J.B. Byrd Alzheimer's Center & Research Institute

Tampa, Florida

November 16, 2011

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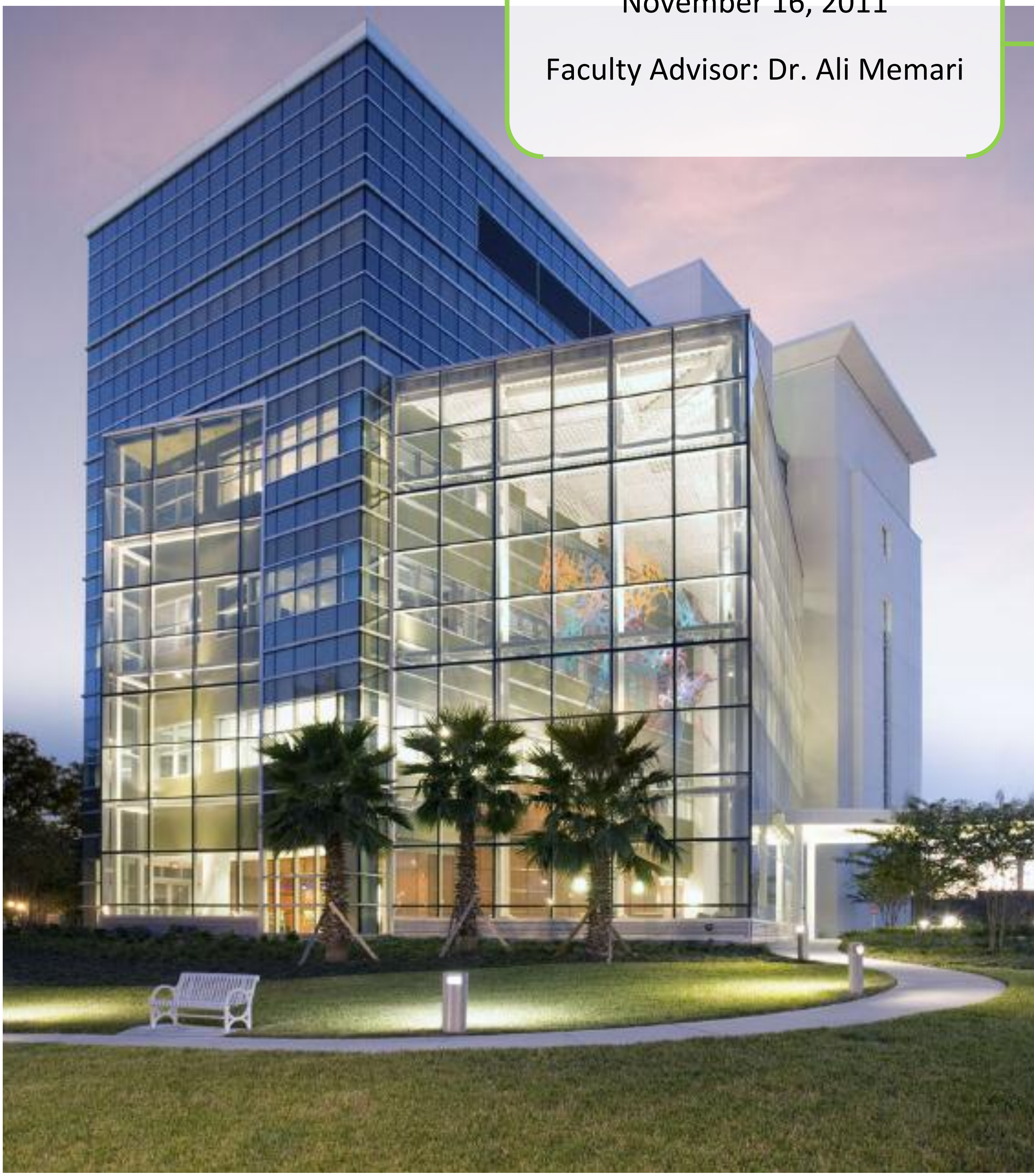


Table of Contents

Executive Summary

The main purpose of this technical report entitled, "Lateral System Analysis and Confirmation Design," is to evaluate the effectiveness of the lateral system of the Alzheimer Center & Research Institute located in Tampa, Florida. The lateral system consists of 11 shear walls placed around elevator shafts and stair cases as well as scattered moment frames throughout the irregularly L-shaped building.

The analysis contained within this technical report began by verifying dead, live, and snow loads used in the structural drawings. Next, both wind and seismic loads were calculated for the building using the Main Wind Force Resisting System procedure and the Equivalent Lateral Force procedure given in Chapters 6 and 12 of the ASCE 7-05. It was found that the wind loads controlled the design of the lateral system by a factor of 3 in the East-West direction (E-W) and 2 in the North-South (N-S) direction in serviceability. Thus with ASCE 7-05 factors, wind will control by 1.6 times more that amount.

Next, a finite element lateral model was built of the Alzheimer Center in ETABS. The first model was built with rigid diaphragms and all gravity elements modeled to accurately represent the stiffness of the structure. The concrete columns and beams were modeled accurately according to their section and were assigned a factor of 0.5 in the property modifier for moment of inertia about 2 and 3 axes to account for concrete cracking. Columns and shear walls were pinned at the base.

Upon completion of the models, all the loads (wind and seismic) were incorporated into the models using load cases for forces in the N-S (x) and E-W (y) directions as well as accidental moments in both directions due to the applied loads. These accidental moments were applied as their own load case to simplify the process of incorporating them into the required load combinations from Chapter 2 of ASCE 7-05. This totaled the load cases to 15.

In order to verify the accuracy of the models, the centers of mass, center of rigidity, shear forces, moments, and drifts were recorded. The centers of mass and rigidity were verified with hand calculations. The center of rigidity could not be replicated by hand due to the fact that only shear walls were taken into account. This was done to study the reliance of the lateral system on shear walls. It was found that the stiffness of the system was 53% due to shear walls and 47% due to moment frames.

The ETABS model was then used to determine the relative stiffness of each frame. A 100 kip load was applied to the top of each individual frame, and the lateral displacement was measured. From these values the relative stiffness of the frames and walls were calculated. With the relative stiffness it was then possible to distribute the lateral load to the building. After confirming the location of the centers of rigidity and mass, both direct and torsional shear must were considered for this building. The hand calculations for shear due to direct shear and torsional shear were 99% accurate.

In lieu of replicating the values, it was chosen to calculate both shear and moment capacities of the lateral force resisting elements. A column in the middle (I-8) and shear wall (P-9) were taken as spot checks. They were found to be more than adequate.

Building Introduction

Magnolia Ave.

Fletcher

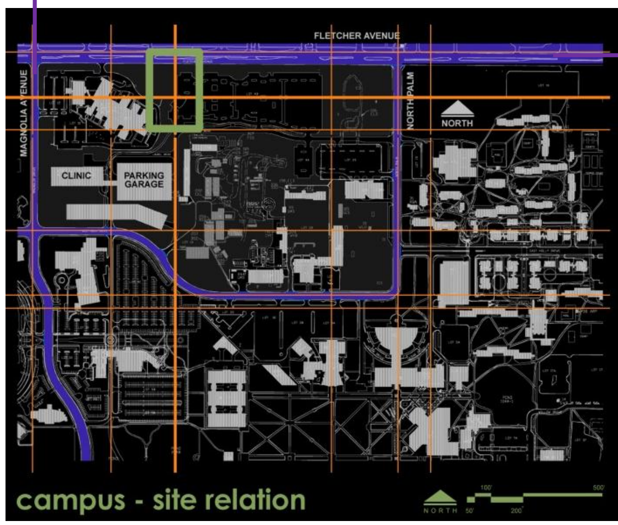


Figure 1- **Site Location on campus of USF**

The Johnnie B. Byrd, Sr. Alzheimer's Center & Research Institute or J.B Alzheimer's center is located in Tampa, Hillsborough, Florida in the University of South Florida's campus. It's located on the intersection of the orange lines on Fletcher Avenue and Magnolia Avenue (See Figure 1). Its occupant is the University of South Florida and it is a business occupancy used for offices and as a research facility. In fact, after its construction the Florida Alzheimer's center and Research facility became one of the largest freestanding facilities of its type in the world specifically devoted to this illness. It is designed to primarily function as a research unit with labs, a hub for clinic trials, and a data collection center for all Alzheimer facilities throughout the state of Florida. It is built on a 2.6 acres site and the size of the building is 108,054 sq ft, gross. It is 9 stories including a basement totally a height 106'10". The actual building cost was $23,602,477. It has been LEED silver accredited after construction. From start to finish the construction dates were from February 7, 2006 to July 9, 2007 hence about a year and a half.

The Owner/Client of the project is Johnnie B. Byrd Alzheimer's Center & Research Institute. The General Contractor + CM were Turner Construction Company. Everything else (i.e. Architecture, Structural Engineering, Mechanical & Electrical & Plumbing Engineering, Civil Engineering, Landscape Architecture, Security & Telecom) were handled by HDR Architecture, Inc. This project was delivered to the owner by a design-bid-build method.

The façade of the building is mainly divided into two parts. The east side consist of curtain wall glazing and Aluminum panels. The west side consists of cement plaster with the same curtain wall like glazing and decorative grille with louver at the top. As for the roof the use of Thermoplastic Membrane roofing was chosen with ¼"per foot slope with Aluminum parapet for architectural reasons.

Structural Overview

Basic construction materials of the building include stone column piers and a spread footing foundation system with below grade footing. The structure is composed of precast joist webs and soffit beam bottoms with concrete shear walls. Exterior walls are constructed of cement plaster and lath on steel stud back up framing. The curtain wall system has a kynar aluminum finish and integrates several glazing types. Mechanical systems include packaged air handlers, on-site chillers, and gas fired boilers.

Initially, HDR Architecture Inc. structural department had designed this building as a composite system composed of steel beams, flanges, columns and a concrete slab on metal floor deck. They had their system pre-designed with specifics. However, all these ideas got tossed away when the Owner and the Contractor decided to use a more economical and efficient concrete system with precast joist webs and soffit beams. The latter exists mainly in Florida. Hence, the use of it will be fairly new to others, which add uniqueness to this building and thesis.

The J.B. Byrd Alzheimer's Center & Research Institute rests on spread footings for columns and continuous strip footings for walls as well as a mat slab foundation system. This was advised by Nodarse & Associates, Inc. because the site lies on a potential sinkhole activity. The lower 7 floors utilize a one way concrete slab with precast joist ribs and soffit beam framing system for floor framing with cast in-place columns. Part of level 7 and level 8 still utilize the same floor framing but with larger spacing as well as concentrated reinforcing bars around roof anchors. The lateral system consists of moment frames with concrete shear walls around the main openings.

The importance factors for all calculations were based on Occupancy category II. This was chosen because the J.B A.C. & R.I. falls under office building.

Design Codes

According to sheet S001, the original building was designed to comply with the following major codes:

- 2001 Florida Building Code with 2003 updates
- 2001 Florida Building Mechanical Code with 2003 updates
- 2001 Florida Building Plumbing Code with 2003 updates
- 2001 Florida Building Fuel Gas Code with 2003 updates
- 2001 Florida Building Accessibility Code as Ch.11 and Energy Code as Ch.13
- 2000 National Fire Protection Association.
- Building code requirements for reinforced concrete (ACI 318)
- AISC Manual of Steel Construction, Allowable Stress Design 9th ED.
- AISC Manual of Steel Construction, Load Resistance Factor Design (LRFD) 1st ED.
- American Welding Society (AWS), D1.1, D1.3, D1.4
- Minimum Design Loads for Buildings and Other Structures (ASCE 7-98)
- Masonry Construction for Buildings (ACI 530-99 AND ACI 530.1-99)

These are also the codes used to complete this technical report:

- Minimum Design Loads for Buildings and Other Structures (ASCE 7-05)
- Building code requirements for reinforced concrete (ACI 318-08)
- 2006 International Building Code (IBC 2006)

Materials Used

Various materials were used on the structure of this project. Below are the main materials derived from Sheet S-001 (see Appendix D).

Concrete		
Usage	Weight	Strength (psi)
Spread footing	Normal	3000
Mat slab foundation	Normal	3000
Precast Joist Webs and soffit beams	Normal	5000
Cast-in-place slab	Normal	4000
Columns, typical	Normal	4000
Columns, as noted	Normal	6000
Precast Masonry Lintels	Normal	5000
Housekeeping Pads	Normal	4000
General Structure Concrete	Normal	4000
Note: Normal weight concrete is at 28 day compressive strength		

Steel		
Usage	Standard	Grade
Reinforcing Steel	ASTM A615	60
Reinforcing Steel (welded)	ASTM A706	60
Welded Wire Fabric	ASTM A185	70
Prestressing Tendons	ASTM A416	270
Wide Flange, S and Tee shapes	ASTM A992	50
Angles Channels and Plates	ASTM A36	36
Tubes	ASTM A500 B	46
Pipes	ASTM A53 B	35
Bolts	ASTM A325	36
Glavanized Roof deck	ASTM A653	33
Note: Welding Electrodes used were E70XX		

Masonary		
Usage	Standard	Strength (psi)
Concrete Masonary Units	ASTM C-90	f'_m= 1500
Mortar	ASTM C270, M	f'c= 2500
Mortar	ASTM C270, S	f'c= 1800
Grout	ASTM C476	f'c= 3000
Joint Reinforcement	ASTM A82, Truss Type	

Figure 2 - **Material Used in building: Concrete, Steel, Masonary**

Foundations

Nodarse & Associates, Inc prepared a report of Preliminary Geotechnical Exploration for this project. The subsurface exploration consisted of a Ground Penetrating Radar (GPR) survey on the site and eight Standard Penetration Test (SPT) borings to depths of 50 to 75 feet below existing site grades.

The borings encountered a relatively uniform subsurface profile consisting of the following respectively with depths: clean sands, medium dense clayey sands, very soft to stiff clays, and weathered to very hard limestone formation. There are indicators in the borings that correlate with the increased risk for sinkhole occurrence. These indicators consist of very soft soils or possibly voids. They estimated that sinkhole could range at the ground level from 10 to 25 feet across. A deep foundation system was not recommended due to the possibility of damage to other adjacent structures from pile-driving vibrations. Also, a cast-in-place deep foundations such as auger cast piles or drilled shafts are not recommended because the presence of joints, fissures, soft zones, and voids within the limestone formation and overburden soils will result in excessive overages of concrete and the need for permanent steel casing. In addition, The University of South Florida expressed concerns about this method as there is the potential of water contamination.

Hence, Nodarse & Associates, Inc recommended, based on their findings the use of a vibro-flotation/stone columns to improve soil conditions so that the building can be supported on a shallow foundation system such as footings and mat slabs (see figure 3 for shallow foundations used). The vibrating probe is intended to pre-collapse potential sinkholes (a total settlement of 1 inch or less) to reduce the possibility of future development. After the dry bottom, stone columns (42" +/-diameter) were installed to a depth of 25 feet. The stone columns were recommended to be crushed stone aggregate a similar gradation to FDOT No. 57 stone. Footings were then designed on a maximum allowable bearing pressure of 6,000psf. The allowable soil bearing capacity is 10,000 psf after soil improvement. Minimum footing widths for columns and wall footings of 36 and 24 inches respectively were used. Footings bear at least 36 inches below finished floor elevations to provide adequate confinement of bearing soils.

The ground water on this project site appears to be below a basement depth of 10 feet below existing grade, making a basement acceptable. Retaining Walls were also designed using a maximum allowable bearing pressure of 2,000 psi.

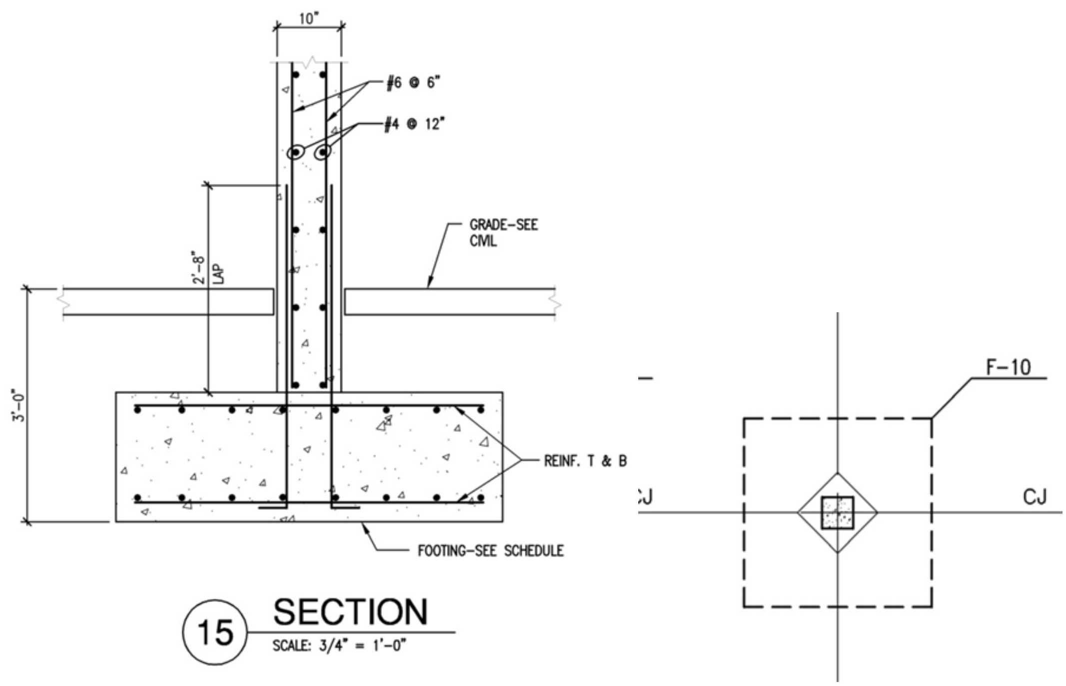


Figure 3- **Foundation section and plan showing footing-column connection and size**

Floor Systems

Even though this building is very architectural and seems like an irregular shape building with a complicated structure it can be divided into 4 simple sections. The sections also correspond to the different uses of the building. Figure 4 shows a typical floor plan with the different bay sizes highlighted with different colors.

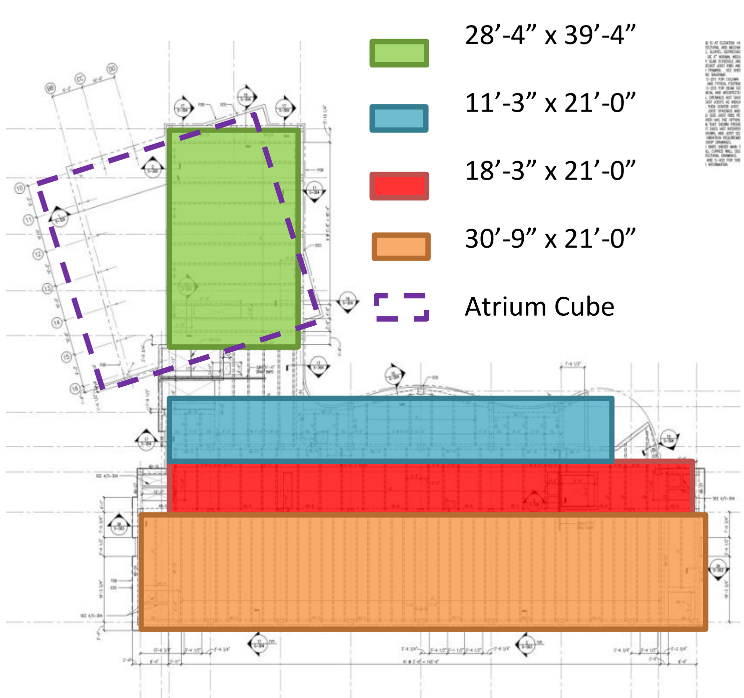


Figure 4- **Floor plan showing different bay sizes**

All the elevated floors of the J.B AC&RI are a hybrid system consisting of a precast joist ribs and soffit beam framing system with cast-in-place to unite the system. In fact, there are 5 main joists that have respectively the following depths: 8", 12', 16", 20", and 28". The entire precast joists and beam soffits are brought on site and lifted to the positions using scaffolding and then they are tied to the structure. Once the structure is erected, the formwork and the rebar reinforcing (if needed) are done then further a 5" concrete slab is casted in place to unite the system (see figure 6). As stated before, 5 different joist depths were used adequately depending on the required spans and uses. For the approximately 40' span, a 20" or J4 was used spaced at 5'-8". That area, corresponding to the green rectangle in figure 4 is typically an office area. For the orange rectangle, where the research labs reside, a J3 or 16" spaced at 5-6" was used for a span of 31'. However in the same area, J4 or 20" spaced at 3'-6" and J5 or 28" at 3'-2" were used to accommodate the PET scans and MRI components respectively (see figure 5).

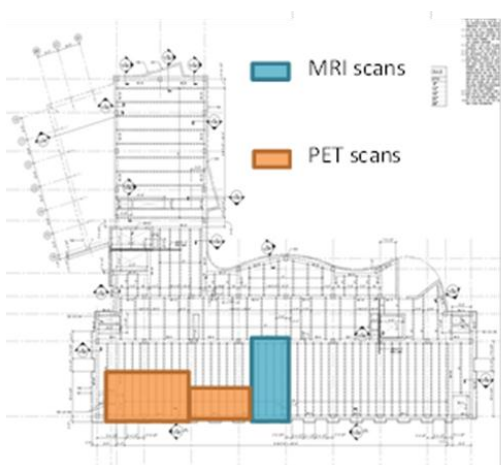
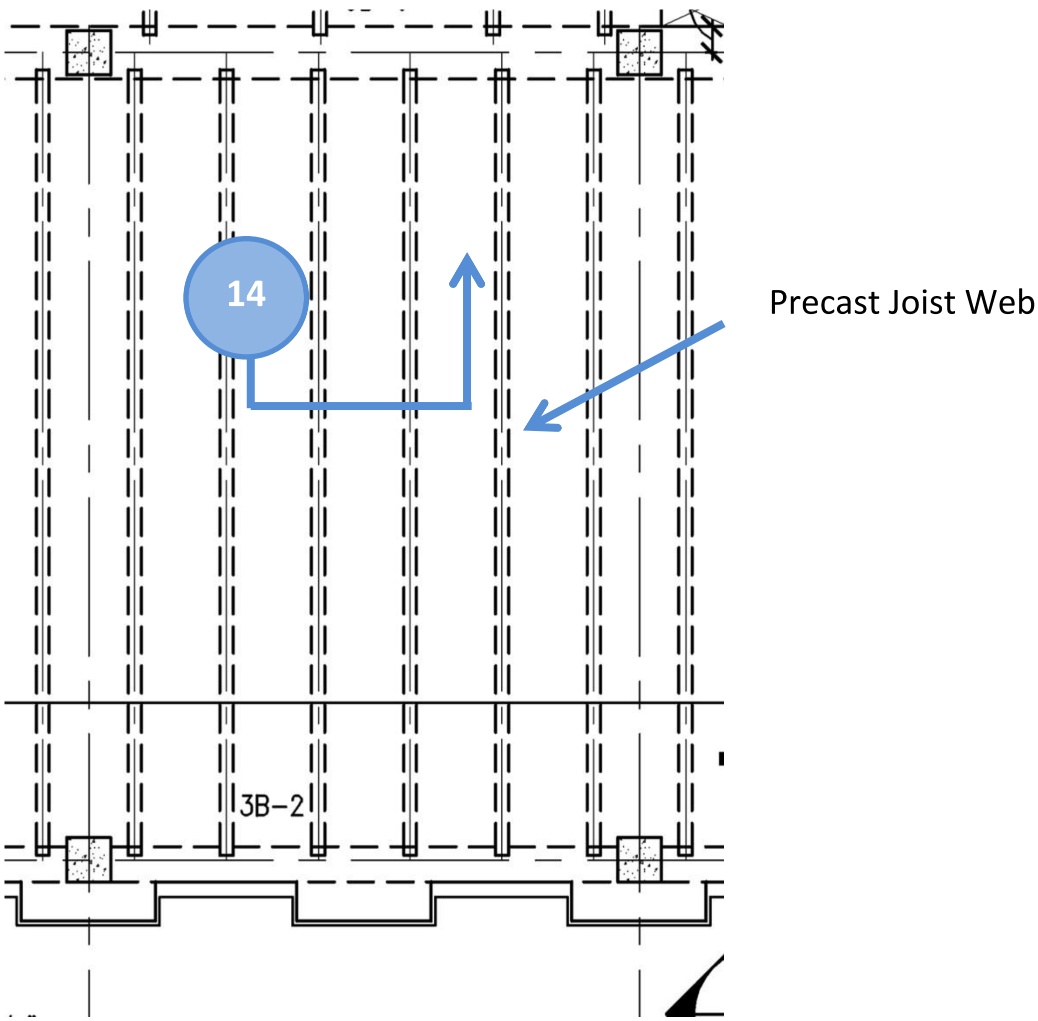


Figure 5- **2nd level floor plan showing MRI/PET scan location**



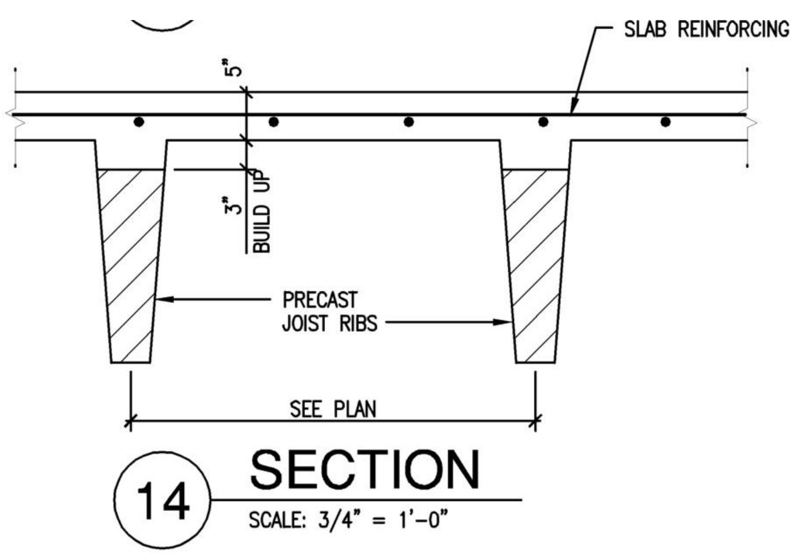
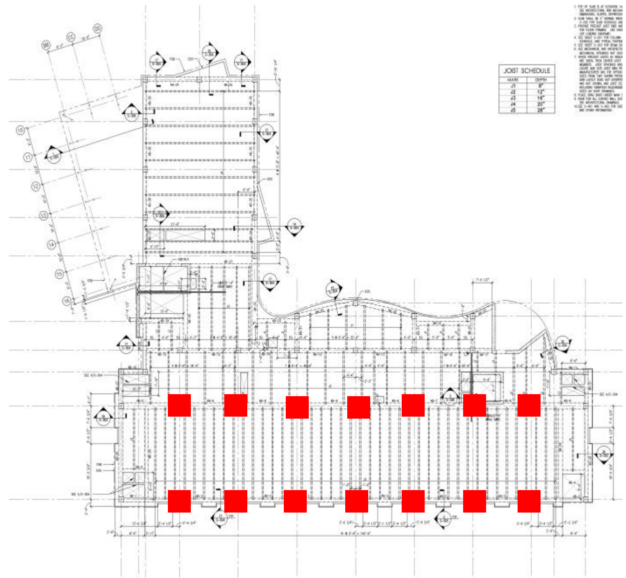


Figure 6- **Plan and section of precast joists**

Framing System



The columns in the lower 7 stories are all cast-in-place concrete. Most of the columns are square and have 4,000psi strength. However, the columns supporting the research labs where the heavy equipment exists and vibration criteria need to be attained a 6,000psi concrete columns were used at the basement and the first floor (see figure 7). All columns are about 20”x20” with reinforcing ranging from 4 to 8 bars except for a few exception that are 20”x30” with 16 bars.

Figure 7- Floor plan showing the 6,000 psi column in basement and 1 floor

Lateral System

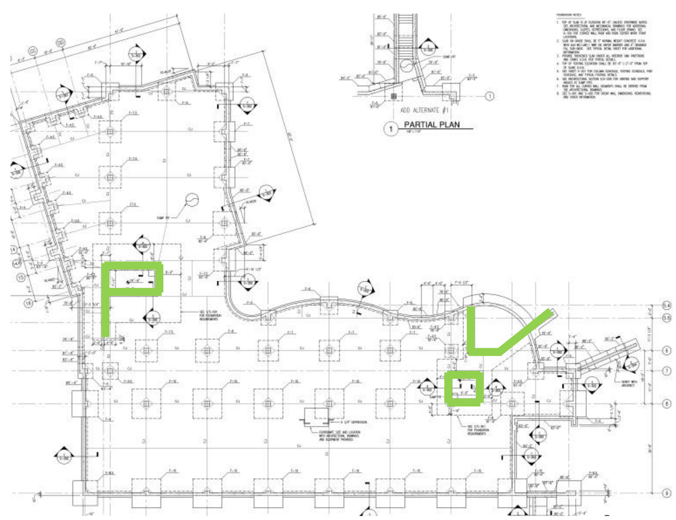


Figure 8- Floor plan showing shear walls

The lateral system is composed of concrete shear walls and moment frames. The shear walls are around the main vertical circulation at both ends of the building (see figure 8). They resist the N-S direction as well as E-W direction for best result and little torsion. All of these walls are cast-in-place and are 12” thick. All of them span from basement to the roof. They are anchored at the base by a mat slab foundation that is 3’-0” thick. An issue not investigated by this report is how much the moment frame resists the loading compared to the shear walls when loaded in both directions.

Atrium Wall Framing / Floor vibration Criteria

The atrium roof is approximately 60 feet above grade. Architectural trusses, approximately 36” deep are designed to support the exterior storefront glazing spanning this 60 feet. The trusses are designed to minimize deflections from hurricane force winds on this wall. The design wind speed for the area is 120mph which yields that the 50’- 60’ range was designed at 31.3 PSF. Truss components are made from structural tubes (ASTM A500, Grade B of Fy= 46Ksi) and pipes (ASTM A53,Grade B Fy= 35Ksi) in this highly visible part of the building.

The vibration control design interfaces with the design of structural, mechanical, architectural, and electrical systems in such a way that those systems do not generate or propagate vibrations detrimental to research activities of the Florida Alzheimer's Center & Research. Vibration criteria have been developed based upon examination of vibration requirements of planned or hypothetical equipment. General labs make up the research facility, and the structure will be designed for vibration amplitude of 2000-4000 µin/s. This accommodates bench microscopes at up to 400x magnification. This last will play a significant role in choosing the members of the system as well as the systems themselves.

Roof Systems

There are two different roof levels: one on the seventh floor and the other on the mechanical level on top of that (See Figure 9). The figure shows a height from level 1 that starts at 100'0" but for simplicity only the true height is shown. This two roof structure consists of the same material and system as the floor system as they hold a great deal of load (mainly mechanical that include packaged air handlers, on-site chillers, and gas fired boilers). However, the slabs were heavily reinforced around the roof anchors. Level 7 has joist spacing of 5'8" in the green section and 3'6" under the red section. On the mechanical level a spacing of 5'-6" is used as loads are minimal. There is also the roof of the atrium cube that is not shown on this figure. That last is at height of 153'-9"and consists of trusses, angles, C shape and HSS bars. In addition to the atrium roof, a canopy at the entrance hangs at a height of 114'-6" and consists of W shape with a 1½" 18 Gage galvanized metal roof deck.

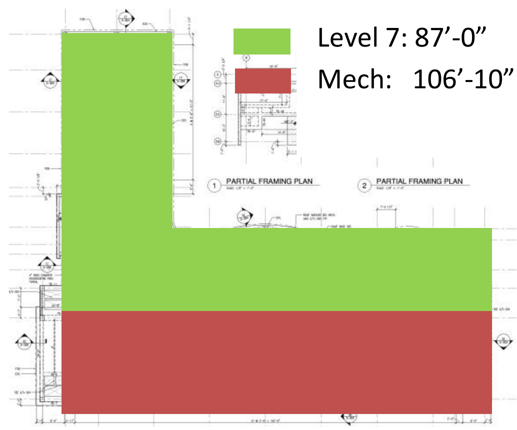


Figure 9- **Showing the different roof levels on the building**

Gravity Loads

Part of this technical report, dead and live loads were calculated and compared to the loads listed on the structural drawings. Snow loads however were not applicable for this project as this building exists in Tampa, Florida. Several gravity member checks were conducted. Detailed calculations for these gravity member checks can be found in Appendix A.

Dead and Live Loads

The structural drawing S001 lists the superimposed dead loads to be used. This is summarized in figure 10. The SP for Ceilings, lighting, plumbing, fire protection, flooring, and HVAC for roof over mechanical levels is higher than usual because all the mechanical system that supplies the research labs that require special feed are situated in that area. These systems include packaged air handlers, on-site chillers, and gas fired boilers.

Also considered in the building weight calculation were the weights of the columns, shear walls, roofs, wall loads, precast joists and soffit beams.

SuperImposed dead loads	
Description	Load
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC all	14 psf
Ceilings, lighting,plumbing, fire protection,flooring,and HVAC for roof over mechanical levels	40 psf
except mechanical	20 psf
allowance for roofing system	20 psf

Figure 10- **Superimposed Dead load on S-001**

The live loads listed below (figure 11) taken from S001 were compared to the live loads in Table 4-1 in ASCE 7-05 based on the usage of the spaces. The result came out to be the same or more than the expected minimum allowed by the code.

There was nothing about Alzheimer research labs or research labs in general hence the provision “Hospitals- Operating Rooms, Laboratories” was used for comparison. The same was done for high density file storage but with the use of two provisions one is based on "Storage-light/heavy" and the other is based on “Libraries-Stack rooms”. Both were in the range or more than the one designed with. The different live loads on each floor are on drawings S-002 and S-003 found in Appendix A. That last one shows on the second level where the MRI and the PET scanner are located special loading was used. A 34kips MRI load distributed to 4 legs then each leg load to 2 joists spaced at 7’-6” apart, center in depression. Also, an 11k scanner load was considered as well as the access path to both the PET and MRI equipment.

One of the last discrepancies, the loadings on S-002 and S-003 are different than the ones stated in the table below. That is due to allow a more flexible building, more stable floors for the vibration and to take into effect the live load reductions.

Floor live loads may be reduced in accordance with the following previsions:

- For live loads not exceeding 100psf for any structural member supporting 150 sq ft or more may be reduced at the rate of 0.08% per sq ft of the area supported. Such reduction shall not exceed 40% for horizontal members, 60% for vertical members, nor R as determined by the following formula:
  R= 23.1 (1+D/L) where D=dead load and L=live load
- A reduction shall not be permitted when the live load exceeds 100psf except that the design live load for columns may be reduced by 20%.

Live Loads			
Area of the building considered	Design Load	ASCE 7-05 Live	Notes
Labratories	125psf	60 psf	Based on "Hospitals-Laboratories"
Offices	50 psf	50 psf	Based on "Office Bldg.-Offices"
Corridors, first floor	100 psf	100 psf	Based on "Office Bldg.-Corridors"
Corridors, above first floor	80 psf	80 psf	Based on "Office Bldg.-Corridors above"
Lobbies	100 psf	100 psf	Based on "Lobbies"
Storage areas	125 psf	125-250 psf	Based on "Storage- light/heavy"
High density file storage	200 psf	125-250 psf	
Mechanical spaces	150 psf	N/A	
Stairs	100 psf	100 psf	Based on "Stairs
Roof	20 psf	20 psf	Based on "Roof- Sloped"

Figure 11- **Live Load comparison to ASCE 7-05**

Snow Loads

No snow load was applicable for this project as it is located in Tampa, Florida. From this following figure 12 taken from ASCE 7-05, the ground snow loads equal zero lb/ft2.

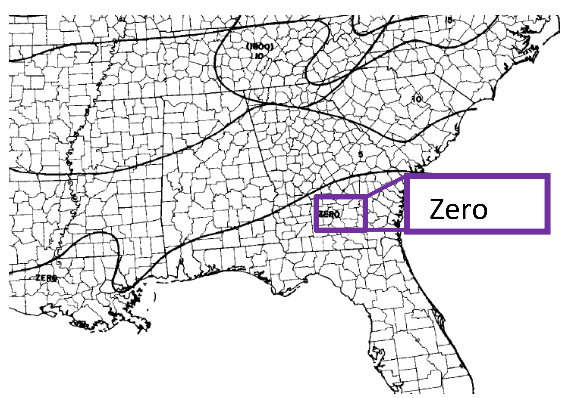


Figure 12- **Diagram showing the ground snow load for Florida**

Lateral Loads

In order to better understand the lateral systems, wind loads and seismic loads were calculated for this technical report. These were calculated by hand, and then applied to a lateral model of the structure created in ETABS. The hand calculations for the wind loads can be found in Appendix B, and the hand calculations for the seismic loads can be found in Appendix C.

Wind Loads

In Technical Report 1, "Existing Conditions and Design Concepts," wind loads were calculated with method 2 Main Wind Force Resisting System (MWRFS) procedure identified in ASCE 7-05 Chapter 6. In order to be able to use this procedure, several simplifying assumptions had to be made. First, the building was modeled with a single roof height of 107'. Next, the surface areas were projected onto North-South (N-S) and East-West (E-W) axes and the projected lengths were used to calculate wind pressures. Using these projected lengths for the calculation of L and B would be conservative. Also, since the new projected shape looks like an L shape, it is assumed that there wouldn't be a buildup in pressure where the void in the L-shape exists. The same forces were used in this technical report.

From technical report 1, it was found that wind loads were greater than seismic by a factor of about 3.6 in the East-West direction and 2.5 in the North-South direction. The design base shear in the North-South direction was calculated to be 682kip, and in the East-West direction was calculated to be 892 kip. Thus, it is expected that wind will control over seismic however this still needs to be checked due to the different load combinations and factors that exist in ASCE 7-05.

Most calculations were performed using Microsoft Excel to simplify a potentially repetitive process. Wind pressures, including windward, leeward, sidewall, and internal pressure were found. These were then used to calculate the story forces at each level. It should be noted that the story forces include windward and leeward pressures, but not internal pressure, because internal pressure is effectively self-cancelling.

The wind loads on this building are collected by the curtain wall glazing and cement plaster walls on the exterior of the building. The walls and the glazing in return transfer these loads to the slabs that they are anchored to. This then transfers the loads into the slabs, which then carry the load to the shear walls and moment frames in relative to their stiffness. These return the loads to their foundations which are mat slabs and footings respectively.

For this technical report, accidental moments were also calculated. This was achieved through the use of the four load cases for torsion due to wind, given in Figure 6-9 of ASCE 7-05 and included as Figure 13. This was done due to the nature of the geometry of the building (L-shaped) that is susceptible to torsion and may control.

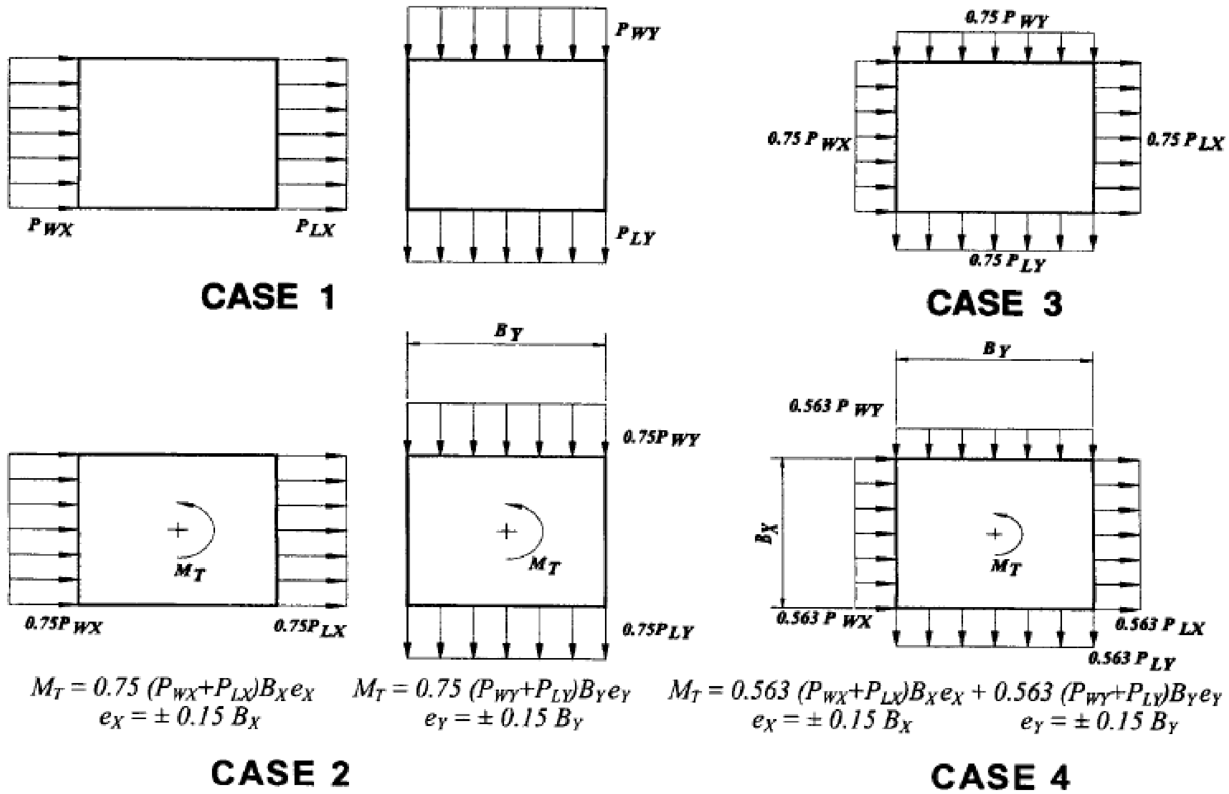


Figure 13- **Figure 6-9 in ASCE 7-05 showing all the torsional wind load cases**

For simple and not iterative process, each load case was represented and labeled differently. They were entered into the model in four basic static load cases: wind forces in the N-S direction (WX), wind forces in the E-W direction (WY), accidental moments due to the N-S loads (WXMX), and accidental moments due to the E-W loads (WYMY). After establishing the formulas and retrieving the corresponding MT, a total of 11 wind cases were established and reported in figure 14. These were then taken as serviceability loads (no factor was incorporated) and analyzed to acquire drifts.

This was done as a first step to determine which of the cases controlled in each direction and in return are then compared to the earthquake loads. This methodology came from the fact that the load factor of wind in ASCE 7-05 is 1.6 much greater than the 1.0 factor used for earthquake meaning the wind forces are magnified. Thus, a simple serviceability comparison would yield the controlling case since the wind forces are greater than earthquake load in both directions. This reasoning produced 13 load combinations detailed in figure 14 (11 with wind and 2 with earthquake).

Serviceability using a factor of 1.0					
Load combinations			Legend		
Wind (total of 11 cases)	Case 1 (2)	$P_{Wx}+P_{Lx}$	Eccentricity	$e_x = \pm 0.15B_x$
		$P_{Wy}+P_{Ly}$		$e_y = \pm 0.15B_y$
	Case 2 (4)	$.75P_{Wx}+.75P_{Lx} \pm M_T$	where M_T=	$0.75(P_{Wx}+P_{Lx})B_xe_x$
		$.75P_{Wy}+.75P_{Ly} \pm M_T$		$0.75(P_{Wy}+P_{Ly})B_ye_y$
	Case 3 (1)	$.75\ (P_{Wx}+P_{Lx)} + .75\ (P_{Wy}+P_{Ly})$	Bx= width of building in x-direction By= width of building in y-direction	
	Case 4 (4)	$.563\ (P_{Wx}+P_{Lx)} + .563\ (P_{Wy}+P_{Ly}) + M_T$	where M_T=	$\pm 0.563(P_{Wx}+P_{Lx})B_xe_x \pm 0.563(P_{Wy}+P_{Ly})B_ye_y$
Earthquake (total of 4)	Case 1 (2)	$1.0\ E_x \pm M_{zx}$		
	Case 2 (2)	$1.0\ E_y \pm M_{zy}$		

Figure 14- **The 11 cases retrieved from figure 6-9 ASCE 7-05 and inputted in ETABS to acquire drifts.**

"Px" or "Py" are the story force at a given level in the direction under consideration and Bx or By are the building dimension in the direction under consideration. The subscripts "W" and "L" represent windward and leeward pressures. The accidental moments are shown under M_T and are shown how they are calculated in the legend of figure 14.

The wind pressures in the N-S direction are listed and diagramed in Figure 15. These were resolved into wind forces in the N-S direction, which are listed and diagramed in Figure 16. The resulting base shear is 682k.

In addition, the wind pressures in the E-W direction are listed and diagramed in Figure 17. These were resolved into wind forces in the E-W direction, which are listed and diagramed in Figure 18. The resulting base shear is 892k.

Wind pressures calculated were able to be compared with the engineer's calculations. In fact, discrepancies of windward and leeward calculations were only 5%. This minor difference was due to the fact that the engineer had used a larger leeward pressure at the altitude of 120'. This height is higher than the building and did not take a simplified roof like it was done in this report.

To see the engineer's calculations and diagrams to compare please refer to pages 38-39.

Desgin wind pressure for MWFRS in N-S Direction						
Type	Level	Height / distance	qz/ qh	Wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
Windward walls	1	0'	21.0	14.3	-6.1	34.7
	2	14'-6"	21.0	14.3	-6.1	34.7
	3	29'	25.5	17.3	-3.1	37.8
	4	43'-6"	28.7	19.5	-0.9	39.9
	5	58'	31.0	21.1	0.7	41.5
	6	72'-6"	33.2	22.6	2.1	43.0
	7	87'	35.1	23.8	3.4	44.3
	Roof	107'	37.1	25.3	4.8	45.7
Leeward walls	All	All	37.1	-13.8	-34.3	6.6
Sidewalls	All	All	37.1	-22.1	-42.5	-1.7
Roof		0-53.5	37.1	-29.9	-50.4	-9.5
		53.5-107	37.1	-27.7	-48.1	-7.2
		107-214	37.1	-16.5	-37.0	3.9

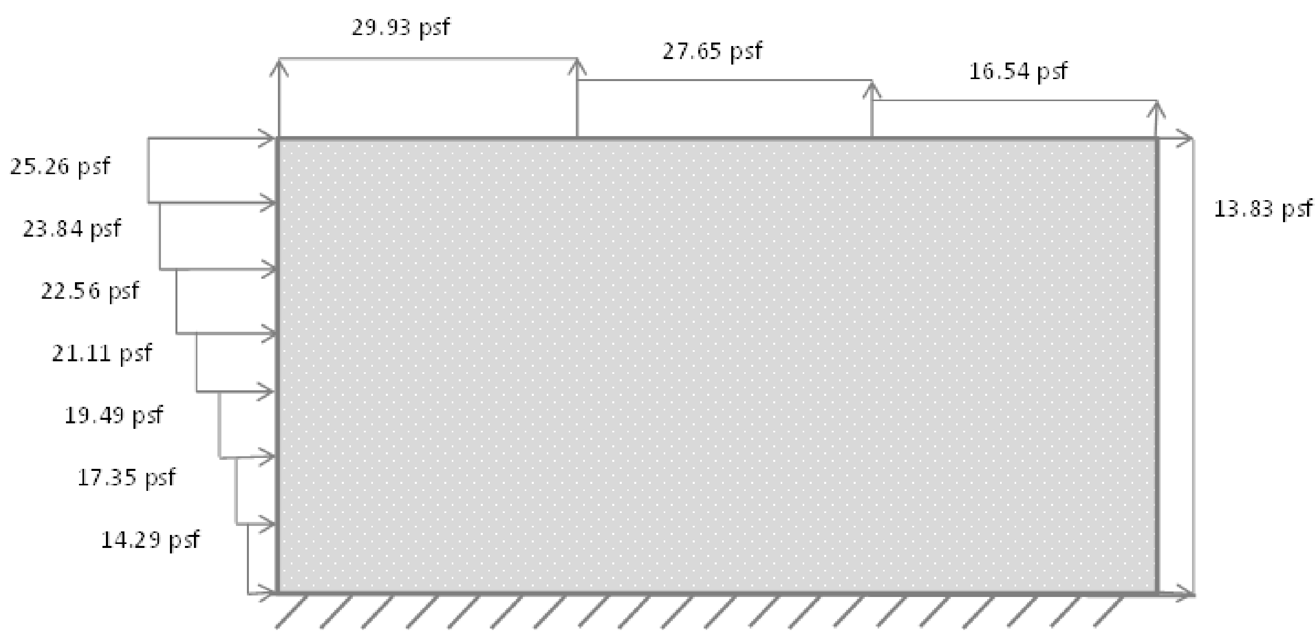


Figure 15 - **List and diagram showing the wind pressure on the building in N-S direction**

Wind Forces- N-S Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0	8	1095	73	682	0
2	14.5	7	1022	8	1095	77	609	1111
3	29	7	1022	8	1095	82	532	2383
4	43.5	7	1022	8	1095	86	450	3748
5	58	7	1022	8	1095	89	364	5186
6	72.5	7	1022	8	1095	92	274	6693
7	87	7	1022	8	1095	115	182	10020
Roof	107	10	1460	10	1460	67	67	7137
							Total base shear=	682 k
							Total overturning Moment=	36276 k

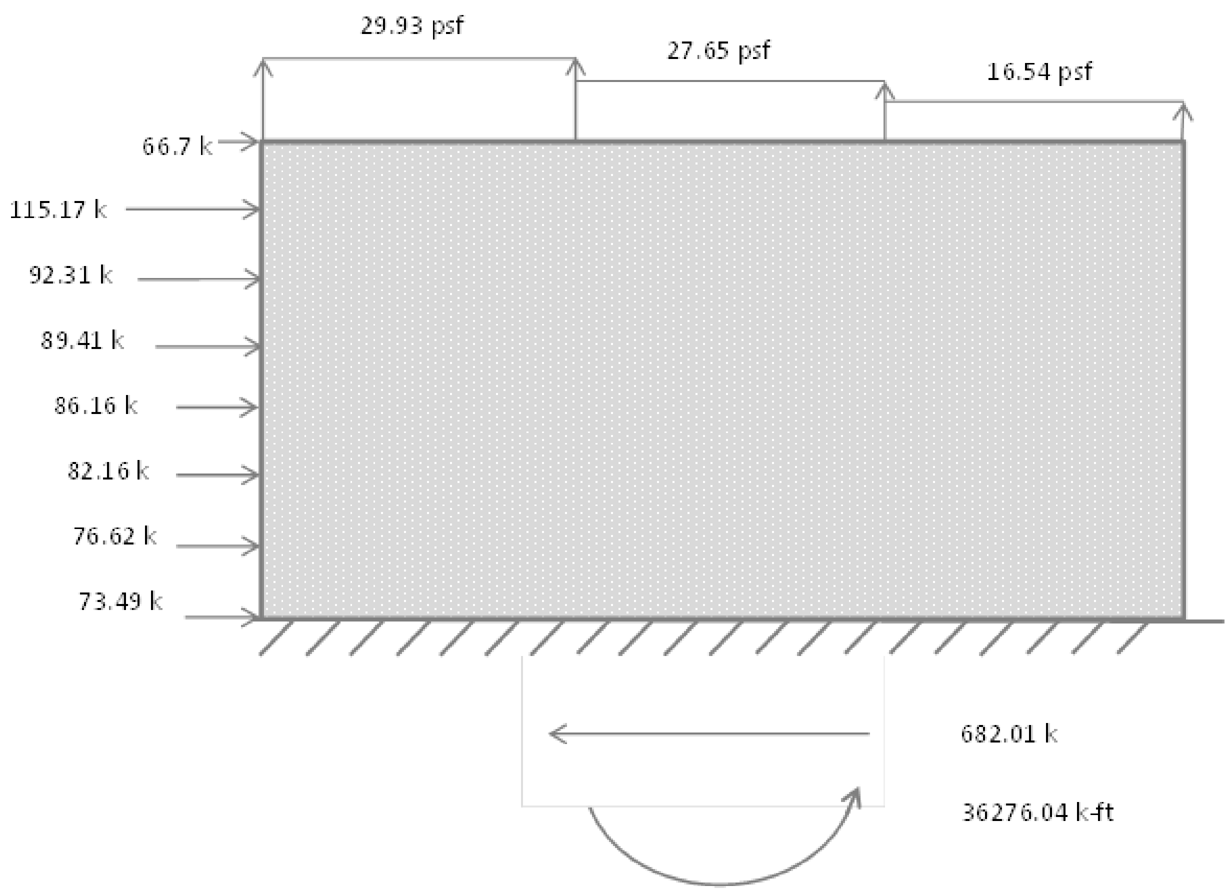


Figure 16 - **List and diagram showing the wind forces on the building in the N-S direction**

Desgin wind pressure for MWFRS in E-W Direction						
type	Level	Height / distance	qz/ qh	Wind pressure (psf)	Net pressure	
					(+)GCPi	(-)GCPi
Windward walls	1	0'	21.0	14.3	-6.1	34.7
	2	14'-6"	21.0	14.3	-6.1	34.7
	3	29'	25.5	17.3	-3.1	37.8
	4	43'-6"	28.7	19.5	-0.9	39.9
	5	58'	31.0	21.1	0.7	41.5
	6	72'-6"	33.2	22.6	2.1	43.0
	7	87'	35.1	23.8	3.4	44.3
	Roof	107'	37.1	25.3	4.8	45.7
Leeward walls	All	All	-16.5	-15.8	-36.2	4.6
Sidewalls	All	All	37.1	-22.1	-42.5	-1.7
Roof		0-53.5'	37.1	-34.2	-54.6	-13.8
		53.5'-107'	37.1	-25.5	-45.9	-5.1
		107'-214'	37.1	-18.7	-39.1	1.7

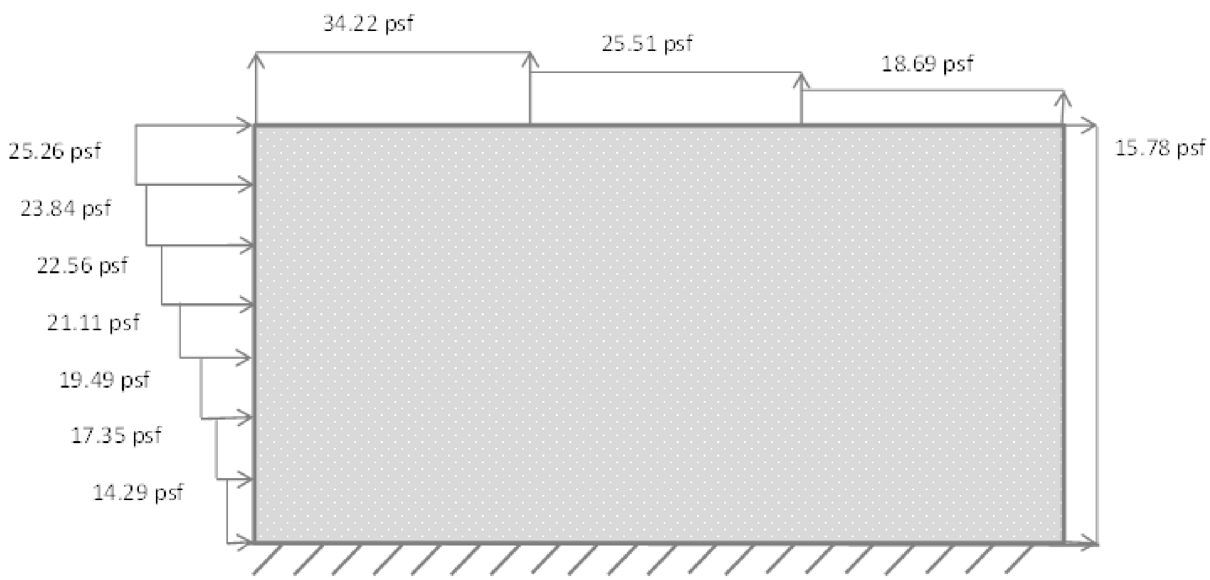


Figure 17 - **List and diagram showing the wind pressure on the building in E-W direction**

`Wind Forces - E-W Direction								
Floor level	Height / distance	Tributary below		Tributary above		Story force (K)	Story Shear (K)	Overturning Moment (k-ft)
		Height (ft)	Area (ft^2)	Height (ft)	Area (ft^2)			
1	0'	N/A	0	8	1433	96	892	0
2	14.5	7.00	1337	8	1433	100	796	1453
3	29	7.00	1337	8	1433	107	696	3117
4	43.5	7.00	1337	8	1433	113	588	4903
5	58	7.00	1337	8	1433	117	476	6784
6	72.5	7.00	1337	8	1433	121	359	8755
7	87	7.00	1337	8	1433	151	238	13108
Roof	107	10.00	1910	10	1910	87	87	9336
Total base shear=								892 k
Total overturning Moment=								47457 k

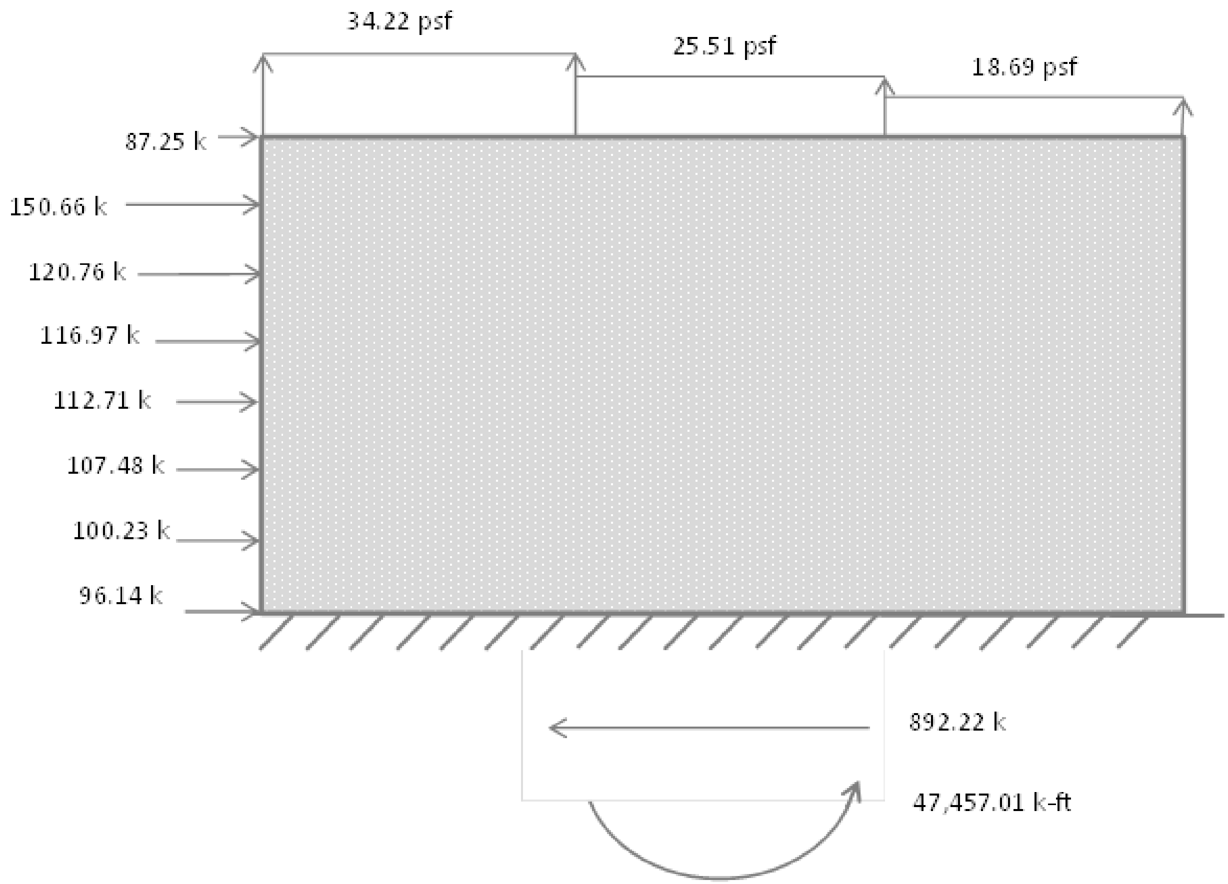


Figure 18 - **List and diagram showing the wind forces on the building in the E-W direction**

Seismic Loads

The engineers who designed this building did not analyze the building for seismic forces as wind always controls in Tampa, Florida. However, Seismic loads were still calculated to check that statement.

Seismic loads were calculated with the Equivalent Lateral Force procedure outlined in Chapters 11 and 12 of ASCE 7-05. This procedure also assumes a simple building footprint. In fact when calculating the weight of the building, 3 sections were considered to simplify the different floor joists system used. Also, an average size of beam of 24”x24”was taken to represent all sizes to simplify the calculations of each weight of the beams.

The loads from seismic forces originate from the inertia of the structure itself, which is related to the mass of the structure. Most of the mass of the structure is locked in the slabs, beams, joists, and columns which are connected to the shear walls. When seismic loads are generated by a ground motion, the slabs transfer the loads directly into the shear walls, which then carry the loads down to the foundations and therefore to grade.

It was assumed that the site is classified as site class E or stiff soil. After calculating the SMs, and S1, the SD1 and SDM were computed which lead to a design category for this structure A. This means that each lateral force at every floor is the weight of the floor multiplied by 0.01.
Seismic forces in the N-S direction are listed and diagramed in Figure 21. The resultant base shear in this direction is 193 k and the overturning moment was 10,819 k-ft. The calculations cannot be compared to those of the engineer’s as no analysis was done.

Furthermore, to follow the ASCE 7-05 and get more accurate loading on the building an accidental moment was computed. In order to compute those moments, a 5% of the building’s length in each direction was taken as eccentricity. Those loads that represent Mzx and Mzy in the load combinations found in Appendix B on pages 49-51 and in figure 14 of the report. In return, the force was multiplied by the eccentricity and a torsional amplification factor, Ax. In fact, that factor is initially assumed to be equal to 1.0 in order to get max and min drifts on each level and recalculate its true value. The maximum and minimum drift per level and Ax were derived according to the figure 12.8-1 from ASCE 7-05 found on figure 19 below.

Seismic Forces - N-S Direction					
Level	Story weight, w_x	height (ft), h_x	Story force (k) Fx=0.01, w_x	Story Shear (k)	Overturning moment (k-ft)
2	2895	15	29	193	420
3	2893	29	29	164	839
4	2893	44	29	135	1258
5	2893	58	29	106	1678
6	2944	73	29	77	2134
7	3133	87	31	48	2726
8	1648	107	16	16	1764
Total=	19299	Base Shear =			193
		Total Overturning moment=			10819

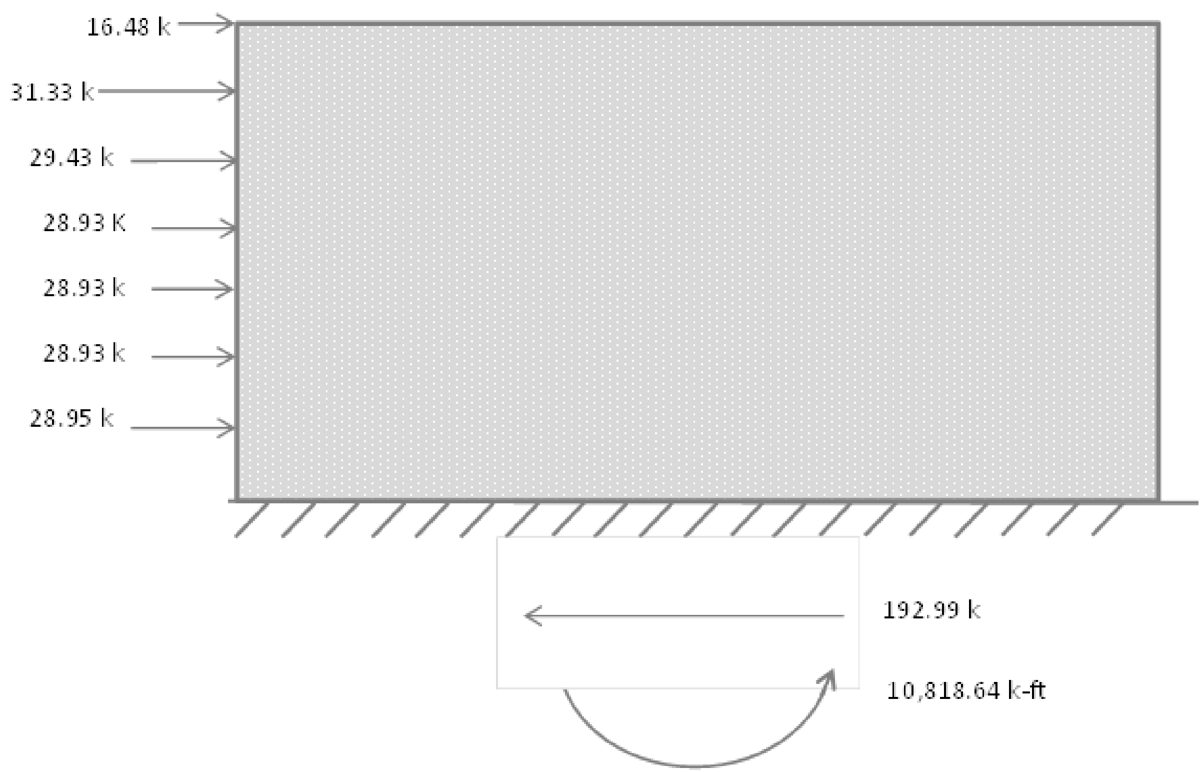


Figure 19 - **List and diagram showing the Seismic forces on the building in the N-S direction**

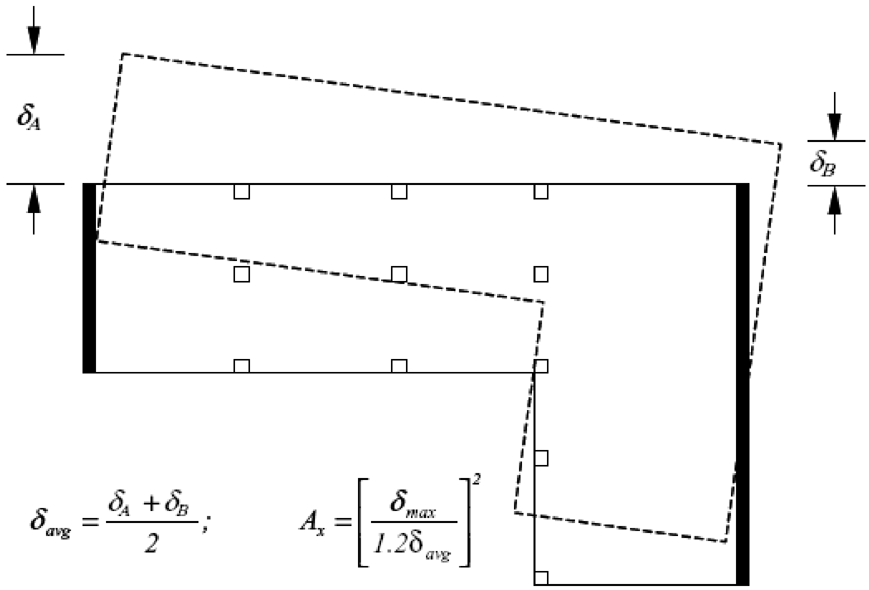


Figure 20 - **Figure showing max/min drift and Torsional Amplification factor, Ax from ASCE 7-05**

After retrieving the true value of Ax, a comparison was made to determine if the building is torsional irregular. Even though, seismic category A does not require this, it was chosen to be completed due to the irregular shape of the building. If Ax was found above 1.2 then it is type 1-a irregular and if Ax is in between 1.2 and 1.4 respectively then it is type 1-b irregular. From table 12.3-1 of ASCE 7-05, type 1-a is torsional irregularity and type 1-b is extreme torsional irregularity. The results came that the building is not torsional irregular in the X-direction however is extreme torsional irregular in the Y-directions. These table and calculations can be found in further details in appendix C.

The story drift was determined according to section 12.8.6 "Story drift determination" in ASCE 7-05. See figure 20.

$$\delta_x = \frac{C_d\,\delta_{xe}}{I} \qquad (12.8\text{-}15)$$

where

C_d = the deflection amplification factor in Table 12.2-1
δ_{xe} = the deflections determined by an elastic analysis
I = the importance factor determined in accordance with Section 11.5.1

Figure 21 - **Story drift determination**

The “I” factor was taken 1.0 and “Cd” was retrieved from table 12.2 -1 as 4 . This amplified the drifts in each direction by 4.0 but it was still under the code allowance of .01hsx. To see in details these calculations please refer to appendix C.

Lateral System Analysis

In order to fully understand the behavior of the Alzheimer Center & Research Institute under lateral loading, an accurate finite element model was built in ETABS. Attempts were made to verify all results using hand calculations and spot checks on specific elements, although this was not always successful due to the complexity of the lateral system. See appendix D for hand calculations.

Computer Modeling Process

Several assumptions were made while creating all of the lateral models that have a significant impact on the final results given by the models. Firstly, it is required by ACI 318-08 section 8.8.2 that stiffness properties be modified to account for concrete cracking. This can be accomplished either by applying different factors to beams and columns, or by applying a sweeping 50% reduction of gross section properties to all concrete elements. For ease of modeling, the first option was chosen. It was achieved by applying a 0.5 factor in the property modifier for moment of inertia about 2 and 3 axes. This was done for each beam and column modeled.

Also, since this is a concrete building, a rigid end offset factor of 0.5 was applied to all beams and columns. This is done as common practice to model the beams from the face of the columns instead of the center. It achieves a more accurate behavior of the building.

Material properties were further modified by eliminating self-mass from the material definitions. In order to better control the results of the modal analysis, the masses were directly assigned using the Additional Area Mass function to the floor areas. Weight, however, was left as self-calculating. See figure below.

Level	weight (Kips)	Area from ETABS (ft^2)	Weight in (K/ft^2)	Mass (kip-in)
2nd	2895	13106	0.221	3.97E-06
3rd	2893	12934	0.224	4.02E-06
4th	2893	12934	0.224	4.02E-06
5th	2893	12934	0.224	4.02E-06
6th	2944	12934	0.228	4.09E-06
7th	3133	12934	0.242	4.35E-06
Roof	1648	7718	0.214	3.84E-06
Total=	19299	85494	1.575	2.83E-05

Roof	Areas	Factor	Mass
Roof A	6993	0.91	3.48E-06
Roof B	232	0.03	1.15E-07
Roof C	493	0.06	2.45E-07
Roof total	7718		

Figure 22 - **Table representing the mass inputted in ETABS**

The next major assumption was to use shell elements rather than membrane to define all shear walls. This choice was made because the model had literally thousands of warnings due to lack of restraint when these elements were modeled as membranes. It is believed that this is related to the fact that several shear walls are on axes which have an oblique angle with respect to the forces applied.

Moreover, to mimic membrane behavior, the elements were given a “Membrane Thickness” equal to their actual thickness and a “Bending Thickness” equal to their actual thickness (i.e. the 12” thick shear walls had a Membrane Thickness of 12”, and a Bending Thickness of 12”). This sufficiently removes the potential for these elements to carry out-of-plane forces while still reducing or eliminating warnings which may render the model less accurate. All shear wall shell elements were meshed into structural elements of a maximum size of 24”, and care was taken to ensure that no portion of the shear wall was divided into less than 2 elements wide or tall. This was important because the program requires at least two elements to calculate both tension and compression in a given bending profile with any degree of accuracy.

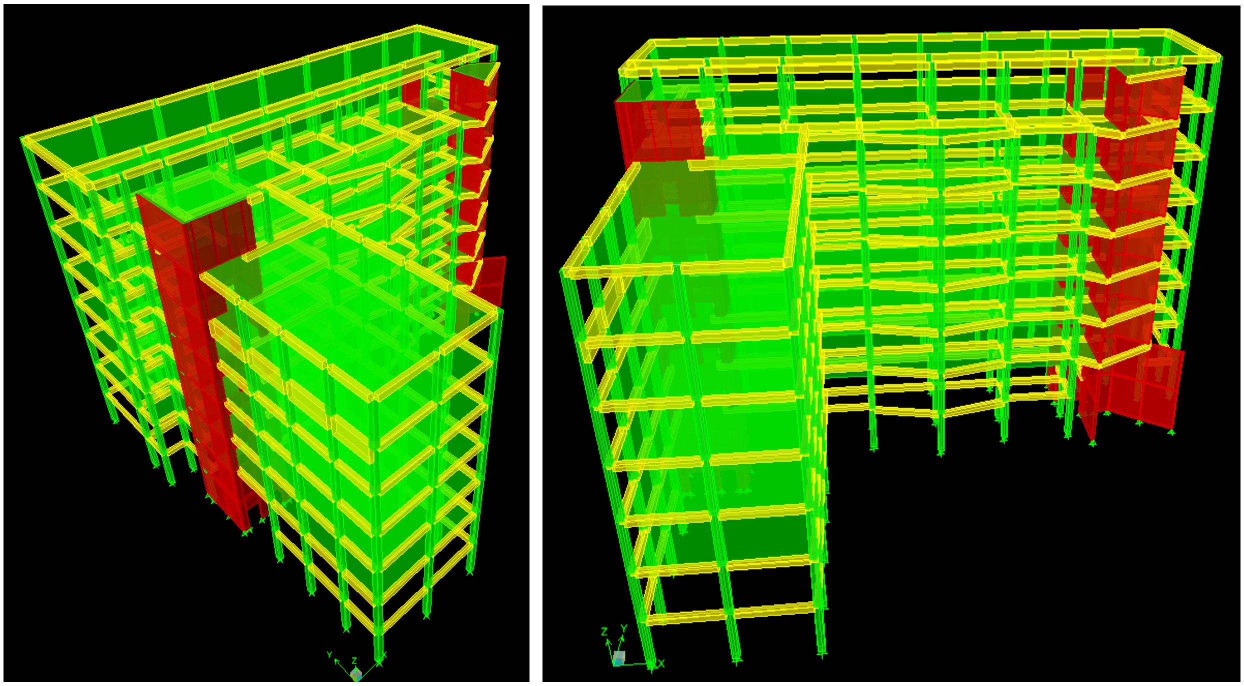


Figure 23 - **3D model of the Alzheimer Center & research Institute**

In addition, the slabs were modeled and assigned as rigid diaphragms. This assumption was made to accurately transfer the loads to the resisting lateral frame representing the true value of stiffness of each frame and wall. This is also done to model the floor as a whole element and be able to displace as a whole. Thus, the rigid diaphragm disregards the stiffness properties of the floor diaphragms, rather considering them rigid bodies, and therefore reports no stresses in the floor diaphragms.

Lastly, although the model was intended only for lateral analysis, it was decided to model all of the beams casted as well. This was primarily driven by the knowledge that they were acting in the lateral system as opposed to the precast joist. This was assumed to put make the answer more accurate however complicating the hand calculations. Thus, a category entitled “other members” was dedicated for those beams that exist and take some lateral force however negligible in hand calculations for simplicity. Also some gravity columns were added in the model. The influence in this decision was the critical nature of the braced frames at the 7th level to the lateral resistance of the Roof. Without the gravity columns spanning from 7th to the 8th Level under the braced frames, the frames were not an accurate representation of the structural behavior.

In total, one accurate model was produced to analyze the AC&RI. In fact, in order to easily and more accurately analyze the building pier labeling for frames was used. The wall pier function was used to easily report forces in the shear walls at all levels. The frame pier function was used to easily report forces in the frame walls at all levels. For shear design, it was important to determine the shear in each individual wall, and therefore each wall was assigned its own individual pier label. The labels given to these walls as well as the pier axes can be seen in Figure 23. Also, the labels of the pier frames are found in figure 24.

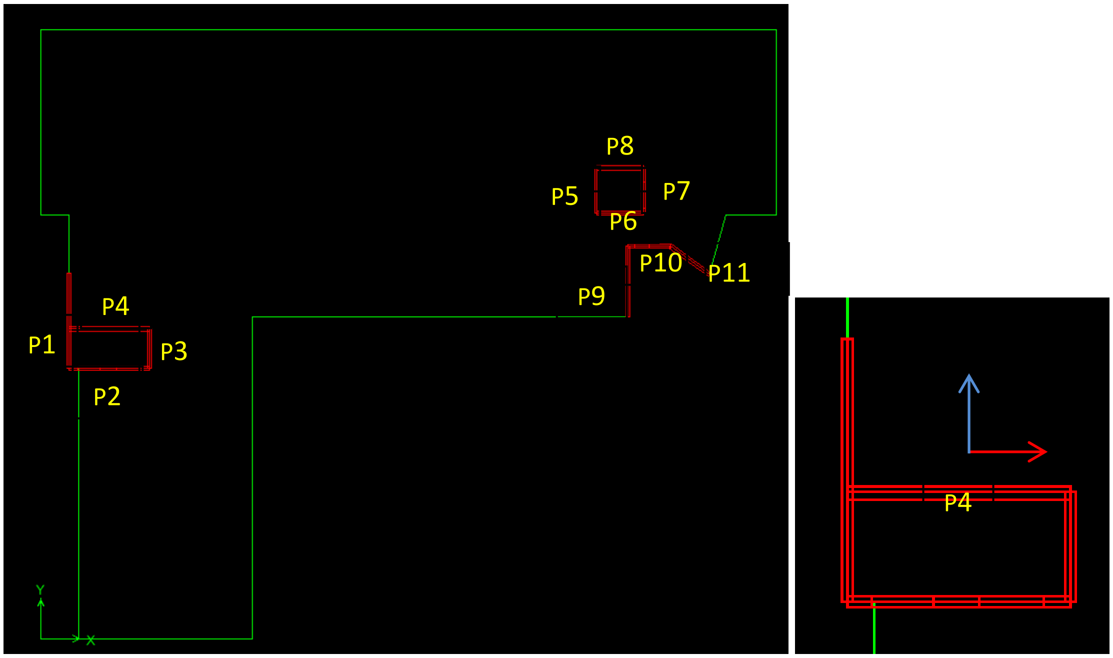


Figure 24 – (a) on the right, represent a floor plan showing shear wall pier labels and (b) on the left represent the axes used to obtain shear results. The red axis corresponds to ETABS' V2 strong axis, and the blue axis corresponds to ETABS' V3 weak axis. The x-y small axis represent (0,0) ft location.

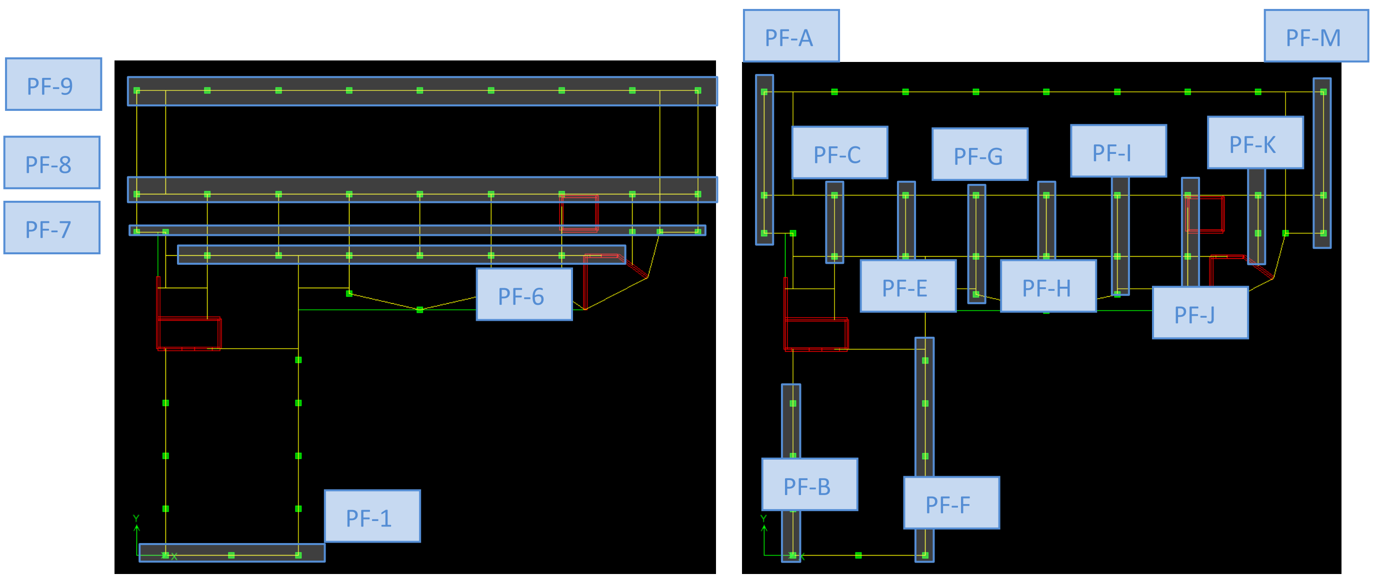


Figure 25- Floor plan 7 representing all the pier frames used to obtain results. (a) on the left represent the x-directions and (b) on the right represent the Y-direction.

Last but not least, looking at the buildings foundations it was assumed that the all the columns and walls to be pinned. This was chosen due to the lack of numerous thick dowels in going in the footings or mat slabs.

Building Properties

In order to produce the most accurate model possible, the center of mass and the center of rigidity were both calculated by hand and then compared to the values given by the rigid diaphragm model in ETABS. The center of mass is the location where all mass could be considered effectively lumped and it would produce a nearly identical effect as the distributed masses of the real building. The center of rigidity is the location at which an applied horizontal load would produce no torsion in a rigid floor diaphragm.

Etabs Results for Center of Mass and Rigidity (ft)						
Floor level	Center of mass		Center of rigidity		Eccenticity (in)	
	XCM	YCM	XCR	YCR	ex	ey
8	83	117	53	81	31	36
7	73	100	53	82	20	18
6	72	96	53	82	18	14
5	71	95	55	82	16	13
4	70	94	56	82	14	12
3	70	94	59	83	11	10
2	70	93	62	86	8	7

Figure 26 – **Summary of ETABS result for Center of Mass and Rigidity**

A hand calculation of the center of mass (CM) and center of rigidity (CR) was done. The CM was found by breaking up the building into representative areas, and then using a spreadsheet to find the weight of each area. The square footage of each area and the individual area centroid locations were found from the drawings. Appendix D shows the area labels used and a typical level's center of mass calculation.

The center of rigidity was calculated using the individual stiffness of the walls. This process proved to be significantly complicated by the fact that the loads were to be applied at axes that were not parallel or perpendicular to all of the walls. When all walls lie on the same two axes as the applied loads, it can be assumed that the walls have no stiffness in out-of-plane bending/shear, and their stiffness for in-plane bending/shear can be found by applying a unit load to the wall and then using the relationship of $P=K\Delta$. Also due to the complexity of the building and the existence of moment frames that are assumed to be heavily dependent on, a simpler method was used.

In fact, a 100Kip load was applied at the top level and was modeled in ETABS to get the corresponding forces in each pier frame and pier wall. In return, whatever load was found in each pier at different level was considered to be its relative stiffness at that story. This is a safe assumption as the model is modeled with rigid diaphragms. The results were used in the hand calculations as stiffness. The results are shown on the next page in figure 27. They were also used to calculate the forces in each pier and verify with ETABS. A discrepancy of 5% was found due to "other members". See appendices B and D.

Relative stiffness of shear walls under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)								
Floor Level	P-2	P-4	P-6	P-8	P-10	P-11	P-11 in X	Total of walls
8	0.0	27.8	5.6	0.0	14.9	13.3	10.8	59.0
7	26.9	7.1	3.2	2.5	12.3	7.8	6.3	58.2
6	31.4	2.8	3.7	1.3	13.7	11.0	8.9	61.8
5	31.5	2.9	3.7	2.7	14.9	12.2	9.9	65.6
4	32.0	3.3	4.5	3.6	17.0	15.1	12.2	72.7
3	28.3	3.3	4.4	1.3	18.8	27.6	22.4	78.5
2	25.1	3.7	5.7	-1.2	13.6	56.7	46.0	92.9
1	25.2	3.7	5.7	-1.2	16.5	56.6	45.9	95.7

Relative stiffness of moment frames under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)						
Floor Level	PF-6	PF-7	PF-8	PF-9	PF-1	Total of frames
8	0.0	15.0	12.0	11.9	0.0	38.9
7	9.7	3.7	11.2	9.1	2.5	36.1
6	8.1	3.5	10.6	8.9	1.8	32.9
5	7.5	3.2	9.6	8.0	1.7	30.0
4	6.4	2.8	8.2	6.8	1.5	25.6
3	5.0	2.3	6.6	5.5	1.4	20.8
2	2.6	0.4	1.2	1.0	0.3	5.6
1	0.3	0.4	1.1	0.8	0.3	2.9

Summary of Relative Stiffness in %				
Floor level	Walls	Frames	Total	Other members*
8	59	39	98	2.1
7	58	36	94	5.7
6	62	33	95	5.3
5	66	30	96	4.4
4	73	26	98	1.7
3	78	21	99	0.8
2	93	6	98	1.5
1	96	3	99	1.4
*:Members ignored in calculations for simplicity				

Figure 27 - **Tables showing relative stiffness of walls and moment frames in order to calculate shear forces by hand as well as CR and CM (see appendices). Also for Y-direction see Appendix B.**

However, the center of rigidity that was calculated by hand was only based on the shear walls. This was done to study the rigidity of the walls compared that of the rest of the building. This was done to demonstrate the reliability of the building's lateral system on shear walls, versus moment frames. It was found that the lateral system is almost equally dependent on both shear walls and moment frames to about 57% to 43% correspondingly. Thus, the assumption to use the simple method calculation for relative stiffness was verified. A sample calculation of CM and CR can be found in Appendix D.

Typically, this stiffness data could also be used to replicate the wall shears found in ETABS using a proportional distribution of direct and torsion-induced shear according to the following equations.

$$V_{Direct,i} = \frac{K_i}{\Sigma K_i} V$$

$$V_{Torsion,i} = \frac{e K_i d_i}{J} V$$

Where e is the eccentricity with respect to the center of mass at which the story shear (V) will be applied and d is the distance to the line of resistance where wall "i" is located. J can be found by summing the product of the stiffness of a wall and its d-value squared. This calculation was attempted to the worst case in the Y-direction and X-direction. The forces found were similar to those found in ETABS, a discrepancy of 0.13% only. This was likely due to the numerous frames modeled and taken in consideration in the hand calculations. However, the center of rigidity of the building was not able to be replicated within a reasonable margin of error since only the shear walls were taken into consideration. See Appendix D Center of Rigidity/Center of Mass.

Upon verifying the model was approximately accurate, modal information was gathered. It was found that the period of mode shape 1 was 1.64 sec. This is much greater than that of the code estimation using the formula N/20 for shear walls where N= number of stories. See table below for the comparison of the rest of the mode shapes. (figure 28)

Mode	Period	N/20
1	1.64	0.4
2	1.38	0.4
3	1.02	0.4
4	0.35	0.4
5	0.34	0.4
6	0.22	0.4
7	0.16	0.4
8	0.15	0.4
9	0.10	0.4
10	0.09	0.4
11	0.09	0.4
12	0.07	0.4

Figure 28 - **Table showing the 12 mode shapes and their periods compared to the N/20**

Results

Upon completing the models and verifying their accuracy, maximum shear, moment and drift values were pulled from ETABS for the rigid model. It was later verified by hand for column I-8 and shear wall P9, the most stressed wall in the worst case wind loading. The use of "sp-column" was used for further verification of the hand calculations. The hand calculations related to these capacity checks can be found in Appendix D.

As mentioned before no factors were added to the loaded as drift are done for serviceability (a factor of 1.0). However the following table shows the load combination for strength.

<table>
<tr><td></td><td colspan="5">Strength using a factor of 1.6 Wind and 1.0 for Earthquake</td></tr>
<tr><td></td><td colspan="3">Load combinations</td><td colspan="2">Legend</td></tr>
<tr><td rowspan="7">x1.6</td><td rowspan="7">Wind (total of 11 cases)</td><td rowspan="2">Case 1 (2)</td><td>$P_{Wx}+P_{Lx}$</td><td rowspan="2">Eccentricity</td><td>$e_x = \pm 0.15B_x$</td></tr>
<tr><td>$P_{Wy}+P_{Ly}$</td><td>$e_y = \pm 0.15B_x$</td></tr>
<tr><td rowspan="2">Case 2 (4)</td><td>$.75P_{Wx}+.75P_{Lx} \pm M_T$</td><td rowspan="2">where M_T=</td><td>$0.75(P_{Wx}+P_{Lx})B_x e_x$</td></tr>
<tr><td>$.75P_{Wy}+.75P_{Ly} \pm M_T$</td><td>$0.75(P_{Wy}+P_{Ly})B_y e_y$</td></tr>
<tr><td>Case 3 (1)</td><td>$.75\ (P_{Wx}+P_{Lx}) + .75\ (P_{Wy}+P_{Ly})$</td><td colspan="2">Bx= width of building in x-direction
By= width of building in y-direction</td></tr>
<tr><td rowspan="2">Case 4 (4)</td><td rowspan="2">$.563\ (P_{Wx}+P_{Lx}) + .75\ (P_{Wy}+P_{Ly}) + M_T$</td><td rowspan="2">where M_T=</td><td>$\pm 0.563(P_{Wx}+P_{Lx})B_x e_x \pm$</td></tr>
<tr><td>$0.563(P_{Wy}+P_{Ly})B_y e_y$</td></tr>
<tr><td rowspan="2">x 1.0</td><td rowspan="2">Earthquake (total of 4)</td><td>Case 1 (2)</td><td>$1.0\ E_x$</td><td></td><td></td></tr>
<tr><td>Case 2 (2)</td><td>$1.0\ E_y$</td><td></td><td></td></tr>
</table>

Figure 29- **table showing the combinations that need to be multiplied by the corresponding factors**

Earthquake Drifts

In order to determine the earthquake drift a 1.0 factor was used to all the forces however added to it was the accidental moment shown in figure 29. It can be seen that neither the inter-story drift nor the overall drift as well as the amplified drift multiplied by Cd, the deflection amplification factor were well under the code allowance of 0.1hsx.

Earthquake Loads X-direction			Earthquake drift		Earthquake interstory drift		Code allows .01h_{sx}	
Story level	Ex (k)	Mzx (k-ft)	δx	δy	Δx	Δy		
2	29	277	0.06	0.02	0.02	0.01	1.74	**Ok**
3	28.9	276	0.12	0.02	0.06	0.00	1.74	**Ok**
4	28.9	276	0.19	0.02	0.07	0.00	1.74	**Ok**
5	28.9	276	0.26	0.01	0.07	0.00	1.74	**Ok**
6	29.4	281	0.33	0.01	0.07	0.00	1.74	**Ok**
7	31.3	299	0.40	0.00	0.07	0.00	1.74	**Ok**
8	16.5	157	0.46	0.00	0.06	0.00	1.74	**Ok**
Max drift is in X= .46 in							1.17	**Ok**

Earthquake Loads Y-direction			Earthquake drift		Earthquake interstory drift		Code allows .01h_{sx}	
Story level	Ey (k)	Mzy (k-ft)	δx	δy	Δx	Δy		
2	29	231	-0.01	0.14	0.00	0.08	1.74	**Ok**
3	28.9	231	-0.02	0.29	-0.01	0.15	1.74	**Ok**
4	28.9	273	-0.04	0.45	-0.02	0.16	1.74	**Ok**
5	28.9	273	-0.06	0.70	-0.02	0.25	1.74	**Ok**
6	29.4	277	-0.08	0.79	-0.02	0.09	1.74	**Ok**
7	31.3	295	-0.10	0.93	-0.02	0.14	1.74	**Ok**
8	16.5	143	-0.12	1.17	-0.02	0.24	1.74	**Ok**
Max drift is in Y= 1.17 in							1.74	**Ok**
multiply max inter story drift 0.24 in by Cd= 4 for the frame of building thus max drift is Y=1.05 in							1.74	**Ok**

Figure 30 - **Tables showing drifts taken from ETABS due to earthquake loading**

The final results at the top level were a maximum drift of 0.46 inches in the X-direction and a maximum drift of 1.17 inches in the Y-direction. This is less than the wind drifts shown in figure 31. These drifts were caused by the seismic forces in figure 30. They are still the same as shown in the seismic loading section however the new accidental moments are shown above in this table as Mzy (k-ft) where Ax was different.

Wind Drifts

Relative displacements and drifts as found in ETABS for the rigid due to wind loading are summarized in tables in Appendix B. These drifts were compared to the typical allowable drift value of H/400 according to ASCE 7-05. Shown below in figure 31 are the two worst cases in the X-direction and the Y-direction separately. This was established through finding the worst inter-story drift which in return yielded the maximum drift at the top level. In fact, case 1 controlled in the X-direction as opposed to case 2 that controlled to the Y-direction. This was expected due the L-shape geometry of the building and the extreme torsional irregularity in that direction.

Wind Forces- Case 1 Loading: $P_{Wx}+P_{Lx}$							
Floor level	Load (K)	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
		δx	δy	Δx	Δy		
2	76.6	0.11	0.00	0.05	0.01	0.44	**Ok**
3	82.2	0.23	-0.02	0.12	-0.03	0.44	**Ok**
4	86.2	0.36	-0.06	0.14	-0.04	0.44	**Ok**
5	89.4	0.51	-0.11	0.14	-0.05	0.44	**Ok**
6	92.3	0.65	-0.15	0.14	-0.04	0.44	**Ok**
7	115.2	0.79	-0.19	0.14	-0.04	0.44	**Ok**
8	66.7	0.92	-0.23	0.13	-0.03	0.44	**Ok**
Overall Worst drift in X = .92 in						3.48	**Ok**

Wind Forces- Case 2 Loading: .75 ($P_{Wy}+P_{Ly}$) + M_T								
Floor level	Load (K)	M_T	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	75.2	1883	-0.03	0.28	0.00	0.00	0.44	**Ok**
3	80.6	2019	-0.08	0.55	-0.05	0.28	0.44	**Ok**
4	84.5	2118	-0.14	0.88	-0.06	0.32	0.44	**Ok**
5	87.7	2198	-0.21	1.22	-0.07	0.34	0.44	**Ok**
6	90.6	2269	-0.28	1.56	-0.07	0.34	0.44	**Ok**
7	113.0	2831	-0.34	1.88	-0.06	0.32	0.44	**Ok**
8	65.4	1639	-0.40	2.19	-0.06	0.31	0.44	**Ok**
where MT=	0.75(PWy+PLy)Byey			and	y= + 0.15By		and By=	167
Overall Worst drift in Y= 2.2 in							3.48	**OK**

Figure 31 - **Tables showing maximum story drifts and inter-story drift in X and Y direction under worst case for each**

The final results at the top level were a maximum drift of 0.92 inches in the X-direction and a maximum drift of 2.2 inches in the Y-direction. This about2 times more than the seismic drifts shown in figure 30. This is expected as the wind forces were greater than seismic by a factor of about 3.6 in the East-West direction and 2.5 in the North-South direction. Thus, before doing load combinations wind controls in both directions as inter-story drifts are greater under wind and are highlighted in Red in the table above. The drifts and displacements for the 9 other cases were computed and are shown in Appendix B. As it can be seen from the above tables and the appendices, all story drift and total drift values were well within the allowable limits. Forces were distributed according to their relative stiffness and are shown in Appendix B.

Spot checks

Spot checks were done on column I-8 and shear wall P-9 at level 6 under the worst factored wind case in each direction. This was done due represent the heaviest loading on those elements and check their strength. The both were found capable of carrying the loads. For details see Appendix D.

Conclusion

After a comprehensive analysis, the lateral system of the Alzheimer Center & Research Institute was found to be adequate to carry the loads that will experience. This conclusion is based upon both hand calculations and finite element computer model analyses using ETABS were conducted for this technical report. The wind forces were found using the Main Wind Force Resisting System method, and the seismic forces were found first with the Equivalent Lateral Force method. The finite element models, constructed in ETABS, had been found to be fairly accurate to the hand calculations done.

Eleven wind load combinations and 4 Earthquake combinations were taken from ASCE 7-05 and considered to determine the controlling load case for this building. The assumption taken from technical report 1 that wind controlled in each direction was confirmed this technical report. In each direction, wind controlled over seismic by a factor of 2. Thus, there was no need to check for the many load combinations to be used as wind would have magnified 1.6 times more than earthquake. However, the factored wind was computed to retrieve the maximum forces in the members of the lateral system.

One fully-inclusive model was built to cover the structural behavior of the building. It was a rigid diaphragm model, in which the shear walls were first individually assigned to piers for ease of reporting shear forces, and then frames were assigned in groups of piers for ease of reporting moments and shear.

Upon completion of the model, shear and moment demands for both models were compared to the hand calculated forces using the traditional lateral force distribution methods. This was done by applying a 100 K at the top of the building to get relative stiffness of each pier (walls and moment frames). In return, the stiffnesses were multiplied by the worst force that the corresponding level and pier will see under extreme loading. Spot checks were then computed to confirm the adequacy of the members.

Two spot checks were calculated: one on a middle column located on I-8 and another for the shear wall P-9. They were considered in extreme X-loading and Y-Loading to their respect resisting direction. They were both found to be sufficient and meet all ACI 318-08 requirements.

Finally, the drift analysis included a strength check of the controlling wind load combination, and a serviceability check of the wind forces acting on the building. Seismic drift values were obtained from the ETABS model and were checked against the allowable story drift and total drift of $0.010h_{sx}$. The wind load drifts were also acquired from ETABS, but they were evaluated against the limit of H/400. All story drift and total drift values were within the allowable limits.

Through hand checks and computer models, The Alzheimer Center & Research Institute was proven to be adequate for strength and serviceability requirements as expected.

Appendices

Appendix A: Typical Plans

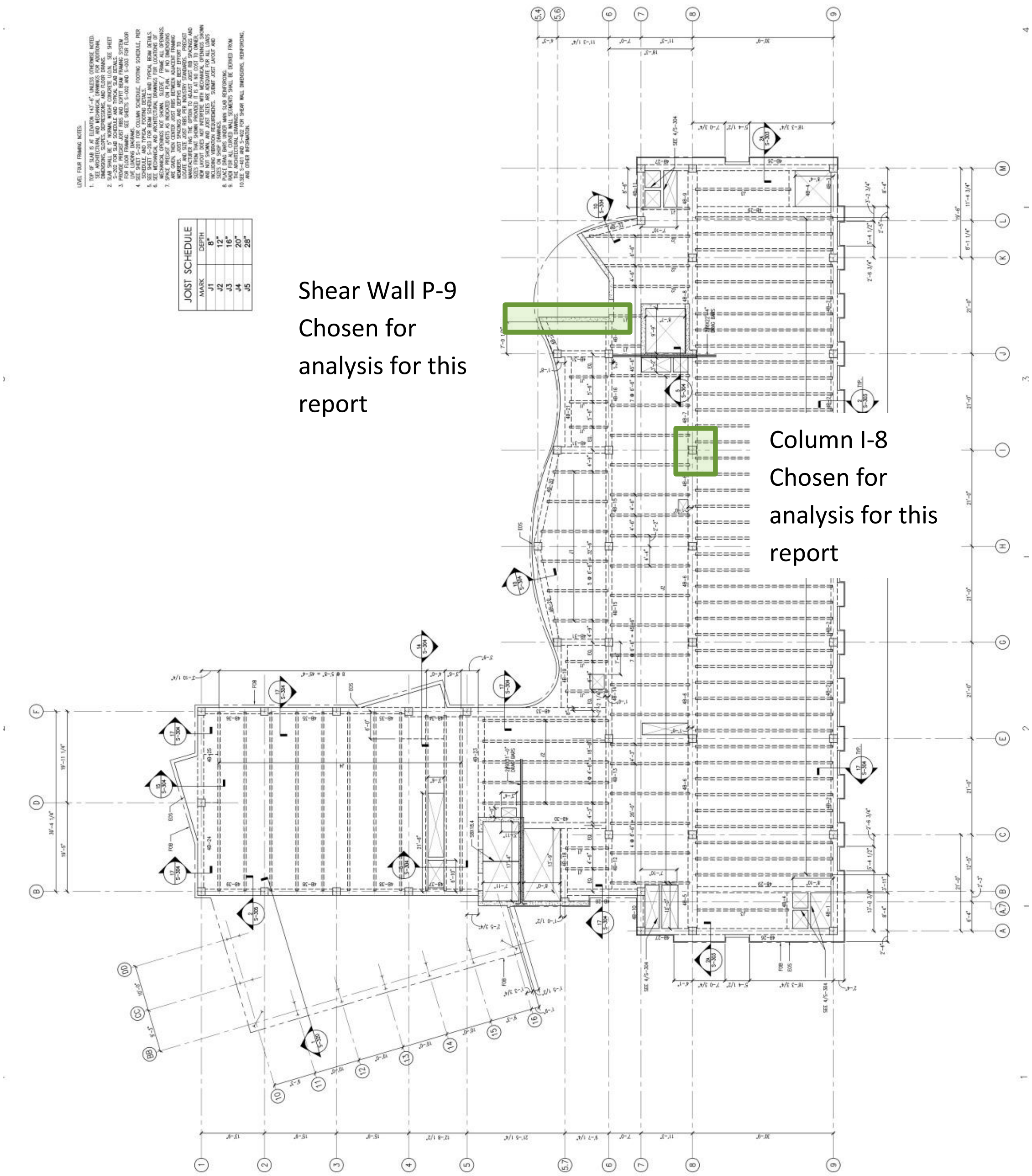


Figure 32 - **Typical floor plan taken from S-104 N-S**

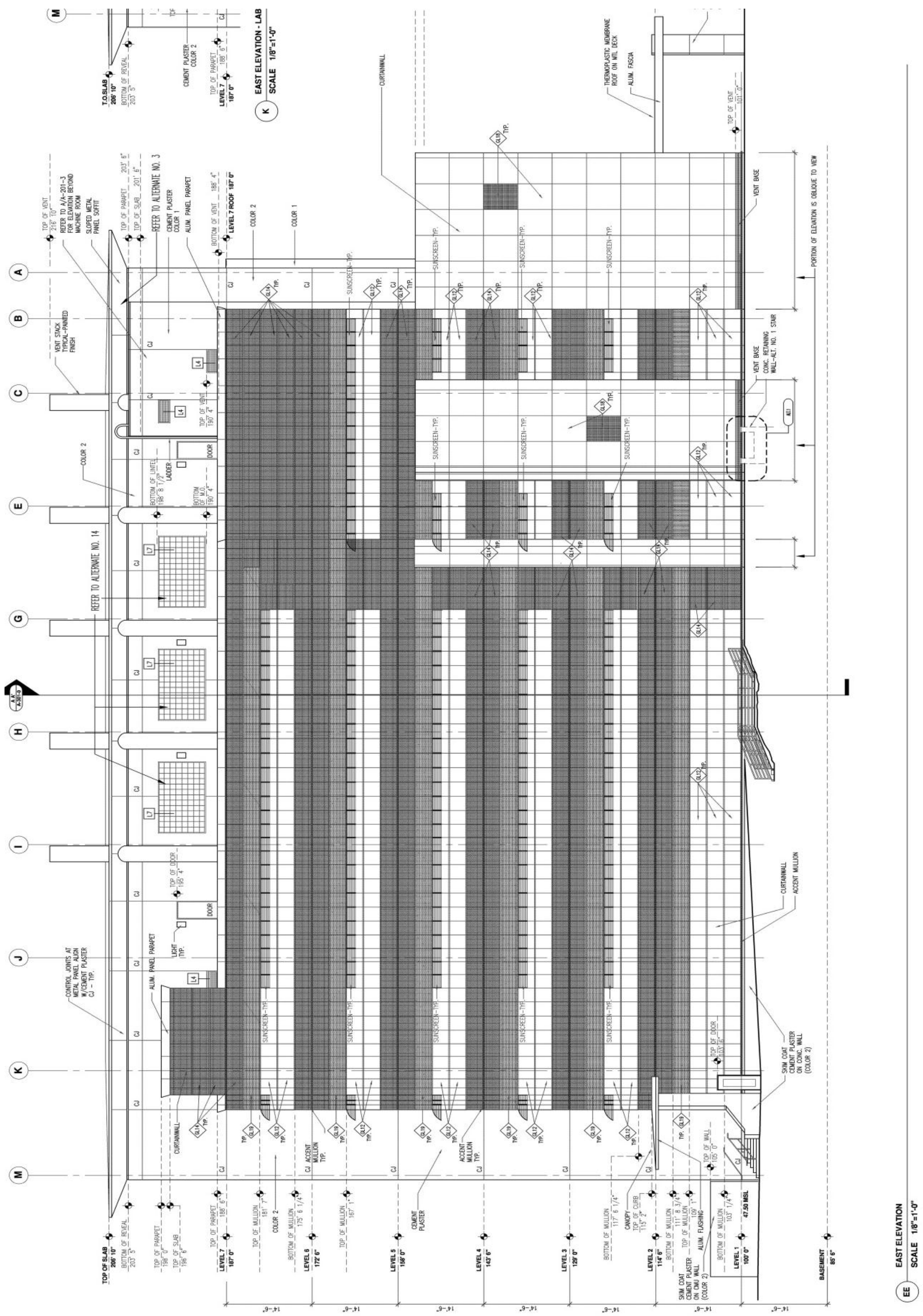


Figure 33 - Elevation of the building showing the different floor heights from A -201- 0

Appendix B: Wind Load Calculations

HDR | ONE COMPANY *Many Solutions*

Project: ALZHEIMER'S	Computed: MP	Date: 12/2/7	
Subject: DESIGN LOADS	Checked:	Date:	
Task: WIND LOAD	Page:	of:	
Job #: 06400	No:		

MWFRS

$p = q\,GC_p - q_i\,(GC_{pi})$

$G = .85$

L/B N-S $= 170/143 = 1.19$

L/B E-W $= 143/170 = 0.84$

C_p WINDWARD $= .8$

LEEWARD N-S $= -.3$

LEEWARD E-W $= -.5$

SIDE WALL $= -.7$

$GC_{pi} = \pm .55$

THIS PRESSURE IS THE INTERNAL PRESSURE AND THE OVERALL TOTAL LAT LOAD IS LOOKED AT, IE WHEN WINDWARD + LEEWARD IS ADDED TOGETHER, THIS PRESSURE CANCELS EACH OTHER OUT IN THE END

WINDWARD N-S & E-W

$P_{0-15'} = 17.9(.85)(.8) - 26.6(\pm.55) = 12.2$ PSF ± 14.6 PSF

15-20 =	19.4	= 13.2	± 14.6
20-25 =	20.7	= 14.1	± 14.6
25-30 =	21.9	= 14.9	± 14.6
30-40 =	23.8	= 16.2	± 14.6
40-50 =	25.4	= 17.3	± 14.6
50-60 =	26.6	= 18.1	± 14.6
60-70 =	27.9	= 19.0	± 14.6
70-80 =	29.1	= 19.8	± 14.6
80-90 =	30.1	= 20.5	± 14.6
90-100 =	31.0	= 21.1	± 14.6
100-120' =	32.6	= 22.2	± 14.6

LEEWARD N-S

$p = \cancel{31.0}\ 32.6\,(.85)(-.3) - 26.6(\pm.55)$ $= \cancel{-7.9\text{ PSF}}\ -8.3$ PSF ± 14.6 PSF

LEEWARD E-W

$p = \cancel{31.0}\ 32.6\,(.85)(-.5) - 26.6(\pm.55)$ $= \cancel{-13.2\text{ PSF}}\ -13.9$ PSF ± 14.6 PSF

SIDE WALL

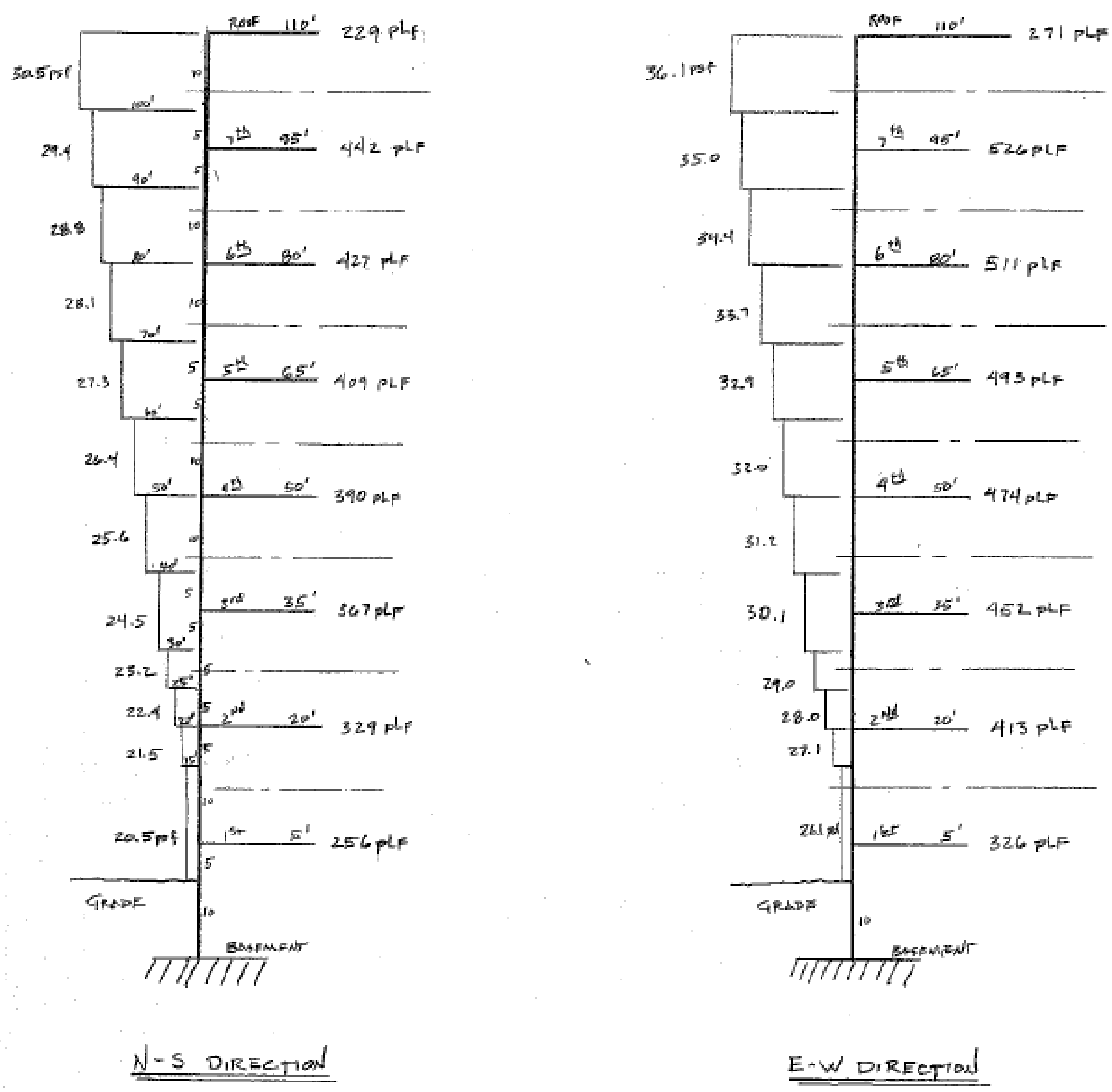
$p = \cancel{31.0}\ 32.6\,(.85)(-.7) - 26.6(\pm.55)$ $= \cancel{-18.4\text{ PSF}}\ -19.4$ PSF ± 14.6 PSF

HDR Computation

Project: ALZHEIMER'S CLINIC — Computed: BGD — Date: 5/5/04

Subject: LATERAL DESIGN LOADS / DISTRIBUTION — Checked — Date

Task: WIND PROFILE — Sheet — Of



This is including leeward pressure

Drifts for 11 wind cases:

Wind Forces- Case 1 Loading: $P_{Wx}+P_{Lx}$							
Floor level	Load (K)	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
		δx	δy	Δx	Δy		
2	76.6	0.11	0.00	0.05	0.01	0.44	**Ok**
3	82.2	0.23	-0.02	0.12	-0.03	0.44	**Ok**
4	86.2	0.36	-0.06	0.14	-0.04	0.44	**Ok**
5	89.4	0.51	-0.11	0.14	-0.05	0.44	**Ok**
6	92.3	0.65	-0.15	0.14	-0.04	0.44	**Ok**
7	115.2	0.79	-0.19	0.14	-0.04	0.44	**Ok**
8	66.7	0.92	-0.23	0.13	-0.03	0.44	**Ok**
Overall Worst drift in X = .92 in						3.48	**Ok**

Wind Forces- Case 1 Loading: $P_{Wy}+PL_y$							
Floor level	Load (K)	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
		δx	δy	Δx	Δy		
2	100.2	0.01	0.26	0.00	0.12	0.44	**Ok**
3	107.5	0.00	0.52	-0.01	0.26	0.44	**Ok**
4	112.7	-0.02	0.83	-0.02	0.31	0.44	**Ok**
5	117.0	-0.05	1.15	-0.03	0.33	0.44	**Ok**
6	120.8	-0.08	1.48	-0.03	0.33	0.44	**Ok**
7	150.7	-0.11	1.79	-0.03	0.31	0.44	**Ok**
8	87.3	-0.13	2.10	-0.03	0.30	0.44	**Ok**
Max drift is in Y= 2.1 in						3.48	**Ok**

Wind Forces- Case 2 Loading: .75 ($P_{Wx}+P_{Lx}$) + M_T								
Floor level	Load (K)	M_T	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	57.5	1250	0.06	0.06	0.02	0.02	0.44	**Ok**
3	61.6	1340	0.12	0.09	0.06	0.03	0.44	**Ok**
4	64.6	1405	0.19	0.12	0.07	0.03	0.44	**Ok**
5	67.1	1459	0.27	0.15	0.08	0.03	0.44	**Ok**
6	69.2	1506	0.34	0.18	0.08	0.03	0.44	**Ok**
7	86.4	1879	0.42	0.21	0.07	0.03	0.44	**Ok**
8	50.0	1088	0.49	0.24	0.07	0.03	0.44	**Ok**
where MT=	0.75(PWx+PLx)Bxex		and	ex= +0.15Bx			and Bx=	145'
Max drift is in X= .49 in							3.48	**Ok**

Wind Forces- Case 2 Loading: .75 (P_{Wy}+P_{Ly}) + M_T								
Floor level	Load (K)	M_T	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	75.2	1883	-0.03	0.28	0.00	0.00	0.44	**Ok**
3	80.6	2019	-0.08	0.55	-0.05	0.28	0.44	**Ok**
4	84.5	2118	-0.14	0.88	-0.06	0.32	0.44	**Ok**
5	87.7	2198	-0.21	1.22	-0.07	0.34	0.44	**Ok**
6	90.6	2269	-0.28	1.56	-0.07	0.34	0.44	**Ok**
7	113.0	2831	-0.34	1.88	-0.06	0.32	0.44	**Ok**
8	65.4	1639	-0.40	2.19	-0.06	0.31	0.44	**Ok**
where MT=	0.75(PWy+PLy)Byey			and	y= + 0.15By		and By=	167
Overall Worst drift in Y= 2.2 in							3.48	**OK**

Wind Forces- Case 2 Loading: .75 (P_{Wx}+P_{Lx}) - M_T								
Floor level	Load (K)	MT	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	57.5	-1250	0.11	-0.05	0.00	0.00	0.44	**Ok**
3	61.6	-1340	0.22	-0.13	0.11	-0.07	0.44	**Ok**
4	64.6	-1405	0.36	-0.22	0.13	-0.09	0.44	**Ok**
5	67.1	-1459	0.50	-0.32	0.14	-0.10	0.44	**Ok**
6	69.2	-1506	0.63	-0.41	0.14	-0.10	0.44	**Ok**
7	86.4	-1879	0.77	-0.50	0.13	-0.09	0.44	**Ok**
8	50.0	-1088	0.89	-0.58	0.12	-0.08	0.44	**Ok**
where MT=	0.75(PWx+PLx)Bxex			and	ex=- 0.15Bx		and Bx=	145'
Max drift is in X= 0.89 in							3.48	**Ok**

Wind Forces- Case 2 Loading: .75 (P_{Wy}+P_{Ly}) - M_T								
Floor level	Load (K)	M_T	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	75.2	-1883	0.05	0.18	0.02	0.09	0.44	**Ok**
3	80.6	-2019	0.08	0.33	0.03	0.15	0.44	**Ok**
4	84.5	-2118	0.11	0.50	0.03	0.17	0.44	**Ok**
5	87.7	-2198	0.14	0.68	0.03	0.18	0.44	**Ok**
6	90.6	-2269	0.16	0.86	0.02	0.17	0.44	**Ok**
7	113.0	-2831	0.18	1.02	0.02	0.17	0.44	**Ok**
8	65.4	-1639	0.20	1.19	0.02	0.17	0.44	**Ok**
where MT=	0.75(PWy+PLy)Byey			and	y= + 0.15By		and By=	167
Max drift in Y= 1.19 in							3.48	**Ok**

Wind Forces- Case 3 Loading: .75 (P_{Wx}+P_{Lx}) + .75 (P_{wy}+P_{Ly})								
Floor level	Load in X (K)	Load in Y (K)	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
			δx	δy	Δx	Δy		
2	57.5	75.2	0.11	0.14	0.04	0.07	0.44	**Ok**
3	61.6	80.6	0.21	0.27	0.10	0.13	0.44	**Ok**
4	64.6	84.5	0.33	0.41	0.12	0.14	0.44	**Ok**
5	67.1	87.7	0.45	0.56	0.12	0.15	0.44	**Ok**
6	69.2	90.6	0.57	0.70	0.12	0.14	0.44	**Ok**
7	86.4	113.0	0.69	0.83	0.12	0.14	0.44	**Ok**
8	50.0	65.4	0.80	0.97	0.11	0.13	0.44	**Ok**
where MT=	0.75(PWx+PLx)Bxex		and	ex=- 0.15Bx			and Bx=	145'
Max drift is in Y= .97 in							3.48	**Ok**

Wind Forces- Case 4 Loading: 0.563 (PWx+PLx) + 0.563 (PWy+PLy) + MT											
Floor level	Load in X (K)	Load in Y (K)	MTx (+)	Mty (+)	MT	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
						δx	δy	Δx	Δy		
2	43.1	56.4	938	1414	2352	0.02	0.25	0.01	0.12	0.44	**Ok**
3	46.3	60.5	1006	1516	2522	0.03	0.48	0.01	0.23	0.44	**Ok**
4	48.5	63.5	1055	1590	2645	0.04	0.75	0.01	0.27	0.44	**Ok**
5	50.3	65.9	1095	1650	2745	0.04	1.03	0.01	0.28	0.44	**Ok**
6	52.0	68.0	1130	1703	2834	0.05	1.31	0.01	0.28	0.44	**Ok**
7	64.8	84.8	1410	2125	3535	0.06	1.57	0.01	0.26	0.44	**Ok**
8	37.6	49.1	817	1231	2047	0.06	1.82	0.01	0.25	0.44	**Ok**
where MT=		$0.563(P_{Wx}+P_{Lx})B_xe_x$ $+0.563(P_{Wy}+P_{Ly})B_ye_y$			and	ex=+ 0.15Bx ey=+ 0.15By			and	Bx= By=	145' 167'
Max drift is in Y= 1.82 in										3.48	Ok

Wind Forces- Case 4 Loading: 0.563 (PWx+PLx) + 0.563 (PWy+PLy) ± MT											
Floor level	Load in X (K)	Load in Y (K)	MTx (+)	Mty (-)	MT	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
						δx	δy	Δx	Δy		
2	43.1	56.4	938	-1414	-475	0.08	0.13	0.03	0.07	0.44	**Ok**
3	46.3	60.5	1006	-1516	-510	0.15	0.25	0.07	0.12	0.44	**Ok**
4	48.5	63.5	1055	-1590	-535	0.23	0.39	0.08	0.13	0.44	**Ok**
5	50.3	65.9	1095	-1650	-555	0.30	0.52	0.08	0.14	0.44	**Ok**
6	52.0	68.0	1130	-1703	-573	0.38	0.66	0.07	0.13	0.44	**Ok**
7	64.8	84.8	1410	-2125	-715	0.45	0.79	0.07	0.13	0.44	**Ok**
8	37.6	49.1	817	-1231	-414	0.52	0.91	0.07	0.13	0.44	**Ok**
where MT=		$0.563(P_{Wx}+P_{Lx})B_xe_x$ $+0.563(P_{Wy}+P_{Ly})B_ye_y$			and	ex=+ 0.15Bx ey=- 0.15By			and	Bx= By=	145' 167'
Max Drift is in Y= .91 in										3.48	Ok

Wind Forces- Case 4 Loading: 0.563 (PWx+PLx) + 0.563 (PWy+PLy) ± MT											
Floor level	Load in X (K)	Load in Y (K)	MTx (-)	Mty (+)	MT	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
						δx	δy	Δx	Δy		
2	43.1	56.4	-938	1414	475	0.06	0.17	0.02	0.08	0.44	**Ok**
3	46.3	60.5	-1006	1516	510	0.11	0.32	0.05	0.15	0.44	**Ok**
4	48.5	63.5	-1055	1590	535	0.16	0.49	0.05	0.17	0.44	**Ok**
5	50.3	65.9	-1095	1650	555	0.22	0.68	0.05	0.18	0.44	**Ok**
6	52.0	68.0	-1130	1703	573	0.27	0.86	0.05	0.18	0.44	**Ok**
7	64.8	84.8	-1410	2125	715	0.32	1.04	0.05	0.17	0.44	**Ok**
8	37.6	49.1	-817	1231	414	0.36	1.21	0.05	0.17	0.44	**Ok**
where MT=		$0.563(P_{Wx}+P_{Lx})B_xe_x$ $+0.563(P_{Wy}+P_{Ly})B_ye_y$			and	ex=- 0.15Bx ey=+ 0.15By			and	Bx= By=	145' 167'
Max Drift is in Y= 1.21 in										3.48	**Ok**

Wind Forces- Case 4 Loading: 0.563 (PWx+PLx) + 0.563 (PWy+PLy) - MT											
Floor level	Load in X (K)	Load in Y (K)	MTx (-)	Mty (-)	MT	Wind drift		Wind Interstory drift		Code allows L/400 for max drift	
						δx	δy	Δx	Δy		
2	43.1	56.4	-938	-1414	-2352	0.12	0.18	0.05	0.09	0.44	**Ok**
3	46.3	60.5	-1006	-1516	-2522	0.23	0.34	0.11	0.16	0.44	**Ok**
4	48.5	63.5	-1055	-1590	-2645	0.35	0.51	0.12	0.18	0.44	**Ok**
5	50.3	65.9	-1095	-1650	-2745	0.48	0.70	0.13	0.18	0.44	**Ok**
6	52.0	68.0	-1130	-1703	-2834	0.60	0.87	0.12	0.18	0.44	**Ok**
7	64.8	84.8	-1410	-2125	-3535	0.71	1.04	0.11	0.17	0.44	**Ok**
8	37.6	49.1	-817	-1231	-2047	0.82	1.20	0.11	0.16	0.44	**Ok**
where MT=		$0.563(P_{Wx}+P_{Lx})B_xe_x$ $+0.563(P_{Wy}+P_{Ly})B_ye_y$			and	ex=- 0.15Bx ey=- 0.15By			and	Bx= By=	145' 167'
Max Drift is in Y = 1.20 in										3.48	**Ok**

Forces due to extreme Load cases in X and Y direction correspondingly:

Load Combination 1.6 Worst Wind Force in X-Direction-		
Floor level	Load (Kip)	1.6*Load (Kip)
8	67	107
7	115	184
6	92	148
5	89	143
4	86	138
3	82	131
2	77	123
1	73	118

Distribution of forces in shear walls under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)								
Floor Level	P-2	P-4	P-6	P-8	P-10	P-11	P-11 in X	Total of walls
8	0.0	29.7	6.0	0.0	15.8	14.2	11.5	63.0
7	49.5	13.1	5.8	4.6	22.6	14.4	11.6	107.3
6	46.4	4.2	5.4	1.9	20.2	16.2	13.2	91.3
5	45.1	4.1	5.3	3.8	21.4	17.5	14.2	93.9
4	44.2	4.5	6.3	4.9	23.5	20.8	16.9	100.2
3	37.2	4.3	5.8	1.6	24.8	36.3	29.4	103.1
2	30.8	4.5	7.0	-1.5	16.7	69.5	56.4	113.9
1	29.6	4.3	6.7	-1.5	19.4	66.5	54.0	112.6

Distribution of forcesin the moment frames under a 100 Kip load in X-Direction at the center of rigidity in percentage (%)						
Floor Level	PF-6	PF-7	PF-8	PF-9	PF-1	Total of frames
8	0.0	16.0	12.8	12.7	0.0	41.5
7	17.9	6.8	20.6	16.7	4.5	66.6
6	12.0	5.2	15.7	13.1	2.7	48.6
5	10.7	4.6	13.7	11.5	2.4	42.9
4	8.8	3.8	11.3	9.4	2.0	35.2
3	6.5	3.1	8.7	7.2	1.8	27.3
2	3.2	0.5	1.5	1.2	0.4	6.8
1	0.4	0.4	1.3	1.0	0.3	3.4

Summary of distribution of forces in Kips				
Floor level	Walls	Frames	Total	Other members*
8	63	41	104	2
7	107	67	174	10
6	91	49	140	8
5	94	43	137	6
4	100	35	135	2
3	103	27	130	1
2	114	7	121	2
1	113	3	116	2
*:Members ignored in calculations for simplicity				

Y direction: (For relative stiffness see page 47)

Load Combination 1.6 Worst Wind Force in Y-Direction- Case 2 Loading: .75 $(P_{Wy}+P_{Ly}) + M_T$				
Floor level	Load (Kip)	M_T	1.6*Load (Kip)	1.6*M_T(ft-kip)
8	65	1639	105	2623
7	113	2831	181	4529
6	91	2269	145	3630
5	88	2198	140	3516
4	85	2118	135	3388
3	81	2019	129	3231
2	75	1883	120	3013
1	72	1806	115	2890

Summary of Distribution of forces in Kips				
Floor level	Walls	Frames	Total	Other members*
8	78	27	105	1
7	78	94	173	-8
6	83	52	135	-10
5	95	41	135	-5
4	99	33	132	-3
3	94	29	123	-6
2	117	6	123	3
1	113	5	118	3
*:Members ignored in calculations for simplicity				

See below for detailed distribution

Distribution of Forces based on Relative stiffness of the shear walls in the Y-Direction (Kips)									
Floor Level	Force	P-1	P-3	P-5	P-7	P-9	P-11	P-11 in Y	Total of Walls
8	105	46.7	0.0	12.9	0.0	18.6	0.5	0.3	78.4
7	181	32.0	10.0	12.5	8.2	10.9	8.3	4.9	78.4
6	145	32.1	6.8	15.3	2.9	25.3	1.0	0.6	83.0
5	140	40.3	10.9	15.3	2.3	24.5	2.8	1.6	94.8
4	135	41.9	11.4	17.0	2.3	25.0	2.4	1.4	98.9
3	129	43.0	10.0	15.8	1.8	22.0	2.2	1.3	93.8
2	120	57.4	12.2	15.7	1.5	18.4	19.8	11.6	116.8
1	115	55.7	11.7	15.0	1.4	17.6	19.0	11.1	112.6

Distribution of Forces based on Relative stiffness of the frame piers chosen in the Y-direction (Kips)													
Floor Level	Force	PF-A	PF-B	PF-C	PF-E	PF-F	PF-G	PF-H	PF-I	PF-J	PF-K	PF-M	Total of frames
8	105	6.2	0.0	2.2	1.7	0.0	1.8	1.6	1.9	3.6	1.1	6.8	27.0
7	181	5.8	19.4	7.2	5.6	19.8	9.1	4.9	10.4	1.8	2.8	7.4	94.2
6	145	4.9	21.7	1.9	0.0	12.8	2.6	0.0	2.3	1.8	-0.1	4.1	52.0
5	140	4.4	8.6	2.5	1.0	11.6	3.2	0.5	3.1	1.4	0.3	3.9	40.6
4	135	3.7	8.0	1.9	0.5	9.7	2.5	0.1	2.4	1.3	0.2	3.2	33.4
3	129	3.3	7.0	1.8	0.8	8.6	2.3	0.3	2.2	0.2	0.2	2.8	29.5
2	120	0.8	1.5	0.3	0.0	2.3	0.3	0.0	0.4	0.2	0.0	0.4	6.1
1	115	0.4	1.1	0.2	0.1	1.4	0.3	0.0	0.2	1.2	0.0	0.3	5.4

Relative Stiffness in Y direction:

Relative stiffness of the shear walls in the Y-direction under a 100 Kip load at the center of rigidity. Reported in percentage (%)								
Floor Level	P-1	P-3	P-5	P-7	P-9	P-11	P-11 in Y	Total of walls
8	44.6	0.0	12.3	0.0	17.7	0.5	0.3	74.9
7	17.7	5.5	6.9	4.5	6.0	4.6	2.7	43.4
6	22.2	4.7	10.5	2.0	17.5	0.7	0.4	57.3
5	28.7	7.7	10.9	1.6	17.4	2.0	1.2	67.6
4	31.0	8.4	12.5	1.7	18.5	1.8	1.1	73.2
3	33.3	7.8	12.2	1.4	17.0	1.7	1.0	72.7
2	47.7	10.2	13.0	1.2	15.3	16.5	9.6	97.1
1	48.2	10.2	13.0	1.2	15.3	16.5	9.6	97.6

Relative stiffness of the moment frames in the Y-direction under a 100 Kip load at the center of rigidity. In percentage (%)											
Floor Level	PF-A	PF-B	PF-C	PF-E	PF-F	PF-G	PF-H	PF-I	PF-J	PF-K	PF-M	Total of frames
8	6.0	0.0	2.1	1.6	0.0	1.7	1.6	1.8	3.4	1.1	6.5	25.8
7	3.2	10.7	4.0	3.1	10.9	5.0	2.7	5.8	1.0	1.6	4.1	52.1
6	3.4	15.0	1.3	0.0	8.8	1.8	0.0	1.6	1.2	0.0	2.8	35.9
5	3.2	6.1	1.8	0.7	8.3	2.3	0.3	2.2	1.0	0.2	2.8	28.9
4	2.8	6.0	1.4	0.4	7.2	1.8	0.0	1.8	0.9	0.1	2.4	24.7
3	2.6	5.5	1.4	0.6	6.6	1.8	0.2	1.7	0.1	0.2	2.2	22.9
2	0.7	1.3	0.2	0.0	1.9	0.2	0.0	0.3	0.2	0.0	0.3	5.1
1	0.4	1.0	0.2	0.1	1.2	0.3	0.0	0.2	1.1	0.0	0.3	4.7

Summary of Relative Stiffness in %				
Floor level	Walls	Frames	Total	Other members*
8	75	26	101	-0.7
7	43	52	95	4.5
6	57	36	93	6.9
5	68	29	96	3.5
4	73	25	98	2.2
3	73	23	96	4.4
2	97	5	102	-2.1
1	98	5	102	-2.3
*:Members ignored in calculations for simplicity				

Appendix C: Seismic Load Calculations

Step 1													
Seismic Forces													
Level	Story weight, w_x	Story height (ft), h_x	Story force (k) Fx=0.01, w_x	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Ax	M_{zy} (k-ft)	By (ft)	5% By	Ax	M_{zx} (k-ft)
2	2895	14.5	29	193.0	420	145	7.25	1.0	210	191	9.55	1.0	277
3	2893	29	28.9	164.0	839	145	7.25	1.0	210	191	9.55	1.0	276
4	2893	43.5	28.9	135.1	1258	145	7.25	1.0	210	191	9.55	1.0	276
5	2893	58	28.9	106.2	1678	145	7.25	1.0	210	191	9.55	1.0	276
6	2944	72.5	29.4	77.3	2134	145	7.25	1.0	213	191	9.55	1.0	281
7	3133	87	31.3	47.8	2726	145	7.25	1.0	227	191	9.55	1.0	299
8	1648	107	16.5	16.5	1764	145	7.25	1.0	120	191	9.55	1.0	157
∑ =	19298.906			Base Shear =	193								
				Total Overturning moment=	10819			∑ Mzy=	1399			∑ Mzx=	1843

Step 2								
Earthquake Loads X-direction			Earthquake drift		Earthquake interstory drift		Code allows .01h_{sx}	
Story level	Ex (k)	Mzx (k-ft)	δx	δy	Δx	Δy		
2	29	277	0.06	0.02	0.02	0.01	1.74	**Ok**
3	28.9	276	0.12	0.02	0.06	0.00	1.74	**Ok**
4	28.9	276	0.19	0.02	0.07	0.00	1.74	**Ok**
5	28.9	276	0.26	0.01	0.07	0.00	1.74	**Ok**
6	29.4	281	0.33	0.01	0.07	0.00	1.74	**Ok**
7	31.3	299	0.40	0.00	0.07	0.00	1.74	**Ok**
8	16.5	157	0.46	0.00	0.06	0.00	1.74	**Ok**
Max drift is in X= .46 in							1.17	**Ok**

Step 3								
Earthquake Loads Y-direction			Earthquake drift		Earthquake interstory drift		Code allows .01h_{sx}	
Story level	Ey (k)	Mzy (k-ft)	δx	δy	Δx	Δy		
2	29	210	-0.01	0.14	0.00	0.08	1.74	**Ok**
3	28.9	210	-0.02	0.28	-0.01	0.14	1.74	**Ok**
4	28.9	210	-0.04	0.44	-0.02	0.16	1.74	**Ok**
5	28.9	210	-0.06	0.61	-0.02	0.17	1.74	**Ok**
6	29.4	213	-0.08	0.77	-0.02	0.16	1.74	**Ok**
7	31.3	227	-0.10	0.92	-0.02	0.15	1.74	**Ok**
8	16.5	120	-0.12	1.07	-0.02	0.15	1.74	**Ok**
Max drift is in Y= 1.07 in							1.17	**Ok**

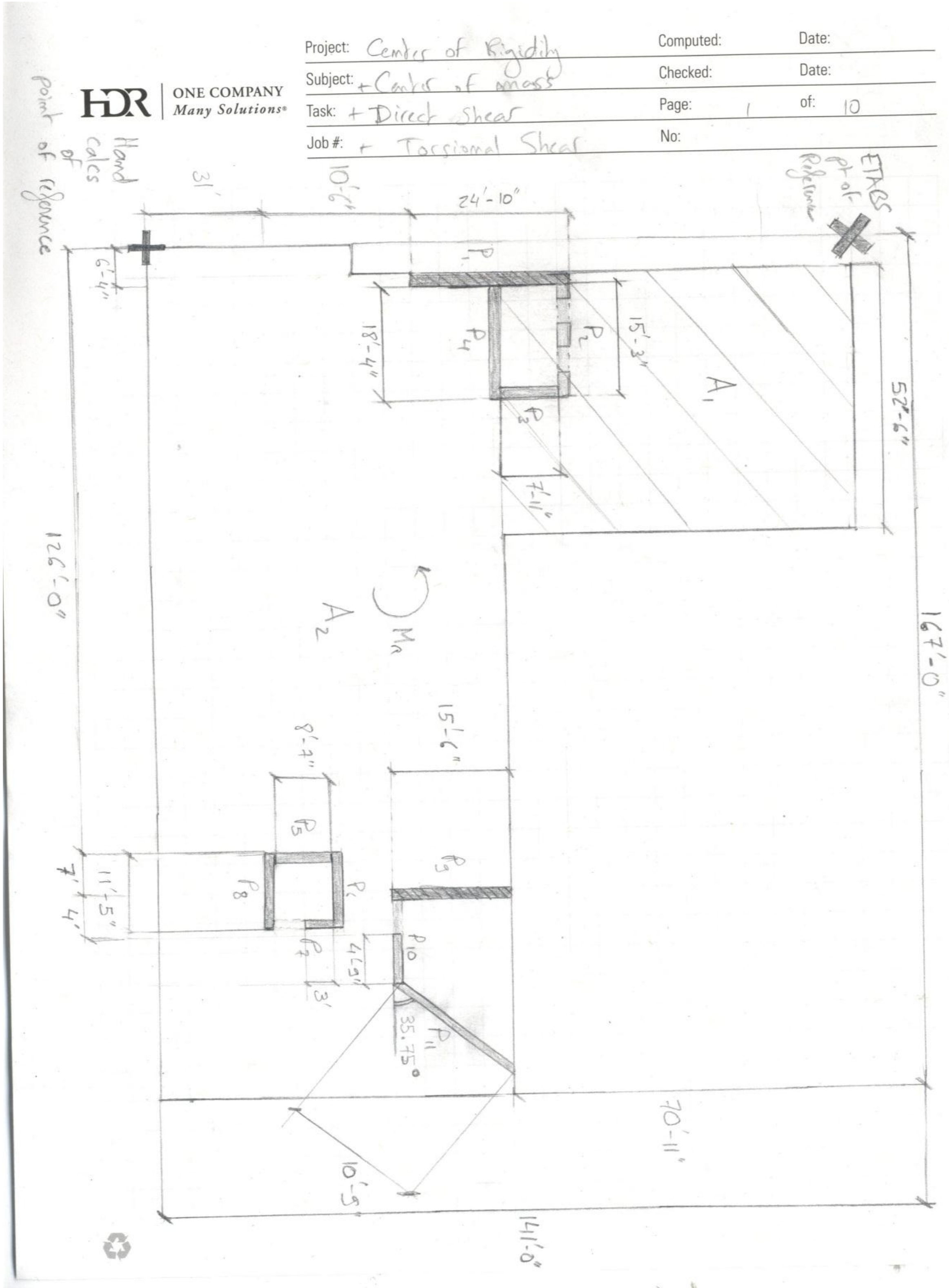
Step 4								
Rigid Diaphragm					Determining type 1-a/ 1-b irregularity			Horizontal Irregularity
X-Direction Loading	Level	$\delta_{X\,MAX}$	$\delta_{X\,MIN}$	A_{XX}	$A_{XX\,chosen}$	$A_{XY} > 1.2$	$1.2 > A_{XY} > 1.4$	
	2	0.058	0.1	0.8	1.0	NO	NO	No Torsional Irregularity
	3	0.117	0.1	0.8	1.0	NO	NO	
	4	0.187	0.2	0.9	1.0	NO	NO	
	5	0.260	0.2	0.9	1.0	NO	NO	
	6	0.332	0.3	0.9	1.0	NO	NO	
	7	0.400	0.3	1.0	1.0	NO	NO	
	8	0.464	0.3	1.0	1.0	NO	NO	

Step 5								
Rigid Diaphragm					Determining type 1-a/ 1-b irregularity			Horizontal Irregularity
Y-Direction Loading	Level	$\delta_{Y\,MAX}$	$\delta_{Y\,MIN}$	A_{XY}	$A_{XX\,chosen}$	$A_{XY} > 1.0$	$1.2 > A_{XY} > 1.4$	
	2	0.282	0.150	1.18	1.1	YES	NO	Extreme Torsional Irregularity 1-b
	3	0.441	0.220	1.24	1.1	YES	NO	
	4	0.608	0.320	1.19	1.3	NO	YES	
	5	0.772	0.462	1.09	1.3	NO	YES	
	6	0.924	0.657	1.00	1.3	NO	YES	
	7	1.071	0.831	1.00	1.3	NO	YES	
	8	1.105	1.015	1.00	1.2	YES	NO	

Step 6												
Seismic Forces												
Level	Story weight, w_x	Story height (ft), h_x	Story force (k) Fx=0.01, w_x	Story Shear (k)	Overturning moment (k-ft)	Bx (ft)	5% Bx	Axy	M_{zy} (k-ft)	By (ft)	5% By	Ax	M_{zx} (k-ft)
2	2895	14.5	29	193.0	420	145	7.25	1.1	231	191	9.55	1.0	277
3	2893	29	28.9	164.0	839	145	7.25	1.1	231	191	9.55	1.0	276
4	2893	43.5	28.9	135.1	1258	145	7.25	1.3	273	191	9.55	1.0	276
5	2893	58	28.9	106.2	1678	145	7.25	1.3	273	191	9.55	1.0	276
6	2944	72.5	29.4	77.3	2134	145	7.25	1.3	277	191	9.55	1.0	281
7	3133	87	31.3	47.8	2726	145	7.25	1.3	295	191	9.55	1.0	299
8	1648	107	16.5	16.5	1764	145	7.25	1.2	143	191	9.55	1.0	157
$\sum$ =	19298.906			Base Shear =	193								
				Total Overturning moment=	10819			$\sum$ Mzy=	1723			$\sum$ Mzx=	1843

Step 7								
Earthquake Loads Y-direction			Earthquake drift		Earthquake interstory drift		Code allows $.01h_{sx}$	
tory lev	Ey (k)	Mzy (k-ft)	δx	δy	Δx	Δy		
2	29	231	-0.01	0.14	0.00	0.08	1.74	**Ok**
3	28.9	231	-0.02	0.29	-0.01	0.15	1.74	**Ok**
4	28.9	273	-0.04	0.45	-0.02	0.16	1.74	**Ok**
5	28.9	273	-0.06	0.70	-0.02	0.25	1.74	**Ok**
6	29.4	277	-0.08	0.79	-0.02	0.09	1.74	**Ok**
7	31.3	295	-0.10	0.93	-0.02	0.14	1.74	**Ok**
8	16.5	143	-0.12	1.17	-0.02	0.24	1.74	**Ok**
Max drift is in Y= 1.17 in							1.74	**Ok**
multiply max inter story drift 0.24 in by Cd= 4 for the frame of building thus max drift is Y=1.05							1.74	**Ok**

Appendix D: Spot checks



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Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page: 2	of: 10
Job #:	No:	

Center of mass: Area × thickness × weight (pcf) x-value y-value

Slabs

Slab A_1: $(70.92')(52.5')\left(\frac{5''}{12''}\right)(150) = 232.71\text{ K}$ 26.25' 105.5'

Slab A_2: $(70.08')(167)\left(\frac{5''}{12''}\right)(150) = 731.46\text{ K}$ 83.5' 35'

Walls

length × t × weight × height x-value y-value

$P_1 = (24.83')\underbrace{(1')(150)(14.5')}_{=2{,}175\text{ plf}} = 54\text{ K}$ 5.58' 64.1'

$P_2 = (15.25)(2.175) = 33.17\text{ K}$ 13.93' 77.1'

P_3	= (7.92)(2.175)	= 17.23 K	24.2'	70.1'
P_4	= (18.3)(2.175)	= 39.8 K	13.93'	66.1'
P_5	= (8.58)(2.175)	= 18.66 K	126.5'	35'
P_6	= (11.42)(2.175)	= 24.84 K	131.7'	40'
P_7	= P_5	= 18.66 K	137'	35'
P_8	= P_6	= 24.84 K	131.7'	31'
P_9	= (15.5)(2.175)	= 33.71 K	133.5'	55.25'
P_{10}	= (8.75)(2.175)	= 19.03 K	139.4'	49'
P_{11}	= (10.75)(2.175)	= 23.38 K	150.5'	52.1'

total = 1271.5 K

X-Direction

$$\bar{X} = \frac{(232.71)(26.25)+(731.46)(83.5)+(54)(5.52)+(33.17)(13.93) + (17.23)(24.2)+(39.8)(13.93)+(18.66)(126.5)+[(24.84)(131.7)]2 + (18.66)(137)+(33.71)(133.5)+(19.03)(139.4)+(23.38)(150.5')}{1271.5}$$

$$\Rightarrow \bar{x} = \frac{91070.2}{1271.5} = 71.62' \quad \therefore \quad \text{ETABS: } 71.61'$$

Project:	Computed:	Date:
Subject:	Checked:	Date:
Task:	Page: 3	of: 10
Job #:	No:	

Y-direction:

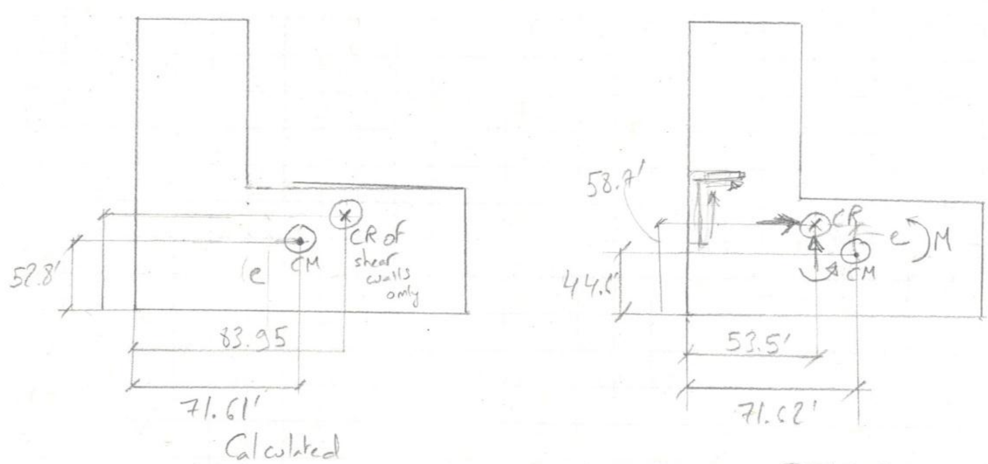
$$\bar{y} = \frac{(232.71)(105.\ \ ) + (731.46)(35) + (54)(64.1) + (33.17)(74.1) + (17.23)(70.1) + (39.8)(66.1) + (18.66)(35.3) + (24.84)(40) + (24.84)(31) + (33.71)(55.75) + (15.03)(58.5) + (23.38)(52.65)}{1271.5}$$

$$\Rightarrow \bar{y} = \frac{67098}{1271.5} = 52.8' \Rightarrow 141' - 52.8' = 88.2'$$

$\therefore$ ETABS $= 96.4'$

$\Rightarrow$ Difference of $8.2'$

$$\frac{8.2'}{141'} = 5.8\ \%\ \text{difference}$$



the center of Rigidity of shear walls is calculated on the next page.

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 4	of: 10
Job #:		No:	

Center of Rigidity: (Shear walls only)

R:

		X-Value:	Y-value:
P_1 :	.222	5.92'	64.1'
P_2 :	.314	13.93'	77.1'
P_3 :	.047	24.2'	70.1'
P_4 :	.028	13.93'	66.1'
P_5 :	.105	126.5'	35.13
P_6 :	.037	131.7'	40'
P_7 :	.02	137'	35.3'
P_8 :	.013	131.7'	31'
P_9 :	.175	133.5'	55.25'
P_{10} :	.137	139.4'	58.5'
P_{11} in x :	.089	150.5'	52.1'
P_{11} in y :	.004≈0		

$$X_r = \frac{(.222)(5.92)+(.047)(24.2)+(.105)(126.5)+(.02)(137)+(.175)(133.5)+(.089)(150.5)}{(.222+.047+.105+.02+.175+.089)}$$

$X_r = 83.95$

∴ ETABS: $X_r = 53.478'$

$83.95 - 53.478 = 30.47'$ $\qquad 30.47/167 = 18.25\%$

from ETABS total wall contribution to resistance is 57.3%

Thus the frames take ≈43% of the force in the x-direction

Hence, moment frames in this system are heavily relied on due to the building's shape and wall placement.

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 5	of: 10
Job #:		No:	

$y_r = (.314)(77.1) + (.028)(66.1') + (.037)(40) + (.013)(31) + (.137)(58.5') \,/\, (.314 + .028 + .037 + .013 + .137)$

$y_r = 67.97'$

$\therefore$ ETABS $\quad Y_r = 82.32' \qquad 141 - 82.32' = 58.68$

$67.97 - 58.68 = 9.29' \qquad 9.29/141 = 6.6\%$

Thus, the walls contribute more to the center of rigidity in the Y-direction than in the x-direction this is due to the lack of big moment frames resisting in that direction as opposed to the X-direction

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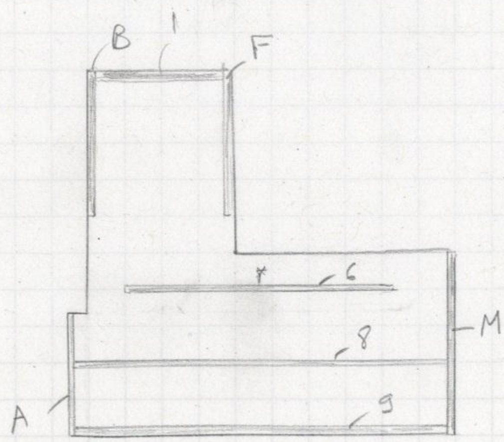
Project: Computed: Date:
Subject: Checked: Date:
Task: Page: 6 of: 10
Job #: No:

Torsional Rigidity : $J = \Sigma R_i d_i^2$ (d_i = distance from CR to wall i)

Using ETABS 's $x_r = 53.5'$ / $y_r = 58.7'$

R

PF-6 : .081 PF-8 : .106 PF-F : .088
PF-1 ≈ 0.02 PF-9 : .089 PF-B : .15
PF-7 = 0.04 PF-A : .034 PF-M : .028



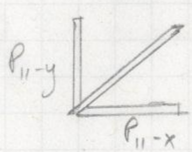
d_i of B = 47'
d_i of F = 7.75'
d_i of A = 53.5'
d_i of M = 113.5'
d_i of 6 = -9.7'
d_i of 8 = -28'
d_i of 9 = -58.7'
d_i of 1 = 82.3'
d_i of 7 = 16.7'

d_i of P_1 = 47.5
d_i of P_2 = 22.3'
" P_3 = 23.7'
" P_4 = 14.3'

d_i of P_5 = 73'
" P_6 = 18.7'
" P_7 = 83.5'
" P_8 = 27.7'

d_i of P_9 = 80'
" P_{10} = 9.7'

Decomposing the slanted wall into two walls for simplicity



d_i of P_{11-x} = 9.7'
d_i of P_{11-y} = 89'

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 7	of: 10
Job #:		No:	

$J = RP_1(47.5)^2 + RP_2(22.3)^2 + RP_3(29.1)^2 + RP_4(14.3)^2 + RP_5(73)^2$
$+ RP_6(18.7')^2 + RP_7(83.5')^2 + RP_8(27.7')^2 + RP_9(80)^2 + RP_{10}(9.7)^2$
$+ RP_{11-x}(9.7)^2 + RP_{11-9}(89')^2 + RPF6(9.7)^2 + RPF8(28)^2 + RPF9(58.7)^2$
$+ RPF\text{-}A(53.5)^2 + RPF\text{-}B(47)^2 + RPF\text{-}F(7.75)^2 + RPF\text{-}M(113.5)^2 + RPF_1(823)^2$

$J = 3938 \ (K/in)(ft^2)$

Direct shear: 93.2 in x-direction using worst case

$V_{P_2} = .32(93.2) = 29.82$
$V_{P_4} = .028(93.2) = 2.6$
$V_{P_6} = .037(93.2) = 3.45$
$V_{P_8} = .013(93.2) = 1.21$
$V_{P_{10}} = .137(93.2) = 12.77$
$V_{P_{11-x}} = .09(93.2) = 8.4$
$V_{PF-6} = .081(93.2) = 7.55$
$V_{PF-8} = .11(93.2) = 10.3$
$V_{PF-9} = .09(93.2) = 8.4$
$V_{PF-1} = .02(93.2) = 1.86$
$V_{PF-7} = .04(93.2) = 3.73$

0.97 — 90.1 K

All forces are acting in this direction ($\leftarrow$)

this not equal to 1 as there are the other members not taken into consideration

A 3.1 K difference is due to the minor contributing frames not considered

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 8	of: 10
Job #:		No:	

Torsional Shear in Walls:

93.2^K → CR, e, CM

$e_y = 58.7' - 44.6' = 14.1$ using ETABS

$$V_{P8} = \frac{(93.2)(14.1)(27.7)(.013)}{3538} = .12\text{ K} \quad \leftarrow$$

$$V_{P2} = \frac{(93.2)(14.1)(22.3)(.314)}{3538} = 2.33\text{ K} \quad \rightarrow$$

$$V_{P4} = \frac{(93.2)(14.1)(14.3)(.028)}{3538} = .14\text{ K} \quad \rightarrow$$

$$V_{P6} = \frac{(93.2)(14.1)(18.7)(.037)}{3538} = .25\text{ K} \quad \leftarrow$$

$$V_{P10} = \frac{(93.2)(14.1)(9.7)(.137)}{3538} = .44\text{ K} \quad \rightarrow$$

$$V_{P11} = \frac{(93.2)(9.7)(14.1)(.09)}{3538} = .3\text{ K} \quad \rightarrow$$

$$V_{PF-6} = \frac{(93.2)(14.1)(9.7)(.08)}{3538} = .26\text{ K} \quad \leftarrow$$

$$V_{PF-8} = \frac{(93.2)(14.1)(28)(.106)}{3538} = 1.0\text{ K} \quad \leftarrow$$

$$V_{PF-5} = \frac{(93.2)(14.1)(58.7)(.09)}{3538} = 1.81\text{ K} \quad \leftarrow$$

$$V_{PF1} = \frac{(93.2)(14.1)(82.3)(.02)}{3538} = 0.55\text{ K} \quad \rightarrow$$

$$V_{PF7} = \frac{(93.2)(14.1)(16.7)(.04)}{3538} = .22\text{ K} \quad \rightarrow$$

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Project: Computed: Date:
Subject: Checked: Date:
Task: Page: 9 of: 10
Job #: No:

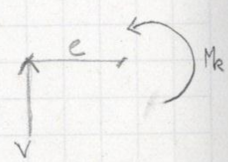
Total shear: Direct shear + torsional shear:

$V_{P_2} = 29.82 - 2.33 = 27.49^k$
$V_{P_4} = 2.6 - .14 = 2.46^k$
$V_{P_6} = 3.45 + .25 = 3.7^k$
$V_{P_8} = 1.21 + .12 = 1.33^k$
$V_{P_{10}} = 12.77 - .44 = 12.33^k$
$V_{P_{11-X}} = 8.4 - .3 = 8.1^k$
$V_{PF-6} = 7.55 + .26 = 7.81^k$
$V_{PF-8} = 10.3 + 1 = 11.3^k$
$V_{PF-9} = 8.4 + 1.81 = 10.21^k$
$V_{PF-1} = 1.86 - .55 = 1.31^k$
$V_{PF-7} = 3.73 - .22 = 3.52^k$

P_2 is worst case in x-direction $V_{P_2} = 27.49^k$

Y-direction:

$P = 145^k$ $M_T = 3630$
$e_x = 18.12'$ from ETABS



Direct shear:

One of these two walls will control in Y direction

$V_{P_1} = (.222)(145) = 32.2^k$
$V_{P_9} = (.175)(145) = 25.4^k$

Torsional Shear:

$$V_{P_1} = \frac{[(145)(18.12) + 3630](47.5)(.222)}{3938} = 16.76^k$$

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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 10	of: 10
Job #:		No:	

$$V_{P_3} = \frac{[(145)(18.12)+3630](80)(.175)}{3938} = 22.25^K \downarrow$$

Total shear

$$V_{P_1} = 32.2 - 16.76 = 15.44 \downarrow$$

$$V_{P_3} = 25.4^K + 22.25 = 47.65^K \downarrow$$

Worst case in Y-direction is wall P_3 $\quad V_{P_3} = 47.65^K \downarrow$

From ETABS, for the same load case, this wall yielded a value of -47.71 K thus a difference of only 0.13%

This same shear wall was then checked through Excell under ACI-318-08.

Spreadsheet is shown on the page after.

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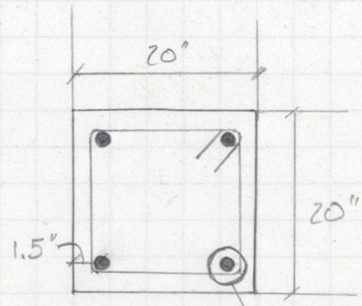
Project:		Computed:	Date:
Subject:	Hand verification for	Checked:	Date:
Task:	the use of sp column	Page: 1	of: 2
Job #:		No:	

Column Spot Check:



$f'_c = 4{,}000$ psi

$f_y = 60{,}000$ psi

$A_s = 4 \times 1.0 = 4.0\ in^2$

(4) #9 w/ #3 tie @ 18"

a) Axial Strength:

$P_o = 0.85 f'_c \cdot A_c + A_s f_y = 0.85(4) \times [20 \times 20 - 4] + 4(60)$

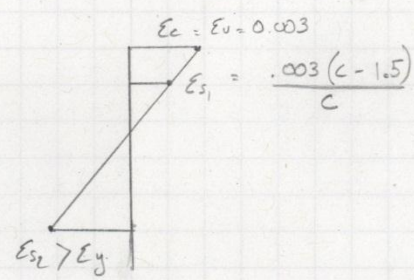
$P_o = 1586^K$

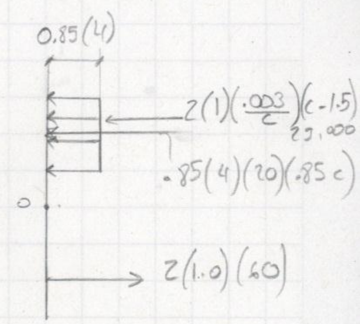
$\varepsilon_y = \dfrac{f_y}{E_s} = \dfrac{60}{29{,}000} = .00207 < .005 \Rightarrow$ this is a compression controlled column

thus $\phi P_n = .65\, P_o$

$\phi P_n = .65\,(1586) = 1031^K$

b)





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Project:		Computed:	Date:
Subject:		Checked:	Date:
Task:		Page: 2	of: 2
Job #:		No:	

From $T = C$ $\quad 120^K = 174\left(\frac{c-1.5}{c}\right) + 57.8c$

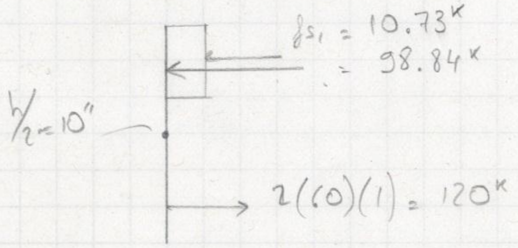
$$120c = 174c - 1.5(174) + 57.8c^2$$

$$\Rightarrow \quad 57.8c^2 + 54c - 261 = 0$$

$$c = 1.71''$$

$$\Rightarrow \varepsilon_{s_1} = \frac{0.003(1.71-1.5)}{1.71} = 0.00037 \qquad \varepsilon_{s_2} = \frac{.003(1.71-18.5)}{1.71} = -.0295 < -\varepsilon_y$$

$$f_{s_1} = \varepsilon_{s_1} \cdot E_s = +10.73\ (C) \qquad f_{s_2} = -f_y = -60$$



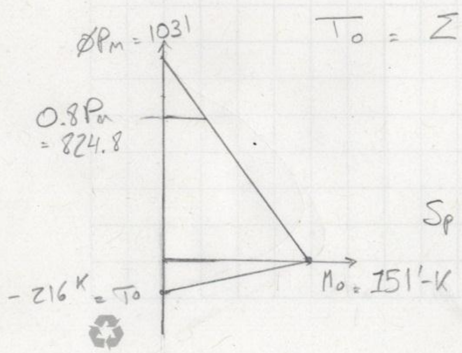
$$M_o = 98.84\left(10 - \frac{.85(1.71)}{2}\right) + 10.73(10-1.71) - 120(10-18.5)$$
$$\quad\ 916.57 \qquad\qquad\qquad 88.95$$

$$M_o = 2025.5\ \text{in-K} = 168\ '\text{-K} \quad \Rightarrow \phi M_n = 151.2\ '\text{-K}$$

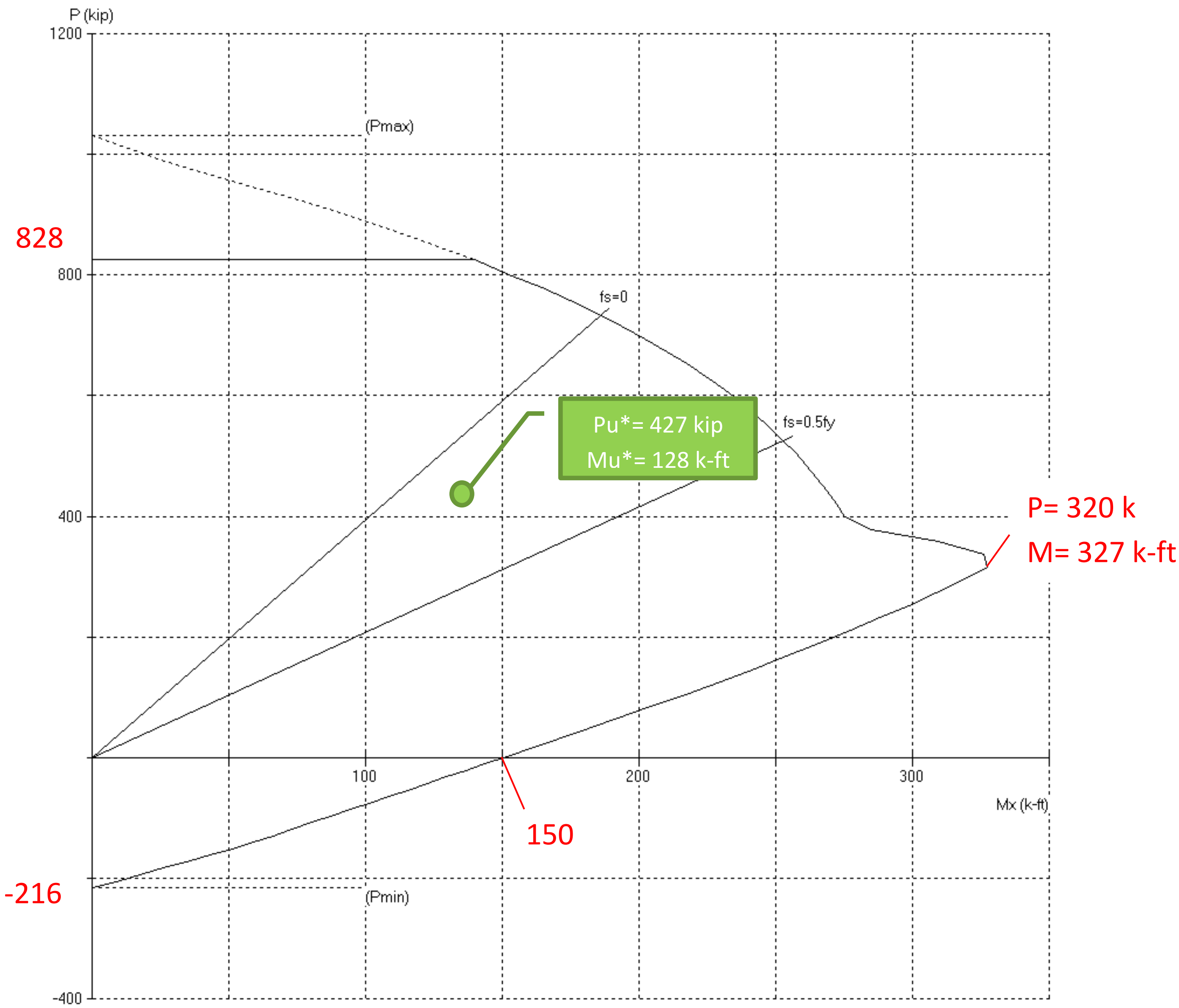
c) Pure Tension:

$$T_o = \Sigma A_{s_i} f_{s_i} = 4 \times 1.0 \times (-60) = -240^K$$

$$\phi T_o = 0.9(240) = -216^K$$



Sp column was used to determine the full and more precise diagram :



*: Pu is taken from technical assignment one under dead and live loads.

*: Mu is taken from ETABS to verify modeling process and adequacy of the member.

==

spColumn v4.60 (TM)

Computer program for the Strength Design of Reinforced Concrete Sections

==

General Information:

Project: Spot Check
Column: I-8 Code: ACI 318-08 Units: English
Run Option: Investigation Slenderness: Not considered
Run Axis: X-axis Column Type: Structural

Material Properties:

f'c = 4 ksi fy = 60 ksi
Ec = 3605 ksi Es = 29000 ksi
Ultimate strain = 0.003 in/in
Beta1 = 0.85

Section:

Rectangular: Width = 20 in Depth = 20 in
Gross section area, Ag = 400 in^2
Ix = 13333.3 in^4 Iy = 13333.3 in^4
rx = 5.7735 in ry = 5.7735 in
Xo = 0 in Yo = 0 in

Reinforcement:

Bar Set: ASTM A615

Confinement: Tied; #3 ties

phi(a) = 0.8, phi(b) = 0.9, phi(c) = 0.65

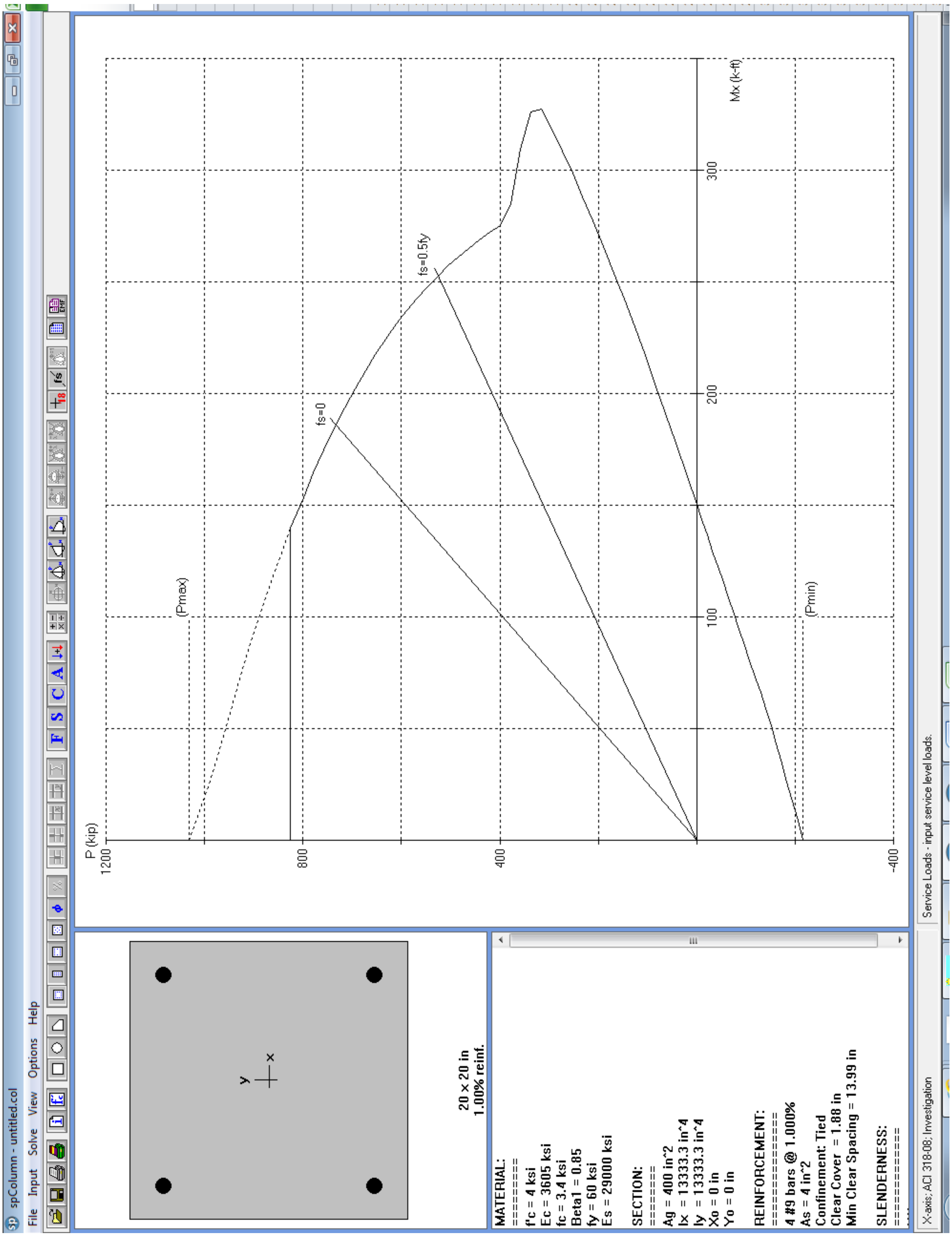
Layout: Rectangular

Pattern: All Sides Equal (Cover to transverse reinforcement)

Total steel area: As = 4.00 in^2 at rho = 1.00%

Minimum clear spacing = 13.99 in

4 #9 Cover = 1.5 in


spColumn - untitled.col
File Input Solve View Options Help
P (kip)
Mx (k-ft)
(Pmax)
(Pmin)
fs=0
fs=0.5fy
1200
800
400
-400
100
200
300
20 x 20 in
1.00% reinf.
MATERIAL:
f'c = 4 ksi
Ec = 3605 ksi
fc = 3.4 ksi
Beta1 = 0.85
fy = 60 ksi
Es = 29000 ksi
SECTION:
Ag = 400 in^2
Ix = 13333.3 in^4
Iy = 13333.3 in^4
Xo = 0 in
Yo = 0 in
REINFORCEMENT:
4 #9 bars @ 1.000%
As = 4 in^2
Confinement: Tied
Clear Cover = 1.88 in
Min Clear Spacing = 13.99 in
SLENDERNESS:
X-axis; ACI 318-08; Investigation
Service Loads - input service level loads.

Control Points according to sp-column			Axial Load P	X-Moment	Y-Moment	NA depth	Dt depth	eps_t	Phi
Bending about			kip	k-ft	k-ft	in	in		
X	@	Max compression	1031	0	0	56.6	17.56	-0.00207	0.65
	@	Allowable comp.	825	140	0	19.7	17.56	-0.00032	0.65
	@	fs = 0.0	733	186	0	17.6	17.56	0.00000	0.65
	@	fs = 0.5*fy	525	253	0	13.1	17.56	0.00103	0.65
	@	Balanced point	386	277	0	10.4	17.56	0.00207	0.65
	@	Tension control	327	332	0	6.6	17.56	0.00500	0.90
	@	Pure bending	0	151	0	2.3	17.56	0.02008	0.90
	@	Max tension	-216	0	0	0.0	17.56	9.99999	0.90
-X	@	Max compression	1031	0	0	56.6	17.56	-0.00207	0.65
	@	Allowable comp.	825	-140	0	19.7	17.56	-0.00032	0.65
	@	fs = 0.0	733	-186	0	17.6	17.56	0.00000	0.65
	@	fs = 0.5*fy	525	-253	0	13.1	17.56	0.00103	0.65
	@	Balanced point	386	-277	0	10.4	17.56	0.00207	0.65
	@	Tension control	327	-332	0	6.6	17.56	0.00500	0.90
	@	Pure bending	0	-151	0	2.3	17.56	0.02008	0.90
	@	Max tension	-216	0	0	0.0	17.56	9.99999	0.90

*** End of output ***

Excel Spreadsheet for Shear Wall P-9 spot check: Spreadsheet is available upon request for further details. This check was done on the 6th level where hand calcs were 99.87% accurate.

				Input				
Material								
	f'c =	4.0 Ksi	- concrete strength					
	fy =	60 Ksi	- steel reinforcement yield strength					
	Es=	29000 Ksi						
wall left end						wall right end		
			Lw					
Wall								
	Lw =	186 in	- wall lentgh					
	tw =	12 in	- wall thickness					
	hw =	174 in	- wall height					
	cw =	0.8 in	- concrete cover @ wall					
Reinforcement								
Vertical	# curtains							
	2							
- Lw				# bars/curtain		actual	Max	
bar size	spacing	db	As	in	As total	spacing	Spacing	Meet max
#4	12 in	0.5	0.2	15	3	11.50	18	spacing
- wall left end (vertical)								
#4	0.8 in	0.5	0.2	1	0.2	1 in	18	Meet max spacing
- wall right end (vertical)								
#4	0.8 in	0.5	0.2	1	0.2	1 in	18	Meet max spacing
		total # bars/curtain		17				
		As=	6.8		ρ_l		0.30%	Meet min
		Ac=	2232		ρ minum ACI11.9.9.4		0.27%	reinf
Horizontal	# curtains							
	2							
- wall (horizontal)							Max	
bar size	spacing	db	As				Spacing	
#4	12 in	0.5	0.2				18	Meet max
		$\rho\tau$		0.28%	Meet			spacing
		ρ minum ACI11.9.9.2		0.25%	min reinf			
Loads								
	Mu=	9517800	in-lb					
	Vu=	54700	lb					
	Pu=	5.3	kip					

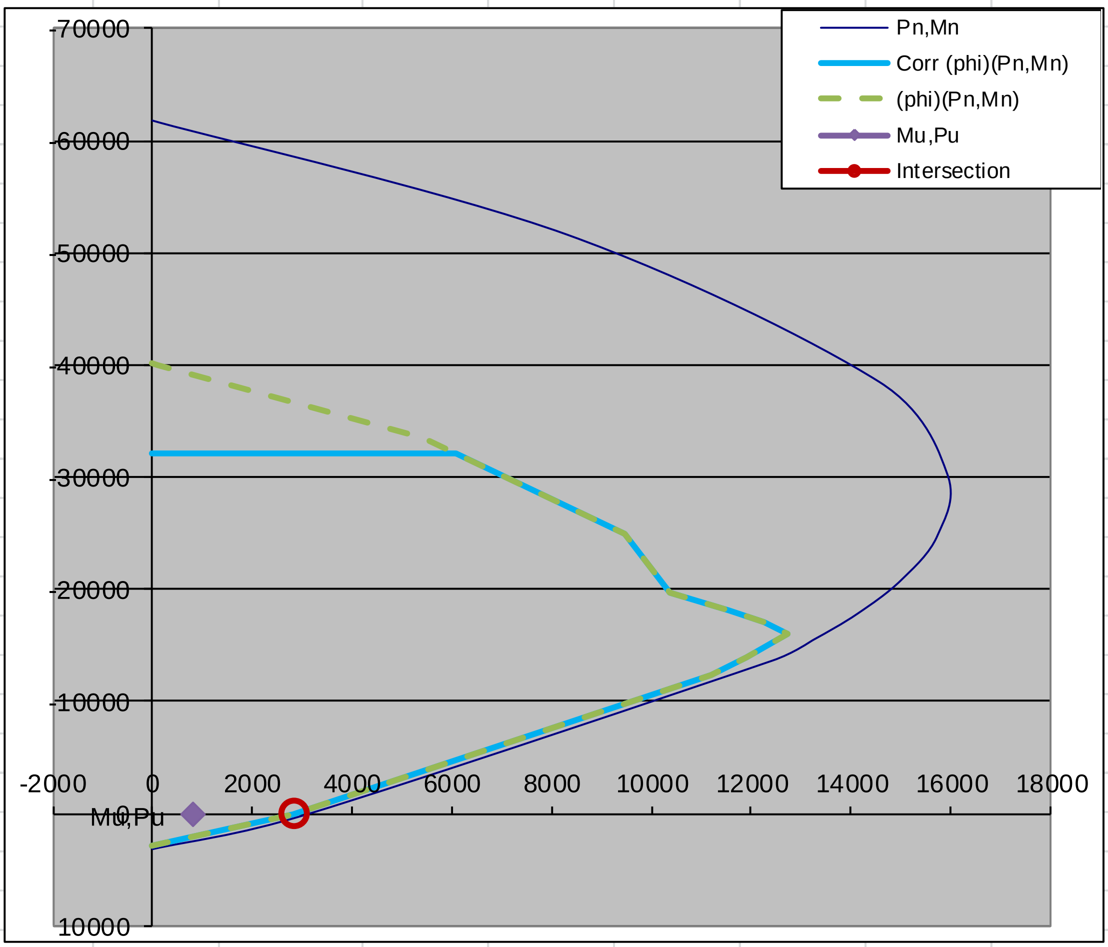
	Pu=	5.3	kip	

Trial and error to find Cu of pure flexure

c_u	10	Change untill Pn=0
Pn	-2	

Results

Shear	Vc=	225.86252	kips			
	ϕ0.5Vc=	84.698445	kips	>Vu	Vs not needed	
	Av=	0.4	in^2			
	Vs=	0	kips			
	Vn=Vc+Vs	225.86252	<	1129.313	kips	ACI section 11.9.3
	Vn=	225.86252	kips			
	ϕVn=	169.39689	kips			
	Vu<ϕVn		PASS			



DCR=	0.2793	<1	PASS