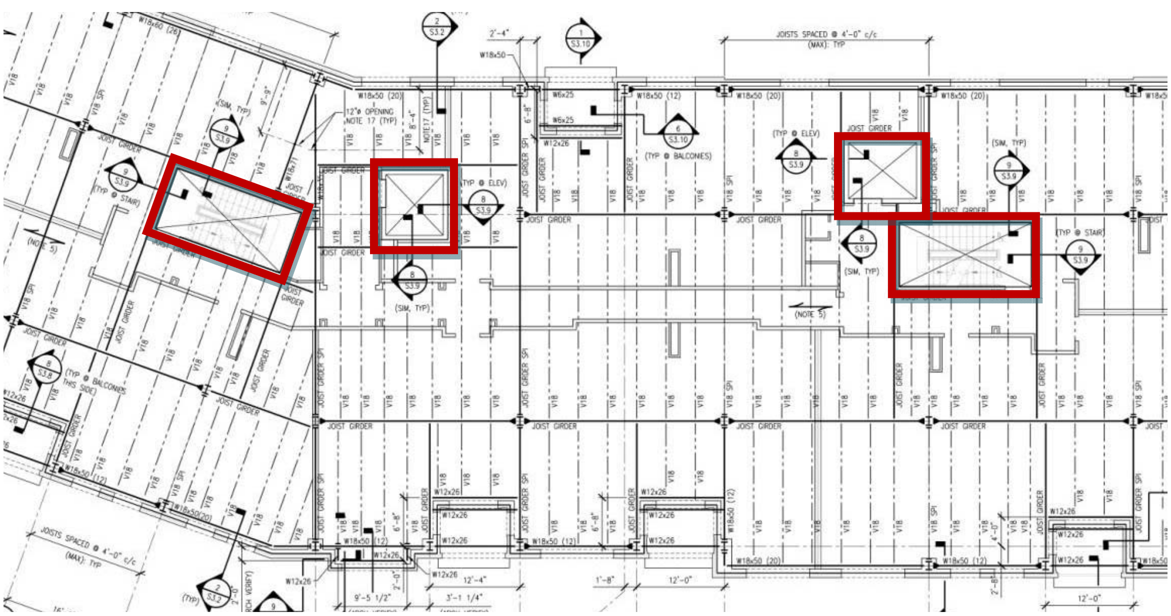


Figure 15: Design Shear Wall Locations

To begin the design of the shear walls the controlling loads on each of the shear walls needed to be determined. To find these controlling loads the new concrete lateral structure was constructed in ETABS using two separate models, one for the seismic loading and the other for the wind loading. In each model, the shear walls were constructed as piers starting at the base of the structure and reaching to the top of the building, since all of the shafts in the building go from the bottom floor to the top floor. Since, each shaft has an opening in it for a doorway it is essential to design coupling beams that span above the individual openings to the bottom of the floor above. These coupling beams will be discussed at more length when they are designed later in this section.

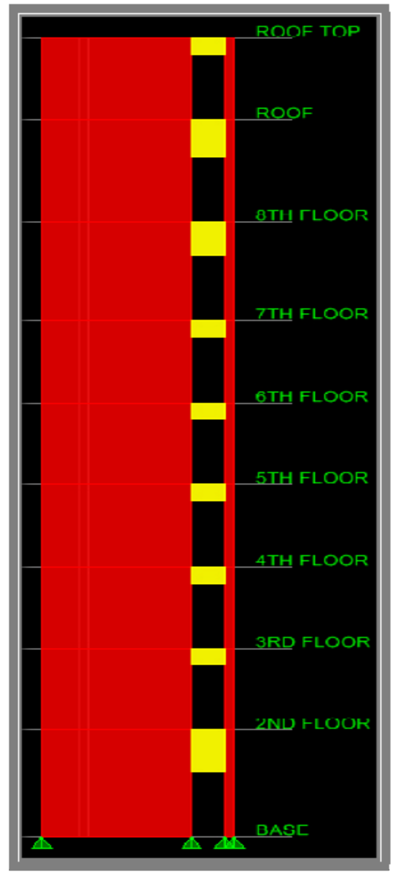


Figure 16: Shear Wall Elevation from ETABS

The reason for using two separate models is because of the different stiffness modifiers required when analyzing shear walls for different types of loading. For the wind model, the moment of inertia about the main bending axis for the coupling beam was reduced to 0.5. Similarly, in the seismic model the stiffness modifier about the minor bending axis was reduced to 0.7 and the moment of inertia about the main axis for the coupling beam was reduced to 0.35. Then using ETABS ability to generate seismic and wind loading based on code, six seismic and twelve wind cases were generated. In terms of the seismic loading, the first two cases correspond to the story forces found by manual calculation using the equivalent lateral force procedure. These forces are found to be in the x and y direction. The last four cases include seismic forces found by using the code provision stating the need for forces in the x and y direction ± 5% eccentricity for torsional considerations. Also being considered in the seismic analysis is the overall weight of the building. For this an area mass of 160 lb/ft^2 was assigned to each floor diaphragm, which are considered to be rigid, to account for the dead and superimposed dead load found in each floor. For the wind loading, the twelve load cases that were analyzed and applied to the building can be found using the design wind load cases found on Figure 6-9 in ASCE 7-05. These wind cases take into account all directional wind, along with torsional considerations and leeward wind forces.

To determine the exact loading in each wall, the shafts were each split up into four individual walls so that the max axial, shear and bending moment could be found for each shear wall. A plan labeling each individual wall is located in Appendix C. The below tables show examples of the controlling loads found in each wall for the corresponding wind or seismic load cases. The maximum load between both cases is highlighted in orange.

Seismic							
	Load Case	**Axial (P)**	**Shear (V2)**	**Shear (V3)**	**Torsion (T)**	**Moment (M2)**	**Moment (M3)**
SR1 (Max P)	SEISMICXY	-339.55	-2.08	1.6	-12.843	-4.26	197.018
SR1 (Max V2)	SEISMICYX	18.04	45.34	-0.65	-18.770	-1.5	786.986
SR1 (Max M3)	SEISMICYX	18.04	45.34	-0.65	-18.770	-1.5	786.986
SL1 (Max P)	SEISMICXYY	371.44	32.28	0.21	3.140	-3.536	237.072
SL1 (Max V2)	SEISMICYX	59.19	75.07	0.19	22.075	-0.3713	757.023
SL1(Max M3)	SEISMICYX	59.19	75.07	0.19	22.075	-0.3716	757.023
SB1 (Max P)	SEISMICYX	576.64	-25.37	-4.54	11.176	4.083	140.879
SB1 (Max V2)	SEISMICXYY	144.02	49.2	-0.72	111.594	16.952	1351.247
SB1(Max M3)	SEISMICXYY	159.99	44.42	-0.27	-28.135	1.726	1734.023
ST1 (Max P)	SEISMICYX	-653.87	28.59	-9.03	-6.075	4.208	272.408
ST1 (Max V2)	SEISMICXY	-67.7	117.69	0.48	-80.483	4.063	1201.721
ST1(Max M3)	SEISMICXYY	-216.9	98.71	-3.58	38.009	1.557	2538.317

Table 16: Controlling Seismic Loads for Shear Wall #1

Wind							
	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Moment (M2)	Moment (M3)
SR1 (Max P)	DCON2	225.39	98.16	-0.25	-10.057	-0.349	1240.895
SR1 (Max V2)	DCON11	123.94	116.03	-0.77	-19.615	-2.36	1609.218
SR1 (Max M3)	DCON11	123.94	116.03	-0.77	-19.615	-2.36	1609.218
SL1 (Max P)	DCON7	238.17	93.89	-1.42	13.024	-2.255	891.857
SL1 (Max V2)	DCON11	83.88	134.71	0.11	26.715	-0.548	1364.967
SL1(Max M3)	DCON11	83.88	134.71	0.11	26.715	-0.548	1364.967
SB1 (Max P)	DCON11	908.89	-25.28	1.02	14.912	8.111	192.363
SB1 (Max V2)	DCON13	177.82	-81.56	0.5	-51.098	7.612	152.627
SB1(Max M3)	DCON3	30.64	36.1	0.52	-17.975	0.4736	1135.531
ST1 (Max P)	DCON12	1116.71	-22.78	3.39	8.767	-8.575	-363.268
ST1 (Max V2)	DCON8	372.38	-126.92	-3.7	59.445	-10.164	-604.321
ST1(Max M3)	DCON3	-53.99	74.88	-0.4	17.788	3.824	1598.308

Table 17: Controlling Seismic Loads for Shear Wall #1

After determining the controlling loads that are applied, each wall was designed with necessary flexural and shear reinforcement. To do this both PCAColumn and hand calculations were completed. The hand calculations were needed because PCAColumn can only design walls based on axial loading and flexural moments. Therefore, only those loads were applied to each wall while they were being analyzed using PCAColumn. During the design using PCAColumn, it was determined that there was a need for the incorporation of boundary elements in coordination with ACI 21.9.6.4. Because the walls are part of shafts, they can be individually modeled as C shaped shear walls using the perpendicular connecting walls as boundary elements. Analyzing the shear walls as C shapes gave them extra flexural capacity and accounts for the fact that in reality the shear walls will act together as a shaft to resist the forces. The flexural analysis of the C shaped shear walls yielded a design using the minimum flexural reinforcement of #5 @ 12" throughout the entire wall. An example of this design is shown below in Figure 18.

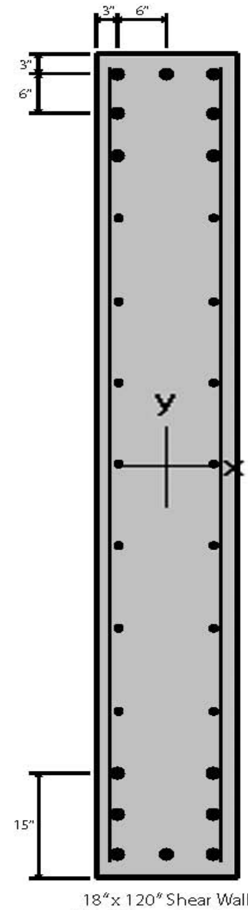


Figure 17: Shear Wall SR1 Detail

The addition of the boundary elements also eliminated the need for extra reinforcement at the ends of the walls because of the large in plane moments. Without the C shape analysis of the walls, a pseudo- boundary element would need to be created at each end of the wall. These boundary elements consist of larger bars spaced closer together. In the case of shear wall SR1, (7) #7 bars spaced at 6" were used as shown in Figure 17. This reinforcement would add the needed flexural capacity for the walls.

For the shear design of each of the walls, hand calculations were performed. The calculations were used to find the vertical and horizontal shear reinforcement needed. To calculate this, the maximum shear load case for each wall for both the seismic and wind loads was found. This controlling case was used to design the shear reinforcement using equations for minimum shear reinforcement in a wall. These equations can be found in Chapter 11 of ACI 318-08. Since, the structure is located in Seismic Class B; there is no need to design special or intermediate reinforced

SR1					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	116.03	V_u	116.03
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-63.87	V_s	-63.87
V_u	116.03	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

Table 18: Shear Reinforcement for Shear Wall SR1

concrete shear walls as is discussed in Chapter 21 of ACI 381-08. The shear calculations for the wall yielded a minimum area of steel for shear reinforcing of 0.180 in^2. In this case using minimum reinforcement of (2) #5 spaced at 18" would be an adequate design. The results for the design can be found above in Table 18.

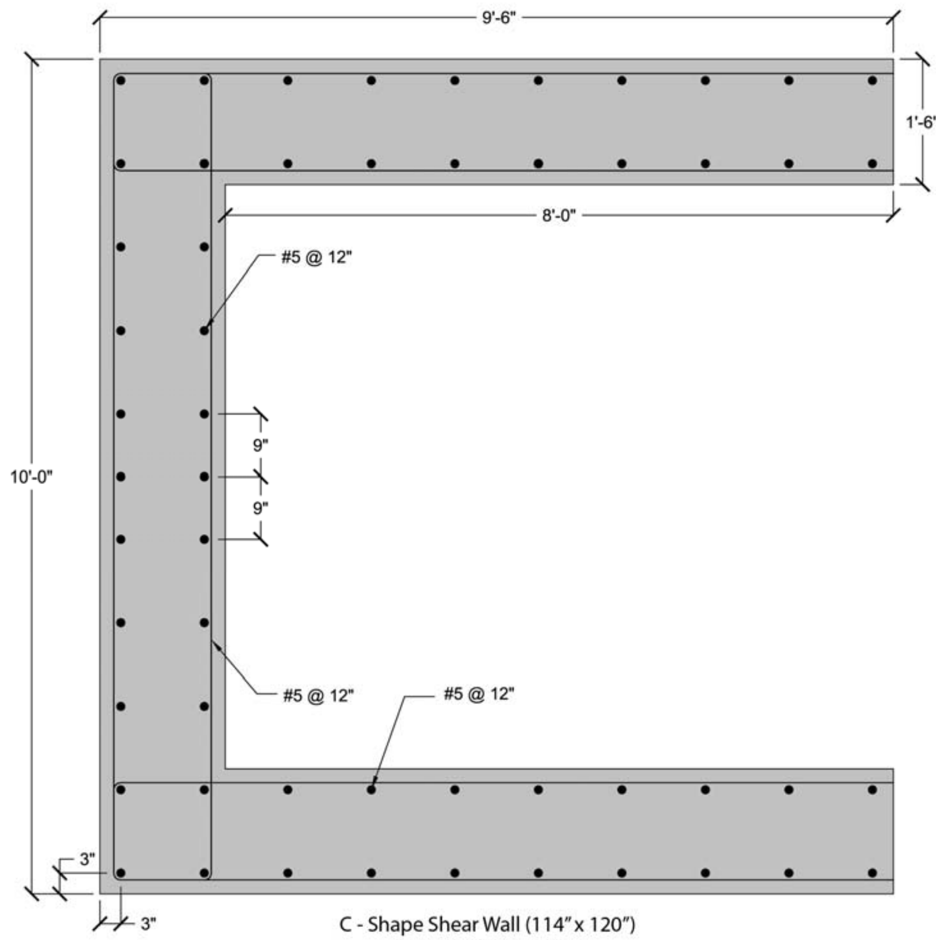


Figure 18: C-Shape Shear Wall SR1 Detail

However, it was decided for the purpose of continuity in the design and ease of construction that (2) #5 spaced @ 12" would be used instead. This was done because the flexural reinforcement needed found using PCASlab was (2) #5 spaced @ 12". This allows for more consistent and even reinforcement throughout the entire wall as well as being a conservative design. As for the rest of the shear walls, their controlling forces, reinforcement details and sizes can be found in Appendix C.

Design Process – Reinforced Concrete Coupling Beams

Coupling beams connecting structural shear walls provide stiffness and energy dissipation. They are designed to crack before the shear walls and to act as plastic hinges in the building. To account for this, the effective moment of inertia about the main bending axis was reduced for both the seismic and wind cases. As stated above, the value used in the wind design is $I_{eff} = 0.5$ where as the value used in the seismic design is $I_{eff} = 0.35$. These values are different because in the case of seismic loading, the correct design accounts for less shear reaction forces in the coupling beams. These different values for effective moment of inertia were used in the both seismic and wind ETABS models. The same load cases for both seismic and wind were used to design the coupling beams as well. Table 19 is an example of the controlling loads found in each beam for the corresponding wind or seismic load cases. The design is then based off of these values when computing the flexural and shear capacity of the beam along with the required reinforcement. The load tables for the remaining coupling beams are located in Appendix D.

Beam 3 (Stair 1)						
Story	**Spandrel**	**V2**	**T**	**T (k-ft)**	**M3**	**M3 (k-ft)**
ROOF	B3	13.84	14.6	1.22	350.214	29.18
8TH	B3	17.94	17.529	1.46	426.54	35.55
7TH	B3	25.26	20.065	1.67	577.218	48.10
6TH	B3	26.29	16.408	1.37	596.334	49.69
5TH	B3	28.39	17.294	1.44	645.447	53.79
4TH	B3	29.69	17.778	1.48	675.866	56.32
3RD	B3	29.63	17.589	1.47	674.034	56.17
2ND	B3	30.05	19.578	1.63	713.036	59.42
1ST	B3	17.98	22.762	1.90	398.137	33.18

Table 19: Coupling Beam 3 (Stair 1) Loading

The coupling beams to be designed were first chosen to be 18" thick, so that they would have the same thickness as the shear walls. After choosing the thickness of the beams, ACI 318-08 Chapter 21.9.7 states that any coupling beam with a span-depth ratio (l_n/h) ≤ 2 and $Vu \geq 4A_{cw}\sqrt{f'c}$ to be reinforced using two intersecting lines of diagonal reinforcement. For each coupling beam designed, the span-depth ratio was less than two. However, the maximum shear found in the beams was far less than $Vu = 4A_{cw}\sqrt{f'c}$. With this being the case, the coupling beams do not meet the criteria of ACI 318-08 21.9.7 and therefore will be designed according to 21.9.4, which states that $V_n \leq 10A_{cw}\sqrt{f'c}$. For the design of shear reinforcement in the beams, the overall depth of the individual beams needed to be considered. Beams that are smaller than 36" deep can be designed using flexural reinforcement according to ACI 318-08 Chapter 10.5.1 and shear reinforcement in accordance with ACI 318-08 Chapter 11.4.6. Beams that were deeper than 36" must be designed as deep beams. In deep beams the flexural reinforcement can also be designed in accordance with ACI 318-08 Chapter 10.5.1; however the shear reinforcement in the beam must be designed differently. Because of this, the minimum area of steel for shear in the direction perpendicular to the flexural reinforcement is $A_{s,min}$ = $0.0025b_ws$. Similarly, the minimum area of steel for shear in the direction perpendicular to the flexural reinforcement is $A_{s,min} = 0.0015b_ws$. Another consideration for deep beams is skin reinforcement, which is provided throughout the entire height of the beam. The skin reinforcement shall be #4 bars spaced at $s = 15\left(\frac{40{,}000}{f_s}\right) - 2.5c_c$. Shown below in Table 20 and Figures 19 and 20 are examples of the final reinforcement design and corresponding detailing for the coupling beams. The final designs for all of the coupling beams as well as the loading used to determine those designs can be found in Appendix D.

Coupling Beam 3 (Stair 1)											
Story	**l_n**	**h**	**l_n/h**	**A_{cw}**	**V_n**	**d**	**As_{min}**	**Av_{min}**	**Flexural Reinf.**	**Shear/Transverse Reinf.**	**Skin Reinf.**
ROOF	40	58	0.69	1044	660.3	55	4.40	0.495	(3) #8 @ 6" T & B	(3) Legs of #4 @ 6"	#4 @ 6.5"
8TH	40	54	0.74	972	614.8	51	4.08	0.459	(3) #8 @ 6" T & B	(3) Legs of #3 @ 6"	#4 @ 6.5"
7TH	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
6TH	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
5TH	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
4TH	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
3RD	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
2ND	40	26	1.54	468	295.9	23	1.84	0.320	(3) #5 @ 6" T & B	(3) Legs of #3 @ 6"	None
1ST	40	66	0.61	1188	751.4	63	5.04	0.567	(3) #9 @ 6" T & B	(3) Legs of #4 @ 6"	#4 @ 6"

Table 20: Coupling Beam 3 (Stair 1) Reinforcement

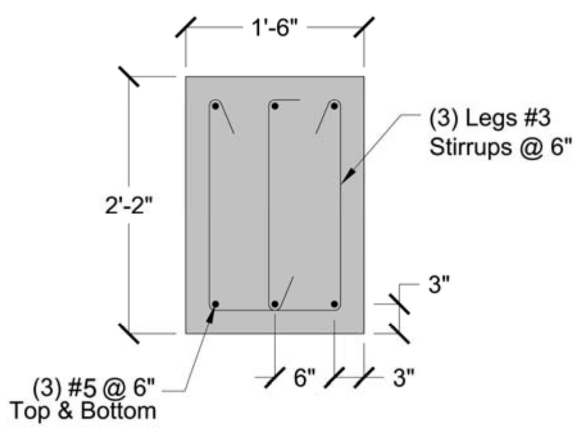


Figure 19: 26" Deep Coupling Beam Detail

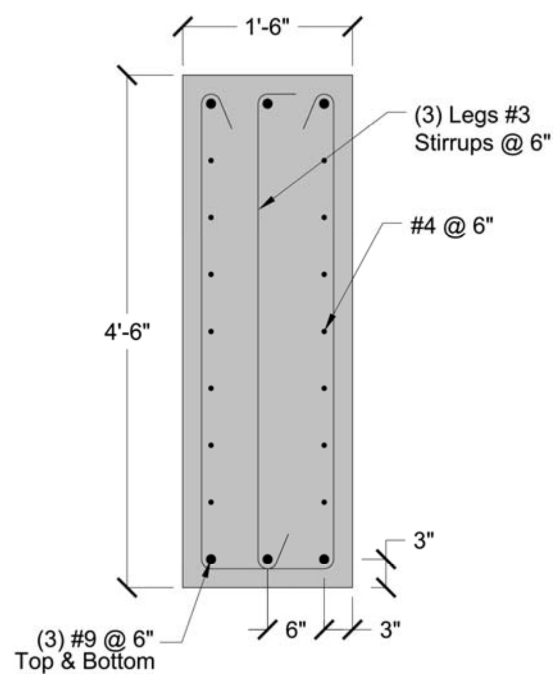


Figure 20: 54" Deep Coupling Beam Detail

Design Process – Dynamic ETABS Analysis (Mode Shapes & Period)

After the design of the reinforced concrete shear walls and the included coupling beams the structure then needed to be analyzed dynamically for drift and the approximate fundamental period. To do this ASCE 7-05 was used to calculate the approximate fundamental period, Ta. Using 12.8.2.1 the period can be calculated by using the equation, . In the case of Washington Park Condominiums, C_t = 0.02, x = 0.75 and h_n = the height of the building above grade. Therefore, according to the minimum design requirements the fundamental period of the structure is 1.046 seconds. The values for the seismic parameters of Washington Park Condominiums can be found in Table 7 above. Once the value for the fundamental period was determined according to code, ETABS was used to determine the period based on the structure that had been designed in the program. ETABS uses an eigenvalue-eigenvector pair called a natural vibration mode. The modes are identified by numbers 1 through n in order in which the modes are found by the program. For the structure of Washington Park Condominiums, only the first four modes are needed within the analysis. The four modes used in the analysis can be found in Figure 21 below.

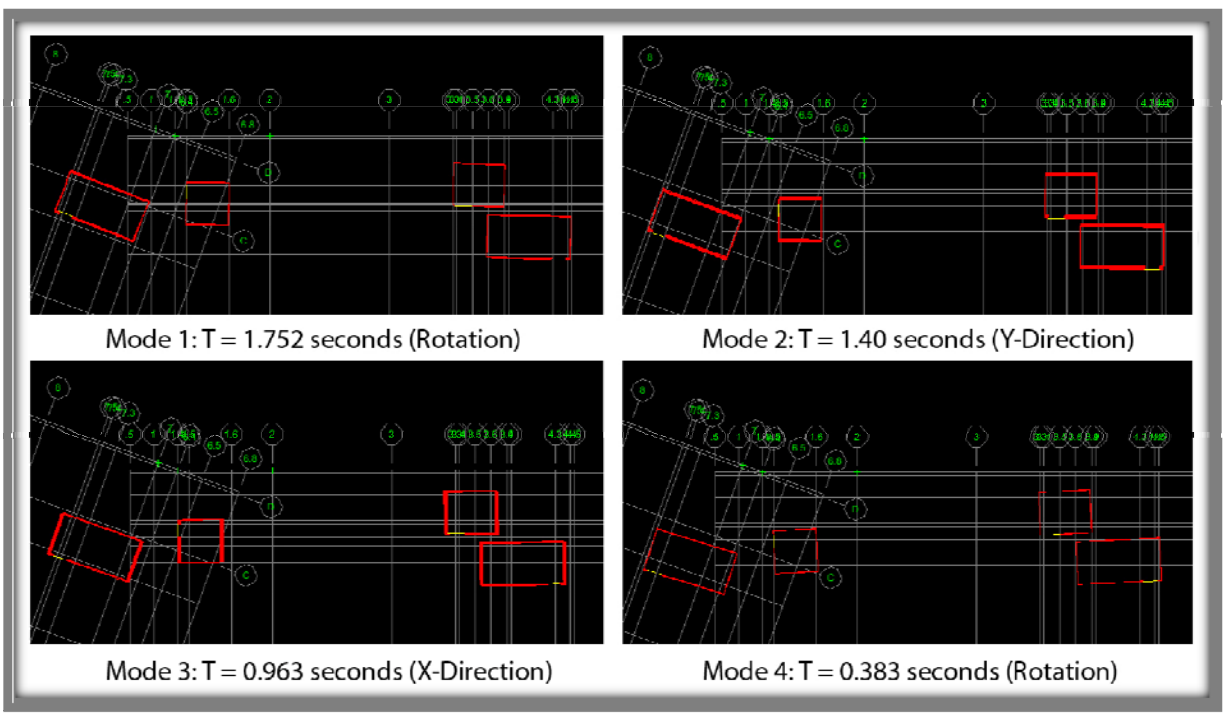


Figure 21: Modal Response Spectral Analysis

After the modes and periods were determined in ETABS they were compared to the calculated fundamental period found from the code values. Both of the values for the period in the y-direction mode and the rotation mode are above the approximate period determined by code. This leads to the decision that is twofold; first, there currently is an issue with rotational moments and torsion created by the asymmetrical nature of the building along with the difference between the center of mass and center of rigidity. Second, the building is deemed as too flexible and stiffness needs to be added so that the period can be reduced. There are two options to adding stiffness and rotational moment resistance to the building. First is adding more shear walls to the design. After this solution was researched it was determined that it was not feasible within the current parameters of the design because there are no practicable locations for additional shear walls that don't interrupt the floor plan or exterior architecture of the building. Because of this, a second design alternative needed to be found. To meet the requirements of both the architecture and structure, concrete moment frames were chosen as a system to add stiffness and lateral load resistance to the building.

Design Process – Dynamic ETABS Analysis (Torsional Amplification)

In addition to the inherent stiffness problems within the building, there may also be issues due to accidental torsion. This is apparent by the fact that the largest period in the building occurs during a torsional mode shape. To determine if this is the case ASCE 7-05 Section 12.8.4.3 was used to calculate the amplification of accidental torsional moments. To begin the calculation ETABS was used to obtain the maximum deflections using both the seismic in the x and seismic in the y loads. These loads were used to find the δ_{AVG} for each load case, and were then compared to the δ_{MAX} found using the seismic loads in the x and y directions that included an eccentricity ratio of 5%. Using those values, the amplification factor, A_x was found using the following equation, $A_x = \left(\frac{\delta_{MAX}}{1.2\delta_{AVG}}\right)^2$. The results for the amplification of the torsional moments in both the x and y directions are given in Table 21 below.

<table>
<tr><th colspan="5">Torsional Amplification Factor</th></tr>
<tr><th>Loading</th><th>δ_A</th><th>δ_B</th><th>δ_{MAX}</th><th>A</th></tr>
<tr><td>Seismic X</td><td>0.5582</td><td>0.3474</td><td>-</td><td rowspan="2">1.534</td></tr>
<tr><td>Seismic XXY (5% Ecc)</td><td>-</td><td>-</td><td>0.67293</td></tr>
<tr><td>Seismic XXY (7.67% Ecc)</td><td>-</td><td>-</td><td>0.6822</td><td>1.576</td></tr>
<tr><td>Seismic XXY (7.9% Ecc)</td><td>-</td><td>-</td><td>0.6836</td><td>1.583</td></tr>
<tr><td>Seismic Y</td><td>0.8155</td><td>0.5587</td><td>-</td><td rowspan="2">1.004</td></tr>
<tr><td>Seismic YX</td><td>-</td><td>-</td><td>0.8262</td></tr>
</table>

Table 21: Torsional Amplification Factor

The results show that the torsional moments within the building are about 3% higher than desired. Although this doesn't seem like a big deal, the larger torsional moments coupled with the large period in a rotation mode shape lead to the fact that the building needs to be designed for these rotational and torsional considerations. This will be accomplished by the addition of exterior concrete moment frames in hopes that the period of the building, along with the torsional effects will be reduced.

Design Process – Concrete Moment Frame Analysis using ETABS

Concrete moment frames can be used as part of the lateral force resisting system in buildings to add stiffness and reduced lateral stresses in members. Beams, columns and beam-column joints in moment frames are designed and detailed to resist flexural, axial and shearing actions that result as the building resists lateral loads. The concrete moment frames are most effective in resisting these loads as a result of the modes of displacement during earthquake ground movement. In the case of Washington Park Condominiums the concrete moment frames will be designed primarily to increase the stiffness of the building and therefore, reduce the period found by ETABS in the modal analysis.

To complete the design of the concrete moment frames, the geometry of the structure was added to the ETABS model that already included the placement of the shear walls stated above. The figure below shows the layout of the new exterior concrete moment frames. The moment frames were determined to be most efficient on the exterior of the building because the current lateral resisting elements are already located more centrally therefore creating torsion in the building. The exterior moment frames reduce these torsional forces and therefore reduce the difference between the center of mass and center of rigidity of the building.

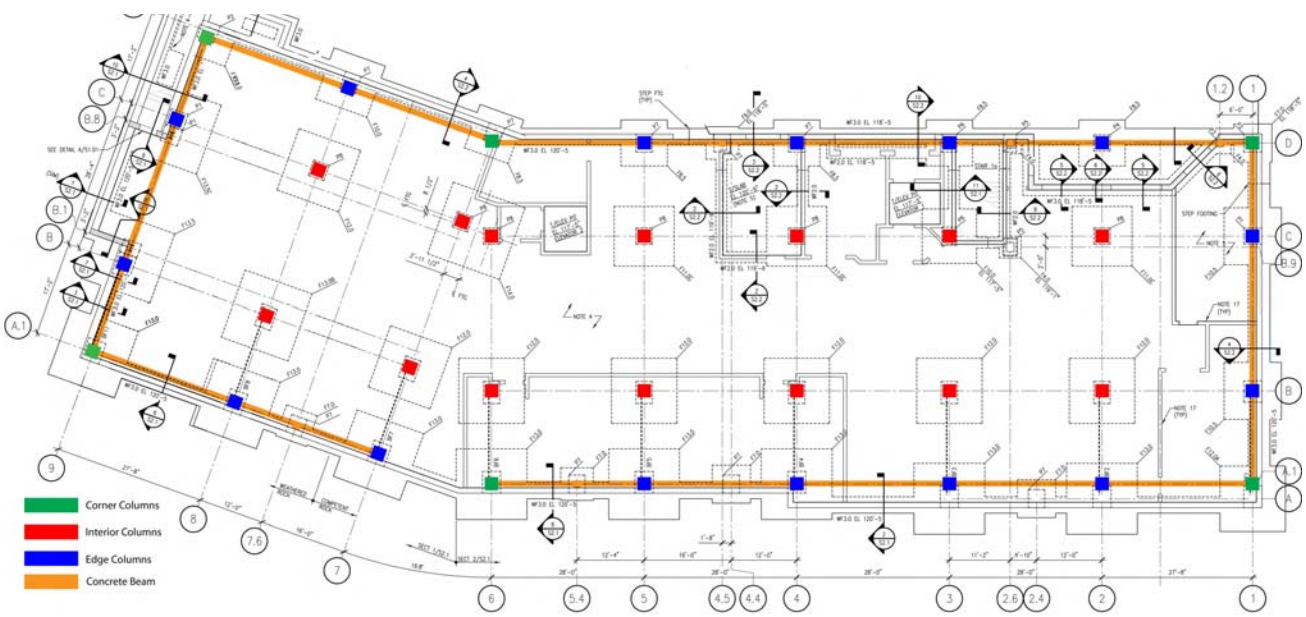


Figure 22: Beams within Concrete Moment Frame

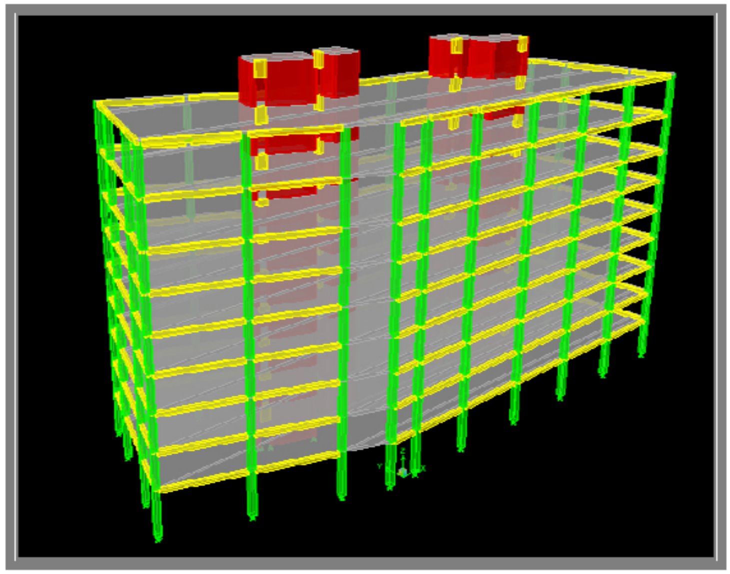
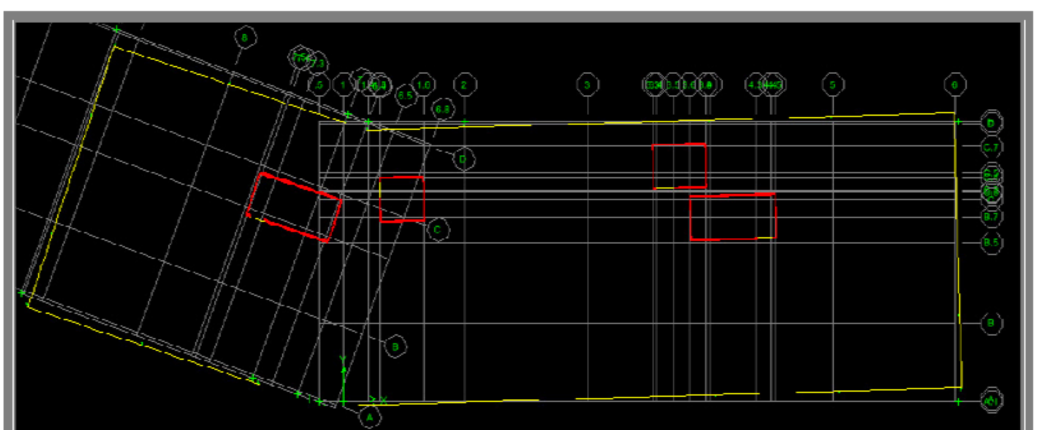


Figure 23: 3D ETABS model with Moment Frames

After adding the necessary columns and beams to the ETABS model end length offsets were added to each of the lateral beams. This was done because when two members such as a beam and a column are connected at a point, there is some overlap of the cross sections. When a beam is added in ETABS it is assumed that origination of that beam is at the centroid of the column. This isn't always the case in real structures and therefore adding end length offsets allows for a more realistic analysis. For all of the beams an end length offset of 12" was defined because the columns that the beams frame into are all 24" x 24". In addition to defining end length offsets, a rigid zone factor was also identified for the beams. The rigid zone factor specifies the fraction of each end offset assumed to be rigid for bending and shear deformations. In the case of Washington Park Condominiums the rigid zone factor was set to be 0.5 so that 6" of the beam is assumed to be rigid for bending and shear deformations. Another modification made to the lateral beams in the concrete moment frames was to reduce the effective moment of inertia about the major and minor axes to 0.35. This was done to comply with ACI 10.10.4.1 which states that for beams the effective moment of inertia is equal to $0.35I_g$. After these modifiers were all changed, the structure was analyzed to obtain new forces in both the shear walls and the concrete moment frames, along with new values for the fundamental period and building drift. The new periods obtained by ETABS with the addition of the exterior concrete moment frames are shown below for each of the four modes.



Mode 1: T= 1.43 seconds (Rotation)

Figure 24: Mode Shape #1 (Rotation)

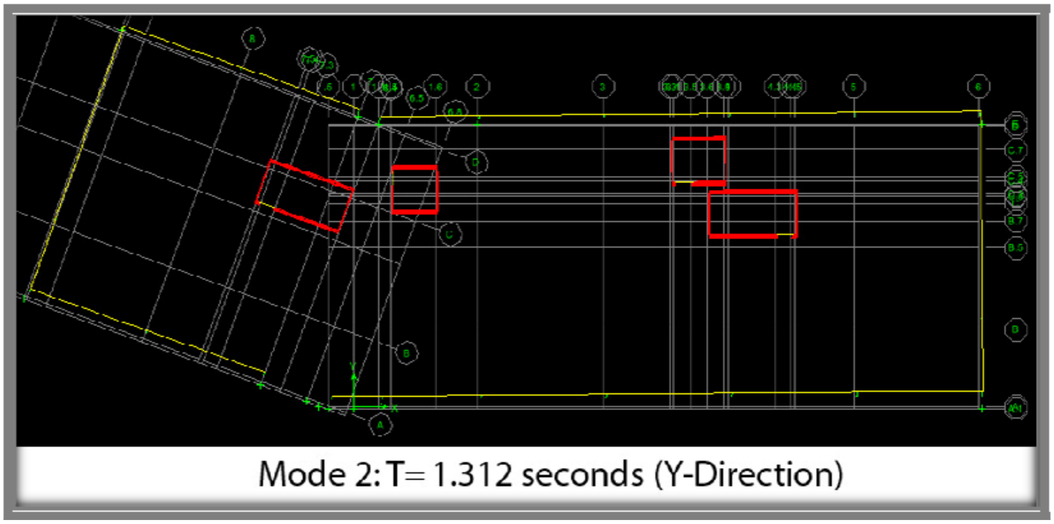


Figure 25: Mode Shape #2 (Y-Direction)

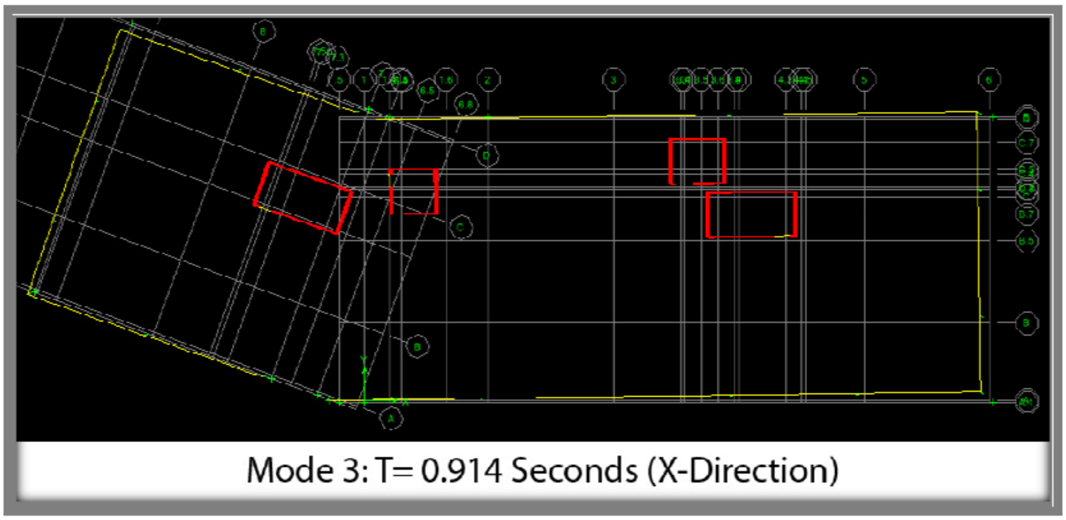


Figure 26: Mode Shape #3 (X-Direction)

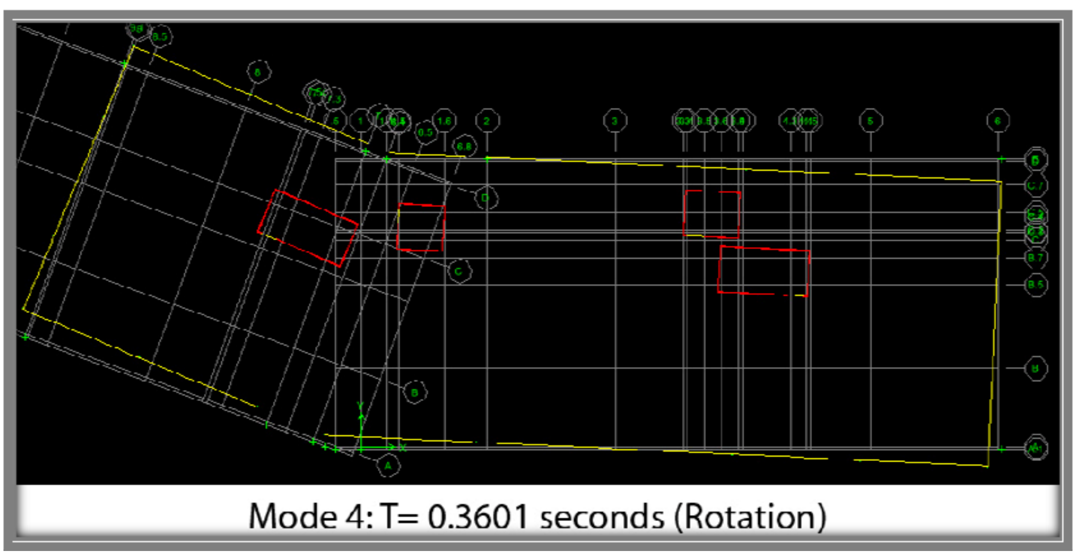


Figure 27: Mode Shape #4 (Rotation)

The addition of the concrete moment frames caused the fundamental period of the building to be reduced as shown in Table 22. Mode #1 which has the highest natural period has been reduced to 1.43, which for design purposes is deemed acceptable. The addition of concrete moment frames also changes the approximate fundamental period calculated by code. The new approximation uses $C_t = 0.016$ and $x = 0.9$, therefore allowing the fundamental period of the building to be $T_a = 1.659$ seconds. This new upper limit for the fundamental period is above the calculated period found using ETABS and therefore the calculated periods found in ETABS will be used. With that being said, Mode #1 is still a rotational mode which points to the fact that torsion could be an issue in the design. Because of this both the shear walls and the concrete moment frames shall be design with torsional reinforcement where considered necessary by code.

Fundamental Period Comparison			
Mode Shape	**Shear Wall Only Design**	**Shear Wall and Moment Frame Design**	**Difference**
1 (Rotation)	T = 1.752 seconds	T = 1.43 seconds	0.322 seconds
2 (Y-Direction)	T = 1.40 seconds	T = 1.312 seconds	0.088 seconds
3 (X-Direction)	T = 0.963 seconds	T = 0.914 seconds	0.049 seconds
4 (Rotation)	T = 0.383 seconds	T = 0.360 seconds	0.023 seconds

Table 22: Fundamental Period Comparison

Design Process – Concrete Moment Frame Design (Columns)

For the purpose of higher education and better proficiency in reinforced concrete design the moment frames in Washington Park Condominiums will be designed as part of an intermediate moment frame as characterized in ACI 21.3. This design will allow for more in depth detailing of the reinforcement in both the columns and beams within the moment frame and while also reducing the risk of the system's failure due to shear in the beams and columns.

The addition of moment frames to the exterior of the building, led to the redesign of both the exterior and corner columns shown above in Figure 12. These columns can no longer be designed solely as gravity columns because they will now take lateral load, and therefore are no longer governed or designed by the controlling gravity load combination. The loads that are acting on these columns were determined using the ETABS analysis denoted above. For simplicity, only a corner column and exterior column on each of the short and long directions of the building were designed. The table below shows the maximum loading results that were found in ETABS for one of the exterior columns. The details of all of the columns that were analyzed along with their respective load tables can be found in Appendix E.

Column C65 (Column Line 9) for Seismic								
Story	**Floor**	**Load**	**P**	**V2**	**V3**	**T**	**M2**	**M3**
Max P	1st	SEISMICXYY	20.28	-0.71	0.17	0.00	-0.23	0.96
Max V2	Roof	SEISMICXYY	2.34	1.5	0.57	1.44	2.89	6.47
Max T	6th	SEISMICXYY	10.36	1.36	0.45	1.58	2.77	7.54
Max M2	2nd	SEISMICYX	3.73	0.06	0.59	0.29	11.88	-1.20
Max M3	2nd	SEISMICXYY	19.24	1.3	0.33	0.95	2.84	14.96

Table 23: Loading on Exterior Column C65

Using the loads provided by ETABS the controlling load combinations were able to be determined. In the case of the corner and short direction exterior column, the controlling load case included wind loading rather than seismic loading. However, in the long direction the controlling load case includes the seismic load. The loadings, shown in Table 24 below, were then used to design the reinforcement using both PCAColumn and hand calculations that adhere to the intermediate moment frame provisions of ACI 21.3.

Concrete Column Loading								
Type	**Area**	**Self Wt.**	**Dead**	**Live**	**Quake**	**Wind**	**Snow**	**LC**
Corner (C55)	118.75	49.027	194.750	53.438	9.320	42.520	2.731	415.37
Exterior 1 (C65)	314.71	49.027	516.124	141.620	20.280	20.200	7.238	855.74
Exterior 2 (C80)	240.33	49.027	394.141	108.149	14.600	13.020	5.528	663.55

Table 24: Controlling Loads on Exterior Columns

To complete the design of the columns, both PCAColumn and various hand calculations were utilized to determine the appropriate amount of flexural, shear and transverse reinforcement required by ACI 318-08. First the columns were entered into PCAColumn for the design of necessary flexural reinforcement. The column size remained the same as the interior columns for ease of design and analysis along with ease in constructability because of the conformity in design between both the gravity and lateral columns.

Using PCAColumn allowed for the input of both axial loads and flexural moments along the major and minor axes. Once the loads were entered into the program, the design yielded the following interaction diagram shown in Figure 28. Once the flexural reinforcement was designed using PCAColumn, the shear and transverse reinforcement using ACI 21.3 needed to be designed. For columns in intermediate concrete moment frames, hoops need to be provided at the beginning and end of each column. In this particular design, these hoops begin and end at a distance 4" from the column/beam joint face. In addition to the hoops, ties are provided throughout the rest of the column for transverse and shear reinforcement. The reinforcement specifics and details used to adhere to ACI 21.3 can be found in Table 25 and Figures 29 and 30 below.

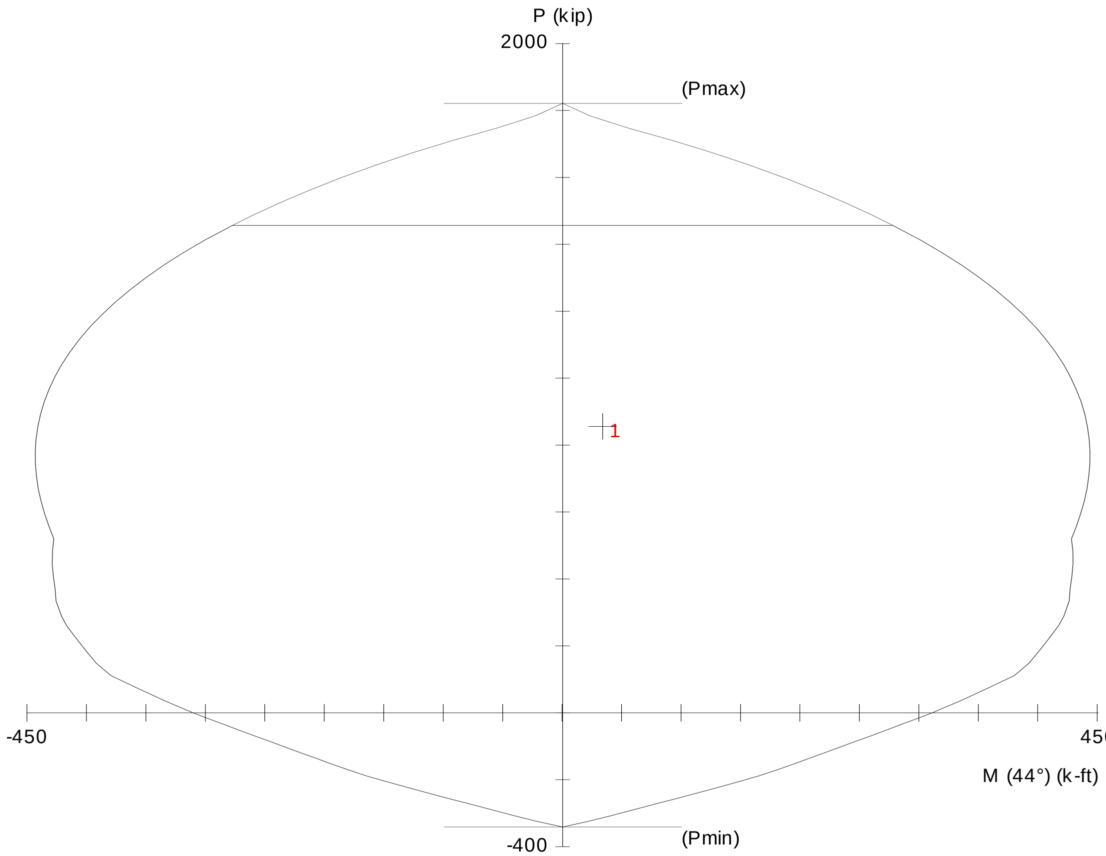


Figure 28: Interaction Diagram for Exterior Column C65

Type	Flexural Reinf.	Shear Reinf.		Transverse Reinf.
		Av_{min}	0.240	
Corner (C55)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"
Exterior 1 (C65)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"
Exterior 2 (C80)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"

Table 25: Required Reinforcement for Moment Frame Columns

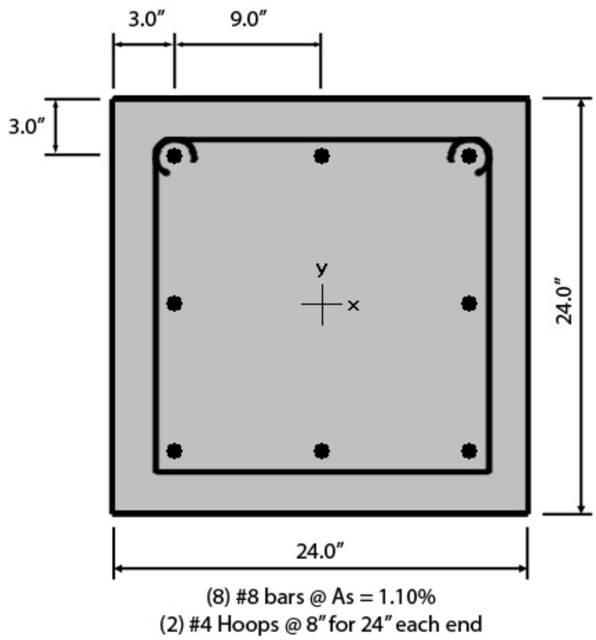
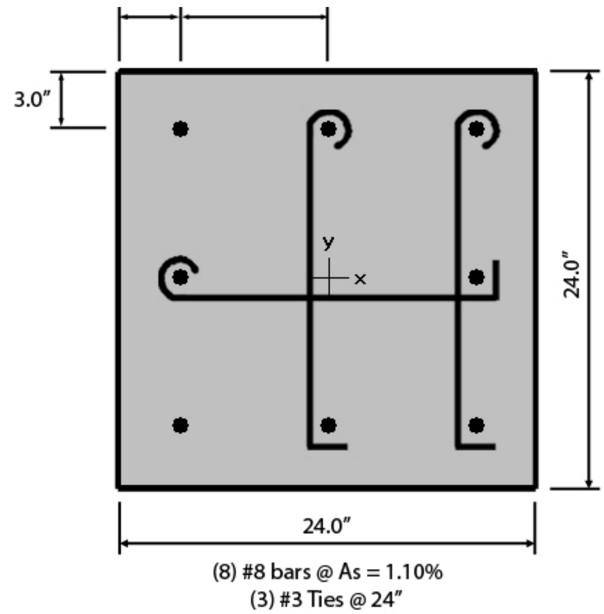


Figure 29: Exterior Column Detail (Column Ends)



Figure

Design Process – Concrete Moment Frame Design (Beams)

The concrete beams that are added as a part of the moment frames are designed to provide stiffness to the structure as well as resist lateral forces caused by the wind and seismic loading on the building. The beams also serve as an integral connection between the concrete moment frames and the two way flat plate slab because the beams carry both lateral loads and transfer gravity loads to the exterior columns of the building. To begin the design of the concrete beams, the loads were once again identified using ETABS. For simplicity purposes two exterior beams were identified as beams that could be designed and then reproduced throughout the remainder of the building. The beams represent the two typical beam lengths found throughout the building. However, it was determined that both beams would require the same reinforcement and therefore the results for only Beam 9 will be discussed and replicated throughout the entire concrete moment frame.

The table below shows the maximum loading results that were found in ETABS for Beam 9. First, a preliminary sizing was completed to obtain a rough estimate for the size of the beams. The width of the beam was first chosen to be 12" and then using the equation $bd^2 \geq 20M_u$, the d for each beam was determined. From this the maximum d found was 9", however this rough estimation only takes into account the moment in the beam and does not account for the fact that the beam will also serve as an edge beam in the two way flat plate concrete slab system. Because of this it was determined that the beam shall be designed to be deeper. The beam also will experience additional shear and axial forces because it is part of a moment frame and therefore cannot be designed solely on the moment alone. Both of these issues led to the increase in the depth of the beams to 18". This approximation is deemed adequate and will be verified by hand calculations providing the design of the necessary reinforcement.

Beam 9 (Earthquake)				Beam 9 (Wind)			
Story	**V2**	**T (k-ft)**	**M3 (k-ft)**	**Story**	**V2**	**T (k-ft)**	**M3 (k-ft)**
ROOF	3.86	0.10	29.52	**ROOF**	3.95	0.11	30.68
8TH	1.82	0.17	13.79	**8TH**	4.91	0.24	37.51
7TH	1.72	0.17	13.06	**7TH**	5.04	0.23	38.55
6TH	1.72	0.18	13.04	**6TH**	5.27	0.26	40.22
5TH	1.63	0.18	12.39	**5TH**	5.3	0.26	40.48
4TH	1.5	0.17	11.41	**4TH**	5.19	0.26	39.66
3RD	1.35	0.15	9.89	**3RD**	4.83	0.25	36.88
2ND	1.05	0.13	5.98	**2ND**	4.22	0.22	32.26
1ST	0.62	0.08	4.74	**1ST**	2.75	0.14	21.04

Table 26: Concrete Beam Forces – Beam #9

After a cross section was chosen and the design loads were identified the flexural, transverse and shear reinforcement needed in the beams was designed using provisions for intermediate concrete moment frames found in ACI 21.3. First, the flexural reinforcement necessary in the beam was sized using the equation $A_s = \frac{3\sqrt{f'_c} b_w d}{f_y}$. Next, the need for torsional reinforcement was checked and it was determined that torsional reinforcement is not necessary in the beam. After that, the design of the shear and transverse reinforcement using ACI 21.3 was completed by first checking if $V_s \leq 4\sqrt{f'_c} b_w d$ and then designing for minimum shear reinforcement in beams using $A_{v,min} = \frac{50 b_w s}{f_y}$. After determining the area of reinforcement necessary, ACI 21.3 indicates that hoops need to be provided in beams at a spacing of s = d/4 for a distance of h/2 beginning and ending no more than 2" from the face of the supporting member. In addition to the hoops, stirrups are provided throughout the rest of the column for transverse and shear reinforcement. The reinforcement specifics and details used to adhere to ACI 21.3 can be found below in Table 27 as well as Figures 31 and 32. The remainder of the beams designed can be located in Appendix E along with their respective load tables.

Fifth Floor B9					
Given:		Estimation of d		Torsional Reinforcement	
M_u	40.48	$bd^2 \ge 20M_u$	8.214	$Tu \le \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	5.3	Use d =	15.5	Reinf. Needed?	no
T_u	0.26	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$Vu \ge \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \le 4\sqrt{f'_c}bd$	yes	$V_s \le 4\sqrt{f'_c}bd$	yes
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Stirrups @ 8" throughout length**	

Table 27: Beam B9 Design – Fifth Floor

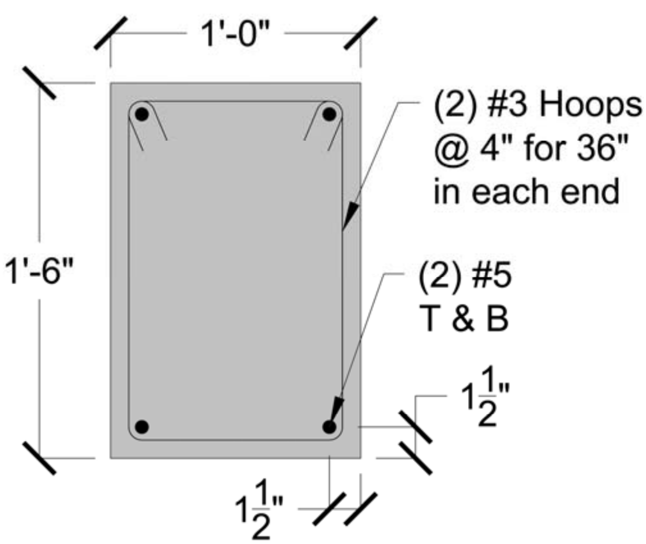


Figure 31: Beam B9 Detail - End of Beam

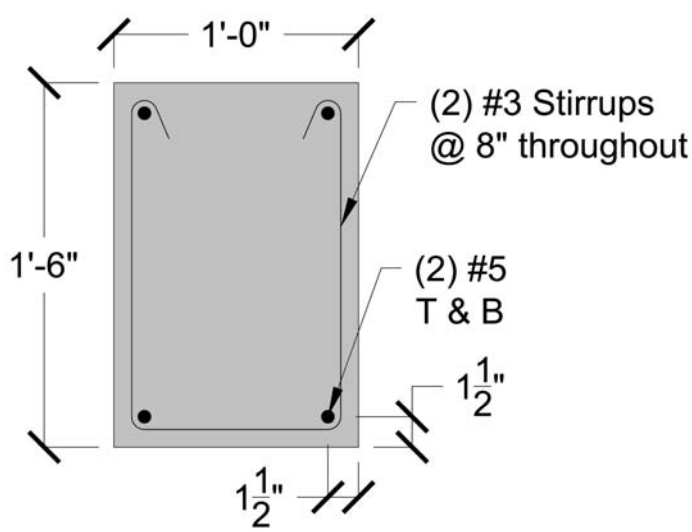


Figure 32: Beam B9 Detail – Throughout Beam

Design Process – Drift Analysis using ETABS

Using ETABS it was determined that the maximum seismic drift in the shear walls and concrete moment frame was well below the allow limit of $\Delta = 0.020h_{sx}$. The maximum drift was found using the seismic load in the y direction with an added 8% eccentricity for accidental torsion considerations. For comparison purposes the maximum drift found when analyzing the system using only the shear walls is also shown. The results of the maximum seismic story drift determined are illustrated in Tables 28 and 29 as well as Figure 33 below.

Controlling Seismic Drift - Shear Walls Only										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$		
Second	14.33	14.33	0.3268	<	3.44	Acceptable	0.3268	<	3.44	Acceptable
Third	11.00	25.33	0.2836	<	2.64	Acceptable	0.6104	<	6.08	Acceptable
Fourth	11.00	36.33	0.3428	<	2.64	Acceptable	0.9532	<	8.72	Acceptable
Fifth	11.00	47.33	0.386	<	2.64	Acceptable	1.3392	<	11.36	Acceptable
Sixth	11.00	58.33	0.4144	<	2.64	Acceptable	1.7536	<	14.00	Acceptable
Seventh	13.17	69.33	0.4408	<	3.16	Acceptable	2.1944	<	16.64	Acceptable
Eighth	13.50	82.67	0.5468	<	3.24	Acceptable	2.7412	<	19.84	Acceptable
Roof	13.83	96.33	0.5636	<	3.32	Acceptable	3.3048	<	23.12	Acceptable

Table 28: Controlling Seismic Drift – Shear Walls Only Design

Controlling Seismic Drift - Moment Frames Included										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$			Total Drift (in)	Allowable Story Drift (in) $\Delta_{SEISMIC} = .020h_{sx}$		
Second	14.33	14.33	0.3024	<	3.44	Acceptable	0.3024	<	3.44	Acceptable
Third	11.00	25.33	0.2576	<	2.64	Acceptable	0.56	<	6.08	Acceptable
Fourth	11.00	36.33	0.308	<	2.64	Acceptable	0.868	<	8.72	Acceptable
Fifth	11.00	47.33	0.35	<	2.64	Acceptable	1.218	<	11.36	Acceptable
Sixth	11.00	58.33	0.3824	<	2.64	Acceptable	1.6004	<	14.00	Acceptable
Seventh	13.17	69.33	0.3976	<	3.16	Acceptable	1.998	<	16.64	Acceptable
Eighth	13.50	82.67	0.4916	<	3.24	Acceptable	2.4896	<	19.84	Acceptable
Roof	13.83	96.33	0.5056	<	3.32	Acceptable	2.9952	<	23.12	Acceptable

Table 29: Controlling Seismic Drift – Shear Walls and Moment Frame Design

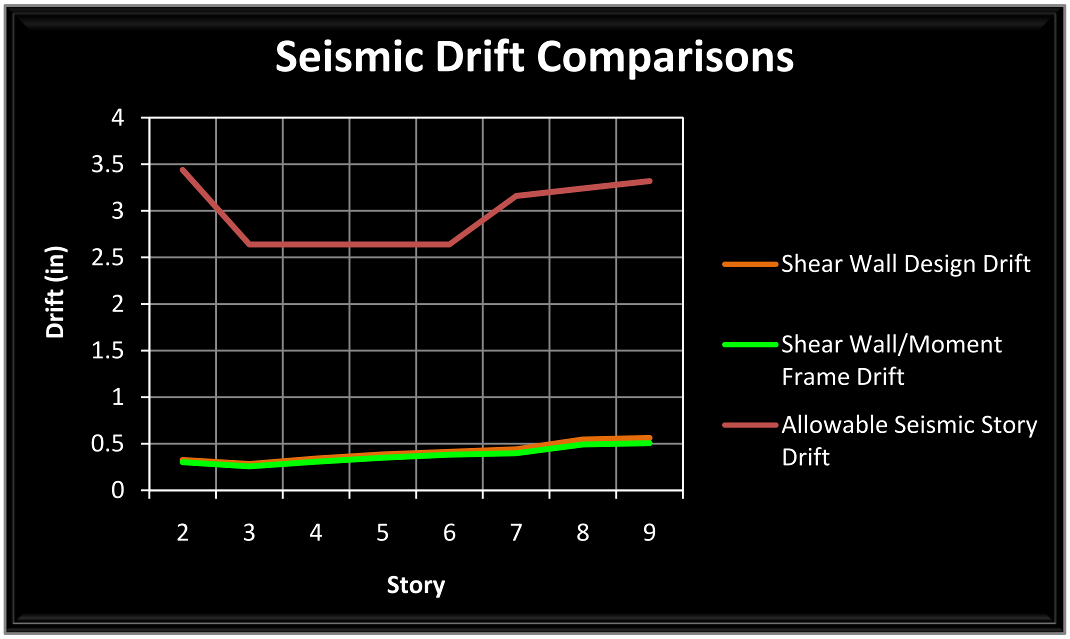


Figure 33: Actual versus Allowable Seismic Story Drift

The drift due to wind in the building controlled the design before the amplified story drift using ASCE 7-05 was applied for seismic drift. However, the design in general is controlled by the gravity loading on the concrete. A drift limitation due to wind loading of Δ = H/400 was used as recommended by ASCE 7-05. This limitation was used specifically for the drift in the entire building including the diaphragms. Also used was a drift limitation of Δ = H/500 for wind serviceability considerations to minimize damage on non-structural components like claddings and partitions. For comparison purposes the maximum drift found when analyzing the system using only the shear walls is also shown. The results of the maximum wind story drift determined are illustrated in Tables 30 and 31 as well as Figure 34 below.

Controlling Wind Drift - Shear Walls Only										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{WIND} = H/400			Total Drift (in)	Allowable Story Drift (in) Δ_{WIND} = H/400		
Second	14.33	14.33	0.177	<	0.43	Acceptable	0.177	<	0.43	Acceptable
Third	11.00	25.33	0.137	<	0.33	Acceptable	0.314	<	0.76	Acceptable
Fourth	11.00	36.33	0.1559	<	0.33	Acceptable	0.4699	<	1.09	Acceptable
Fifth	11.00	47.33	0.1674	<	0.33	Acceptable	0.6373	<	1.42	Acceptable
Sixth	11.00	58.33	0.1727	<	0.33	Acceptable	0.81	<	1.75	Acceptable
Seventh	13.17	69.33	0.2103	<	0.33	Acceptable	1.0203	<	2.08	Acceptable
Eighth	13.50	82.67	0.2109	<	0.40	Acceptable	1.2312	<	2.48	Acceptable
Roof	13.83	96.33	0.1648	<	0.41	Acceptable	1.396	<	2.89	Acceptable

Table 30: Controlling Wind Drift – Shear Walls Only Design

Controlling Wind Drift - Moment Frames Included										
Floor	Story Height (ft)	Total Height (ft)	Story Drift (in)	Allowable Story Drift (in) Δ_{WIND} = H/400			Total Drift (in)	Allowable Story Drift (in) Δ_{WIND} = H/400		
Second	14.33	14.33	0.154	<	0.43	Acceptable	0.154	<	0.43	Acceptable
Third	11.00	25.33	0.112	<	0.33	Acceptable	0.266	<	0.76	Acceptable
Fourth	11.00	36.33	0.1264	<	0.33	Acceptable	0.3924	<	1.09	Acceptable
Fifth	11.00	47.33	0.1329	<	0.33	Acceptable	0.5253	<	1.42	Acceptable
Sixth	11.00	58.33	0.1349	<	0.33	Acceptable	0.6602	<	1.75	Acceptable
Seventh	13.17	69.33	0.1616	<	0.33	Acceptable	0.8218	<	2.08	Acceptable
Eighth	13.50	82.67	0.1594	<	0.40	Acceptable	0.9812	<	2.48	Acceptable
Roof	13.83	96.33	0.1214	<	0.41	Acceptable	1.1026	<	2.89	Acceptable

Table 31: Controlling Wind Drift – Shear Walls and Moment Frame Design

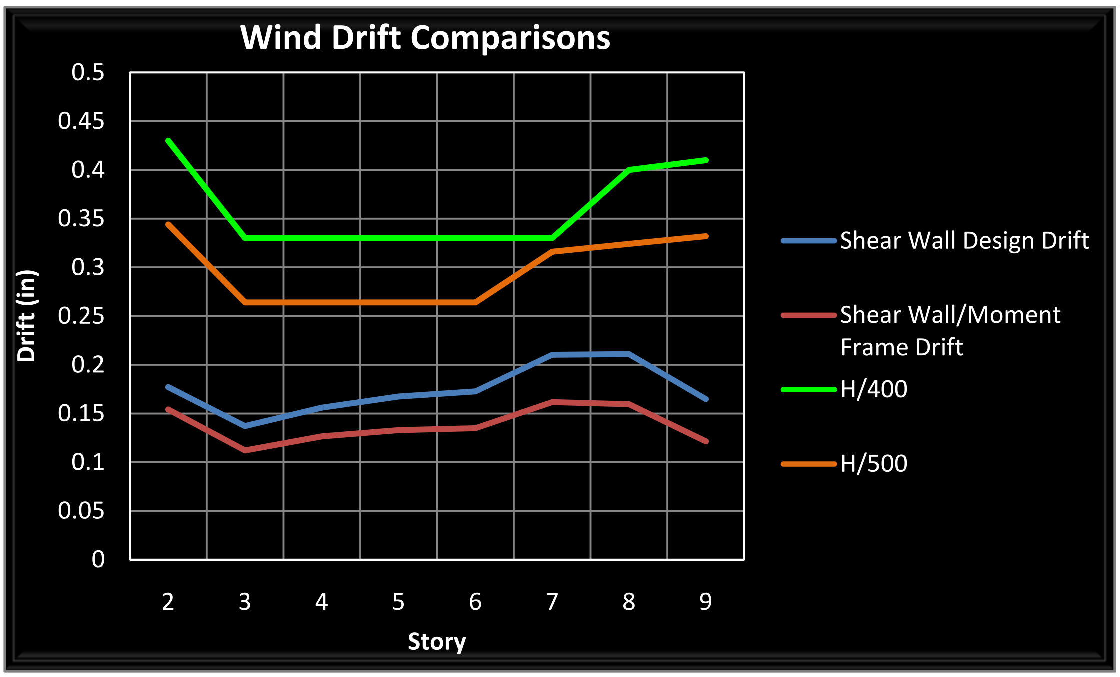


Figure 34: Actual versus Allowable Seismic Story Drift

Structural Depth Conclusions:

Gravity System

The results of the gravity redesign conclude that a two way flat plate concrete slab system with reinforced concrete columns was adequate to support the gravity loads of Washington Park Condominiums and therefore is seen as a viable alternative to the composite steel joist system currently in place. The layout of the floors was designed in such a way that all of the columns remained in the same place and the individual apartment floor plans were not impacted. In additional to the layout of the floors, the floor to floor heights of each floor did not change, however the ceiling heights on the each floors had to be dropped to accommodate the 10" concrete slab and airspace necessary to run all of the needed MEP equipment throughout the ceiling cavity. This will discussed at more length in the architectural detailing breadth. Another impact on the design was that the overall building weight increased because the structure was changed from steel to concrete. This increase led to the resizing of the existing spread footing foundations and the resizing of the piers connecting those foundations to the columns.

Lateral System

For the lateral system a core of "shaft" shear walls was originally designed to replace the system of steel moment frames. The shear walls were laid out around the stair and elevator shafts within the building so that there would be little to no impact on the floor plans. Upon completing a dynamic analysis using ETABS it was determined that the shear wall system was sufficient to resist the lateral loads of the building. In addition to this the story drift and overall building drift of the structure were within the maximum limitations laid out in ASCE 7-05. With that being said there were issues concerning the rotational period of the building as well as the accidental torsion caused by the buildings rectangular shape. This led to the design of exterior concrete moment frames to help add stiffness to the building, while reducing the accidental torsion found in the structure. Although the lateral system of the building may be over designed, the knowledge and understanding that was gained in designing both shear walls and concrete moment frames proves to be invaluable.

Without the use of ETABS in coordination with ASCE 7-05 it would not have been possible to determine the torsional effects that the lateral loads had on the structure. Although the torsional effects did cause for a modification in design, both the gravity and lateral systems within the building were designed based on strength and then checked for drift. It is understood that this will not always be the case especially since a building may be located in an area of high seismic or wind loading. Overall the redesign of the structural system for Washington Park Condominiums proved to be adequate using the given loads found in and on the structure.

Acoustics Breadth Study:

The purpose of the acoustical breadth study is to investigate the sound transmission through various floors and walls within the building. Acoustics was chosen as a breadth study because it was mentioned as a possible option by the owner of the building. The owner expressed concern that the residents would experience unwanted noise in their apartment from other apartments, roof mechanical equipment, first floor retail spaces and the elevators. This concern stems from the fact that many of these louder spaces in the building share walls with the apartments, therefore creating the possibility for unwanted noise. This concern also led to the owner commissioning an acoustics study to be performed for the existing structure of the building by the Sextant Group. The Sextant Group performed an acoustics study for the current wall and ceiling constructions in various locations in the building with the purpose of determining the estimated STC rating for each assembly. The estimated STC ratings were then compared to the minimum sound and impact isolation requirements for wall and floor assemblies found in IBC 2006.

The new acoustics study is necessary because of the change in structural system. A change in the structural system caused the wall and floor systems to also change and therefore modifies the STC rating of each individual assembly. The study will also perform transmission loss and STC calculations on each of the assemblies studied in the previous acoustics study as well as studying various other assemblies. To begin the analysis, the spaces that share common walls and/or floors and have a high expected noise level will be identified. The figure to the right shows the different spaces in the building, with red signifying the spaces that have a high expected noise level, blue signifying the apartments and green signifying spaces that were not considered in the study. From this figure it is clear to see that the main focus of the study is how the noise from the spaces with high expected noise levels transmits through the wall and floor assemblies and into the apartment units. The spaces that are

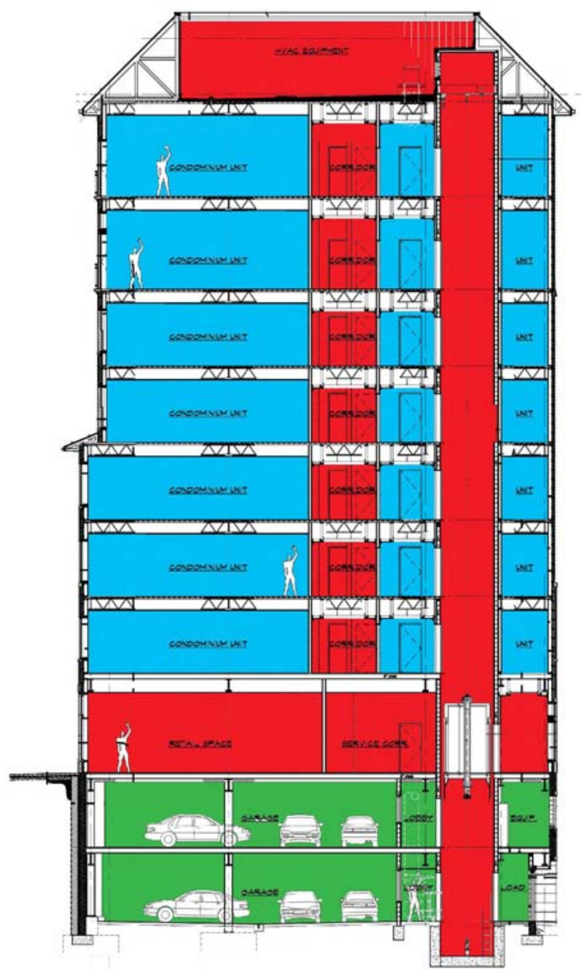


Figure 35: Spaces in building based on noise level.

signified in red include; the retail space, the elevator shaft, the corridor and the roof top mechanical equipment. Each space will be analyzed separately for how sound is transmitted from the space through the shared wall or floor assembly and into the apartment.

To begin the analysis of each individual space a few different parameters needed to be determined. These parameters include; the expected noise level in the adjacent space, the expected background noise of the space being studied and the sound absorption of the wall assembly. All of these values were found using Architectural Acoustics by M. David Egan. Architectural Acoustics also set forth the process of designing the necessary transmission loss and STC required, isolating each apartment from the various noise sources. Once those values were identified they were used to set the basis for the design along with the required noise reduction and transmission loss in each assembly. After the required transmission loss for each assembly was established, a wall of floor construction could be developed to meet the required level. Each assembly has different materials used in the wall construction based on the purpose of the assembly. The values for the transmission loss and STC of each material were found using Architectural Acoustics as well as Noise Control in Buildings by Cyril M. Harris. These values were added together to obtain the overall transmission loss and STC in each wall and floor assembly. This process was completed a total of five different times for five different wall and floor assemblies throughout the building. The results for the five separate assemblies are listed in Tables 32 thru 36 below.

Floor Sound Isolation Assembly used above First Floor Retail Space							
	125	**250**	**500**	**1000**	**2000**	**4000**	**STC**
Expected Noise Level in Retail Space	72	83	82	82	80	75	
Minus expected background noise in Apartment (RC-30)	45	40	35	30	25	20	
Required NR	**27**	**43**	**47**	**52**	**55**	**55**	**50**
Minus 10 log a2/S	-1	-1	-1	-1	-1	-1	
Required TL	**28**	**44**	**48**	**53**	**56**	**56**	**50**
Finding an Adequate Wall Construction:							
3/4" Wood Flooring on 1" glass fiber	0	1	0	1	1	1	-
10" Reinforced Concrete Slab	44	48	55	58	63	67	-
18" Airspace	12	12	14	15	16	8	-
1/2" Gypsum Wall Board Finished Ceiling	15	20	25	29	32	27	-
Total TL of Wall Construction	**71**	**81**	**94**	**103**	**112**	**103**	**95**
Difference between Actual and Required Transmission Loss	43	37	46	50	56	47	45

Table 32: Floor Sound Isolation between 1st Floor Retail Space and 2nd Floor Apartments.

Floor Sound Isolation Assembly used on Floors 3 thru 8							
	125	**250**	**500**	**1000**	**2000**	**4000**	**STC**
Expected Noise Level in Apartments	62	64	67	70	68	63	
Minus expected background noise in Apartment (RC-30)	45	40	35	30	25	20	
Required NR	**17**	**24**	**32**	**40**	**43**	**43**	**50**
Minus 10 log a2/S	-1	-1	-1	-1	-1	-1	
Required TL	**18**	**25**	**33**	**41**	**44**	**44**	**50**
Finding an Adequate Wall Construction:							
3/4" Wood Flooring on 1" glass fiber	0	1	0	1	1	1	-
10" Reinforced Concrete Slab	44	48	55	58	63	67	-
18" Airspace	12	12	14	15	16	8	-
1/2" Gypsum Wall Board Finished Ceiling	15	20	25	29	32	27	-
Total TL of Wall Construction	**71**	**81**	**94**	**103**	**112**	**103**	**95**
Difference between Actual and Required Transmission Loss	53	56	61	62	68	59	45

Table 33: Floor Sound Isolation between Floors on Apartment Levels 3 thru 8

Floor Sound Isolation Assembly used below Rooftop Mechanical Equipment							
	125	**250**	**500**	**1000**	**2000**	**4000**	**STC**
Likely noise level from Rooftop Mechanical Equipment	93	89	85	80	75	69	
Minus expected background noise in Apartment (RC-30)	45	40	35	30	25	20	
Required NR	**48**	**49**	**50**	**50**	**50**	**49**	**50**
Minus 10 log a2/S	-1	-1	-1	-1	-1	-1	
Required TL	**49**	**50**	**51**	**51**	**51**	**50**	**50**
Finding an Adequate Wall Construction:							
24 gauge decking with 1 3/8" sprayed insulation	17	22	26	30	35	41	-
10" Reinforced Concrete Slab	44	48	55	58	63	67	-
18" Airspace	12	12	14	15	16	8	-
1/2" Gypsum Wall Board Finished Ceiling	15	20	25	29	32	27	-
Total TL of Wall Construction	**88**	**102**	**120**	**132**	**146**	**143**	**105**
Difference between Actual and Required Transmission Loss	39	52	69	81	95	93	55

Table 34: Floor Sound Isolation between Penthouse Apartments and Rooftop Mechanical Equipment

Wall Sound Isolation Assembly to be used between Elevator and Apartment							
	125	**250**	**500**	**1000**	**2000**	**4000**	**STC**
Likely noise level from Elevator	86	85	84	83	82	80	
Minus expected background noise in Apartment (RC-30)	45	40	35	30	25	20	
Required NR	**41**	**45**	**49**	**53**	**57**	**60**	**50**
Minus 10 log a2/S	-1	-1	-1	-1	-1	-1	
Required TL	**49**	**50**	**51**	**51**	**51**	**50**	**50**
Finding an Adequate Wall Construction:							
18" Concrete Masonry Unit Wall	31	40	44	51	57	61	-
Resilient channels	1	1	2	2	1	1	-
2 Layers of 1/2" Gypsum Wall Board	19	26	30	32	29	37	-
Total TL of Wall Construction	**51**	**67**	**76**	**85**	**87**	**99**	**75**
Difference between Actual and Required Transmission Loss	9	21	26	31	29	38	25

Table 35: Wall Sound Isolation between Elevator Shaft and Apartment

Wall Sound Isolation Assembly to be used Between Apartments							
	125	**250**	**500**	**1000**	**2000**	**4000**	**STC**
Likely noise level in Apartments	62	64	67	70	68	63	
Minus expected background noise in Apartment (RC-30)	45	40	35	30	25	20	
Required NR	**17**	**24**	**32**	**40**	**43**	**43**	**50**
Minus 10 log a2/S	-1	-1	-1	-1	-1	-1	
Required TL	**18**	**25**	**33**	**41**	**44**	**44**	**50**
Finding an Adequate Wall Construction:							
2 Layers of 1/2" Gypsum Wall Board (each side)	19	26	30	32	29	37	-
2 Layers of 3 5/8" Steel Studs @ 24" O.C.	2	4	5	6	7	6	-
1/2" Air Gap	1	1	0	2	3	1	-
2 Layers of 3 1/2" fiberglass insulation	10	18	22	18	10	22	-
Total TL of Wall Construction	**32**	**49**	**57**	**58**	**49**	**66**	**61**
Difference between Actual and Required Transmission Loss	14	24	24	17	5	22	11

Table 36: Wall Sound Isolation between Two Apartments and Apartments and Corridors

After the five separate assemblies were designed, their acoustical performance was compared to that of the existing assemblies as provided by the Sextant Group. The results of the comparison can be found below in Table 37.

Acoustic Performance Comparison				
Assembly	**STC - HUD Noise Control Guide**	**STC - Existing Design**	**STC - New Design**	**Difference**
Floor Assembly between 1st Floor Retail and 2nd Floor Apartment	STC - 56	STC - 62	STC - 95	+33
Floor Assembly between Floors on Apartment Levels 3 thru 8	STC - 56	STC - 58	STC - 95	+37
Floor Assembly between Penthouse Apartment and Rooftop Mechanical Equipment	STC - 56	STC - 62	STC - 105	+43
Wall Assembly between Elevator Shaft and Apartments	STC - 56	STC - 55	STC - 75	+20
Wall Assembly between two Apartments and an Apartment and a Corridor	STC - 56	STC - 57	STC - 61	+4

Table 37: Acoustical Study Comparisons

Based on the calculations and the comparison between the acoustical study performed by Sextant Group and the one done above, it is obvious that all new wall and floor assemblies outperform both the requirements set forth by U.S. Department of Housing and Urban Development and the existing design. This can be attributed to the fact that the structural floor system of the building was changed to reinforced concrete which has extremely high TL and STC values when compared to the existing VESCOM composite joist system. Overall, the acoustical study met the requirements of the owner by increasing the TL and STC values in the wall/floor assemblies therefore creating better sound isolation between spaces.

Although the study is comprehensive it does not focus on vibrations caused by any of the mechanical equipment. This is because the vibrations from the rooftop mechanical equipment would be controlled by using Kinetics Model FDS free-standing spring isolators and Kinetics Model KSR Isolation Rails. Both of these isolation systems were discussed by the Sextant Group in the existing acoustical report and would be used if they were necessary in the new structure. For reference, the cut sheets for both of those systems are included in Appendix F.

Architectural Detailing Breadth Study:

The purpose of the architectural detailing breadth study is to examine how the ceiling cavity of the building will change due to the redesign of the structural floor system. The existing composite steel joist system was used as an architectural element that allowed the ceiling to be installed directly to the joist bottom chord. The 18" depth of the joists also allowed the mechanical systems (HVAC, plumbing, fire protection, electrical and telecommunications) to be installed within the joist system, saving space and allowing for higher ceilings and floor to floor height within the apartments. The figure right is an example of how the ceiling cavity is used as a space to run all of the necessary mechanical equipment.

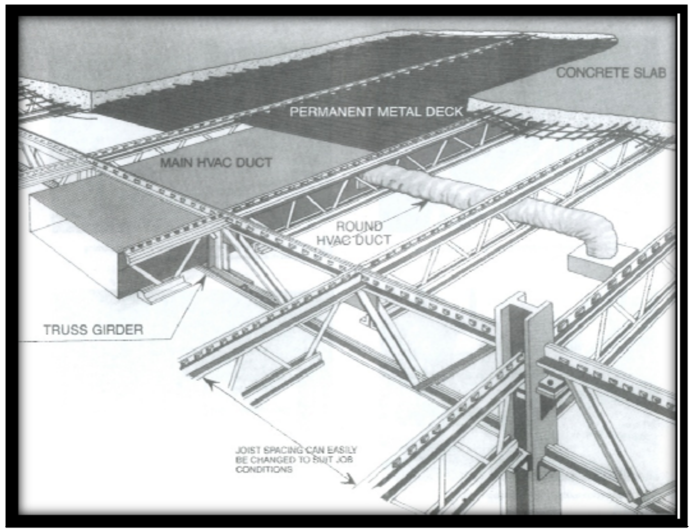


Figure 36: Existing Apartment Ceiling Cavity

Although there are many benefits that make the composite joist system effective, there are also issues that arise from having an inconsistent depth of space for the mechanical equipment within the ceiling cavity. One of the main issues with the existing system is the fact that the placement of the ductwork must coordinate with either openings in the joists or the 4 ft spaces between joists. The current system allows for circular ductwork of up to 12" in diameter to be run through the joists along with rectangular ductwork up to 12" x 6" and square ductwork of 9" x 9". Where the design demands larger sizes, the ductwork must be situated within the 4 ft spaces between joists. These spaces are open and allow a maximum duct depth of 18". An example of larger ductwork running parallel to the joists through the ceiling cavity can be seen in Figure 37 below.

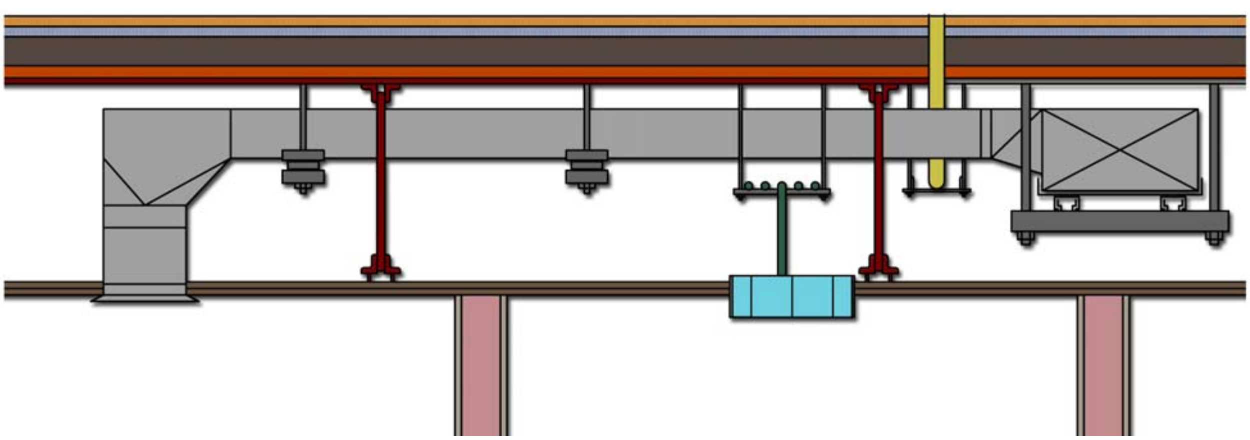


Figure 37: Ceiling Cavity in Apartment

The composite joist floor system has many benefits other than just the allowable depth within the ceiling cavity. One of the main benefits of the system is the fact that the ductwork and other equipment can easily be attached and hung from the top chord of the joist. This allows for more simplified steel to steel connections that are typical to all steel construction. Figure 38 below shows how the ductwork is currently connected to the bottom chord of the joist or the bottom of the steel decking. The connections consist of either a bolted or welded stud used to anchor and support the hanger straps of the ductwork. These connections will be discussed more below when the connection necessary for hanging the ductwork from the new concrete slab is discussed.

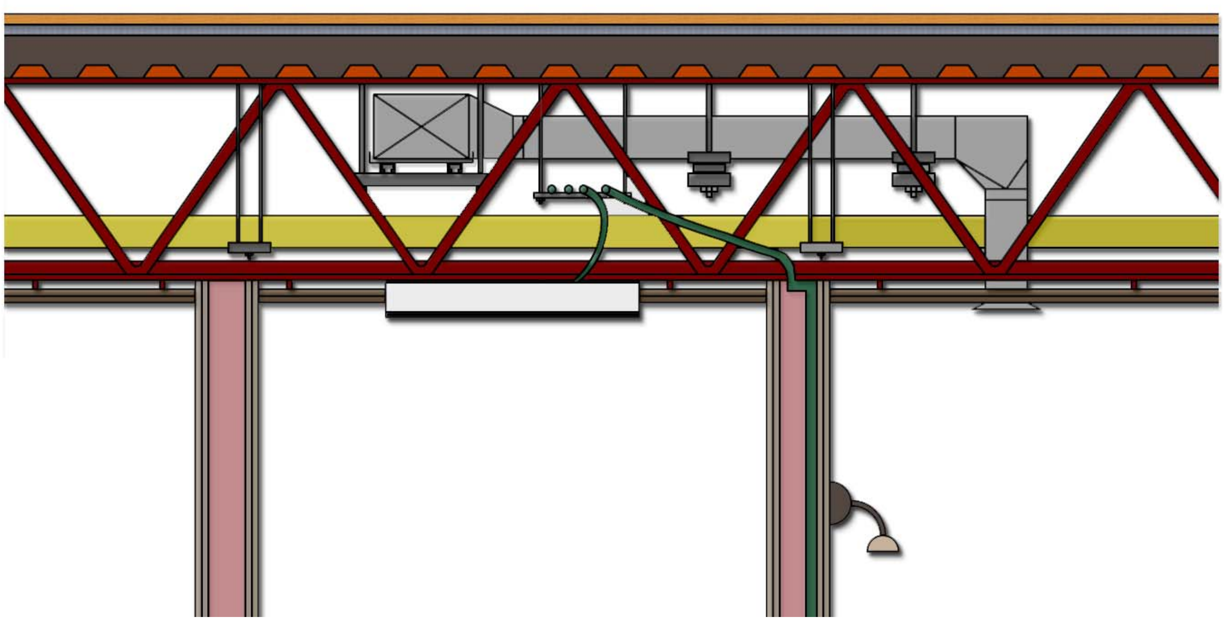


Figure 38: Ceiling Cavity in Corridor/Apartment

Overall, the composite joist system is extremely effective in reducing the amount of space that is necessary for the ceiling cavity because it is able to integrate the structural system and mechanical systems into the same space.

To begin the analysis of the ceiling cavity in the new reinforced concrete floor system, it is essential to determine the amount of open space that is necessary for the ductwork of the building. Since the depth of the slab is uniform throughout the building, the cavity depth will not change and therefore the placement of the ductwork will not have to be modified like it did in the existing composite joist system. Once the amount of available space is determined, it will also be necessary to decide whether or not any of the ductwork in the building needs to be resized or moved based on the fact that there is less space in the ceiling.

To begin the design, the coordination of the systems and design requirements of the building were taken into account to determine the allowable depth of the ceiling cavity. Based on the floor to ceiling heights of the building as set forth by the architect along with the acoustical performance study as shown above, the depth of the airspace between the bottom of the concrete slab and the top of the ceiling is 18". This size allows for optimal acoustical performance of the floor system as well as maintaining the finished floor to ceiling height within the apartments of 8'-9". The 18" airspace chosen for the ceiling cavity is adequate for all of the ductwork in the building since there are no ducts that have a depth greater than 15". The added 3" of depth allows for the addition of insulation to the ductwork if it is deemed necessary by the mechanical contractor or the architect. For the purpose of this study, and the acoustics study, no insulation was added. The figure below shows a typical cavity design for the space between the corridor and the apartments, using the new structural concrete floor system.

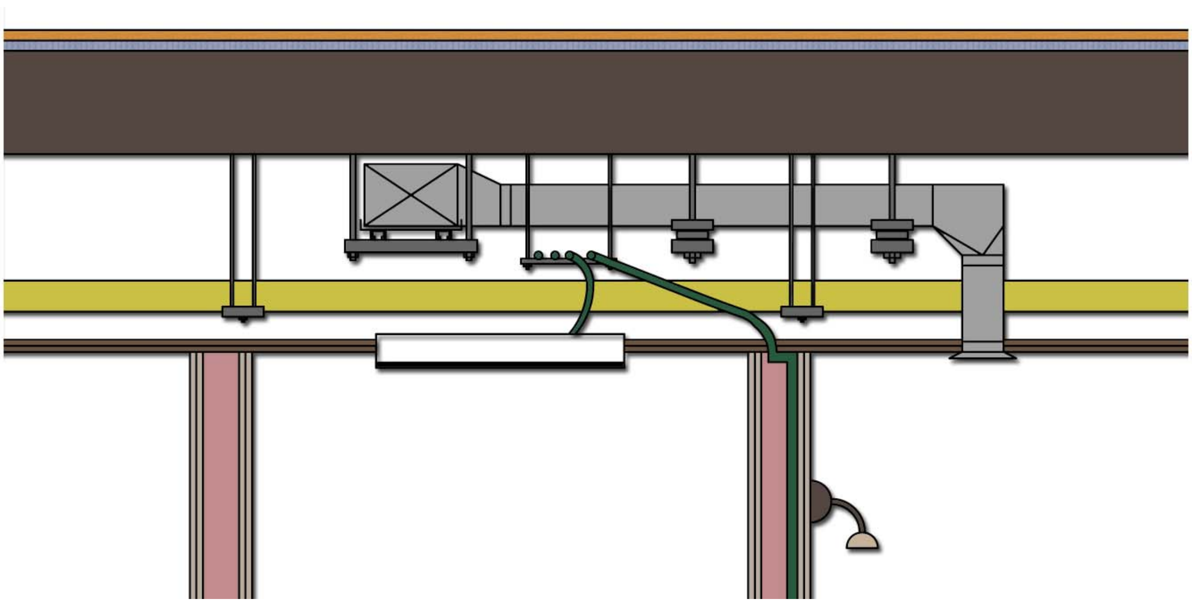


Figure 39: New Corridor/Apartment Ceiling Cavity

To continue the design it was essential to establish acceptable spacing between ductwork and other mechanical equipment in the cavity. For this design, the minimum spacing between all of the equipment has been set at 3.0". A spacing of 3.0" allows for the addition of 1" of insulation to the ductwork and piping, while still creating clearance of at least 1". Another design issue that needed to be revised was the connections between the hanger straps of the ductwork and the concrete slab. This connection would require a lag screw along with an expansion shield being drilled into the slab so that the straps could be attached. The below detail shows the difference between the connections required when hanging ducts from the existing steel decking and the new concrete slab.

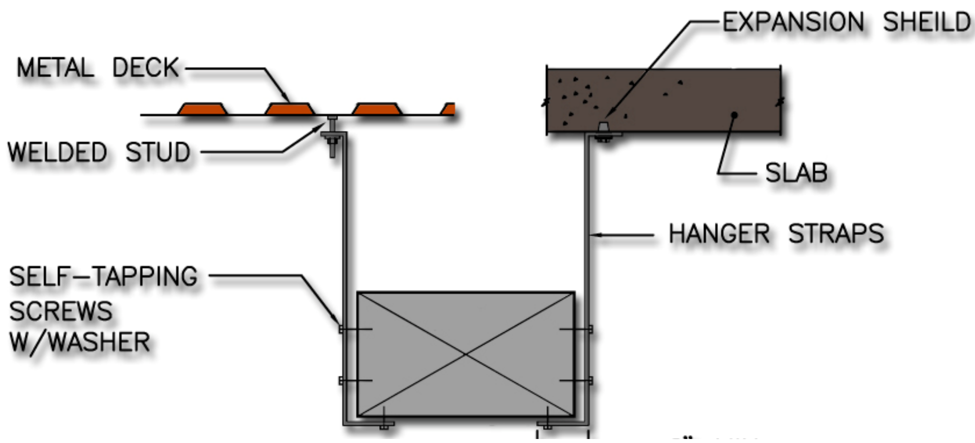


Figure 40: Ductwork to Slab Connections

For the new connection, ¼" diameter TITEN® Concrete Screws were chosen to attach the straps to the concrete. The screws in the connection are 3 ¼" long and are covered by a lag screw expansion shield. The entire connection has a tension capacity of 726 lbs and a shear capacity of 900 lbs. The cut sheets for both the concrete screw and the lag screw expansion shield are included for reference in Appendix G.

Based on a comparison between the existing composite steel joist system and the new reinforced concrete system it is apparent that both systems are adequate for the design and placement of the mechanical equipment within the ceiling cavity. Even though both systems are sufficient in their design, there are benefits and drawbacks inherent in the coordination between each system and the mechanical equipment present in the design. There appears to be no significant cost difference in terms of the placement of the mechanical equipment because the ceiling cavity of the existing design has already been optimized to fit all of the ductwork and piping. The only noticeable benefit apparent using the concrete slab system is the fact that the airspace used to run the mechanical equipment is deeper and more uniform. Overall, both structures provide sufficient ceiling cavities that allow the mechanical, electrical, plumbing and other systems the opportunity to be designed and constructed with maximum coordination and minimal interference.

Final Conclusions and Recommendations:

The main goal of a designing a new structural system for Washington Park Condominiums proved to be both effective as well as ineffective. In many ways changing the structural steel system to that of a system completely made from concrete holds some inherent benefits. For one, many apartment and condominium buildings in general are built using concrete because of the material's intrinsic properties of heaviness and mass, which allows for more lateral stiffness and resistance of horizontal movement. Because of this, occupants in tall buildings experience less building motion as well as less noise and vibration from other parts of the building.

One of the main issues in the building was the fact that every beam and column in the structure was used as part of the moment frame. This seemed extremely inefficient and was one of the main causes for the lateral system redesign. With that in mind, shear walls and concrete moment frames were designed to resist the lateral loads on the building. Although the shear walls proved to be sufficient in resisting these loads, the exterior concrete moment frame was designed to reduce the period and add stiffness to the building. In some respect, the moment frames can be viewed as over design, however the value found in designing various lateral resistance systems proves to be particularly beneficial.

When recalling the goals set forth at the onset of this report, it is apparent that most of them have been accomplished to some degree. Structurally the building satisfies all requirements found in IBC 2003 and ASCE 7-05. More specifically, the flat plate slab, columns and shear walls were all able to be designed with minimal to no impact on the building's architecture and floor plan. Lastly, the ultimate goal of exploring reinforced concrete as a structural system as well as learning of its similarities and differences from a steel system was accomplished.

In terms of breadth studies, both the acoustics and architectural detailing studies are viewed as relevant. The acoustics study requested by the owner, allows for better sound isolation between apartments as well as better isolation between the apartments and spaces which have high noise levels. The detailing study showed that the change in structural system would not significantly impact the ceiling cavity or the necessary coordination between the structure and the mechanical systems of the building.

The final conclusions and recommendations of this study are that the current structural system of the building is the most efficient. However, the benefits to designing a reinforced concrete system in mid to high rise apartment buildings are obvious and well documented throughout the industry. Overall, it is concluded that a project such as this takes years of experience and understanding to design properly so that the structures efficiency is maximized. From this report it has be determined that the design completed by the engineers is both practical and capable of meeting the design requirements, making it the correct design for the building.

Works Cited:

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Das, Braja M. Principles of Foundation Engineering. 6th ed. Belmont: Thomson-Engineering, 2006.

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Appendix A: Flat Plate Reinforce Concrete Slab Design

Direct Design Method

Total Static Moments at Factored Loads

Frame A		Frame B	
w_u	0.2824	w_u	0.2824
l_2	28.333	l_2	28
l_n	28	l_n	28.333
M_o	676.10	M_o	685.38
l_2/l_1	1.01	l_2/l_1	0.988
α	0	α	0
b_{Col}	168	b_{Col}	168
b_{Mid}	168	b_{Mid}	172
d_{eff}	8.3125	d_{eff}	8.9375

Frame A				Frame B			
	M^-_{int}	M^+	M^-_{int}		M^-_{int}	M^+	M^-_{int}
Total Moment	-439.47	236.64	-439.47	Total Moment	-445.50	239.88	-445.50
Column Strip	-329.60	141.98	-329.60	Column Strip	-334.12	143.93	-334.12
Middle Strip	-109.87	94.65	-109.87	Middle Strip	-111.37	95.95	-111.37

Frame A - Reinforcement		Frame B - Reinforcement	
M^- (Column Strip)	#5 @ 5.5" O/C	M^- (Column Strip)	#5 @ 6" O/C
M^+ (Column Strip)	#5 @ 14" O/C	M^+ (Column Strip)	#5 @ 14" O/C
M^- (Middle Strip)	#5 @ 15.25" O/C	M^- (Middle Strip)	#5 @ 15.75" O/C
M^+ (Middle Strip)	#5 @ 15.25" O/C	M^+ (Middle Strip)	#5 @ 15.75" O/C

Interior Moments

Frame A		Frame B	
M^- (0.65M_o)	439.47	M^- (0.65M_o)	445.50
M^+ (0.35M_o)	236.64	M^+ (0.35M_o)	239.88

Endspan Moments

Frame A		Frame B	
M^-_{int}	473.27	M^-_{int}	479.77
M^+	338.05	M^+	342.69
M^-_{ext}	202.83	M^-_{ext}	205.61

Shear Capacity in Slab

V_u	52.41	OK
ϕV_c	142.44	

Punching Shear Capacity in Slab

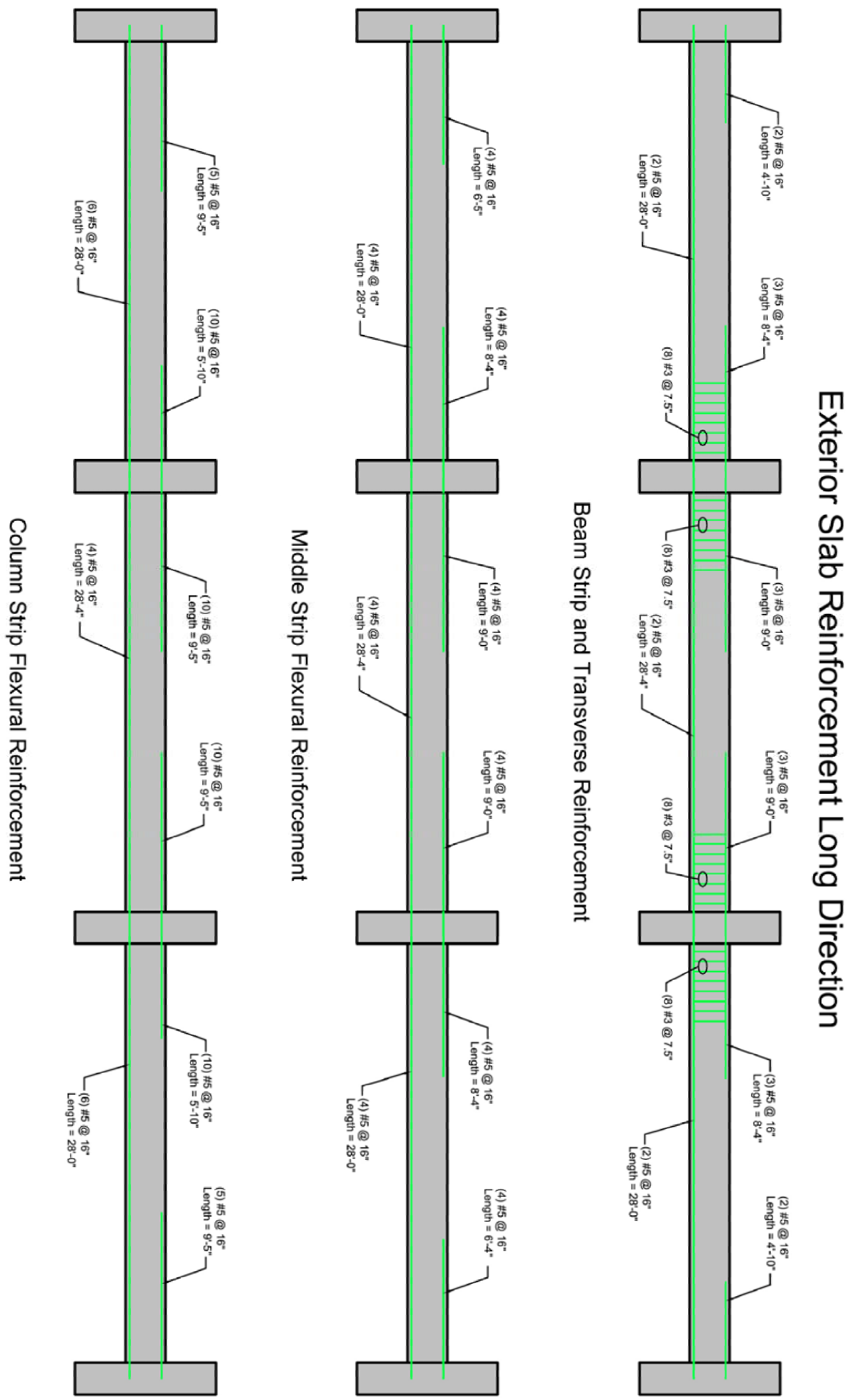
V_u	222.9050976	NO GOOD
ϕV_c	128.715832	

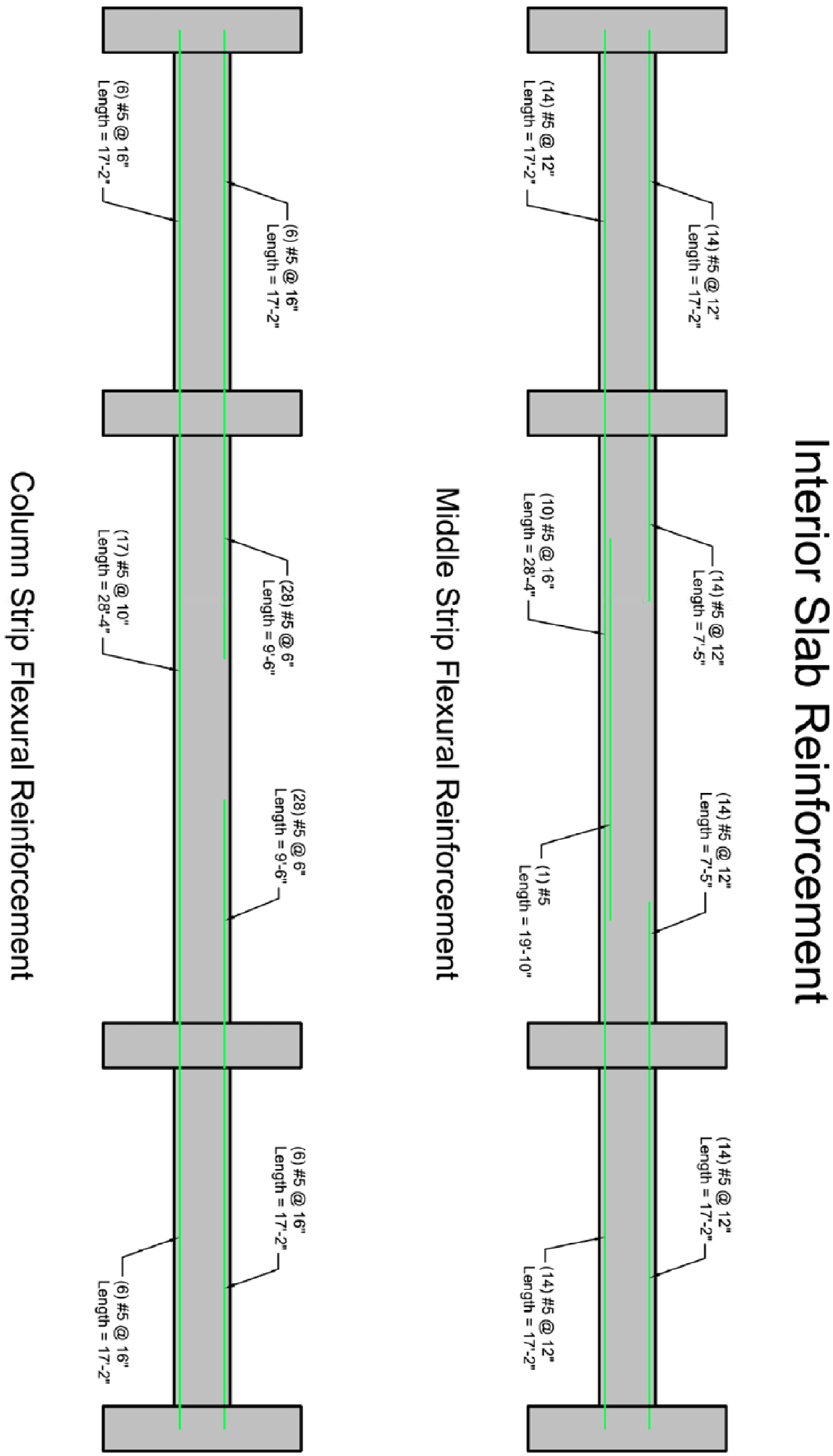
Shear Reinforcement

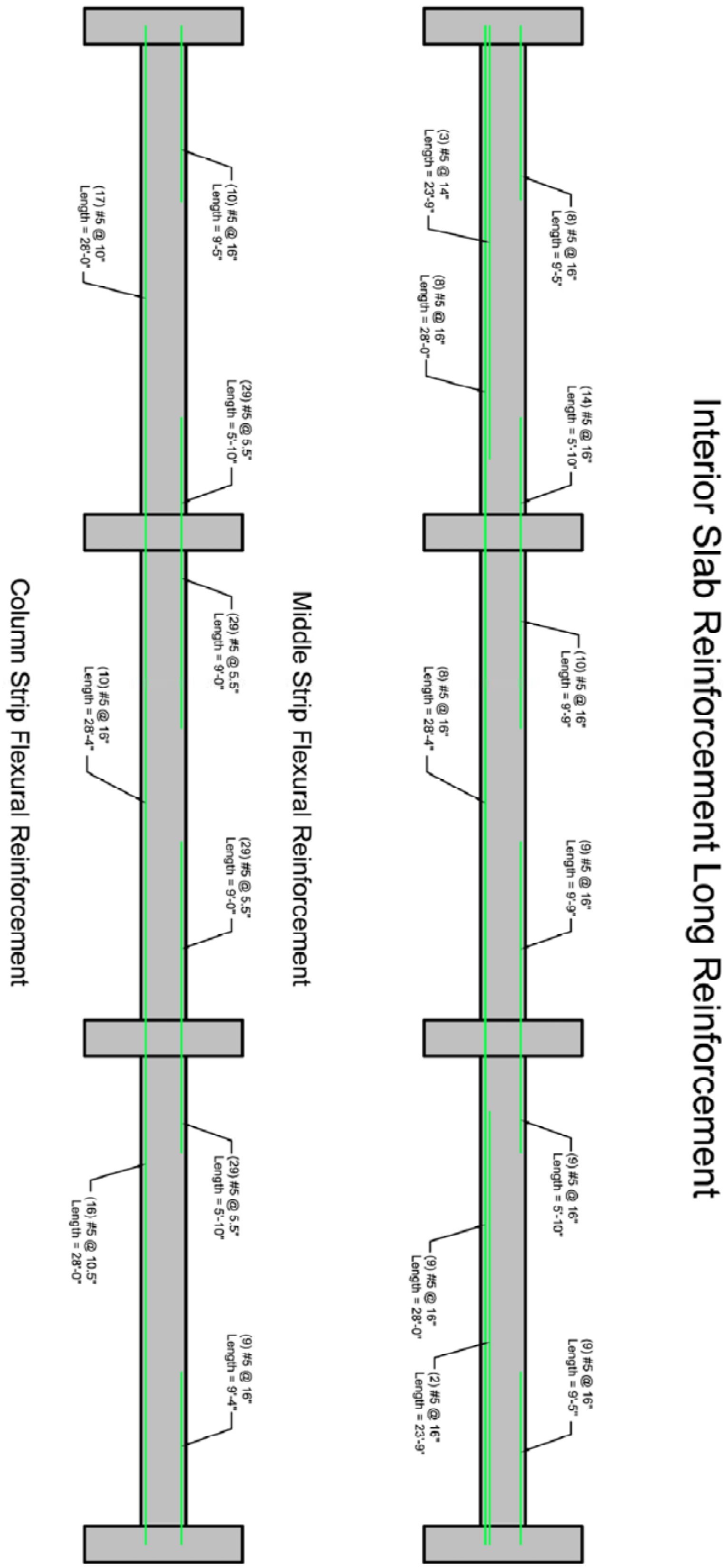
Bar/Wire Limit - V_c	244.19
$V_u \le V_c$	USE BAR/WIRE
V_s	188.68
s = d/2	4.5
A_v	1.57
Use (15) #3 Stirrups @ 4.5"	

Frame A		Column Strip		Middle Strip	
Description		M^-	M^+	M^-	M^+
1	Moment	-329.60	141.98	-109.87	94.65
2	Width of Strip (b)	168	168	168	168
3	Effective d	8.3125	8.3125	8.3125	8.3125
4	Mn = Mu/ϕ	-366.22	157.76	-122.07	105.17
5	Mn*12/b	-26.16	11.27	-8.72	7.51
6	R = Mn/bd^2	-378.58	163.08	-126.19	108.72
7	ρ (Table A.5)	0.0067	0.0026	0.00213	0.0018
8	A_s = ρbd	9.357	3.631	2.975	2.514
9	A_{smin} = .002bt	3.36	3.36	3.36	3.36
10	N = Larger of 8 & 9/0.31	30	12	11	11
11	N_{min} = b/2t	8	8	8	8

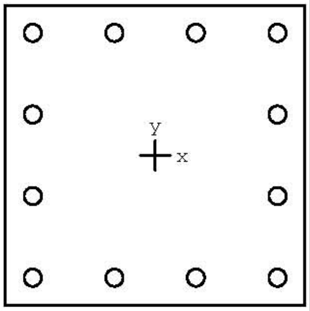
Frame B		Column Strip		Middle Strip	
Description		M^-	M^+	M^-	M^+
1	Moment	-334.12	143.93	-111.37	95.95
2	Width of Strip (b)	168	168	172	172
3	Effective d	8.9375	8.9375	8.9375	8.9375
4	Mn = Mu/ϕ	-371.25	159.92	-123.75	106.62
5	Mn*12/b	-26.52	11.42	-8.63	7.44
6	R = Mn/bd^2	-331.98	143.00	-108.08	93.12
7	ρ (Table A.5)	0.00583	0.00245	0.00188	0.0021
8	A_s = ρbd	8.754	3.679	2.890	3.228
9	A_{smin} = .002bt	3.36	3.36	3.44	3.44
10	N = Larger of 8 & 9/0.31	28	12	11	11
11	N_{min} = b/2t	8	8	9	9


Exterior Slab Reinforcement Long Direction
Beam Strip and Transverse Reinforcement
(2) #5 @ 16" Length = 4'-10"
(2) #5 @ 16" Length = 28'-0"
(3) #5 @ 16" Length = 8'-4"
(8) #3 @ 7.5"
(8) #3 @ 7.5"
(3) #5 @ 16" Length = 9'-0"
(2) #5 @ 16" Length = 28'-4"
(3) #5 @ 16" Length = 9'-0"
(8) #3 @ 7.5"
(8) #3 @ 7.5"
(3) #5 @ 16" Length = 8'-4"
(2) #5 @ 16" Length = 28'-0"
(2) #5 @ 16" Length = 4'-10"
Middle Strip Flexural Reinforcement
(4) #5 @ 16" Length = 6'-5"
(4) #5 @ 16" Length = 28'-0"
(4) #5 @ 16" Length = 8'-4"
(4) #5 @ 16" Length = 9'-0"
(4) #5 @ 16" Length = 28'-4"
(4) #5 @ 16" Length = 9'-0"
(4) #5 @ 16" Length = 8'-4"
(4) #5 @ 16" Length = 28'-0"
(4) #5 @ 16" Length = 6'-4"
Column Strip Flexural Reinforcement
(5) #5 @ 16" Length = 9'-5"
(6) #5 @ 16" Length = 28'-0"
(10) #5 @ 16" Length = 5'-10"
(10) #5 @ 16" Length = 9'-5"
(4) #5 @ 16" Length = 28'-4"
(10) #5 @ 16" Length = 9'-5"
(10) #5 @ 16" Length = 5'-10"
(6) #5 @ 16" Length = 28'-0"
(5) #5 @ 16" Length = 9'-5"

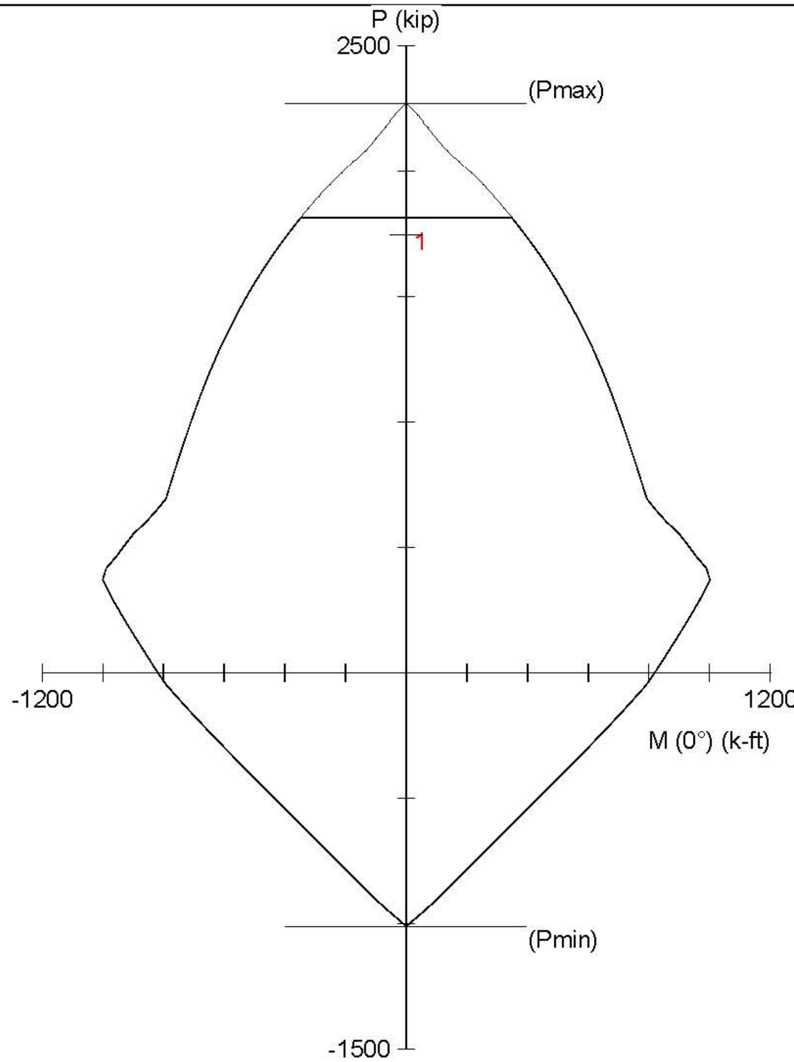

Interior Slab Reinforcement
Middle Strip Flexural Reinforcement
(14) #5 @ 12"
Length = 17'-2"
(14) #5 @ 12"
Length = 17'-2"
(10) #5 @ 16"
Length = 28'-4"
(14) #5 @ 12"
Length = 7'-5"
(1) #5
Length = 19'-10"
(14) #5 @ 12"
Length = 7'-5"
(14) #5 @ 12"
Length = 17'-2"
(14) #5 @ 12"
Length = 17'-2"
Column Strip Flexural Reinforcement
(6) #5 @ 16"
Length = 17'-2"
(6) #5 @ 16"
Length = 17'-2"
(17) #5 @ 10"
Length = 28'-4"
(28) #5 @ 6"
Length = 9'-6"
(28) #5 @ 6"
Length = 9'-6"
(6) #5 @ 16"
Length = 17'-2"
(6) #5 @ 16"
Length = 17'-2"


Interior Slab Reinforcement Long Reinforcement
Middle Strip Flexural Reinforcement
(8) #5 @ 16" Length = 9'-5"
(3) #5 @ 14" Length = 23'-9"
(8) #5 @ 16" Length = 28'-0"
(14) #5 @ 16" Length = 5'-10"
(10) #5 @ 16" Length = 9'-9"
(8) #5 @ 16" Length = 28'-4"
(9) #5 @ 16" Length = 9'-9"
(9) #5 @ 16" Length = 5'-10"
(9) #5 @ 16" Length = 28'-0"
(2) #5 @ 16" Length = 23'-9"
(9) #5 @ 16" Length = 9'-5"
Column Strip Flexural Reinforcement
(10) #5 @ 16" Length = 9'-5"
(17) #5 @ 10" Length = 28'-0"
(29) #5 @ 5.5" Length = 5'-10"
(29) #5 @ 5.5" Length = 9'-0"
(10) #5 @ 16" Length = 28'-4"
(29) #5 @ 5.5" Length = 9'-0"
(29) #5 @ 5.5" Length = 5'-10"
(16) #5 @ 10.5" Length = 28'-0"
(9) #5 @ 16" Length = 9'-4"

Appendix B: Gravity Column Design & Foundations



24 x 24 in



Code: ACI 318-02

Units: English

Run axis: Biaxial

Run option: Design

Slenderness: Not considered

Column type: Structural

Bars: ASTM A615

Date: 03/29/09

Time: 18:41:58

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

File: Y:\~One Last Final Last Time\PCA Column\Interior Column.col

Project: Washington Park Condominiums

Column:		Engineer:	
f'c = 5 ksi	fy = 60 ksi	Ag = 576 in^2	12 #11 bars
Ec = 4031 ksi	Es = 29000 ksi	As = 18.72 in^2	Rho = 3.25%
fc = 4.25 ksi	fc = 4.25 ksi	Xo = 0.00 in	Ix = 27648 in^4
e_u = 0.003 in/in		Yo = 0.00 in	Iy = 27648 in^4
Beta1 = 0.8		Clear spacing = 5.12 in	Clear cover = 1.50 in
Confinement: Tied	phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65		

General Footing Design

File: C:\Users\Benjamin\Documents\ENERCALC Data Files\washingtonpark.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21
Lic. # : Evaluation Version License Owner : Evaluation Version
Description : Interior Column Spread Footing

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	.00140	
Min Allow % Temp Reinf.	=	.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Overturning Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	Yes	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	9.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

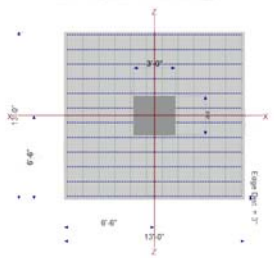
Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Dimensions

Width along X-X Axis	=	13.0	ft
Length along Z-Z Axi	=	13.0	ft
Footing Thicknes	=	36.0	in
Load location offset from footing center...			
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in
Pedestal dimensions...			
px : Along X-X Axis	=	36.0	in
pz : Along Z-Z Axis	=	36.0	in
Height	=	36.0	in
Rebar Centerline to Edge of Concrete.. at Bottom of footing	=	3.0	in



Reinforcing

Bars along X-X Axis		
Number of Bars	=	12.0
Reinforcing Bar Size	=	# 9
Bars along Z-Z Axis		
Number of Bars	=	12.0
Reinforcing Bar Size	=	# 9

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation		n/a
# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	1,080.0	140.0	282.450	14.4360				k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing Design

File: C:\Users\Benjamin\Documents\ENERCALC Data Files\washingtonpark.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21
Lic. # : Evaluation Version License Owner : Evaluation Version
Description : Corner Column Spread Footing (C55)

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	.00140	
Min Allow % Temp Reinf.	=	.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Overturning Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	Yes	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	9.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

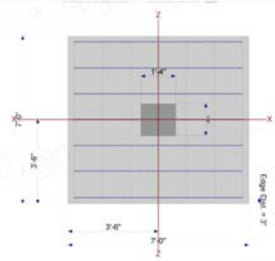
Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Dimensions

Width along X-X Axis	=	7.0	ft
Length along Z-Z Axi	=	7.0	ft
Footing Thicknes	=	18.0	in
Load location offset from footing center...			
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in
Pedestal dimensions...			
px : Along X-X Axis	=	16.0	in
pz : Along Z-Z Axis	=	16.0	in
Height	=	36.0	in
Rebar Centerline to Edge of Concrete.. at Bottom of footing	=	3.0	in



Reinforcing

Bars along X-X Axis		
Number of Bars	=	7.0
Reinforcing Bar Size	=	# 7
Bars along Z-Z Axis		
Number of Bars	=	7.0
Reinforcing Bar Size	=	# 7

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation		n/a
# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	245.0	35.0	53.50	2.740	42.520	9.320		k
OB : Overburden	=								ksf
M-xx	=					37.060	11.70		k-ft
M-zz	=					5.350	7.670		k-ft
V-x	=					0.580	0.50		k
V-z	=					4.440	0.870		k

General Footing Design

File: C:\Users\Benjamin\Documents\ENERCALC Data Files\washingtonpark.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21
Lic. # : Evaluation Version License Owner : Evaluation Version
Description : Exterior Column Spread Footing (C65)

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	.00140	
Min Allow % Temp Reinf.	=	.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Overturning Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	Yes	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	9.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

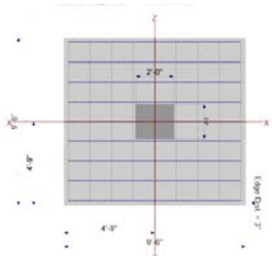
Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Dimensions

Width along X-X Axis	=	9.50	ft
Length along Z-Z Axi	=	9.50	ft
Footing Thicknes	=	28.0	in
Load location offset from footing center...			
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in
Pedestal dimensions...			
px : Along X-X Axis	=	24.0	in
pz : Along Z-Z Axis	=	24.0	in
Height	=	36.0	in
Rebar Centerline to Edge of Concrete.. at Bottom of footing	=	3.0	in



Reinforcing

Bars along X-X Axis		
Number of Bars	=	9.0
Reinforcing Bar Size	=	# 8
Bars along Z-Z Axis		
Number of Bars	=	9.0
Reinforcing Bar Size	=	# 8

Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation		n/a
# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	570.0	75.0	142.0	7.40	20.20	20.280		k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

General Footing Design

File: C:\Users\Benjamin\Documents\ENERCALC Data Files\washingtonpark.ec6
ENERCALC, INC. 1983-2008, Ver: 6.0.21
Lic. # : Evaluation Version License Owner : Evaluation Version
Description : Exterior Column Spread Footing (C80)

General Information

Calculations per IBC 2006, CBC 2007, ACI 318-05

Material Properties

f'c : Concrete 28 day strength	=	4.0	ksi
Fy : Rebar Yield	=	60.0	ksi
Ec : Concrete Elastic Modulus	=	3,122.0	ksi
Concrete Density	=	145.0	pcf
φ Values Flexure	=	0.90	
Shear	=	0.850	

Analysis Settings

Min Steel % Bending Reinf.	=	.00140	
Min Allow % Temp Reinf.	=	.00180	
Min. Overturning Safety Factor	=	1.50	: 1
Min. Overturning Safety Factor	=	1.50	: 1
AutoCalc Footing Weight as DL	:	Yes	
AutoCalc Pedestal Weight as DL	:	No	

Soil Design Values

Allowable Soil Bearing	=	9.0	ksf
Increase Bearing By Footing Weight	=	No	
Soil Passive Resistance (for Sliding)	=	250.0	pcf
Soil/Concrete Friction Coeff.	=	0.30	

Increases based on footing Depth

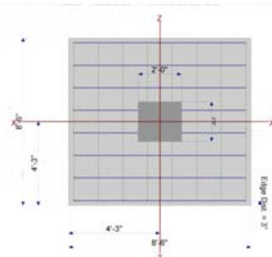
Reference Depth below Surface	=		ft
Allow. Pressure Increase per foot of depth	=		ksf
when base footing is below	=		ft

Increases based on footing Width

Allow. Pressure Increase per foot of width	=		ksf
when footing is wider than	=		ft

Dimensions

Width along X-X Axis	=	8.50	ft
Length along Z-Z Axi	=	8.50	ft
Footing Thicknes	=	24.0	in
Load location offset from footing center...			
ex : Along X-X Axis	=	0	in
ez : Along Z-Z Axis	=	0	in
Pedestal dimensions...			
px : Along X-X Axis	=	24.0	in
pz : Along Z-Z Axis	=	24.0	in
Height	=	12.0	in
Rebar Centerline to Edge of Concrete.. at Bottom of footing	=	3.0	in



Reinforcing

Bars along X-X Axis		
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 8
Bars along Z-Z Axis		
Number of Bars	=	8.0
Reinforcing Bar Size	=	# 8

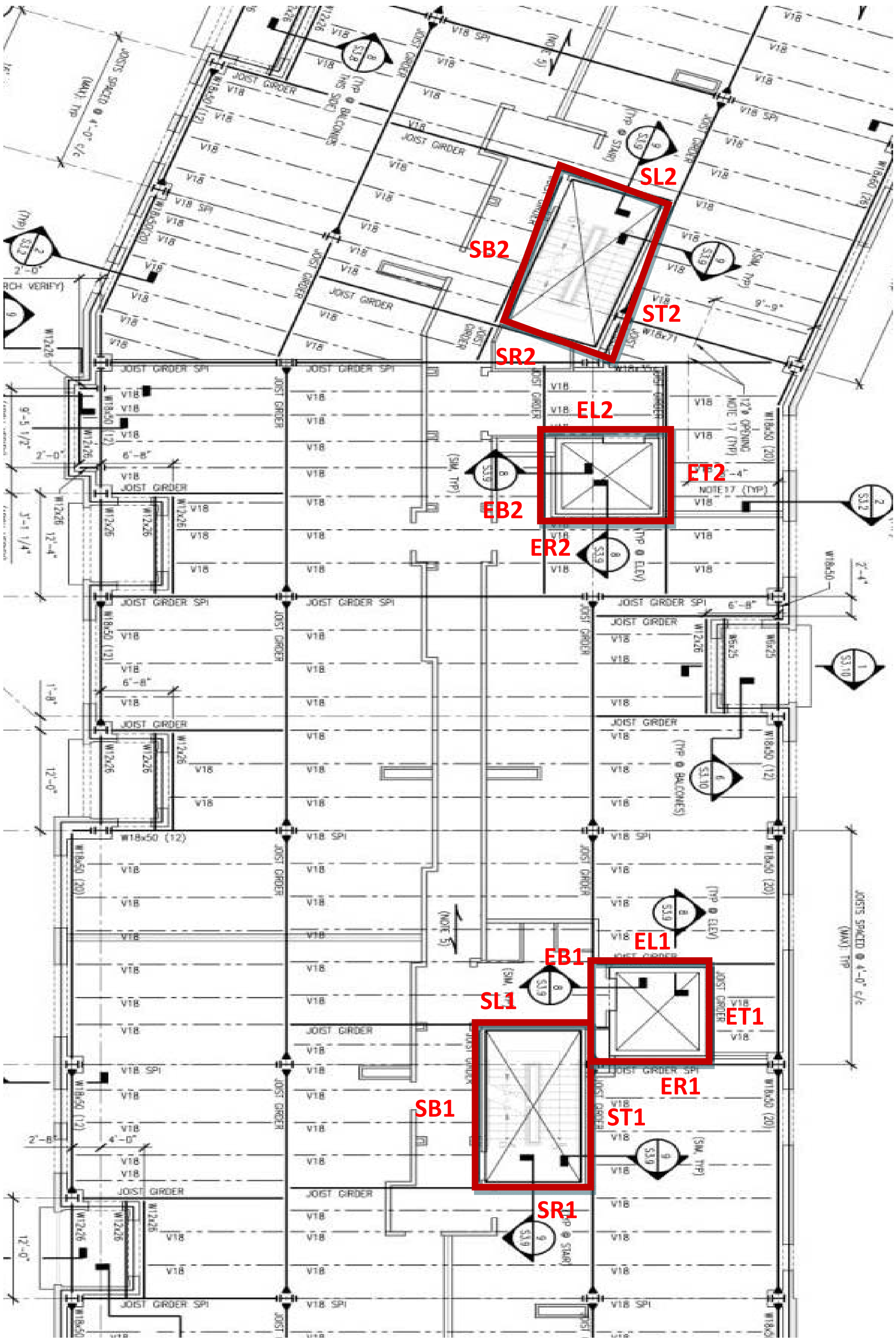
Bandwidth Distribution Check (ACI 15.4.4.2)

Direction Requiring Closer Separation		n/a
# Bars required within zone	=	n/a
# Bars required on each side of zone	=	n/a

Applied Loads

		D	Lr	L	S	W	E	H	
P : Column Load	=	440.0	60.0	110.0	5.50	12.020	14.60		k
OB : Overburden	=								ksf
M-xx	=								k-ft
M-zz	=								k-ft
V-x	=								k
V-z	=								k

Appendix C: Reinforced Concrete Shear Wall Design


SL2
SB2
ST2
SR2
EL2
ET2
EB2
ER2
EL1
EB1
SL1
ET1
ER1
SB1
ST1
SR1

Seismic

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
SR1 (Max P)	SEISMICXY	-339.55	-2.08	1.6	-154.116	-12.843	-51.14	2364.212	197.018
SR1 (Max V2)	SEISMICYX	18.04	45.34	-0.65	-225.241	-18.770	-17.943	9443.836	786.986
SR1 (Max M3)	SEISMICYX	18.04	45.34	-0.65	-225.241	-18.770	-17.943	9443.836	786.986
SL1 (Max P)	SEISMICXYY	371.44	32.28	0.21	37.681	3.140	-42.439	2844.864	237.072
SL1 (Max V2)	SEISMICYX	59.19	75.07	0.19	264.901	22.075	-4.456	9084.273	757.023
SL1(Max M3)	SEISMICYX	59.19	75.07	0.19	264.901	22.075	-4.456	9084.273	757.023
SB1 (Max P)	SEISMICYX	576.64	-25.37	-4.54	134.114	11.176	48.999	1690.543	140.879
SB1 (Max V2)	SEISMICXYY	144.02	49.2	-0.72	1339.123	111.594	203.426	16214.961	1351.247
SB1(Max M3)	SEISMICXYY	159.99	44.42	-0.27	-337.621	-28.135	20.712	20808.273	1734.023
ST1 (Max P)	SEISMICYX	-653.87	28.59	-9.03	-72.904	-6.075	50.491	3268.891	272.408
ST1 (Max V2)	SEISMICXY	-67.7	117.69	0.48	-965.801	-80.483	48.758	14420.651	1201.721
ST1(Max M3)	SEISMICXYY	-216.9	98.71	-3.58	456.107	38.009	18.692	30459.802	2538.317

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
ER1 (Max P)	SEISMICXY	-215.4	7.17	2.69	-96.846	-8.071	-28.592	1818.786	151.566
ER1 (Max V2)	SEISMICYX	2.05	63.14	0.19	-259.607	-21.634	-5.197	8976.801	748.067
ER1 (Max M3)	SEISMICYX	2.05	63.14	0.19	-259.607	-21.634	-5.197	8976.801	748.067
EL1 (Max P)	SEISMICXY	234.04	19.85	2.72	-13.398	-1.117	-46.475	1337.336	111.445
EL1 (Max V2)	SEISMICYX	74.46	40.25	1.89	193.955	16.163	10.347	8893.151	741.096
EL1(Max M3)	SEISMICYX	74.46	40.25	1.89	193.955	16.163	10.347	8893.151	741.096
EB1 (Max P)	SEISMICY	281.68	12.86	-2.61	-112.166	-9.347	36.039	1237.315	103.110
EB1 (Max V2)	SEISMICXYY	100.31	21.14	-1.28	-148.705	-12.392	13.041	3519.549	293.296
EB1(Max M3)	SEISMICXY	49.05	17.18	-1.05	-168.69	-14.058	6.809	3796.498	316.375
ET1 (Max P)	SEISMICYX	-414.81	-3.86	-6.99	102.986	8.582	40.663	971.678	80.973
ET1 (Max V2)	SEISMICXY	-67.69	44.62	-0.99	188.964	15.747	9.423	7727.589	643.966
ET1(Max M3)	SEISMICXY	-67.69	44.62	-0.99	188.964	15.747	9.423	7727.589	643.966

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
ER2 (Max P)	SEISMICYXX	-201.78	-3.89	2.95	40.727	3.394	-26.736	-771.408	-64.284
ER2 (Max V2)	SEISMICY	-34.06	68.26	0.59	-227.941	-18.995	-3.207	8783.707	731.976
ER2 (Max M3)	SEISMICY	-34.06	68.26	0.59	-227.941	-18.995	-3.207	8783.707	731.976
EL2 (Max P)	SEISMICXYY	176.51	-13.03	1.39	-20.225	-1.685	-25.443	-379.311	-31.609
EL2 (Max V2)	SEISMICY	35.92	18.43	-0.46	237.647	19.804	-4.973	4692.472	391.039
EL2(Max M3)	SEISMICY	35.92	18.43	-0.46	237.647	19.804	-4.973	4692.472	391.039
EB2 (Max P)	SEISMICY	352.3	17.83	-6.12	81.942	6.829	46.645	442.158	36.847
EB2 (Max V2)	SEISMICXYY	-11	37.87	0.61	-143.531	-11.961	-7.401	5179.077	431.590
EB2(Max M3)	SEISMICXYY	-11	37.87	0.61	-143.531	-11.961	-7.401	5179.077	431.590
ET2 (Max P)	SEISMICY	-354.17	-17.32	-6.03	85.993	7.166	64.236	975.804	81.317
ET2 (Max V2)	SEISMICYX	-217.12	-22.56	1.77	119.972	9.998	234.993	13.388	1.116
ET2(Max M3)	SEISMICXY	12.59	22.35	0.81	114.25	9.521	3.023	4869.599	405.800

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
SR2 (Max P)	SEISMICYXX	-381.85	-11.67	0.08	-59.82	-4.985	-45.568	48.112	4.009
SR2 (Max V2)	SEISMICY	190.75	82.3	-0.1	-229.747	-19.146	20.446	8985.545	748.795
SR2 (Max M3)	SEISMICY	190.75	82.3	-0.1	-229.747	-19.146	20.446	8985.545	748.795
SL2 (Max P)	SEISMICXYY	342.88	1.58	2.38	-107.826	-8.986	-53.481	-1333.371	-111.114
SL2 (Max V2)	SEISMICY	-111.99	43.53	0.01	263.148	21.929	35.504	9521.877	793.490
SL2(Max M3)	SEISMICY	-111.99	43.53	0.01	263.148	21.929	35.504	9521.877	793.490
SB2 (Max P)	SEISMICY	566.88	7.15	-3.92	30.375	2.531	50.525	-9455.641	-787.970
SB2 (Max V2)	SEISMICXYY	21.77	62.71	0.36	1370.778	114.232	-25.687	16639.196	1386.600
SB2(Max M3)	SEISMICXYY	34.44	60.66	-1.35	-409.844	-34.154	-6.166	22283.546	1856.962
ST2 (Max P)	SEISMICY	-645.64	-70.67	-8.81	-103.792	-8.649	51.625	-14426.134	-1202.178
ST2 (Max V2)	SEISMICXYY	-17.67	109.19	-0.01	-1079.338	-89.945	20.401	18927.567	1577.297
ST2(Max M3)	SEISMICXYY	4.54	105.65	0.71	505.164	42.097	-3.403	32186.766	2682.231

Wind									
	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
SR1 (Max P)	DCON22	225.39	98.16	-0.25	-120.687	-10.057	-4.19	14890.736	1240.895
SR1 (Max V2)	DCON11	123.94	116.03	-0.77	-235.382	-19.615	-28.314	19310.616	1609.218
SR1 (Max M3)	DCON11	123.94	116.03	-0.77	-235.382	-19.615	-28.314	19310.616	1609.218
SL1 (Max P)	DCON17	238.17	93.89	-1.42	156.283	13.024	-27.058	10702.278	891.857
SL1 (Max V2)	DCON11	83.88	134.71	0.11	320.575	26.715	-6.576	16379.607	1364.967
SL1(Max M3)	DCON11	83.88	134.71	0.11	320.575	26.715	-6.576	16379.607	1364.967
SB1 (Max P)	DCON11	908.89	-25.28	1.02	178.942	14.912	97.327	2308.359	192.363
SB1 (Max V2)	DCON13	177.82	-81.56	0.5	-613.175	-51.098	91.345	1831.518	152.627
SB1(Max M3)	DCON3	30.64	36.1	0.52	-215.701	-17.975	5.684	13626.37	1135.531
ST1 (Max P)	DCON12	1116.71	-22.78	3.39	105.204	8.767	-102.905	-4359.221	-363.268
ST1 (Max V2)	DCON18	372.38	-126.92	-3.7	713.341	59.445	-121.967	-7251.846	-604.321
ST1(Max M3)	DCON3	-53.99	74.88	-0.4	213.458	17.788	3.824	19179.699	1598.308

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
ER1 (Max P)	DCON26	196.09	12	0.86	198.889	16.574	29.825	-2924.067	-243.672
ER1 (Max V2)	DCON11	47.51	134.92	-0.23	-252.975	-21.081	-3.728	16499.134	1374.928
ER1 (Max M3)	DCON11	47.51	134.92	-0.23	-252.975	-21.081	-3.728	16499.134	1374.928
EL1 (Max P)	DCON17	269.7	69.45	1.38	72.966	6.081	-19.986	11521.463	960.122
EL1 (Max V2)	DCON5	206.5	80.06	1.79	154.854	12.905	6.638	14899.96	1241.663
EL1(Max M3)	DCON11	81.4	79.89	2.98	289.333	24.111	28.705	16311.792	1359.316
EB1 (Max P)	DCON11	604.5	33.19	1.71	3.901	0.325	82.061	948.66	79.055
EB1 (Max V2)	DCON59	428.83	38.4	0.78	247.125	20.594	169.56	397.374	33.115
EB1(Max M3)	DCON25	81.4	6.77	-0.62	-215.461	-17.955	9.46	4156.354	346.363
ET1 (Max P)	DCON12	733.41	25.47	2.08	-207.526	-17.294	-78.558	-138.927	-11.577
ET1 (Max V2)	DCON74	151.6	-66.61	1.3	-60.198	-5.017	-10.413	-7166.857	-597.238
ET1(Max M3)	DCON74	151.6	-66.61	1.3	-60.198	-5.017	-10.413	-7166.857	-597.238

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
ER2 (Max P)	DCON18	147.18	-108.12	-0.46	215.701	17.975	18.675	-13588.956	-1132.413
ER2 (Max V2)	DCON5	-58.35	141.24	0.59	-298.35	-24.863	-4.49	17910.506	1492.542
ER2 (Max M3)	DCON5	-58.35	141.24	0.59	-298.35	-24.863	-4.49	17910.506	1492.542
EL2 (Max P)	DCON17	118.66	10.55	-2.15	593.977	49.498	-69.353	4478.591	373.216
EL2 (Max V2)	DCON13	-9.33	53.46	-0.92	286.033	23.836	-7.837	7945.794	662.150
EL2(Max M3)	DCON5	30.19	42.85	-1.1	355.635	29.636	-9.778	8756.796	729.733
EB2 (Max P)	DCON5	692.49	27.98	-2.08	190.049	15.837	98.334	355.225	29.602
EB2 (Max V2)	DCON17	292.51	49	1.87	336.019	28.002	114.352	1038.699	86.558
EB2(Max M3)	DCON3	17.41	27.35	-0.22	-55.3	-4.608	-0.608	3508.527	292.377
ET2 (Max P)	DCON6	664.33	30.36	3.27	-267.122	-22.260	-125.391	-2849.345	-237.445
ET2 (Max V2)	DCON5	-509.11	-40.47	0.91	173.139	14.428	363.865	-38.908	-3.242
ET2(Max M3)	DCON17	-492.62	-8.15	-2.16	241.834	20.153	97.103	4844.513	403.709

	Load Case	Axial (P)	Shear (V2)	Shear (V3)	Torsion (T)	Torsion (T)	Moment (M2)	Moment (M3)	Moment (M3)
SR2 (Max P)	DCON20	470.46	146.44	2.83	-238.606	-19.884	62.256	15768.584	1314.049
SR2 (Max V2)	DCON13	440.82	190.09	2.46	-338.927	-28.244	57.654	20921.703	1743.475
SR2 (Max M3)	DCON13	440.82	190.09	2.46	-338.927	-28.244	57.654	20921.703	1743.475
SL2 (Max P)	DCON15	269.95	-72.41	-1.47	-351.921	-29.327	-73.499	-15275.676	-1272.973
SL2 (Max V2)	DCON61	-109.89	159.6	0.82	411.263	34.272	85.731	26739.088	2228.257
SL2(Max M3)	DCON61	-109.89	159.6	0.82	411.263	34.272	85.731	26739.088	2228.257
SB2 (Max P)	DCON13	1078.89	-44.82	2.71	5.955	0.496	135.276	-23324.788	-1943.732
SB2 (Max V2)	DCON20	633.11	-72.23	5.38	-1170.244	-97.520	327.926	-18686.789	-1557.232
SB2(Max M3)	DCON68	793.77	-61.35	2.44	112.489	9.374	104.515	-25840.677	-2153.390
ST2 (Max P)	DCON14	1409.83	147.11	4.28	180.263	15.022	-140.453	34039.847	2836.654
ST2 (Max V2)	DCON15	515.75	171.31	-5.5	-976.881	-81.407	-177.286	17604.327	1467.027
ST2(Max M3)	DCON19	1064.65	153.98	3.4	273.862	22.822	-107.185	37310.669	3109.222

SR1					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	116.03	V_u	116.03
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-63.87	V_s	-63.87
V_u	116.03	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

SR2					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	190.09	V_u	190.09
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	34.88	V_s	34.88
V_u	190.09	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

SL1					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	134.71	V_u	134.71
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-38.96	V_s	-38.96
V_u	134.71	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

SL2					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	159.6	V_u	159.6
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-5.78	V_s	-5.78
V_u	159.6	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

SB1					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	81.56	V_u	81.56
h	96.33	V_c	427.14	V_c	427.14
l_u	19.542	V_s	-318.40	V_s	-318.40
V_u	81.56	$1/2\phi V_c$	160.18	$1/2\phi V_c$	160.18
d	187.6032	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

SB2					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	72.23	V_u	72.23
h	96.33	V_c	427.14	V_c	427.14
l_u	19.542	V_s	-330.84	V_s	-330.84
V_u	72.23	$1/2\phi V_c$	160.18	$1/2\phi V_c$	160.18
d	187.6032	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

ST1					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	126.92	V_u	126.92
h	96.33	V_c	427.14	V_c	427.14
l_u	19.542	V_s	-257.92	V_s	-257.92
V_u	126.92	$1/2\phi V_c$	160.18	$1/2\phi V_c$	160.18
d	187.6032	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

ST2					
Design Conditions		**Horizontal Shear Reinforcement**		**Vertical Shear Reinforcement**	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	171.31	V_u	171.31
h	96.33	V_c	427.14	V_c	427.14
l_u	19.542	V_s	-198.73	V_s	-198.73
V_u	171.31	$1/2\phi V_c$	160.18	$1/2\phi V_c$	160.18
d	187.6032	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

ER1					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	134.92	V_u	134.92
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-38.68	V_s	-38.68
V_u	134.92	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

ER2					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	141.42	V_u	141.42
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-30.02	V_s	-30.02
V_u	141.42	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

EL1					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	80.06	V_u	80.06
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-111.83	V_s	-111.83
V_u	80.06	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

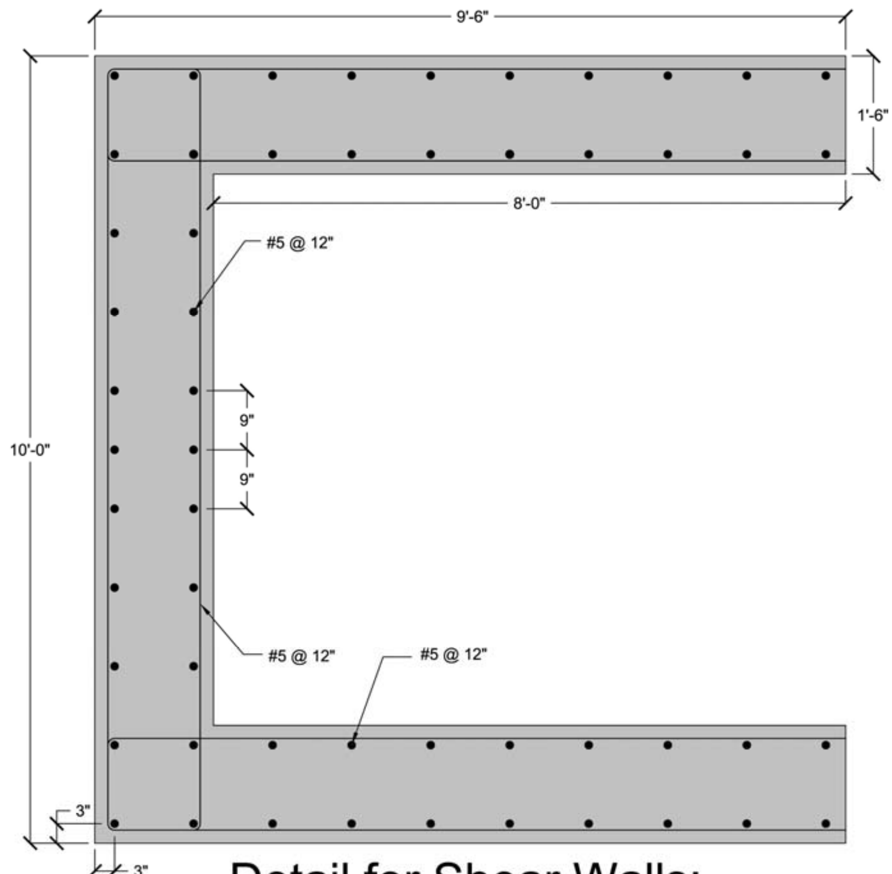
EL2					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	53.46	V_u	53.46
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-147.30	V_s	-147.30
V_u	53.46	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

EB1					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	38.4	V_u	38.4
h	96.33	V_c	262.29	V_c	262.29
l_u	12	V_s	-211.09	V_s	-211.09
V_u	38.4	$1/2\phi V_c$	98.36	$1/2\phi V_c$	98.36
d	115.2	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

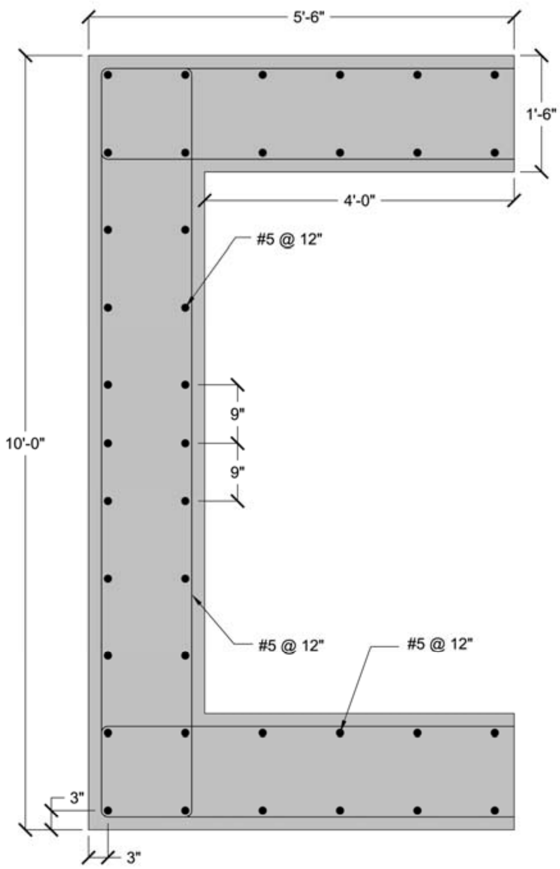
EB2					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	49	V_u	49
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-153.24	V_s	-153.24
V_u	49	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

ET1					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	66.61	V_u	66.61
h	96.33	V_c	262.29	V_c	262.29
l_u	12	V_s	-173.48	V_s	-173.48
V_u	66.61	$1/2\phi V_c$	98.36	$1/2\phi V_c$	98.36
d	115.2	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	

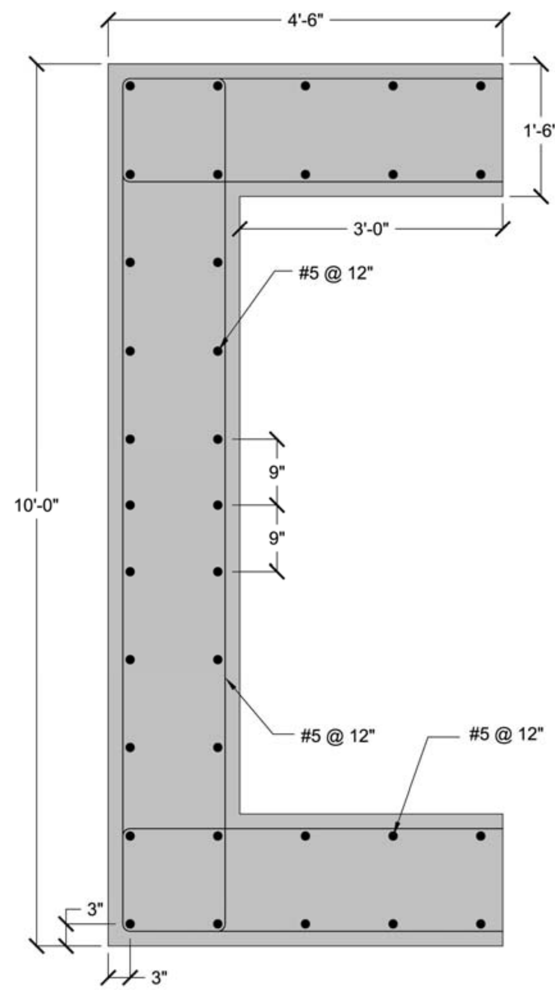
ET2					
Design Conditions		Horizontal Shear Reinforcement		Vertical Shear Reinforcement	
f'_c	4000	ρ_t	0.0025	ρ_t	0.0025
f_y	60000	spacing	18	spacing	18
t	18	V_u	40.47	V_u	40.47
h	96.33	V_c	218.58	V_c	218.58
l_u	10	V_s	-164.62	V_s	-164.62
V_u	40.47	$1/2\phi V_c$	81.97	$1/2\phi V_c$	81.97
d	96	A_v	0.180	A_v	0.180
		Use (2) #5 @ 18"		Use (2) #5 @ 18"	



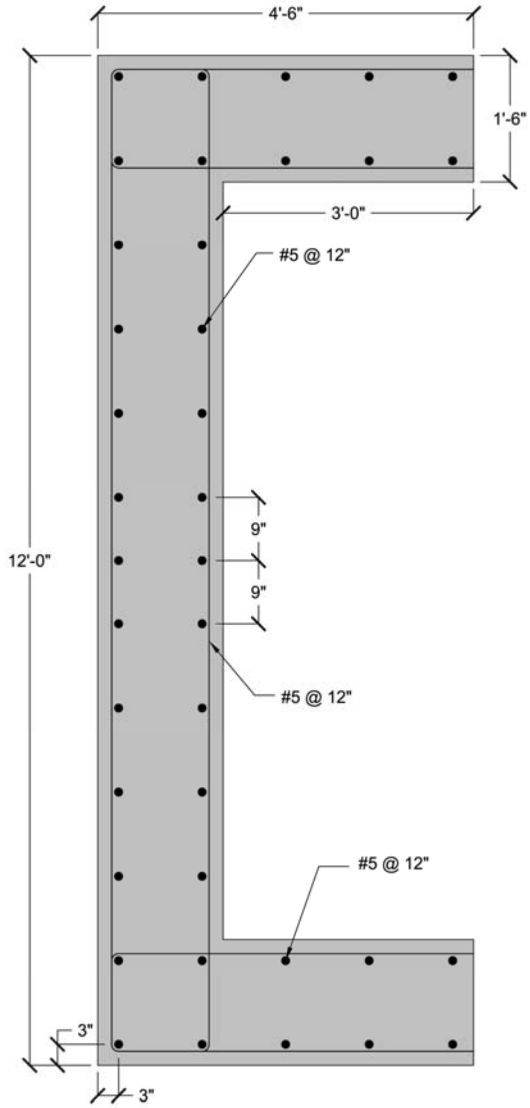
Detail for Shear Walls:
SL1, SR1, SL2, & SR2



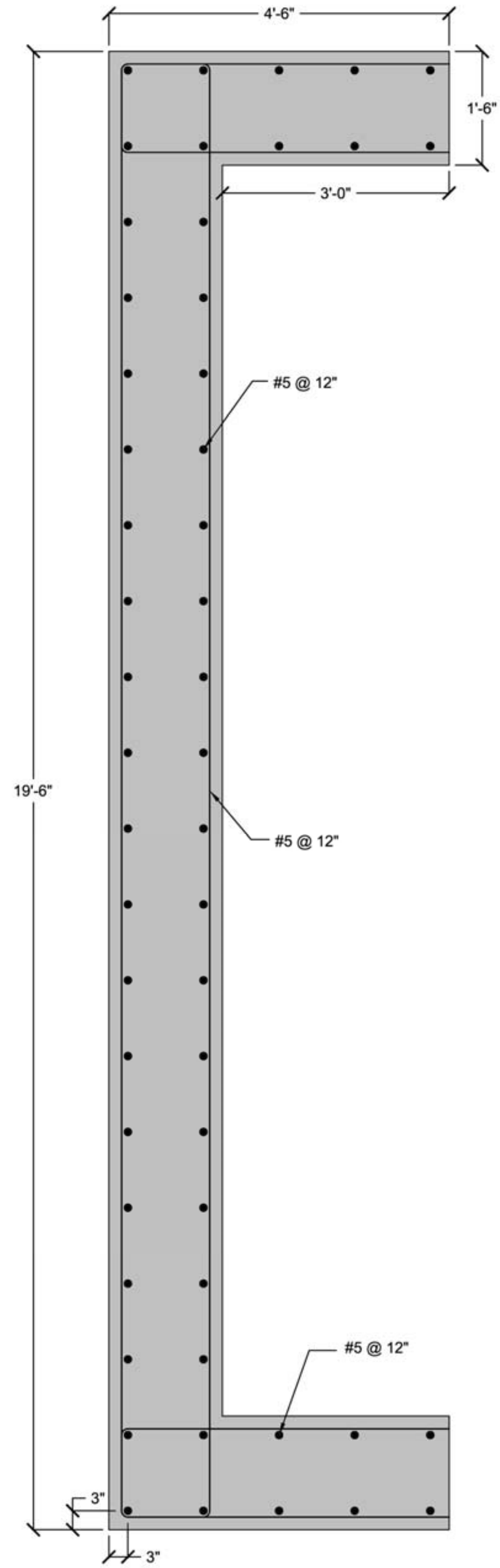
Detail for Shear Walls:
ER1 & EL1



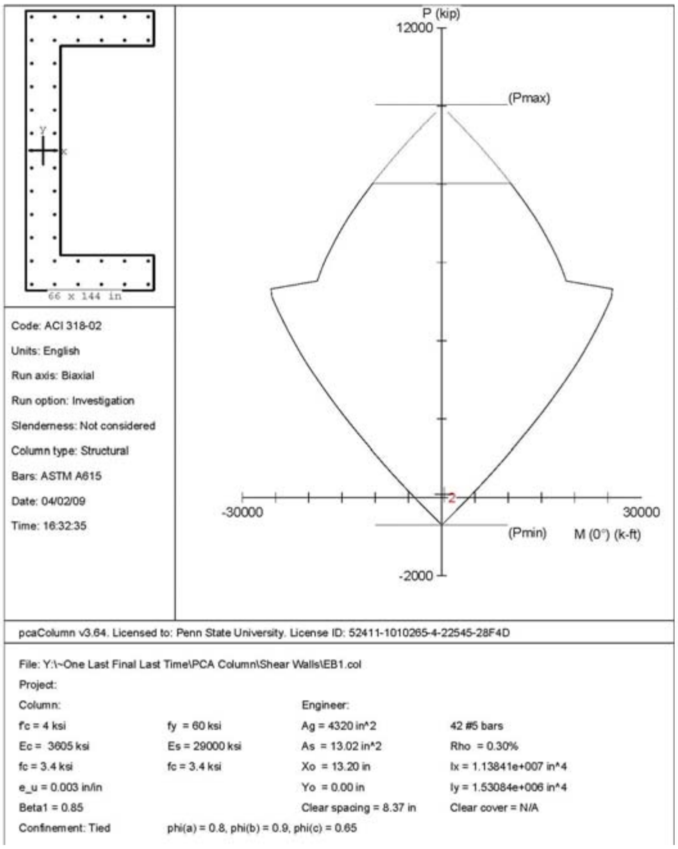
Detail for Shear Walls:
EB2, ER2, EL2, & ET2

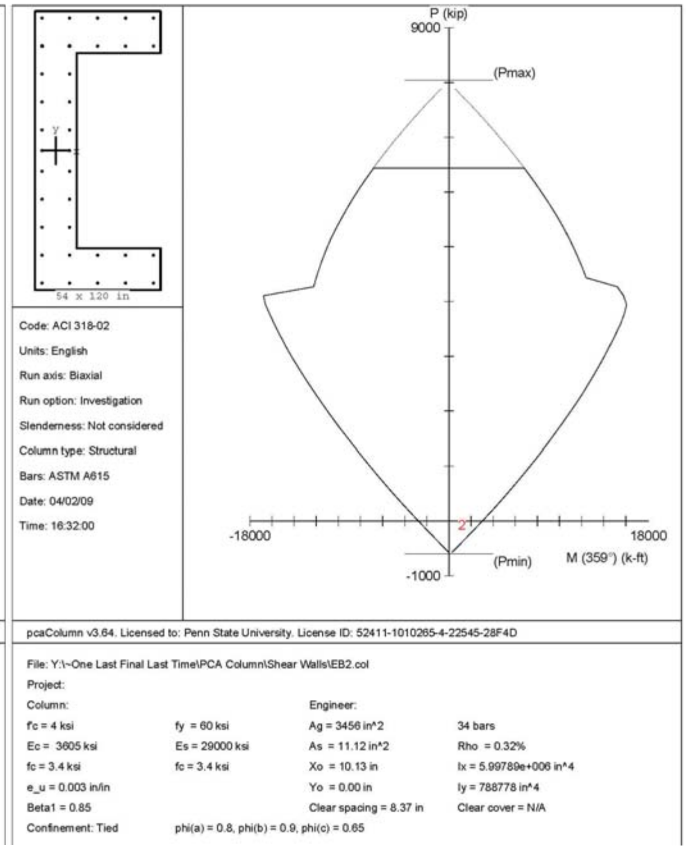


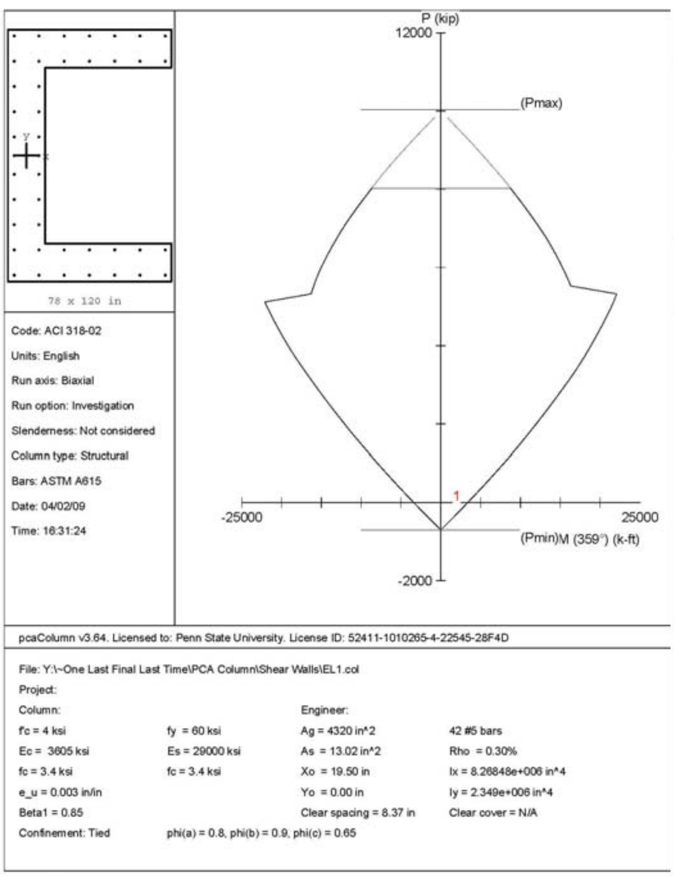
Detail for Shear Walls:
EB1 & ET1

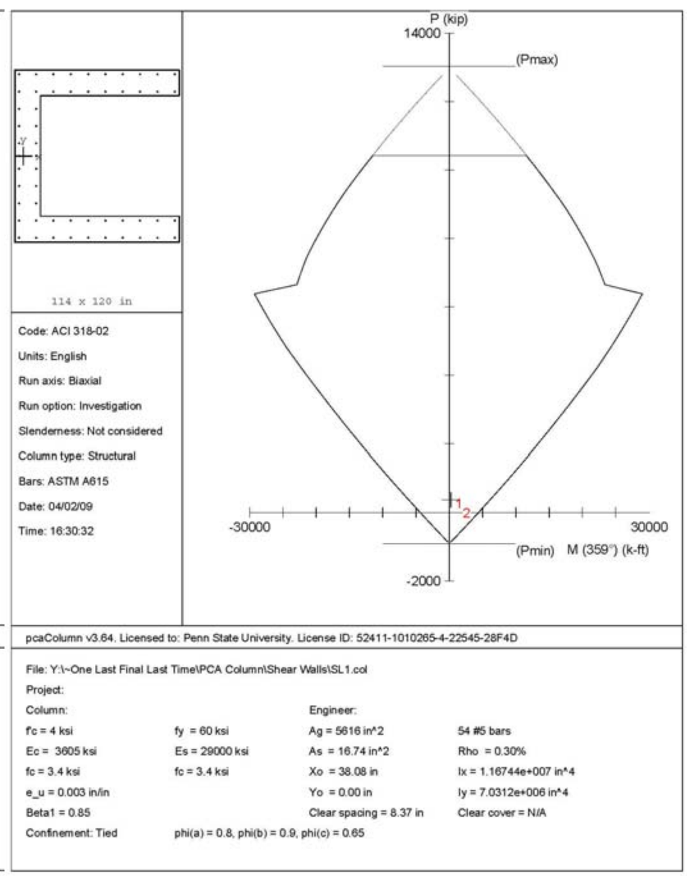


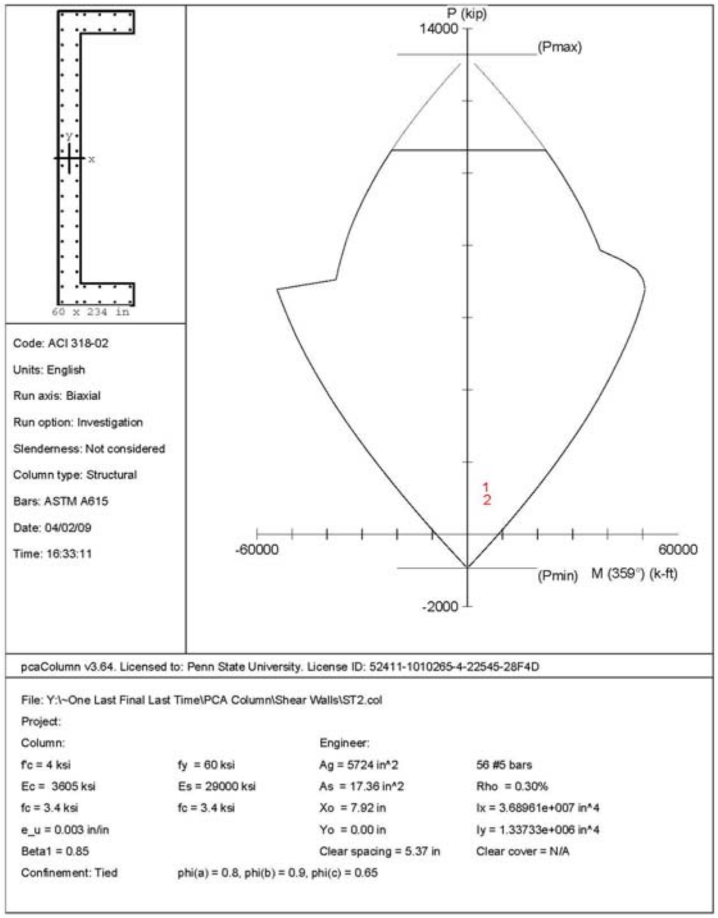
Detail for Shear Walls:
SB1, ST1, SB2 & ST2


66 x 144 in
P (kip)
12000
(Pmax)
-30000
30000
(Pmin)
M (0°) (k-ft)
-2000
Code: ACI 318-02
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 04/02/09
Time: 16:32:35
pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: Y:\~One Last Final Last Time\PCA Column\Shear Walls\EB1.col
Project:
Column:
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 4320 in^2
42 #5 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 13.02 in^2
Rho = 0.30%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 13.20 in
Ix = 1.13841e+007 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 1.53084e+006 in^4
Beta1 = 0.85
Clear spacing = 8.37 in
Clear cover = N/A
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


54 x 120 in
P (kip)
9000
(Pmax)
-18000
18000
(Pmin)
M (359°) (k-ft)
-1000
Code: ACI 318-02
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 04/02/09
Time: 16:32:00
pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: Y:\~One Last Final Last Time\PCA Column\Shear Walls\EB2.col
Project:
Column:
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 3456 in^2
34 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 11.12 in^2
Rho = 0.32%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 10.13 in
Ix = 5.99789e+006 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 788778 in^4
Beta1 = 0.85
Clear spacing = 8.37 in
Clear cover = N/A
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


78 x 120 in
P (kip)
12000
(Pmax)
-25000
25000
(Pmin)M (359°) (k-ft)
-2000
Code: ACI 318-02
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 04/02/09
Time: 16:31:24
pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: Y:\~One Last Final Last Time\PCA Column\Shear Walls\EL1.col
Project:
Column:
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 4320 in^2
42 #5 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 13.02 in^2
Rho = 0.30%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 19.50 in
Ix = 8.26848e+006 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 2.349e+006 in^4
Beta1 = 0.85
Clear spacing = 8.37 in
Clear cover = N/A
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


114 x 120 in
P (kip)
14000
(Pmax)
-30000
30000
(Pmin)
M (359°) (k-ft)
-2000
Code: ACI 318-02
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 04/02/09
Time: 16:30:32
pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: Y:\~One Last Final Last Time\PCA Column\Shear Walls\SL1.col
Project:
Column:
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 5616 in^2
54 #5 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 16.74 in^2
Rho = 0.30%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 38.08 in
Ix = 1.16744e+007 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 7.0312e+006 in^4
Beta1 = 0.85
Clear spacing = 8.37 in
Clear cover = N/A
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65


60 x 234 in
P (kip)
14000
(Pmax)
-60000
60000
(Pmin)
M (359°) (k-ft)
-2000
1
2
Code: ACI 318-02
Units: English
Run axis: Biaxial
Run option: Investigation
Slenderness: Not considered
Column type: Structural
Bars: ASTM A615
Date: 04/02/09
Time: 16:33:11
pcaColumn v3.64. Licensed to: Penn State University. License ID: 52411-1010265-4-22545-28F4D
File: Y:\~One Last Final Last Time\PCA Column\Shear Walls\ST2.col
Project:
Column:
Engineer:
f'c = 4 ksi
fy = 60 ksi
Ag = 5724 in^2
56 #5 bars
Ec = 3605 ksi
Es = 29000 ksi
As = 17.36 in^2
Rho = 0.30%
fc = 3.4 ksi
fc = 3.4 ksi
Xo = 7.92 in
Ix = 3.68961e+007 in^4
e_u = 0.003 in/in
Yo = 0.00 in
Iy = 1.33733e+006 in^4
Beta1 = 0.85
Clear spacing = 5.37 in
Clear cover = N/A
Confinement: Tied
phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

Appendix D: Reinforced Concrete Coupling Beams

Max Wind Loading

Beam 7 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B7	17.42	15.332	1.28	427.857	35.65
8TH	B7	36.62	29.197	2.43	880.943	73.41
7TH	B7	38.92	25.029	2.09	891.973	74.33
6TH	B7	43.56	27.373	2.28	987.327	82.28
5TH	B7	48.16	29.403	2.45	1093.734	91.14
4TH	B7	51.54	30.838	2.57	1171.626	97.64
3RD	B7	52.73	31.164	2.60	1197.096	99.76
2ND	B7	55.02	36.935	3.08	1302.006	108.50
1ST	B7	34.11	37.581	3.13	754.077	62.84

Beam 3 (Stair 1)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B3	13.84	14.6	1.22	350.214	29.18
8TH	B3	17.94	17.529	1.46	426.54	35.55
7TH	B3	25.26	20.065	1.67	577.218	48.10
6TH	B3	26.29	16.408	1.37	596.334	49.69
5TH	B3	28.39	17.294	1.44	645.447	53.79
4TH	B3	29.69	17.778	1.48	675.866	56.32
3RD	B3	29.63	17.589	1.47	674.034	56.17
2ND	B3	30.05	19.578	1.63	713.036	59.42
1ST	B3	17.98	22.762	1.90	398.137	33.18

Beam 6 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B6	9.88	14.189	1.18	221.635	18.47
8TH	B6	21.32	20.515	1.71	467.015	38.92
7TH	B6	23.12	18.848	1.57	481.977	40.16
6TH	B6	25.76	19.859	1.65	530.306	44.19
5TH	B6	28.43	21.238	1.77	585.923	48.83
4TH	B6	30.5	22.02	1.84	628.673	52.39
3RD	B6	31.72	21.938	1.83	652.691	54.39
2ND	B6	34.12	27.837	2.32	726.798	60.57
1ST	B6	24.24	18.973	1.58	490.55	40.88

Beam 1 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B1	9.29	14.429	1.20	271.292	22.61
8TH	B1	17.92	20.702	1.73	510.432	42.54
7TH	B1	16.37	17.192	1.43	440.335	36.69
6TH	B1	16.25	17.558	1.46	432.294	36.02
5TH	B1	15.95	17.813	1.48	425.019	35.42
4TH	B1	15.11	17.425	1.45	402.745	33.56
3RD	B1	13.49	16.345	1.36	358.989	29.92
2ND	B1	12.03	18.038	1.50	335.52	27.96
1ST	B1	5.79	9.032	0.75	147.824	12.32

Max Seismic Loading

Beam 7 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B7	8.1	8.843	0.737	199.78	16.648
8TH	B7	17.13	16.272	1.356	408.416	34.035
7TH	B7	16.22	13.443	1.120	357.852	29.821
6TH	B7	17.52	14.176	1.181	385.435	32.120
5TH	B7	18.65	14.581	1.215	410.713	34.226
4TH	B7	19.17	14.672	1.223	422.046	35.171
3RD	B7	18.78	14.082	1.174	414.241	34.520
2ND	B7	15.98	9.604	0.800	345.729	28.811
1ST	B7	15.96	32.673	2.723	386.997	32.250

Beam 3 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B3	8.93	9.636	0.803	216.517	18.043
8TH	B3	17.94	17.529	1.461	426.54	35.545
7TH	B3	16.38	14.456	1.205	360.657	30.055
6TH	B3	17.34	14.867	1.239	380.642	31.720
5TH	B3	18.15	15.234	1.270	398.658	33.222
4TH	B3	18.35	15.125	1.260	402.991	33.583
3RD	B3	17.63	14.248	1.187	387.947	32.329
2ND	B3	14.66	10.689	0.891	316.09	26.341
1ST	B3	13.97	20.051	1.671	338.212	28.184

Beam 6 (Stair 2)

Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B6	4.61	6.35	0.529	101.482	8.457
8TH	B6	9.89	12.079	1.007	214.281	17.857
7TH	B6	9.71	10.577	0.881	194.939	16.245
6TH	B6	10.45	10.826	0.902	208.99	17.416
5TH	B6	11.1	11.208	0.934	222.179	18.515
4TH	B6	11.38	11.127	0.927	227.52	18.960
3RD	B6	11.22	10.654	0.888	224.81	18.734
2ND	B6	9.84	8.505	0.709	192.466	16.039
1ST	B6	10.51	12.656	1.055	235.97	19.664

Beam 1 (Stair 2)

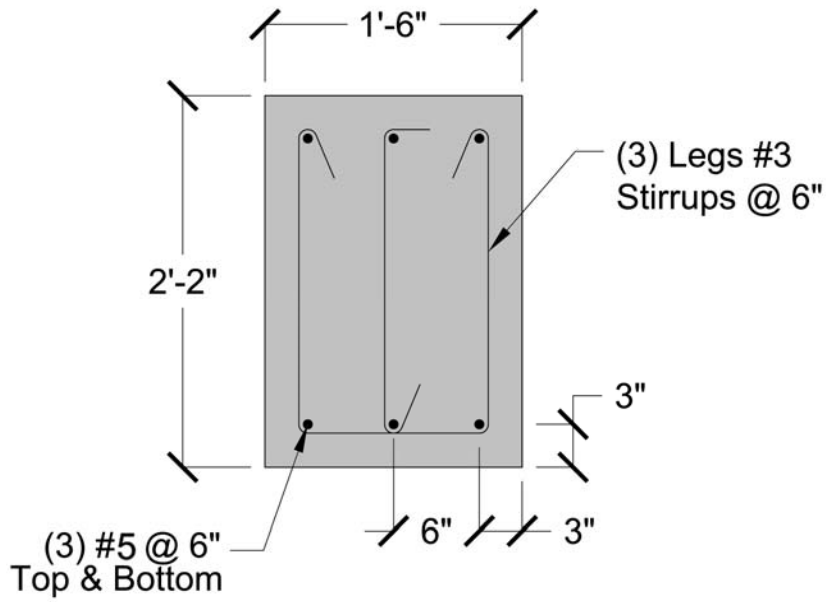
Story	Spandrel	V2	T	T (k-ft)	M3	M3 (k-ft)
ROOF	B1	4.43	7.225	0.602	122.52	10.210
8TH	B1	8.7	12.698	1.058	236.452	19.704
7TH	B1	7.32	10.367	0.864	183.989	15.332
6TH	B1	7.42	10.503	0.875	186.107	15.509
5TH	B1	7.46	10.561	0.880	187.151	15.596
4TH	B1	7.23	10.159	0.847	181.282	15.107
3RD	B1	6.64	9.353	0.779	166.557	13.880
2ND	B1	5.2	7.096	0.591	127.848	10.654
1ST	B1	4.97	6.535	0.545	139.007	11.584

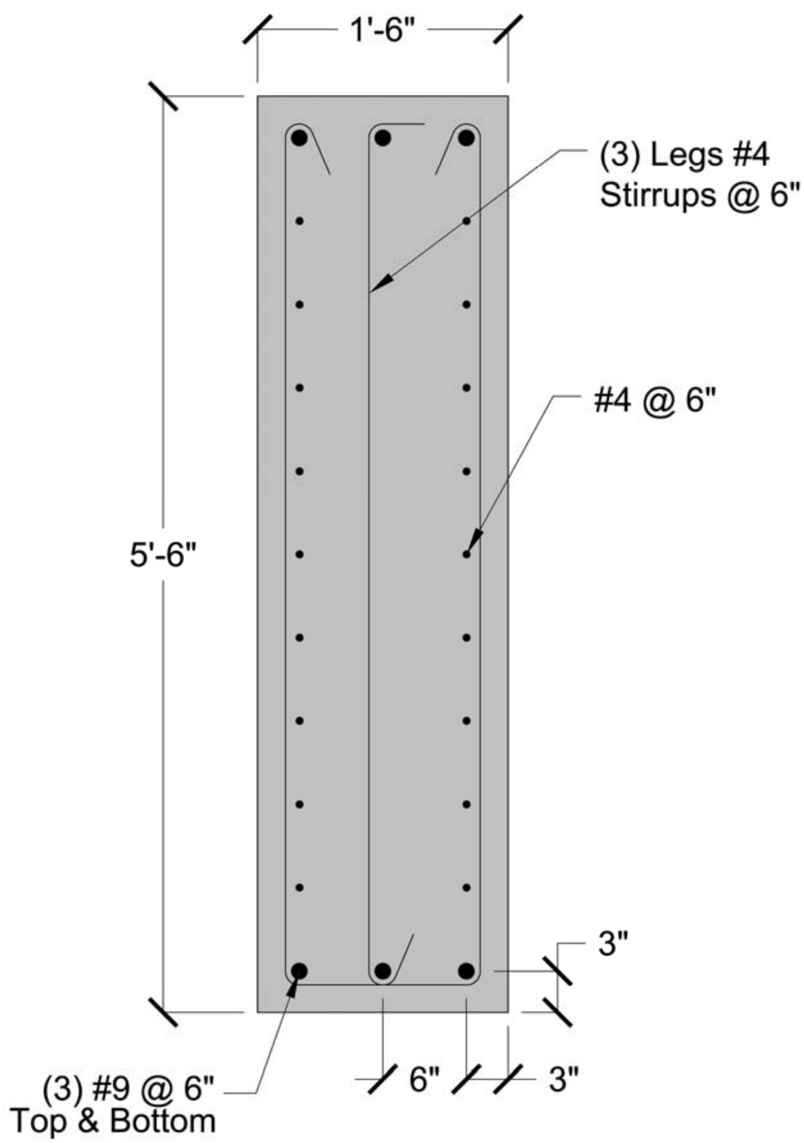
Coupling Beam 7 (Stair 2)											
Story	l_n (in)	h (in)	l_n/h	A_{cw}	V_n	d	As_{min}	Av_{min}	Flexural Reinf.	Shear Reinf.	Skin Reinf.
ROOF	40	58	0.69	1044	264.113	55	4.400	0.495	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6.5"
8TH	40	54	0.74	972	614.747	51	4.080	0.459	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6.5"
7TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
6TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
5TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
4TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
3RD	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
2ND	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
1ST	40	66	0.61	1188	751.357	63	5.040	0.567	(3) #9 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6.5"

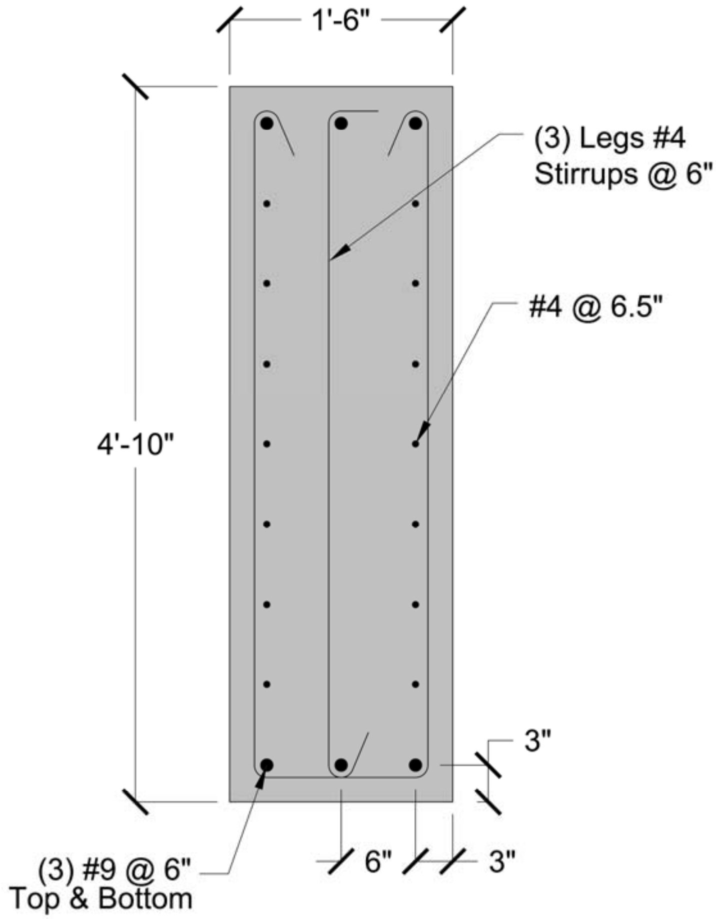
Coupling Beam 6 (Elevator 2)											
Story	l_n (in)	h (in)	l_n/h	A_{cw}	V_n	d	As_{min}	Av_{min}	Flexural Reinf.	Shear Reinf.	Skin Reinf.
ROOF	46	58	0.79	1044	660.284	55	4.400	0.495	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
8TH	46	54	0.85	972	614.747	51	4.080	0.459	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
7TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
6TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
5TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
4TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
3RD	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
2ND	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
1ST	46	66	0.70	1188	751.357	63	5.040	0.567	(3) #9 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"

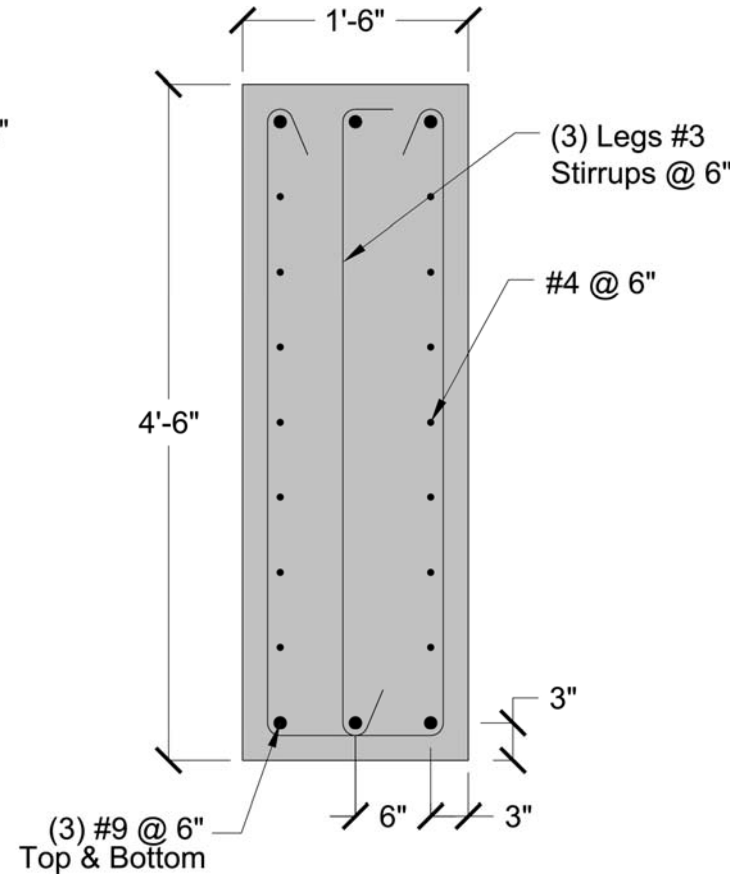
Coupling Beam 3 (Stair 1)											
Story	l_n (in)	h (in)	l_n/h	A_{cw}	V_n	d	As_{min}	Av_{min}	Flexural Reinf.	Shear Reinf.	Skin Reinf.
ROOF	40	58	0.69	1044	660.284	55	4.400	0.495	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
8TH	40	54	0.74	972	614.747	51	4.080	0.459	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
7TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
6TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
5TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
4TH	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
3RD	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
2ND	40	26	1.54	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
1ST	40	66	0.61	1188	751.357	63	5.040	0.567	(3) #9 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"

Coupling Beam 1 (Elevator 1)											
Story	l_n (in)	h (in)	l_n/h	A_{cw}	V_n	d	As_{min}	Av_{min}	Flexural Reinf.	Shear Reinf.	Skin Reinf.
ROOF	46	58	0.79	1044	660.284	55	4.400	0.495	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
8TH	46	54	0.85	972	614.747	51	4.080	0.459	(3) #8 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"
7TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
6TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
5TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
4TH	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
3RD	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
2ND	46	26	1.77	468	295.989	23	1.840	0.320	(3) #5 @ 6" T & B	(2) Legs of #4 @ 16"	None
1ST	46	66	0.70	1188	751.357	63	5.040	0.567	(3) #9 @ 6" T & B	(2) Legs of #5 @ 11"	#4 @ 6"


1'-6"
(3) Legs #3
Stirrups @ 6"
2'-2"
3"
6"
3"
(3) #5 @ 6"
Top & Bottom


1'-6"
(3) Legs #4
Stirrups @ 6"
#4 @ 6"
5'-6"
3"
6"
3"
(3) #9 @ 6"
Top & Bottom


1'-6"
(3) Legs #4
Stirrups @ 6"
#4 @ 6.5"
4'-10"
3"
(3) #9 @ 6"
Top & Bottom
6"
3"


1'-6"
(3) Legs #3
Stirrups @ 6"
#4 @ 6"
4'-6"
3"
(3) #9 @ 6"
Top & Bottom
6"
3"

Appendix E: Concrete Moment Frame – Columns and Beams

Column C55 (Column Line 9) for Seismic								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	SEISMICYX	-9.32	0.24	-0.65	0.00	8.39	-3.13
Max V2	Roof	SEISMICXY	1.27	0.49	0	0.42	0.45	7.67
Max T	6th	SEISMICXYY	2.16	0.38	-0.3	1.58	2.01	-0.40
Max M2	2nd	SEISMICYX	-8.91	-0.26	0.87	0.29	11.70	-3.94
Max M3	2nd	SEISMICXY	1.27	0.49	0	0.42	0.45	7.67

Column C55 (Column Line 9) for Wind								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	DCON14	42.52	-0.12	1.15	0.00	-1.55	0.17
Max V2	Roof	DCON21	0.49	0.58	-0.23	-1.17	1.09	-4.08
Max T	5th	DCON14	25.02	0.13	-4.41	2.57	17.96	-0.22
Max M2	2nd	DCON13	-39.7	-0.18	4.44	-1.84	37.06	-2.30
Max M3	2nd	DCON15	22.83	0.51	-2.19	0.48	-18.57	5.35

Column C80 (Column Line 9) for Seismic								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	SEISMICYXX	-14.6	-0.66	0.11	0.00	-1.44	8.54
Max V2	Roof	SEISMICXYY	-1.64	2.52	0.04	1.44	0.47	11.33
Max T	6th	SEISMICXYY	-7.37	2.16	-0.02	1.58	0.30	-8.11
Max M2	2nd	SEISMICYX	-2.68	0.33	0.55	0.29	11.89	3.28
Max M3	2nd	SEISMICXYY	-13.84	1.74	-0.08	0.95	-1.52	16.89

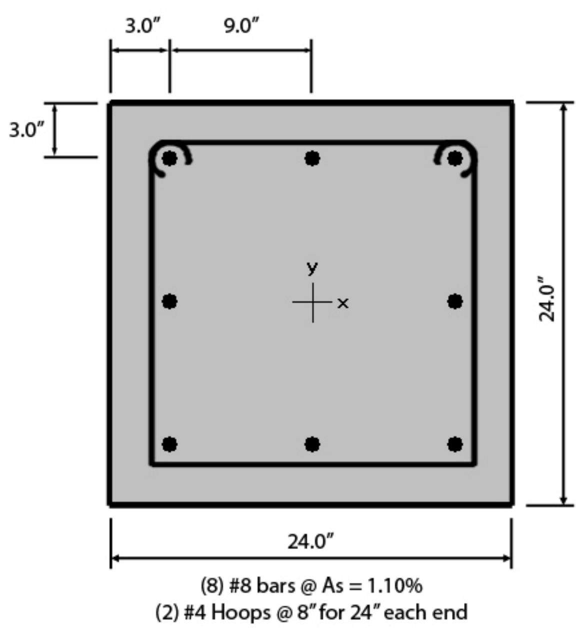
Column C80 (Column Line 9) for Wind								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	DCON20	13.02	0.15	-0.79	0.00	10.29	-1.94
Max V2	5th	DCON19	-6.55	2.34	-0.1	2.22	-0.90	-10.33
Max T	5th	DCON14	-5.22	2	-0.12	2.57	-1.29	-9.03
Max M2	2nd	DCON13	9.95	-1.99	0.79	-1.84	14.07	-13.01
Max M3	2nd	DCON19	-11.89	2.2	-0.62	1.59	-10.88	14.62

Column C65 (Column Line 9) for Seismic								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	SEISMICXYY	20.28	-0.71	0.17	0.00	-0.23	0.96
Max V2	Roof	SEISMICXYY	2.34	1.5	0.57	1.44	2.89	6.47
Max T	6th	SEISMICXYY	10.36	1.36	0.45	1.58	2.77	7.54
Max M2	2nd	SEISMICYX	3.73	0.06	0.59	0.29	11.88	-1.20
Max M3	2nd	SEISMICXYY	19.24	1.3	0.33	0.95	2.84	14.96

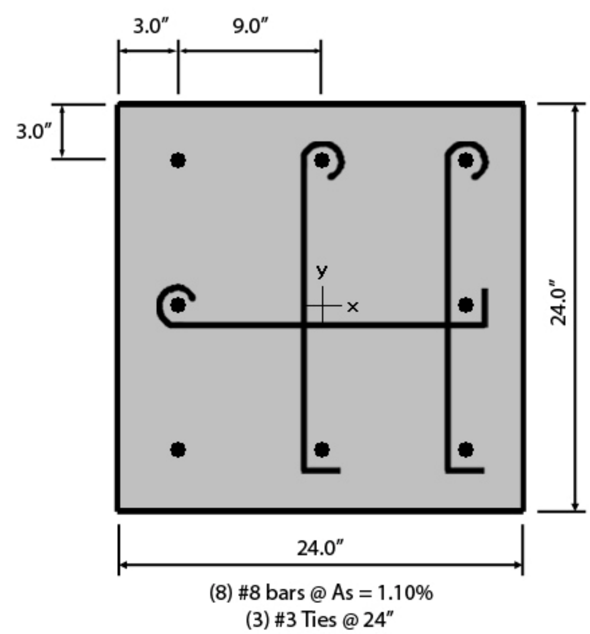
Column C65 (Column Line 9) for Wind								
Story	Floor	Load	P	V2	V3	T	M2	M3
Max P	1st	DCON19	20.2	-0.25	0.88	0.00	-1.18	0.33
Max V2	2nd	DCON19	18.59	1.69	-0.14	1.59	-6.21	-1.01
Max T	5th	DCON14	8.63	1.22	0.27	2.57	-0.57	6.10
Max M2	1st	DCON13	-16.99	0.2	-1.13	0.00	14.64	-2.63
Max M3	2nd	DCON19	18.59	1.69	-0.14	1.59	-7.56	15.03

Concrete Column Loading								
Type	Area	Self Wt.	Dead	Live	Quake	Wind	Snow	LC
Corner (C55)	118.75	49.027	194.750	53.438	9.320	42.520	2.731	415.37
Exterior 1 (C65)	314.71	49.027	516.124	141.620	20.280	20.200	7.238	855.74
Exterior 2 (C80)	240.33	49.027	394.141	108.149	14.600	13.020	5.528	663.55

Type	Flexural Reinf.	Shear Reinf.		Transverse Reinf.
		Av_{min}	0.240	
Corner (C55)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"
Exterior 1 (C65)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"
Exterior 2 (C80)	(8) #8 @ 9"	Use (2) #4 Hoops @ 8" for 24" each end		Use (3) #3 Ties @ 24"



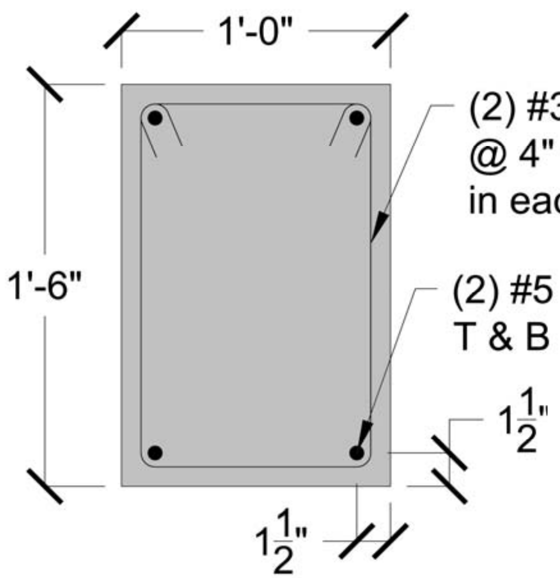
Exterior Column Reinforcement Detail – Column Ends



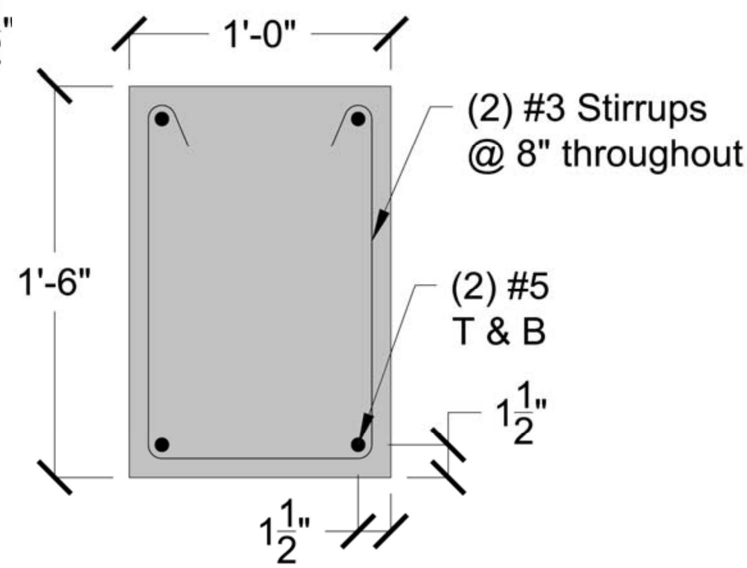
Exterior Column Reinforcement Detail – Throughout Column

Beam 9 (Earthquake)				
Story	Spandrel	V2	T (k-ft)	M3 (k-ft)
ROOF	B9	3.86	0.10	29.52
8TH	B9	1.82	0.17	13.79
7TH	B9	1.72	0.17	13.06
6TH	B9	1.72	0.18	13.04
5TH	B9	1.63	0.18	12.39
4TH	B9	1.5	0.17	11.41
3RD	B9	1.35	0.15	9.89
2ND	B9	1.05	0.13	5.98
1ST	B9	0.62	0.08	4.74

Beam 9 (Wind)				
Story	Spandrel	V2	T (k-ft)	M3 (k-ft)
ROOF	B9	3.95	0.11	30.68
8TH	B9	4.91	0.24	37.51
7TH	B9	5.04	0.23	38.55
6TH	B9	5.27	0.26	40.22
5TH	B9	5.3	0.26	40.48
4TH	B9	5.19	0.26	39.66
3RD	B9	4.83	0.25	36.88
2ND	B9	4.22	0.22	32.26
1ST	B9	2.75	0.14	21.04



Moment Frame Beam Reinforcement Detail – Beam Ends



Moment Frame Beam Reinforcement Detail – Throughout Beam

Roof Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	30.68	$bd^2 \geq 20M_u$	7.151	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	3.95	Use d =	15.5	Reinf. Needed?	no
T_u	0.11	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Fourth Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	39.66	$bd^2 \geq 20M_u$	8.130	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	5.19	Use d =	15.5	Reinf. Needed?	no
T_u	0.26	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Eighth Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	37.51	$bd^2 \geq 20M_u$	7.907	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	4.91	Use d =	15.5	Reinf. Needed?	no
T_u	0.24	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Third Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	36.88	$bd^2 \geq 20M_u$	7.840	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	4.83	Use d =	15.5	Reinf. Needed?	no
T_u	0.25	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Seventh Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	38.55	$bd^2 \geq 20M_u$	8.016	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	5.04	Use d =	15.5	Reinf. Needed?	no
T_u	0.23	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Second Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	32.26	$bd^2 \geq 20M_u$	7.333	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	4.22	Use d =	15.5	Reinf. Needed?	no
T_u	0.22	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Sixth Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	40.22	$bd^2 \geq 20M_u$	8.187	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	5.27	Use d =	15.5	Reinf. Needed?	no
T_u	0.26	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Eighth Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	21.04	$bd^2 \geq 20M_u$	5.922	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	2.75	Use d =	15.5	Reinf. Needed?	no
T_u	0.14	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Fifth Floor B9

Given:		Estimation of d		Torsional Reinforcement	
M_u	40.48	$bd^2 \geq 20M_u$	8.214	$T_u \leq \frac{1}{4}\phi 4\sqrt{f'_c}(A_c^2/P_c)$	36.885
V_u	5.3	Use d =	15.5	Reinf. Needed?	no
T_u	0.26	Calculation of A_s (Flexure)		Transverse Shear Reinforcement	
ϕ	0.9	A_s	0.882	V_c	23.527
b	12	**Use (2) #5 Bars T & B**		$V_u \geq \frac{1}{2}\phi V_c$	no
h	18				
f'_c	4000	Shear Reinforcement			
f_y	60000	$V_s \leq 4\sqrt{f'_c}bd$	yes		
		S_{max} = d/4 = 3.875", use 4"		S_{max} = d/2 = 7.75", use 8"	
		Av_{min}	0.120	Av_{min}	0.120
		Use (2) #3 Hoops @ 4" for 36" @ each end		**Use (2) #3 Strirrups @ 8" throughout length**	

Appendix F: Vibration Damper Cut sheets

KINETICS™
Free Standing Spring Isolators
Model FDS 1 and 2

Description

Kinetics Model FDS Spring Vibration Isolators consist of high deflection, free-standing, unhoused, large diameter, laterally stable steel springs assembled into an upper load plate and leveling assembly. To assure stability, the spring isolators have lateral spring stiffness greater than 1.0 times the rated vertical stiffness and are designed to provide a minimum of 50% overload capacity. Springs are epoxy powder coated, with a 1000-hour salt spray rating per ASTM B-117. In lighter capacities, FDS Spring Isolators have molded neoprene bottom load plate assemblies. In heavier capacities, springs are welded to the load plate assemblies and are furnished with a neoprene noise isolation pad. FDS Isolators have provisions for bolting the isolator to the structure. FDS isolators are available with deflections to 2" (51 mm) and with load capacities to 18,000 lbs. (8165 kg) as standard products. Custom isolators with higher deflection and greater load capacities are also available. Kinetics Model FDS Spring Isolators are highly effective for control of both high and low frequency vibration produced by reciprocating air or refrigeration compressors, pumps, packaged air handling and air conditioning equipment, centrifugal and axial fans, internal combustion engines, etc.

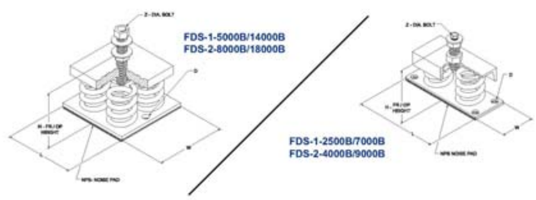
Application

Kinetics Model FDS spring mounts are recommended for use in isolating floor mounted sources of noise and vibration located near critically quiet areas.

Model FDS spring mounts are typically used to reduce the transmission of noise and vibration from low speed mechanical equipment into a building structure. Operating static deflections are available to 2" (51 mm) to compensate for long span floor structures.

Model FDS spring mounts are used in a wide range of applications, some requiring Kinetics equipment bases in addition to spring isolators, and can be used to support and isolate the following equipment types: reciprocating air or refrigeration compressors, close coupled and base mounted pumps, packaged air handling and refrigeration equipment, centrifugal fans, internal combustion engines, and similar equipment. Model FDS Isolators are for use on equipment that is not subject to lateral forces such as wind.

Isolator Type	Spring Color	Rated Capacity		Rated Deflection		Rated O.D.		Free Height		L		W		D		H		Z	
		lbs.	kg	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm	in.	mm
FDS-1-35	Blue	35	16	1.52	39	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-70	Green	70	32	1.36	35	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-120	Grey	120	54	1.18	30	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-220	Brown	220	100	1.07	27	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-370	Orange	370	168	0.96	24	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-500	Beige	500	227	1.00	25	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-600	Chrome	600	272	1.00	25	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-700	Beige/Wh	700	318	1.00	25	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-805	Chr/Wh	805	365	1.01	26	1.75	44	3.19	81	2.63	67	2.63	67	0.69	17	4.19	106	0.38	10
FDS-1-250	Blue	250	113	1.79	45	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-450	Green	450	204	1.54	39	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-625	Black	625	283	1.44	37	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-800	Grey	800	363	1.31	33	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-1000	Red	1000	454	1.15	29	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-1250	Brown	1250	567	1.09	28	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-1700	Orange	1700	771	0.95	24	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-2200	Org/Grey	2200	998	1.00	26	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-2465	Blue	2465	1118	1.00	25	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-2865	Blue/Grey	2865	1300	1.00	25	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-3500	Blue/Brn	3500	1588	1.00	25	3.00	76	4.20	107	3.88	98	3.88	98	1.00	25	5.44	138	0.38	10
FDS-1-120B	Grey	120	54	1.18	30	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-220B	Grey	220	100	1.07	27	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-370B	Grey	370	168	0.96	24	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-500B	Grey	500	227	1.00	25	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-600B	Grey	600	272	1.00	25	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-700B	Grey/Grey	700	318	1.00	25	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-805B	Grey/Grey	805	365	1.01	26	1.75	44	3.19	81	4.50	114	2.50	64	0.56	14	4.25	108	0.38	10
FDS-1-250B	Grey	250	113	1.79	45	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-450B	Grey	450	204	1.54	39	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-625B	Grey	625	283	1.44	37	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-800B	Grey	800	363	1.31	33	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-1000B	Grey	1000	454	1.15	29	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-1250B	Grey	1250	567	1.09	28	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-1700B	Grey	1700	771	0.95	24	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-2200B	Grey/Grey	2200	998	1.00	26	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-2465B	Grey	2465	1118	1.00	25	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-2865B	Grey/Grey	2865	1300	1.00	25	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-3500B	Grey/Grey	3500	1588	1.00	25	3.00	76	4.20	107	7.00	178	4.00	102	0.69	17	5.63	143	0.38	10
FDS-1-2500B	Grey/Grey	2500	1134	1.09	28	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-3400B	Grey/Grey	3400	1542	0.95	24	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-4400B	Grey/Grey	4400	1996	1.00	26	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-4930B	Grey/Grey	4930	2236	1.00	25	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-5730B	Grey/Grey	5730	2599	1.00	25	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-7000B	Grey/Grey	7000	3175	1.00	25	3.00	76	4.20	107	10.00	254	4.00	102	0.69	17	6.25	159	0.50	13
FDS-1-5000B	Grey	5000	2268	1.09	28	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.50	13
FDS-1-6800B	Grey	6800	3084	0.95	24	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.50	13
FDS-1-8800B	Grey/Grey	8800	3992	1.00	26	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.50	13
FDS-1-9860B	Grey	9860	4472	1.00	25	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.63	16
FDS-1-11460B	Grey/Grey	11460	5198	1.00	25	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.63	16
FDS-1-14000B	Grey/Grey	14000	6350	1.00	25	3.00	76	4.20	107	8.00	203	8.00	203	0.69	17	6.88	175	0.63	16



Specifications

Vibration isolators shall be free standing, unhoused, laterally stabile steel springs. Springs shall have a lateral stiffness greater than 1.0 times the rated vertical stiffness and shall be designed to provide a minimum 50% overload capacity.

Springs shall be assembled or welded between top and bottom load plates. The upper load plate shall be provided with steel leveling bolts, lock nut and washer for attachment to the supported equipment. The lower load plate shall incorporate a non-skid noise isolation pad and shall have provisions for bolting the isolator to the supporting structure, as required.

Springs shall be selected to provide operating static deflections shown on the Vibration Isolation Schedule or as otherwise indicated on the project documents. Springs shall be color coded or otherwise identified to indicate load capacity.

Spring isolation mounts for floor-mounted equipment shall be Model FDS, as manufactured by Kinetics Noise Control, Inc.

KINETICS Noise Control

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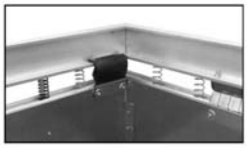
www.kineticsnoise.com
sales@kineticsnoise.com

FDS 1-2 10/07

KINETICS™
Roof Curb Rail
Model KSR



A noise and vibration control system that goes beyond internal isolation.

Description

Kinetics KSR Isolation Rails are the next generation isolation system designed and engineered to isolate packaged rooftop equipment from the roof structure. Designed for easy installation, minimum interference with equipment overhang and with accessible springs, the Kinetics KSR goes well beyond internal isolation by reducing casing-radiated vibration caused by turbulent air flow as well as compressor and fan vibration.

KSR rails have a positive elastomeric air and weather seal permitting the inside of the unit to be used as a return air plenum. The KSR mates with the inside of the manufacturer's curb eliminating any internal interference. The KSR also features an impressive family of options including:

- Aluminum weather seal flashing
- Seismic restraint
- Airborne noise control package
- Duct blockoffs

Application

Kinetics Model KSR Isolation Rails are specifically designed and engineered for use as a noise and vibration isolation system for roof curb-mounted mechanical equipment.

Model KSR isolation rails are compatible with most roof-supported equipment and standard roof curb systems without modification and provide support, noise and vibration isolation, and an air and water seal for supported equipment.

Typical applications include support and isolation for unitary-packaged air-handling and refrigeration equipment, and exhaust fans, ordinarily mounted directly on non-isolated roof curb systems.

Model KSR isolation rails significantly reduce noise and vibration transmitted from rooftop equipment into roof structures by using equipment weight as an inertia mass to load high-deflection, free-standing, stable springs integrated with the continuous aluminum isolation rail system.

Specifications

Spring components shall be (1"/25 mm), (2"/51 mm) deflection, free-standing, unhoused, laterally stable steel springs. Springs shall have a lateral stiffness greater than 1.0 times the rated vertical stiffness and shall be designed for 50% overload to solid.

Springs shall be color coded to indicate load capacity.

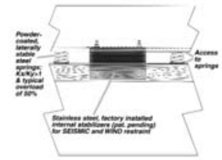
Rails shall provide continuous support for the rooftop equipment and shall be designed to provide isolation against casing-radiated vibration in the rooftop equipment housing and structureborne vibration from rotating and mechanical equipment in the rooftop package.

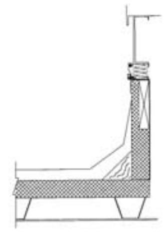
Rail assembly shall consist of extruded aluminum top and bottom members connected by spring isolators and a continuous air- and water-tight seal. The seal shall be a beaded elastomeric material retained in a keyway along the top extrusion. The weather strip shall be sealed along the bottom with an aluminum fascia strip.

Rail assemblies shall incorporate means for attachment to the building and the supported equipment and shall incorporate additional stiffening members if necessary to assure stability.

Vibration isolators shall be selected by the manufacturer for each specific application to comply with deflection requirements as shown on the Vibration Isolation Schedule or as indicated on the project documents.

Roof Curb Rails shall be Model KSR as manufactured by Kinetics Noise Control, Inc.





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KSR 4/05

Appendix G: Concrete Slab Fasteners

TITEN® *Concrete and Masonry Screws*

SIMPSON Strong-Tie ANCHOR SYSTEMS

Titen® screws are 3/16" and 1/4" diameter masonry screws for attaching all types of components to concrete and masonry. Available in hex and phillips head designs in three colors. Use with appropriately sized Titen drill bits included with each box.

Warning: Industry studies show that hardened fasteners can experience performance problems in wet or corrosive environments. Accordingly, use these products in dry, interior and non-corrosive environments only.

MATERIAL: Heat-treated carbon steel

FINISH: Zinc plated with a baked on ceramic coating

CODES: Florida FL 2355.1

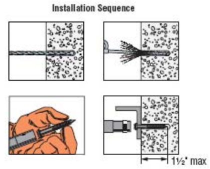
INSTALLATION:

Caution: Oversized holes in the base material will reduce or eliminate the mechanical interlock of the threads with the base material and will reduce the anchor's load capacity.

- Drill a hole in the base material using the appropriate diameter carbide drill bit as specified in the table. Drill the hole to the specified embedment depth plus 1/2" to allow the thread tapping dust to settle and blow it clean using compressed air. Overhead installations need not be blown clean. Alternatively, drill the hole deep enough to accommodate embedment depth and dust from drilling and tapping.
- Position fixture, insert screw and tighten using drill and installation tool fitted with a hex socket or phillips bit.

Preservative-treated wood applications: Suitable for use in non-ammonia formulations of CCA, ACQ-C, ACQ-D, CA-B, SBX/DOT and zinc borate. Use in dry, interior environments only. Use caution not to damage ceramic barrier coating during installation. Recommendations are based on testing and experience at time of publication and may change. Simpson Strong-Tie cannot provide estimates on service life of screws. Contact Simpson Strong-Tie for additional information.

Installation Sequence



Mechanical Anchors

Titen® Tension and Shear Load Values in Normal-Weight Concrete

Titen Dia. in. (mm)	Drill Bit Dia. in.	Embed. Depth in. (mm)	Critical Spacing in. (mm)	Critical Edge Dist. in. (mm)	Tension Load f'c ≥ 2000 psi (13.8 MPa) Concrete Ultimate lbs. (kN)	Tension Load f'c ≥ 2000 psi (13.8 MPa) Concrete Allowable lbs. (kN)	Tension Load f'c ≥ 4000 psi (27.6 MPa) Concrete Ultimate lbs. (kN)	Tension Load f'c ≥ 4000 psi (27.6 MPa) Concrete Allowable lbs. (kN)	Shear Load f'c ≥ 2000 psi (13.8 MPa) Concrete Ultimate lbs. (kN)	Shear Load f'c ≥ 2000 psi (13.8 MPa) Concrete Allowable lbs. (kN)
3/16 (4.8)	5/32	1 (25.4)	2¼ (57.2)	1⅛ (28.6)	500 (2.2)	125 (0.6)	640 (2.8)	160 (0.7)	1,020 (4.5)	255 (1.1)
3/16 (4.8)	5/32	1½ (38.1)	2¼ (57.2)	1⅛ (28.6)	1,220 (5.4)	305 (1.4)	1,850 (8.2)	460 (2.0)	1,670 (7.4)	415 (1.8)
1/4 (6.4)	3/16	1 (25.4)	3 (76.2)	1½ (38.1)	580 (2.6)	145 (0.6)	726 (3.2)	180 (0.8)	900 (4.0)	225 (1.0)
1/4 (6.4)	3/16	1½ (38.1)	3 (76.2)	1½ (38.1)	1,460 (6.5)	365 (1.6)	2,006 (8.9)	500 (2.2)	1,600 (7.1)	400 (1.8)

1. Maximum anchor embedment is 1½" (38.1 mm).
2. Concrete must be minimum 1.5 x embedment.

Titen® Phillips head screw available in white, standard blue and silver

Titen® Tension and Shear Load Values in Face Shell of Hollow and Grout-Filled CMU

Titen Dia. in. (mm)	Drill Bit Dia. in.	Embed. Depth in. (mm)	Critical Spacing in. (mm)	Critical Edge Dist. in. (mm)	Values for 6" or 8" Lightweight, Medium-Weight or Normal-Weight CMU: Tension Load Avg. Ult. lbs. (kN)	Tension Load Allow. lbs. (kN)	Shear Load Avg. Ult. lbs. (kN)	Shear Load Allow. lbs. (kN)
3/16 (4.8)	5/32	1 (25.4)	2¼ (57.2)	1⅛ (28.6)	542 (2.4)	110 (0.5)	1,016 (4.5)	205 (0.9)
1/4 (6.4)	3/16	1 (25.4)	3 (76.2)	1½ (38.1)	740 (3.3)	150 (0.7)	1,242 (5.5)	250 (1.1)

*See page 10 for an explanation of the load table icons

1. The tabulated allowable loads are based on a safety factor of 5.0 for installations under the IBC and IRC. For installations under the UBC use a safety factor of 4.0 (multiply the tabulated allowable loads by 1.25).
2. See notes 1 and 2 above.

Bulk packaging available for large-volume applications

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LSES *Lag Screw Expansion Shield*

SIMPSON Strong-Tie ANCHOR SYSTEMS

The Lag Screw Expansion Shield is a die cast zinc alloy expansion shield for anchoring lag screws in a variety of base materials, including concrete, concrete block, brick and mortar joints. Radial ribs provide additional holding power in softer material.

MATERIAL: Die cast Zamac 3 Alloy

INSTALLATION:

Caution: Oversized holes may make it impossible to set the anchor and will reduce the anchor's load capacity.

- Drill a hole in the base material using the appropriate-diameter carbide drill bit as specified in the table. Drill the hole to the specified embedment depth plus 1/8" for flush mounting and blow it clean using compressed air. Alternatively, drill the hole deep enough to accommodate embedment depth and dust from drilling. Overhead installations need not be blown clean.
- Insert anchor into hole. Tap with hammer until flush with surface of base material.
- Position fixture; insert screw and tighten.

LSES

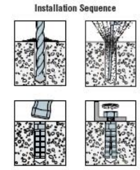
LSES Product Data and Tension Loads in Normal-Weight Concrete

Size (in.)	Model No.	Drill Bit Dia. (in.)	Embed. Depth (in.)	Allowable Tension Load (lbs.) f'c ≥ 3000 psi	Quantity Box	Quantity Carton
1/4 Short	LSES25S	1/2	1	90	100	500
5/16 Short	LSES31S	1/2	1¼	100	100	500
3/8 Short	LSES37S	5/8	1¾	220	50	250
1/2 Short	LSES50S	3/4	2	250	25	125
1/4 Long	LSES25L	1/2	1½	120	50	250
5/16 Long	LSES31L	1/2	1¾	150	50	250
3/8 Long	LSES37L	5/8	2½	260	50	200
1/2 Long	LSES50L	3/4	3	310	25	100

*See page 10 for an explanation of the load table icons

1. The allowable loads listed are based on a safety factor of 4.0.
2. The minimum concrete thickness is 1½ times the embedment depth.
3. Screw is not included.

Installation Sequence



Mechanical Anchors

ESA *Expansion Screw Anchor*

The ESA was the original internally-threaded mechanical anchor design. The malleable lead shield allows for secure mounting.

MATERIAL: Cone – Die Cast Zamac 3 alloy; Expander Shield – 3 - 5% antimonial lead

INSTALLATION:

- Drill a hole in the base material using the appropriate diameter carbide drill bit as specified in the table. Drill the hole to the specified embedment depth plus 1/8" for flush mounting. Blow the hole clean using compressed air. Overhead installations need not be blown clean.
- Insert anchor into hole.
- Using a piloted setting punch, drive expander shield over cone.
- Position fixture; insert fastener and tighten.

CODES: Meets Federal Specifications A-A-1922A, Type 1, except ESA50.

ESA

Piloted Setting Punch

Installation Sequence

ESA Product Data and Tension Loads in Normal-Weight Concrete

Internal Thread Size (dia. - threads per inch)	Model No.	Drill Bit Dia. (in.)	Embed. Depth (in.)	Allowable Tension Load (lbs.) f'c ≥ 3000 psi	Quantity Box	Quantity Carton
#10 - 24	ESA10	3/8	5/8	140	100	1600
1/4 - 20	ESA25	1/2	7/8	190	100	500
3/8 - 16	ESA37	3/4	1¼	380	50	200
1/2 - 13	ESA50	7/8	1½	400	50	200

1. The allowable loads listed are based on a safety factor of 4.0.
2. The minimum concrete thickness is 1½ times the embedment depth.
3. Machine bolt is not included.
4. One piloted setting punch is included in each box.

*See page 10 for an explanation of the load table icons

Piloted Setting Punch Product Data

Model No.	For Use With	Box Qty.
PSP10	ESA10	10
PSP25	ESA25	10
PSP37	ESA37	10
PSP50	ESA50	10

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