Technical Report III

University of Maryland College Park Dorm Building 7

College Park, MD

Prepared By: **Ryan Solnosky** Structural Option

Faculty Advisor: **Dr. Ali Memari**

November 21, 2008

Table of Contents

Executive Summary

Technical Report III is a study to gain an understanding of how lateral loads are distributed among load resisting elements, to verify that a load path exists, and to verify that lateral members have been designed adequately for both strength and serviceability criteria for Building 7. The lateral system for Building 7 consists of 16 shear walls throughout the building which are comprised of reinforced concrete on the lower two sections while the upper 6 floors have light gage shearwalls. The lateral loads from Technical Report I were used and summarized again here in this technical report stating which load case would control.

The distribution of lateral forces was discussed and how the forces move through the lateral system to reach the ground. A computer model was generated for determining the relative stiffness of each shear wall in ETABS and SAP. The center of mass and rigidity were calculated in order to determine the forces in each shear wall on each floor caused by eccentricity in addition to direct shear forces caused by the lateral loads. Torsional moments were determined and converted into shear values acting on lateral shear walls. Torsion had a small impact on the lateral structure in the East-West direction and slightly larger in the North-South direction but not significantly more. This was expected due to the floor plan and the location of the lateral elements.

Building drift requirements for wind loads were considered. Deflections and story drift due to wind were limited to and checked against h/400. Building 7 passed the deflection limits and the story drift limits. Overturning moments caused by wind evaluated using the calculated story shear data from the computer model in each direction for a single shear wall. Overturning issues did arise but based on the complexity of the foundation there overall should not be a problem.

Finally spot checks were performed to confirm the adequacy of selected shearwalls used throughout this report. The hand calculations show that my design/checks match the designers to an extent but they seem to have added extra reinforcing as a safety measure and also for constructability reasons.

Topics covered in detail in this report include but not limited to:

- Gravity and Lateral Loads
- Lateral Force Distribution
- Computer Analysis
- Drift
- Overturning Issues
- Strength Checks

Introduction

The University of Maryland College Park Dorm Building 7 (Building 7) is the final stage of the south campus master plan at the University of Maryland. Building 7 is the corner stone of the south campus entrance for all to take part of as they approach the campus. Building 7 is an eight story residential dorm in the shape of an unsymmetrical-U that compliments the adjacent two existing dorm buildings in architectural styles with its shape and material usage.

This eight story-133,000 square feet residential building, houses 370 bedrooms, study lounges, seminar spaces and resident life offices. The average floor to floor height is 10 feet on each floor with an average floor area of 12,000-15,500 square feet per floor, depending on shifts in the vertical plane. The layout of each floor is such that all of the rooms have an exterior view of the surrounding campus with a central corridor running the length of the building. The roof level houses the mechanical equipment along with the elevator and stair towers.

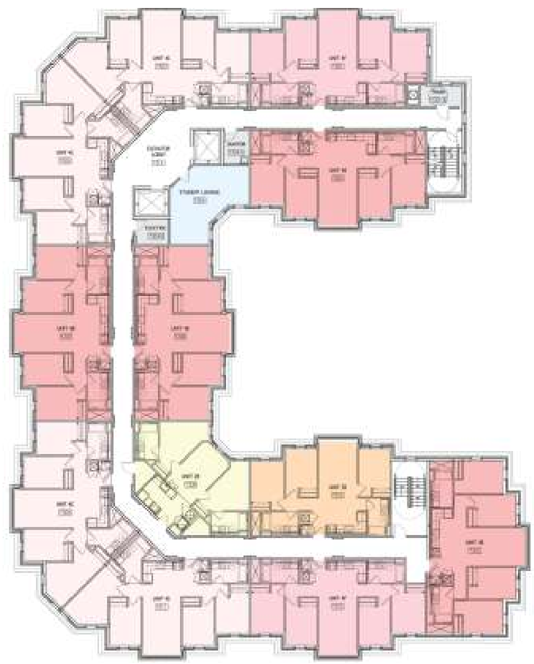


Figure 1. (Typical Floor Plan)

The façade and building envelope is comprised of light gage studs with a brick masonry veneer exterior around the entire building. There is rigid insulation on the exterior of the studs between the veneer with a 1.5 inch air cavity. The walls are filled with batt insulation and covered in drywall.

The windows are fixed casement aluminum windows with cast stone sills to accent them. In the regions where the wall sections are pulled away from the primary facade, the wall system is composed of composite metal panel and cast stone veneer panels. The roof system is an EPDM classification which is a fully adhered system comprised of a waterproof membrane that is bonded to rigid insulation by mechanical and chemical means with appropriate flashing at the base of the parapets and where the brick meets the top of the parapet.

This technical report will examine the existing lateral force-resisting system designed by the engineer for Building 7. The analysis includes a combination of SAP, ETABS, and hand calculations for various considerations. Spot checks are also performed on various lateral elements to verify their adequacy in resisting the loads.

Structural Systems

Foundation

The foundation system is composed of reinforced concrete grade beams 24”x30” with 3#8’s on the top and bottom with number #4 stirrups placed every 14”. The deep foundation portion is auger cast grout piles 16” in diameter. These piles are to be 65’ below elevation and are to be able to carry at 65 ton allowable load capacity. The pile configurations range from 2-4 piles per cap. The slab on grade for the foundation is 4” thick normal weight concrete reinforced with 6x6-1.4xW1.4 welded wire fabric. All foundation concrete is 4ksi except for the SOG which is 3.5 ksi. Due to the site’s soil conditions it was necessary that the differential settlement over the entire building was limited, because of this the allowable soil bearing capacity was held to 500 psf.

Column and Bearing Wall Systems

The concrete columns support the lower two floors of Building 7. They arranged to form a typical bay of 15’x20’. These columns are gravity bearing only due to the type of lateral system in the building. The typical size of the columns range from 18x14 to 64x14 with the reinforcing ranging in each from 4#9’s to 10#9’s for vertical bars with #4 stirrups spaced at 14” O.C.. The concrete compressive strength for the columns is 6 ksi.

The bearing walls in Building 7 support the upper 6 floors and run along the outside perimeter of the building as well as along the corridors. The typical spans for the floor joists are 20’. Dealing with the concerns that the joists may not line up with the studs causing the header to buckle, this problem was solved by placing a distribution tube across the tops of all bearing walls. These walls are also to be designed by the contractor who is given general criteria to follow along with a loading diagram for all the different bearing walls. The general criteria are: a maximum stud spacing of 16” O.C., a minimum G90 galvanized coating, and have a minimum 16 gage thickness.

Roof System

The roof system is made of the same Hambro Composite Floor System bearing on light gage walls. This Hambro Composite Floor System is also to be designed by the contractor instead of the Engineer just as the other floors are to be designed. Here are the criteria for the roof: overall depth of the members is 16” deep typically throughout except in the corridors which it drops to 8”deep with a 3” thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric. The mechanical unit weights are listed and are placed close to the corridors for they are formed by the bearing walls. The elevator towers and stair towers are made of the same light gage studs.

Floor Systems

Lower 2 Floors

The lower two floors are made of reinforced concrete beams spanning between the columns. The intermediate members between these beams are made up of the Hambro Composite Floor System, which includes the steel joists and the slab system. The concrete beams range from 16x36 to 18x18 to 24x36 with the reinforcing ranging in each from 3#5's to 6#10's for longitudinal bars with #4 stirrups spaced from 8" to 16" O.C.

The Hambro Composite Floor System in Building 7 is not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed criteria that the contractor must follow. The following is the criteria: are overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep, the slab on top is to be 5" thick reinforced with 6x6-W4.0xW4.0 welded wire fabric.

Upper 6 Floors

The floor system is made of the same Hambro Floor System but instead of them bearing on concrete girders they bear on light-gage stud bearing walls. This Hambro Floor System is also to be designed by the contractor instead of the Engineer. Here are the criteria for these 7 stories: overall depth of the members is 16" deep typically throughout except in the corridors which it drops to 8"deep with a 3" thick concrete slab reinforced with 6x6-2.9xW2.9 welded wire fabric.

Lateral Systems

The primary lateral system for Building 7 is shear walls. On each floor there are 16 shear walls spanning both directions of the building, 9 in the north-south direction and 7 in the east-west direction. The lower two stories shear walls are 10" thick reinforced concrete with 10#5's on each end for flexure and for shear reinforcement there is #5@12" each way, each face. All concrete shear walls are 6 ksi normal weight concrete. The upper floors shear walls are to be light gage studs with maximum stud spacing of 16" O.C. they are also have a minimum G90 galvanized coating and have a minimum gage of 16 for the studs while the tracks are permitted to have a 20 gage minimum. There is to be bridging at 4' spacing throughout the shear walls. Since these are light gage it was determined that steel strapping was needed and is being provided in an X pattern connecting to the farthest opposite ends. The light-gage shear walls not designed by the Structural Engineer but rather is to be designed by the Contractor. The Structural Engineer has however given detailed loading diagrams of each load and the type of load on every shear wall.

Here is a Typical Floor Plan that will be utilized throughout this technical report. This floor plan was chosen due to the majority of the building is structurally supported in this manner as well as the configuration of the spaces is the same except on the lowest levels. The areas shaded in blue are the locations of the shear walls throughout the building.

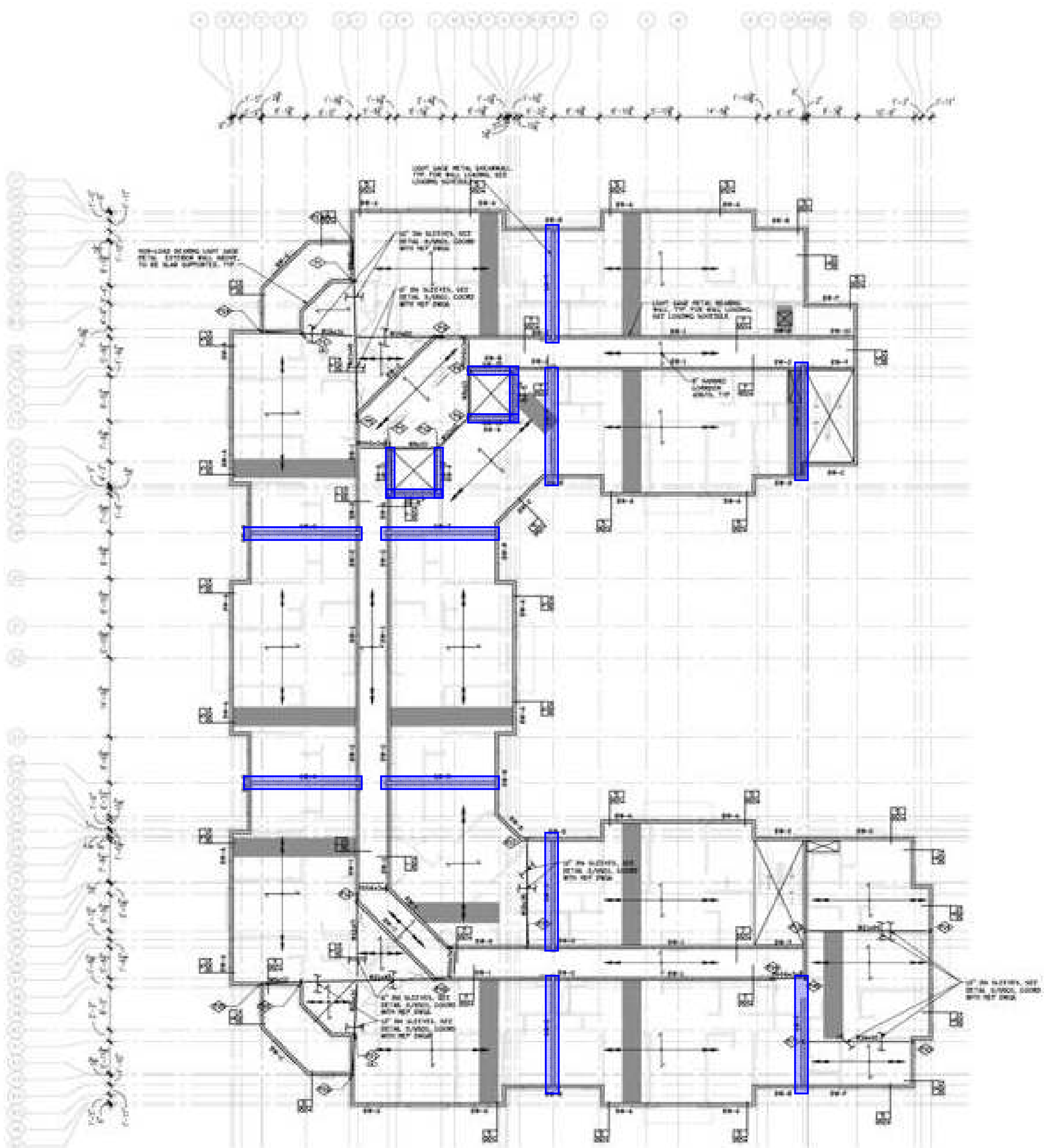


Figure 2.

Figures 3-6 shown below and on the next page are the lateral details given on the construction documents. First three figures show how the walls to be build and how the X-bracing is to be placed on the shear walls along with how the shear walls are to be connected to the floor system. Figure 6 shows all the lateral loads that each shear wall is to carry so that the contractor knows how to design them, it should be noted these loads are unfactored loads so that load combinations can be used with them.

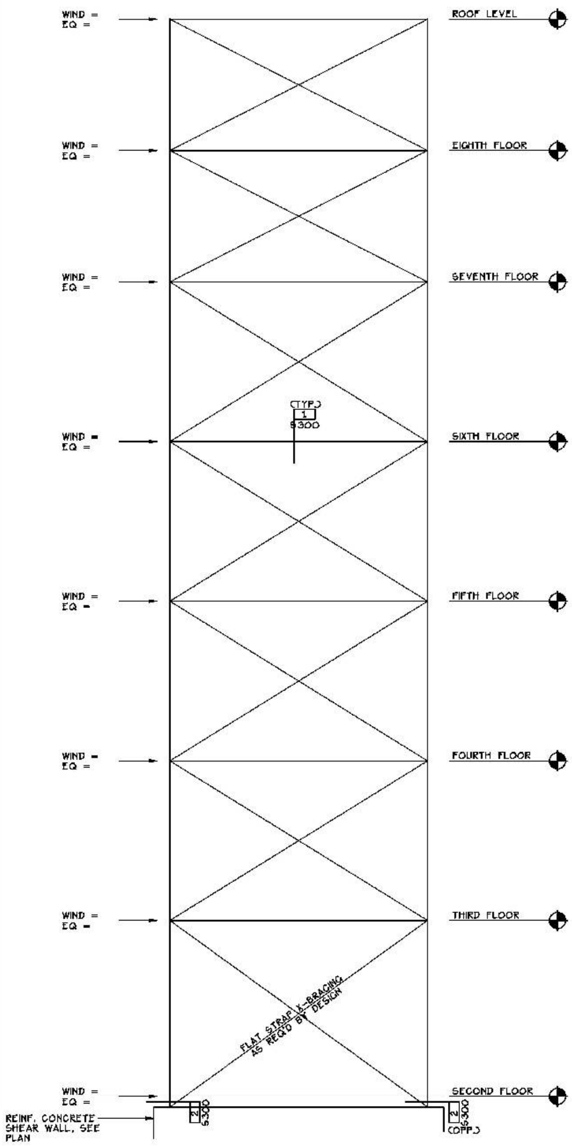


Figure 3 (Typical Detail for Upper Level Shear walls)

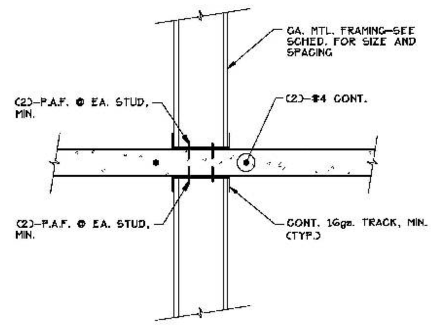


Figure 4 (Slab to Shear wall Connection)

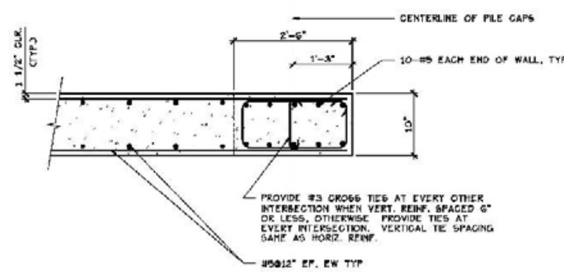


Figure 5 (Concrete Shear wall Detail)

SHEAR WALL LATERAL LOAD SCHEDULE

SW-#	FLOOR / TYPE	HIGH ROOF	ROOF	8	7	6	5	4	3	2
SW-1	WIND	—	0.5 K	1.5 K	2.5 K	2.5 K	2.25 K	2.25 K	2.25 K	2.25 K
	EQ	—	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K	1.5 K
SW-2	WIND	—	0.75 K	2.75 K	4.5 K	4.25 K	4.0 K	4.0 K	3.75 K	3.75 K
	EQ	—	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	2.5 K
SW-3	WIND	—	0.75 K	2.75 K	4.5 K	4.25 K	4.0 K	4.0 K	3.75 K	3.75 K
	EQ	—	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	2.5 K
SW-4	WIND	—	1.25 K	6.0 K	9.5 K	9.25 K	9.0 K	8.75 K	8.25 K	8.0 K
	EQ	—	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	4.0 K
SW-5	WIND	—	1.25 K	6.0 K	9.5 K	9.25 K	9.0 K	8.75 K	8.25 K	8.0 K
	EQ	—	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	2.75 K	4.0 K
SW-6	WIND	—	1.0 K	4.25 K	7.0 K	6.75 K	6.5 K	6.25 K	6.0 K	5.75 K
	EQ	—	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	3.0 K
SW-7	WIND	—	1.0 K	4.25 K	7.0 K	6.75 K	6.5 K	6.25 K	6.0 K	5.75 K
	EQ	—	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	2.0 K	3.0 K
SW-8	WIND	—	0.75 K	2.75 K	4.5 K	4.25 K	4.0 K	4.0 K	3.75 K	3.75 K
	EQ	—	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	2.5 K
SW-9	WIND	—	0.75 K	2.75 K	4.5 K	4.25 K	4.0 K	4.0 K	3.75 K	3.75 K
	EQ	—	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	1.75 K	2.5 K
SW-10	WIND	—	0.5 K	2.5 K	3.75 K	3.75 K	3.5 K	3.5 K	3.25 K	3.25 K
	EQ	—	1.5 K	1.5 K	1.5 K	1.5 K	1.5 K	1.5 K	1.5 K	2.25 K
SW-11	WIND	1.0 K	0.25 K	0.75 K	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.75 K
SW-12	WIND	1.0 K	0.25 K	0.75 K	1.0 K	1.0 K	1.0 K	1.0 K	1.0 K	0.75 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K
SW-13	WIND	1.0 K	0.25 K	1.25 K	1.75 K	1.75 K	1.75 K	1.75 K	1.5 K	1.5 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.75 K
SW-14	WIND	1.0 K	0.25 K	0.75 K	1.25 K	1.25 K	1.0 K	1.0 K	1.0 K	1.0 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K
SW-15	WIND	1.0 K	0.25 K	0.75 K	1.25 K	1.25 K	1.0 K	1.0 K	1.0 K	1.0 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K
SW-16	WIND	1.0 K	0.25 K	0.75 K	1.25 K	1.25 K	1.25 K	1.25 K	1.25 K	1.0 K
	EQ	0.25 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.5 K	0.75 K

SHEAR WALL LATERAL LOAD SCHEDULE NOTES:
1. LOADS PROVIDED ARE SERVICE LATERAL LOADS PER LEVEL PER DIRECTION TO BE USED FOR LIGHT GAUGE MANUFACTURER FOR LIGHT GAUGE SHEAR WALL DESIGN. LATERAL LOADS OCCUR IN EITHER DIRECTION. WALL DESIGN SHALL INCLUDE GRAVITY LOADS BASED ON TRIBUTARY AREA AND LOADING SCHEDULE SHOWN ON S001.
2. SEE PLANS FOR LOCATIONS OF LIGHT GAUGE SHEAR WALLS.

Figure 6 (Designers load values for the GC to design with)

Design Codes & Guides

1. AISC Unified Manual 13th Edition
2. ACI 318-08
3. ASCE 7-05
4. International Building Code (IBC) 2006

Deflection Criteria

Typical live load deflections limited to: L/360
Typical total deflections limited to: L/240
Typical construction load deflections limited to: L/360

Drift Criteria

Allowable Building Drift H/400

Inter-story Drift
Wind h/400 to h/600
Seismic 0.015h

Load Combinations

Listed here are the load combinations that are being considered when generating the computer model and analyzing the lateral system. All of these combinations are based on LRFD design method. These load combinations all come from ASCE 7-05.

* $1.4(D + F)$
* $1.2(D + F + T) + 1.6(L + H) + 0.5(Lr \text{ or } S \text{ or } R)$
* $1.2D + 1.6(Lr \text{ or } S \text{ or } R) + (L \text{ or } 0.8W)$
* $1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$
* $1.2D + 1.0E + L + 0.2S$
* $0.9D + 1.6W + 1.6H$
* $0.9D + 1.0E + 1.6H$

Gravity Loads

Live Loads

Here are the live loads for Building 7 and for a further explanation of how these were obtained please refer to Technical Report I. The loads listed here are the ones used throughout this technical report.

Live Loads			
Occupancy	Design Load	Code Required Loads	
		Load	Code
Corridors	100 psf	100 psf	ASCE 7
Offices	100 psf	50 psf	ASCE 7
Seminar Room	100 psf	40 psf	ASCE 7
Mechanical Room	250 psf	125 psf	Light manufacturing
Partition	15 psf	-	-
Roof	30 psf	20 psf	ASCE 7
Dormitory Rooms	40 psf	40 psf	ASCE 7
Lobby	100 psf	100 psf	ASCE 7

Dead Loads

Here are the dead loads for Building 7 and for a further explanation of how these were obtained please refer to Technical Report I. The loads listed here are the ones used throughout this technical report.

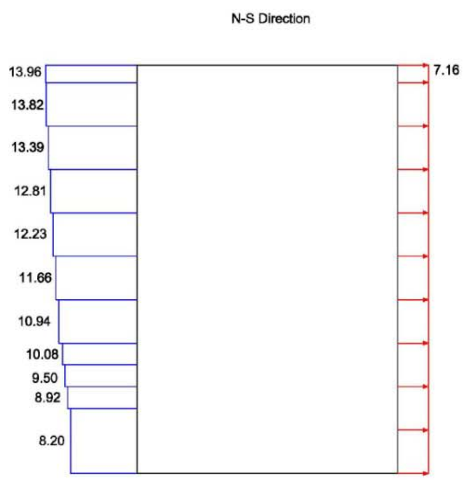
Dead Loads		
Roof Dead Load	Material	Design Weight
	Rigid Insulation	4 psf
	3" Hambro Slab	38 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Roofing Finish	4 psf
	Total Dead Load	54 psf
Typ. Floor Dead Load	Material	Design Weight
	3" Hambro Slab	38 psf
	5" Hambro Slab	63 psf
	M/E/P	5 psf
	Ceiling Finishes	3 psf
	Total Dead Load	46-71 psf

Lateral Loads

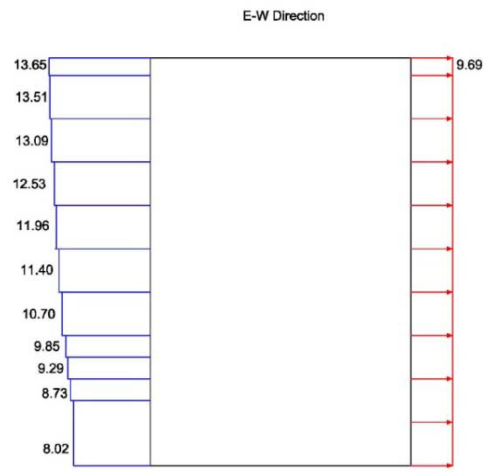
Wind Loads

All wind loads were calculated in accordance with ASCE 7-05, Chapter 6. The analytical method 2 was used to examine lateral wind loads in the North/South direction as well as the East/West direction. Also due to the irregular shape of the building it was necessary to look at the most critical orthogonal for it could possibly control. Since the floor was made of reinforced concrete it was assumed that the building was acting rigid. Thus, wind controlled in the NE/SW direction. These wind loads were calculated and described in Technical Report 1 and should be referred to if a more in-depth explanation is wanted. A brief summary of the loading is listed below. Also refer to Appendix A for more detailed criteria that was used in determining these values

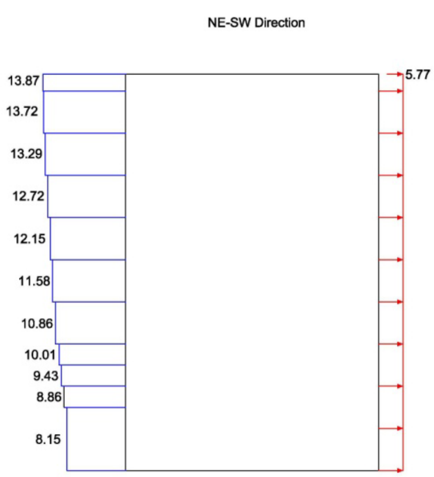
Wind Pressures



Wind Pressure Distribution in the North-South Direction



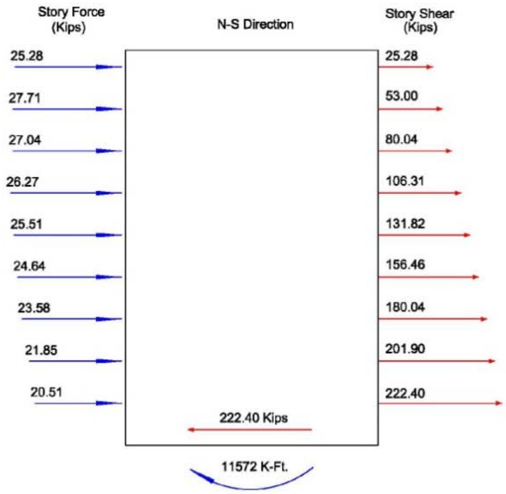
Wind Pressure Distribution in the East-West Direction



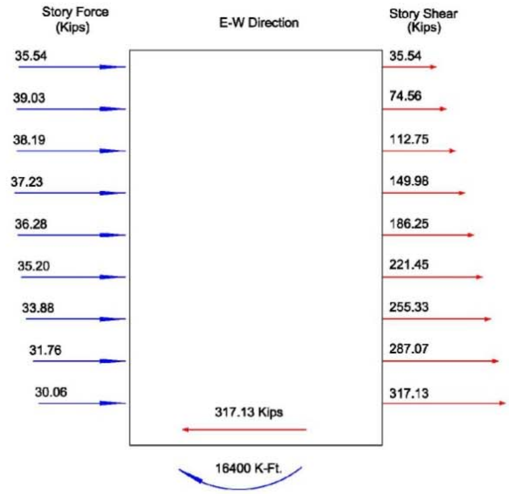
Wind Pressure Distribution in the Northeast-Southwest Direction

All Values on Wind Pressure Step Diagrams are in pounds per square foot (psf). The Blue indicates windward and the red indicate leeward pressures.

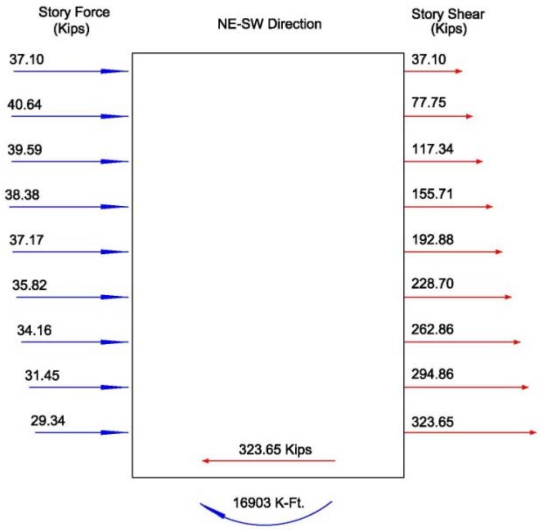
Wind Story Forces and Story Shears



Story Force and Shear in the North-South Direction



Story Force and Shear in the East-West Direction



Story Force and Shear in the Northeast-Southwest Direction

These calculated wind loads were used in the ETABS computer model rather than having ETABS calculate them with its built in function. The reason for this is so that the user knows what variables are being considered and ultimately has more control over the results.

Seismic Loads

The seismic loads were calculated in accordance with ASCE 7-05, Chapter 12 and referencing Chapter 22. After looking at the geotechnical report, it was concluded that the building site is very stiff to hard silty clays at the deep foundation level, resulting in a Site Class C. it was also determined to be Seismic Design Category A. Two simplification assumptions have been made for these calculations: the building is regular in shape and the building is rigid.

ASCE 7 Sect. 11.7 Allows for a simplified procedure because the factors of the site and response allow for a Seismic Design Category A. after looking at both equivalent lateral force procedure (ELF) and the simplified method there are significant differences. These seismic loads were calculated and described in Technical Report 1 and should be referred to if a more in-depth explanation is wanted. A brief summary of the loads is listed below. Refer to Appendix A for more detailed spreadsheets and criteria.

Simplified Base Shear = 1% weight = 119.1 Kips

The simplified procedure was used for the seismic loading in this technical report for it is the lowest and also this is the procedure that the structural designer of Building 7 used. These calculated seismic loads were used in the ETABS computer model rather than having ETABS calculate them with its built in function. The reason for this is so that the user knows what variables are being considered and ultimately has more control over the results.

Controlling Lateral Loads

After completing the wind load analysis and seismic load analysis from Technical Report 1 it can be concluded that wind load clearly will control even without the factors. The Structural Engineer did use the wind load to design the building with the critical combinations that have wind in them. From looking at the load combinations listed earlier in this report the load combination listed below seems to be the most critical and will be used for hand calculations while for modeling all combinations listed in the previous section will be considered.

$1.2D + 1.6W + L + 0.5(Lr \text{ or } S \text{ or } R)$

Modeled Lateral System Analysis

When Building 7 was modeled in ETABS, only the lateral elements and diaphragms were modeled for simplicity and also to reduce possible errors. The shear walls were modeled so that they only resist in-plane shear by the membrane designation. All shear walls were meshed to a maximum dimension of 36” x 36”. The diaphragms were modeled with no materials but instead given a mass equal to that of the total dead weight of the floor. During this process the diaphragms were modeled as so to act perfectly rigid which they would due to them being concrete in Building 7.

Both wind and seismic story forces were applied to each diaphragm at its centroid. A separate static load case was created for each direction so to see the effects more clearly while the LRFD load combinations were used to find the critical loads. Figure 7 below shows the first run of the ETABS model for Building 7’s lateral force resisting system. This model was used to calculate the center of mass and rigidity.

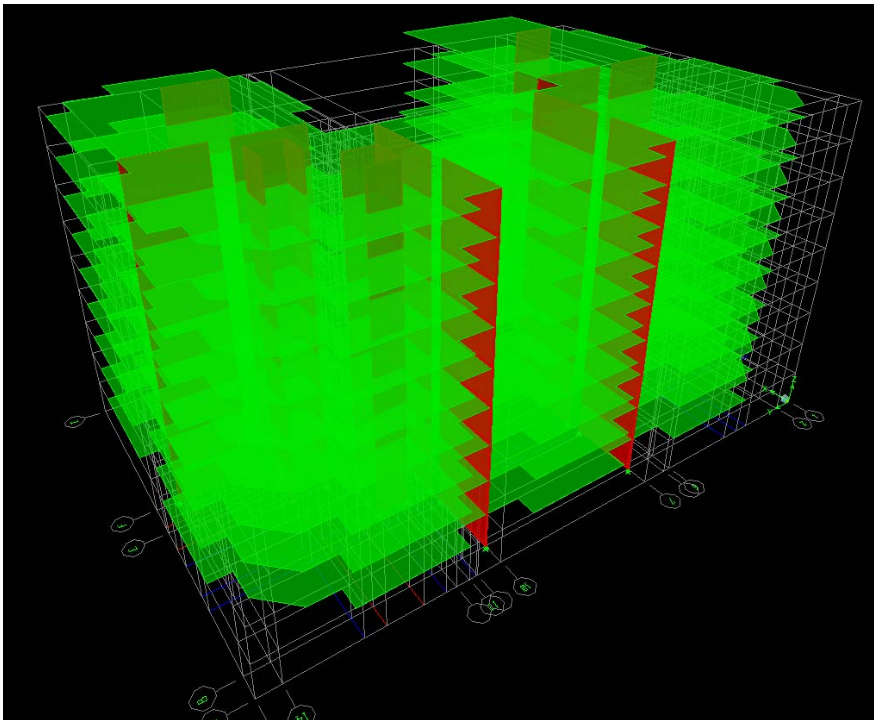


Figure: 7

Similar modeling techniques were used from that of the engineer because they didn't design the upper light-gage shearwalls, they instead modeled each wall the same but only their lengths were adjusted. For this project I based my light-gage design on a simple brace frame, for that's how the engineer based their behavior by for their feasibility study to see if the system would work. From here the typical details and the criteria for the walls and studs were taken and applied it to the shear walls in the model. Below in Figure 8 is the second model from which the rest of this technical report is based on, as well as used to verify hand checks.

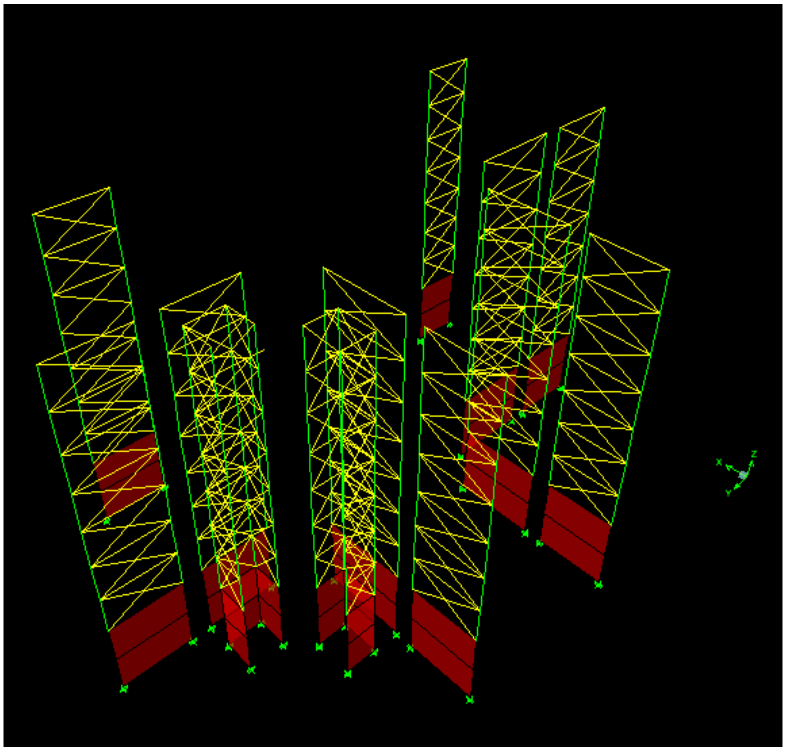


Figure: 8

Due to the number of shear walls in Building 7, key shear walls in each direction are being more closely looked at for simplicity and the results of those walls (3 and 4) are presented in this report on the following pages to come.

Distribution of Lateral Loads

The lateral loads for Building 7 are distributed by the method of relative stiffness. The reason for using relative stiffness is due to the concrete slab and how it acts rigidly. The controlling wind force was resolved into X and Y forces and be applied to the floor. From here the center of mass and center of rigidity were calculated to determine how those forces went into each shear wall. Figure 9 shows the loads path of how the wind force travels through the building and into the lateral system.

The load path for the lateral force through the building is as follows in order:

1. Brick masonry façade
2. Light gage back up studs
3. Concrete slab on the Hambro Floor System
4. Light gage and concrete shear walls
5. Concrete piers and caisson foundation

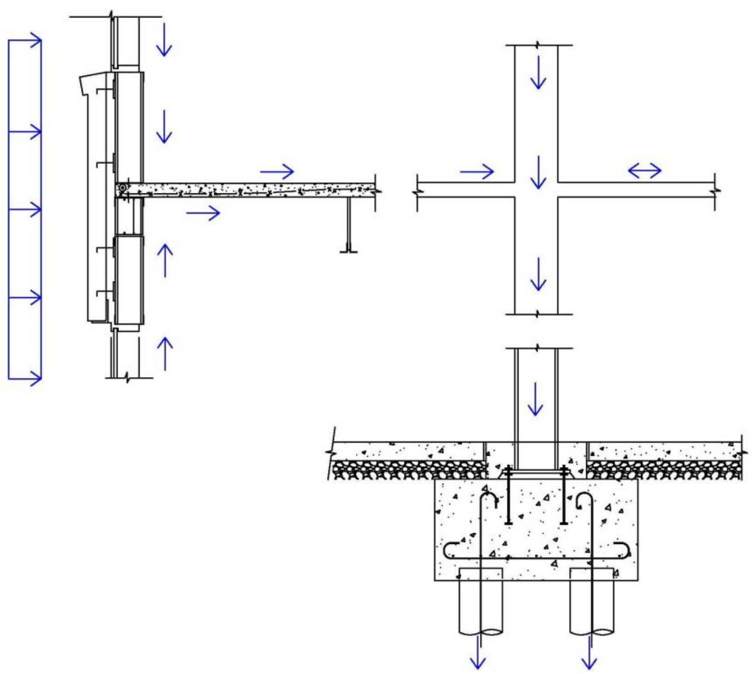


Figure: 9

Shearwall Stiffness

The stiffness of each frame/shear wall is critical in order to determine the forces that go to each due to the diaphragm acting rigidly in plan view. Also the stiffness affects the center of rigidity which in turns determines how much torsion you have on the floor and structure as a whole. The stiffer the shear walls the less deflection you have.

Listed below are the stiffness's for each of the shear walls. Since Building 7 is a low to midrise structure the relative stiffness of each floor shouldn't change much and as a result the overall shear wall stiffness will be used. The process of determining the stiffness was to model each one in SAP and to apply a 1 kip Load at the top and measure its deflection. Stiffness can then be determined by using the equation P=KU.

Shearwall Stiffness's								
Wall	Force	Displacement	Stiffness		Wall	Force	Displacement	Stiffness
SW 1	1 kip	0.68 inch	1.471		SW 9	1 kip	0.38 inch	2.632
SW 2	1 kip	0.38 inch	2.632		SW 10	1 kip	0.38 inch	2.632
SW 3	1 kip	0.38 inch	2.632		SW 11	1 kip	2.37 inch	0.375
SW 4	1 kip	0.38 inch	2.632		SW 12	1 kip	2.37 inch	0.375
SW 5	1 kip	0.38 inch	2.632		SW 13	1 kip	1.65 inch	0.606
SW 6	1 kip	0.38 inch	2.632		SW 14	1 kip	2.37 inch	0.375
SW 7	1 kip	0.38 inch	2.632		SW 15	1 kip	2.37 inch	0.375
SW 8	1 kip	0.38 inch	2.632		SW 16	1 kip	1.65 inch	0.606

Center of Mass and Rigidity

For each diaphragm the center of mass (COM) and center of rigidity (COR) were calculated so that the exact location of the resultant story force was is located. These two points on the diaphragm determine how much eccentricity there will be, which in turn will cause a torsional moment on each floor. A sample calculation was performed on a typical upper level floor plan and with the stiffness's listed above the COM and COR for that floor is:

COM: X= 54.0ft, Y= 83.5ft
COR: X= 70.77ft, Y= 88.17ft

These hand calculated values are very close to the ETABS output and is a check to ensure that the computer model is accurate, which it seems to be. The rest of the COM and COR's were taken from ETABS and are listed in the tables on the next page. The values are almost the same for each floor with respect to each other due mostly to each floor being relatively the same layout and plan area. Refer to Appendix B for more details calculations.

Center of Mass			
Story	Diaphragm	XCM	YCM
ROOF	D1	54.21	83.20
STORY8	D1	53.95	84.06
STORY7	D1	53.95	84.06
STORY6	D1	53.95	84.06
STORY5	D1	53.95	84.06
STORY4	D1	53.95	84.06
STORY3	D1	53.95	84.06
STORY2	D1	53.95	84.06
STORY1	D1	53.95	84.06

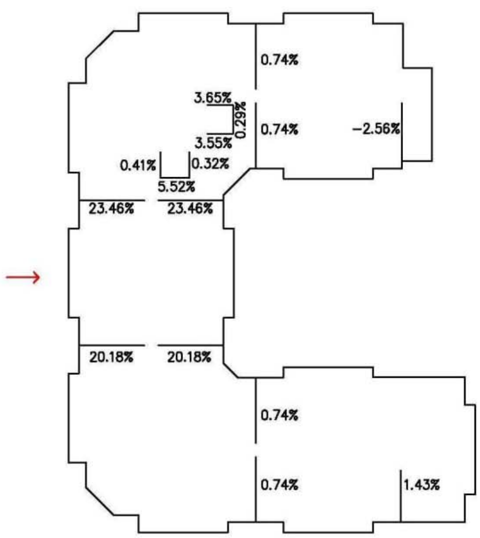
Center of Rigidity			
Story	Diaphragm	XCR	YCR
ROOF	D1	77.28	90.02
STORY8	D1	77.29	90.03
STORY7	D1	77.31	90.03
STORY6	D1	77.34	90.04
STORY5	D1	77.38	90.06
STORY4	D1	77.45	90.11
STORY3	D1	77.56	90.25
STORY2	D1	77.73	90.67
STORY1	D1	77.79	92.27

Torsion Effects

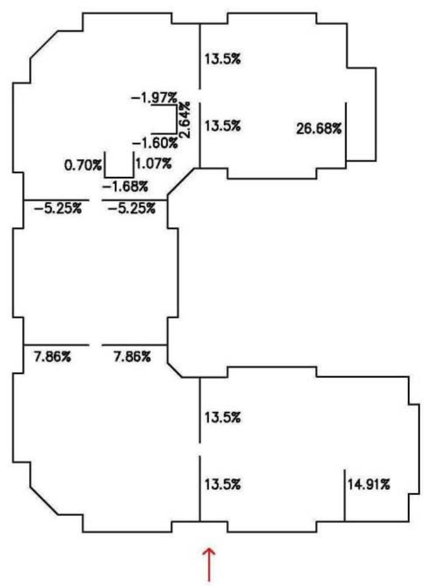
Along with the direct story force applied to each floor, torsion needs to be considered while calculating the lateral forces acting on the system. The COM and COR previously calculated were used to determine how much eccentricity in each direction was there. The Y direction had a close COM and COR resulting in a small torsional moment but the X direction had a larger torsional moment due to the COM and COR being farther apart but not to large in term of the building whole. An analysis was performed to determine the torsional shear on each story caused by wind forces.

With this analysis, a 100 kip load was applied to the floor and the resulting force in each shear wall is the percentage of the total force acting on the diaphragm no matter how large the force is. To find the actual forces the given, wind and seismic loads need to be only multiplied by the correct percentage to get the result now. The Following diagrams show the percentages in each wall along with the direction of the forces for both the N-S direction and the E-W direction.

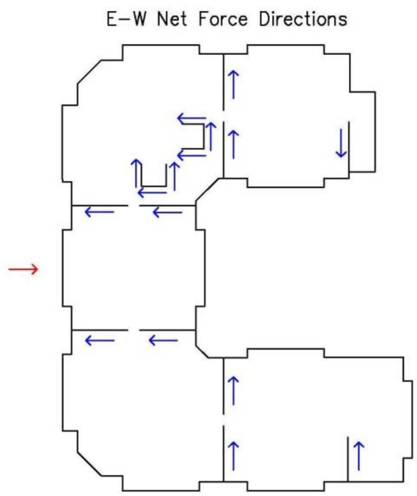
E–W Net Story Force Contributions

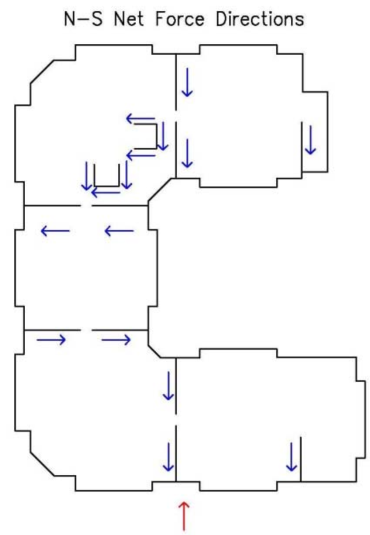


N–S Net Story Force Contributions



Appendix B has the supporting calculations performed. A spreadsheet was created to generate these values due to the large amount of numbers also so there was less chance for error.





Listed here are the forces in each wall for a single floor to show the real loads for the roof level. These loads seem reasonable and look similar to that of the ETABS forces generated. Also listed are the ETABS results for an entire shearwall in the N-S direction and one in the E-W Direction. The loads listed for these include Axial, Shear and Moment on that wall to accurately show what is happening.

Roof Story Shearwall Forces			
	Force		Force
Wall 1	-0.85	Wall 9	0.44
Wall 2	0.44	Wall 10	-1.52
Wall 3	0.44	Wall 11	0.25
Wall 4	11.98	Wall 12	0.19
Wall 5	11.98	Wall 13	3.28
Wall 6	13.93	Wall 14	2.11
Wall 7	13.93	Wall 15	2.16
Wall 8	0.44	Wall 16	0.17
		Sum	59.37

Shearwall 3 Story Forces						
Story	Pier	Load	Location	Axial	Shear	Moment
STORY9	WALL 3	COMB1Y	Bottom	-64.8	11.6	115.5
STORY8	WALL 3	COMB1Y	Bottom	-129.7	21.1	326.4
STORY7	WALL 3	COMB1Y	Bottom	-194.5	31.4	640.3
STORY6	WALL 3	COMB1Y	Bottom	-259.4	41.6	1056.6
STORY5	WALL 3	COMB1Y	Bottom	-324.2	51.6	1572.5
STORY4	WALL 3	COMB1Y	Bottom	-389.1	61.0	2182.9
STORY3	WALL 3	COMB1Y	Bottom	-453.9	69.5	2877.7
STORY2	WALL 3	COMB1Y	Bottom	-518.8	74.6	3623.4
STORY1	WALL 3	COMB1Y	Bottom	-583.6	63.6	4259.8

Shearwall 4 Story Forces						
Story	Pier	Load	Location	Axial	Shear	Moment
STORY9	WALL 4	COMB2X	Bottom	-54.3	8.4	84.4
STORY8	WALL 4	COMB2X	Bottom	-108.6	20.6	290.4
STORY7	WALL 4	COMB2X	Bottom	-162.8	31.8	608.4
STORY6	WALL 4	COMB2X	Bottom	-217.1	42.5	1033.4
STORY5	WALL 4	COMB2X	Bottom	-271.4	52.9	1562.6
STORY4	WALL 4	COMB2X	Bottom	-325.7	63.3	2195.2
STORY3	WALL 4	COMB2X	Bottom	-380.0	74.0	2934.9
STORY2	WALL 4	COMB2X	Bottom	-434.3	86.5	3799.7
STORY1	WALL 4	COMB2X	Bottom	-488.5	93.2	4731.8

Building Drift Results

Deflection is a serviceability issue that should be limited as much as possible while staying within reason. The drift of a building is inversely proportional to the total stiffness of the lateral structure. The maximum building deflection due to wind is limited to h/400 of the total height of the building. The deflection values for Building 7 in this report are taken from ETABS at the center of mass of each floor to give an overall reference to how the building moves due to lateral loads. The following table summarizes the overall building deflection.

In the case of Building 7 the maximum building deflection is 2.88 inches

Story Drift was also calculated and to ensure that no one story drifts to much causing issues on that floor. The same limitation was used for building deflection, h/400. These values were also taken from the center of mass of each floor.

In the case of Building 7 the maximum story drift is 0.30 inches, since all floors have the same height this is valid for all stories.

The table below summarizes the story drift and also the building deflection in both the X and Y direction.

Diaphragm Drifts			
Story	Load	DriftX (inches)	DriftY (inches)
STORY9	Wind X-Dir.	0.17	0.04
STORY9	Wind Y-Dir.	0.03	0.08
STORY8	Wind X-Dir.	0.17	0.05
STORY8	Wind Y-Dir.	0.03	0.09
STORY7	Wind X-Dir.	0.17	0.05
STORY7	Wind Y-Dir.	0.03	0.08
STORY6	Wind X-Dir.	0.16	0.04
STORY6	Wind Y-Dir.	0.02	0.08
STORY5	Wind X-Dir.	0.15	0.04
STORY5	Wind Y-Dir.	0.02	0.07
STORY4	Wind X-Dir.	0.12	0.03
STORY4	Wind Y-Dir.	0.02	0.06
STORY3	Wind X-Dir.	0.08	0.02
STORY3	Wind Y-Dir.	0.01	0.03
STORY2	Wind X-Dir.	0.00	0.00
STORY2	Wind Y-Dir.	0.00	0.00
STORY1	Wind X-Dir.	0.00	0.00
STORY1	Wind Y-Dir.	0.00	0.00
	Total Drift	1.16	0.77

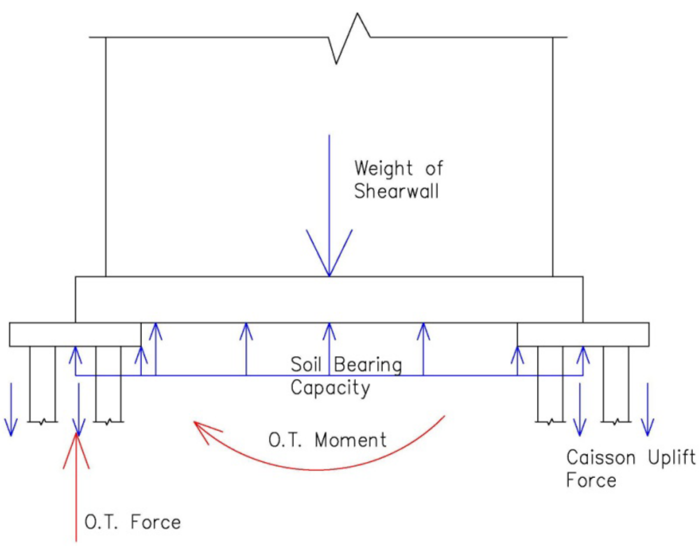
From looking at the Data from ETABS is can be concluded that no one story drifted past the allowable limit. Also the total deflection was within range of the allowable deflection. The building drift is larger in the X-Direction which seems to make sense due to the stiffness in that direction. The overall deflections are within a normal range and seem valid.

Foundation Overturning Inspection

Overturning issues in foundations arise when the forces on the lateral elements are greater than the weight that the lateral element. Also the soil bearing capacity has an effect on overturning by how much load it can take before a strength failure or a bearing failure occurs. When the lateral moments and axial forces are not balanced out by the weight and soil capacity, then the foundation wants to start and tip over inside the ground. One end tends to lift up while the other often likes to sink into the soil. The figure below shows how the forces are interacting with each other. The red loads are trying to force the foundation to rotate as the blue loads are trying to resist the movement.

In the case of Building 7, overturning was looked at on two shear walls, on in the E-W direction and the other in the N-S direction. Upon performing the calculations it was determined that there are overturning issues in respect to the building weight and the lateral overturning moment at the base of the foundation. The lateral moment's resulting axial force was twice as large as the resulting weight axial load.

This does not mean that the shear walls are not stable for the foundation system supporting them are drilled cassions that go down to bedrock. These foundations can support uplift by both friction and also by acting in tension from being supported at the end by being anchored to the bedrock. These more detailed calculations were not performed for time and difficulty reasons. The supporting calculations for overturning are found in Appendix C that shows that there are issues.



Strength Design/Check

Strength checks were performed on the lateral elements of Building 7 to see if they could carry the loads found earlier in this report as well as past reports. Two walls were looked at for strength checks, one in the E-W direction and the other in the N-S direction. The lower more critical sections of these two walls were chosen for they had the highest forces acting on that particular area.

Since the lower levels are made of concrete the check was based off of ACI 318-08 and the previsions for reinforced shear walls. It was concluded that no shear reinforcing was needed from the strength check, but based on a minimum steel ratio, reinforcing was determined and after looking at the construction documents my design was equal to the reinforcing for flexure and shear. The only difference is that I found to only need one layer of shear reinforcing but the designer used 2 and from looking at the wall detail. It is quite possible that this was chosen for construction and the ease of building a cage rather than trying to center a single layer within the wall. Refer to Appendix D for the calculations and assumptions used for the check.

Conclusion

After completing the lateral analysis of the Building 7, it can be concluded that lateral loads are applied in the form of seismic and wind forces which cause shears at each story that are resisted by the shear walls placed throughout the building. The floor diaphragms act rigidly so that the lateral loads to travel through the structure on the basis of relative stiffness. After creating an accurate computer model, the building is acceptable for the drift limitations with regards to wind and since the seismic forces were so low they would not control after being factored. The overall drift of Building 7 was 1.26 inches in the X-Direction and 0.77 inches in the Y direction.

In general, torsional shear does not seem to be a major issue, for the force contribution is not large. A spreadsheet was developed to show much of each story force goes to each shear walls, the results show that the most any shear wall carries is 27% while the minimum is 0.41%. The center of rigidity and mass were fairly close in the N-S direction but improvements could be made in the E-W direction for it was 3.5 times as far apart as the other direction.

Overturning was looked at and there seems to be issues regarding the moment force in the represented shearwalls being larger than the offset weight of the shearwall. A more complete analysis should performed though it is reasonable to say that the rest of the building would help keep the overturning issue down. Also that the bearing capacity of the soil and the caissons can take an uplift force should be more than enough to balance the forces out. Strength checks were looked at for one shear wall in each direction at the base where the forces were the highest. The designed shearwall had the correct amount of flexural reinforcing to resist the forces found and had extra shear reinforcing, most likely for construction issues and ease of building a rebar cage for inside the formwork.

Overall it is felt that by finishing this technical report, a better understanding of lateral load distribution has been gained along with a knowledge of how lateral resisting elements work together. A further investigation would need to be performed in the future depending on what changes will be made and looked at during the spring semester. Perhaps a simpler lateral system with less volume of frames/walls may be more economical to look at or a different material for the lateral system.

Appendices

The pages following this page contain the following Appendices:

A: Wind and Seismic Load Calculations

B: Lateral Analysis and Distribution Checks

C: Foundation Overturning Check

D: Shearwall Strength Design Check

Appendix A: Wind and Seismic Calculations

Wind Criteria & Calculated Variables

Wind Criteria		
Basic Wind Speed (V)	90 mph	figure 6-1C
Wind Exposure Category	B	
Occupancy Factor	II	
Importance Factor	1	table 6-1
Wind Directionality Factor (K_d)	0.85	table 6-4
Topographic factor (K_{zt})	1	sect 6.5.7.1-2
Number of Stories	9	
Building Height (Ft.)	94	
N-S Building Length (Ft.)	169.75	
E-W Building Length (Ft.)	133.5	
NE-SW Building Length (Ft.)	200.75	
NW-SE Building Length (Ft.)	210.75	
L/B in N-S Direction	1.27	
L/B in E-W Direction	0.79	
L/B in NE-SW Direction	1.05	

Gust Factor			
Variable	Wind Direction		
	N-S	E-W	NE-SW
Stiffness	Rigid	Rigid	Rigid
B (Feet)	133.50	169.75	210.75
L (Feet)	169.75	133.50	200.75
h (Feet)	94.00	94.00	94.00
c	0.30	0.30	0.30
Z	56.40	56.40	56.40
I_z	0.27	0.27	0.27
Lz	202.95	159.61	240.02
$\in$	0.33	0.33	0.33
Q	0.77	0.73	0.76
gQ & gv	3.40	3.40	3.40
G	0.80	0.77	0.79

Wind Story Force, Shear and Overturning Moment Spreadsheets

Wind (North-South)							
Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	21.00	25.28	25.28	2275.40	
8	80	10	20.54	27.71	53.00	4492.57	
7	70	10	19.97	27.04	80.04	6385.40	
6	60	10	19.39	26.27	106.31	7961.77	
5	50	10	18.82	25.51	131.82	9237.03	
4	40	10	18.10	24.64	156.46	10222.79	
3	30	10	17.23	23.58	180.04	10930.27	
2	20	10	16.37	21.85	201.90	11367.35	
1	10	10	15.36	20.51	222.40	11572.41	
ground	0	0	0.00	0.00	222.40	11572.41	Total

Wind (East-West)							
Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	23.28	35.54	35.54	3198.50	
8	80	10	22.78	39.03	74.56	6320.54	
7	70	10	22.21	38.19	112.75	8993.51	
6	60	10	21.65	37.23	149.98	11227.08	
5	50	10	21.09	36.28	186.25	13040.86	
4	40	10	20.38	35.20	221.45	14448.76	
3	30	10	19.54	33.88	255.33	15465.23	
2	20	10	18.7	31.73	287.07	16099.92	
1	10	10	17.71	30.06	317.13	16400.55	
ground	0	0	0	0.00	317.13	16400.55	Total

Wind (East-West)							
Level	Height (Feet)	Tributary Area (Feet)	Total (psf)	Story Force (Kips)	Story Shear (Kips)	Overturning Moment (k-ft)	
roof	90	9	19.57	37.10	37.10	3339.42	
8	80	10	19.07	40.64	77.75	6590.87	
7	70	10	18.5	39.59	117.34	9362.13	
6	60	10	17.92	38.38	155.71	11664.78	
5	50	10	17.35	37.17	192.88	13523.07	
4	40	10	16.64	35.82	228.70	14955.75	
3	30	10	15.78	34.16	262.86	15980.62	
2	20	10	14.92	31.45	294.31	16609.71	
1	10	10	13.92	29.34	323.65	16903.08	
ground	0	0	0	0.00	323.65	16903.08	Total

Seismic Loads

Building Weight

The effective weight of the building was first calculated by determining the weight of each of the building's 8 floors and roof. This included the exact weights of all slabs, bearing walls, partitions, exterior brick façade, and the superimposed dead loads. Adding the weights of the floors resulted in the building's effective weight. From here the seismic base shear was calculated.

Roof Weight						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Roof Area
	54	20<30 so NA.	-	47	760	14750
	Total Roof Weight =	1117.98	kips			

Typical Floor Weight Upper Floors						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Floor Area
	46	-	15	47	760	14750
	Total Floor Weight =	1256.95	kips			

Typical Floor Weight Lower Floors						
	Dead Load	Snow Load	Partition Load	Perimeter Wall Load	Perimeter Length	Floor Area
	71	-	15	47	760	14750
	Total Floor Weight =	1625.7	kips			

Appendix B: Lateral Analysis and Distribution Checks

Center of Rigidity Calculation Spreadsheets

Center of Rigidity X-Direction			
	Xi	Ki	XiKi
Wall 1	107.25	1.47	157.72
Wall 2	60.20	2.63	158.42
Wall 3	60.20	2.63	158.42
Wall 4	0	0	0
Wall 5	0	0	0
Wall 6	0	0	0
Wall 7	107.25	2.63	282.24
Wall 8	60.20	2.63	158.42
Wall 9	60.20	2.63	158.42
Wall 10	0	0	0
Wall 11	29.50	0.37	11.05
Wall 12	38.80	0.37	14.53
Wall 13	0	0	0
Wall 14	0	0	0
Wall 15	0	0	0
Wall 16	52.80	0.61	32.00
		15.98	1131.22

COR X	70.7739

Center of Rigidity Y-Direction			
	Yi	Ki	XiKi
Wall 1	0	0	0
Wall 2	0	0	0
Wall 3	0	0	0
Wall 4	60.20	2.63	158.42
Wall 5	60.20	2.63	158.42
Wall 6	106.80	2.63	281.05
Wall 7	106.80	2.63	281.05
Wall 8	0	0	0
Wall 9	0	0	0
Wall 10	0	0	0
Wall 11	0	0	0
Wall 12	0	0	0
Wall 13	114.20	0.61	69.21
Wall 14	128.20	0.37	48.01
Wall 15	137.50	0.37	51.50
Wall 16	0	0	0
		11.88	1047.67

COR Y	88.18

Direct Force and Torsional Forces Calculation Spreadsheets

North South Direction

North South Direction				
Wall	Ki	Di	KiDi	$KiDi^2$
Wall 1	1.471	36.4	53.52941	1948.471
Wall 2	2.632	-10.58	-27.8421	294.569
Wall 3	2.632	-10.58	-27.8421	294.569
Wall 4	2.632	28	73.68421	2063.158
Wall 5	2.632	28	73.68421	2063.158
Wall 6	2.632	-18.7	-49.2105	920.237
Wall 7	2.632	-18.7	-49.2105	920.237
Wall 8	2.632	-10.58	-27.8421	294.569
Wall 9	2.632	-10.58	-27.8421	294.569
Wall 10	2.632	36.4	95.78947	3486.737
Wall 11	0.375	-41.25	-15.4494	637.289
Wall 12	0.375	-31.9	-11.9476	381.127
Wall 13	0.606	-26	-15.7576	409.697
Wall 14	0.375	-40	-14.9813	599.251
Wall 15	0.375	-49.3	-18.4644	910.296
Wall 16	0.606	-17.8	-10.7879	192.024
Sum	27.865			15709.959

North South Direction				
	Ki	F direct (%)	F Torsion(%)	F net (%)
Wall 1	1.471	9.20	5.71	14.91
Wall 2	2.632	16.46	-2.97	13.50
Wall 3	2.632	16.46	-2.97	13.50
Wall 4	0	0	7.86	7.86
Wall 5	0	0	7.86	7.86
Wall 6	0	0	-5.25	-5.25
Wall 7	0	0	-5.25	-5.25
Wall 8	2.632	16.46	-2.97	13.50
Wall 9	2.632	16.46	-2.97	13.50
Wall 10	2.632	16.46	10.21	26.68
Wall 11	0.375	2.34	-1.65	0.70
Wall 12	0.375	2.34	-1.27	1.07
Wall 13	0	0	-1.68	-1.68
Wall 14	0	0	-1.60	-1.60
Wall 15	0	0	-1.97	-1.97
Wall 16	0.606	3.79	-1.15	2.64
Sum	15.984	100.00	-0.052	99.948

M Torsion	1675	K-Ft.

East West Direction

East West Direction				
Wall	Ki	Di	KiDi	$KiDi^2$
Wall 1	1.471	-36.4	-53.5294	1948.471
Wall 2	2.632	10.58	27.84211	294.569
Wall 3	2.632	10.58	27.84211	294.569
Wall 4	2.632	-28	-73.6842	2063.158
Wall 5	2.632	-28	-73.6842	2063.158
Wall 6	2.632	18.7	49.21053	920.237
Wall 7	2.632	18.7	49.21053	920.237
Wall 8	2.632	10.58	27.84211	294.569
Wall 9	2.632	10.58	27.84211	294.569
Wall 10	2.632	-36.4	-95.7895	3486.737
Wall 11	0.375	41.25	15.44944	637.289
Wall 12	0.375	31.9	11.94757	381.127
Wall 13	0.606	26	15.75758	409.697
Wall 14	0.375	40	14.98127	599.251
Wall 15	0.375	49.3	18.46442	910.296
Wall 16	0.606	17.8	10.78788	192.024
Sum	27.865			15709.959

East West Direction				
	Ki	F direct (%)	F Torsion(%)	F net (%)
Wall 1	0	0	-1.43	-1.43
Wall 2	0	0	0.74	0.74
Wall 3	0	0	0.74	0.74
Wall 4	2.632	22.15	-1.97	20.18
Wall 5	2.632	22.15	-1.97	20.18
Wall 6	2.632	22.15	1.32	23.46
Wall 7	2.632	22.15	1.32	23.46
Wall 8	0	0	0.74	0.74
Wall 9	0	0	0.74	0.74
Wall 10	0	0	-2.56	-2.56
Wall 11	0	0	0.41	0.41
Wall 12	0	0	0.32	0.32
Wall 13	0.606	5.10	0.42	5.52
Wall 14	0.375	3.15	0.40	3.55
Wall 15	0.375	3.15	0.49	3.65
Wall 16	0	0	0.29	0.29
Sum	11.881	100.00	0.013	100.013

M Torsion	420	K-Ft.

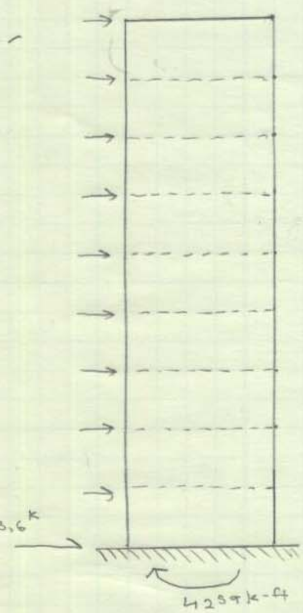
Appendix C: Foundation Overturning Check

Technical Report 3.

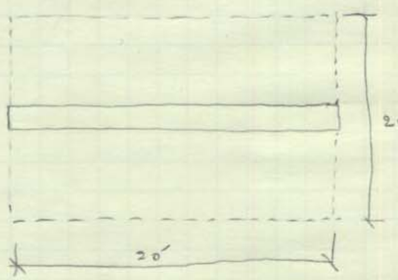
Foundation Overturning.

Shear wall 3

The forces came from earlier calculations / the computer and are allready factored.



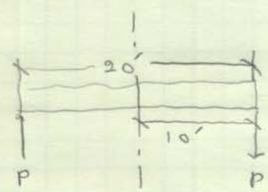
Weight



$$W = 162\,\text{psf}\,(400)\,9$$
$$= 583.6^k$$

Axial caused by moment

$$P = \frac{4259^{k\text{-}ft}}{10'} = 425^k$$



• saying 1/2 axial weight is at each side.

$$\frac{583.6^k}{2} = 292^k$$

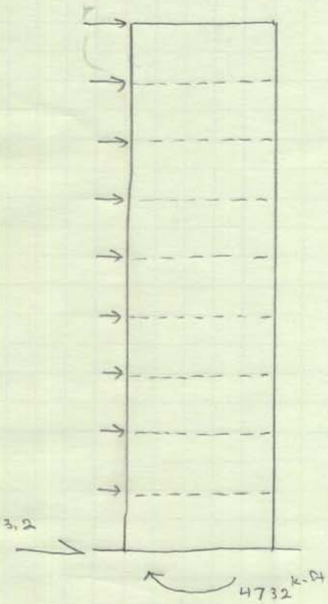
note this $292^k < 425^k$ so overturning would be an issue based on just Forces but the foundation per geo tech report can take uplift.

1/2

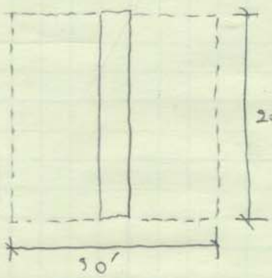
Tech 3

Shear wall 4

These forces come from earlier calculations / the computer model and they are all ready factored.

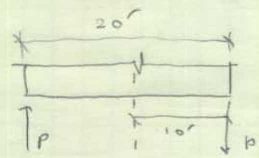


Weight



$W = 90.5(600)9/1000$
$= 488.5^{k}$

Axial caused by moment



$P = \frac{4732}{10'} = 473^{k}$

, saying 1/2 axial weight is at each side

$\frac{488.5}{2} = 244.3^{k}$

note this $244.3^{k} < 473^{k}$, so overturning would be an issue based on just forces but the foundation per geotech code / report the caissons can take uplift.

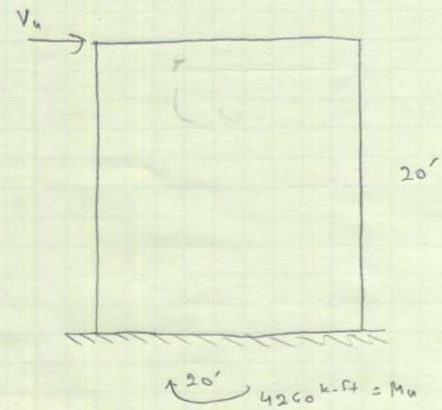
2/2

Appendix D: Shearwall Strength Design Check

Tech Report 3

Strength Check Shearwall 3

$F'_c = 6000$ psi
$F_y = 60$ ksi
$h = 10''$
$V_u = 63.7^k$



• max shear strength

$d = 0.8(20 \times 12) = 192$

$\phi V_n = 0.75(10)\sqrt{6000}\,(10)(192)$
$= 1115.4 > 63.7^k$
so ok.

• shear strength by Vc

$V_c = 2\sqrt{6000}\,(10)(192)$
$= 297.4^k$

$V_u \geq 0.5(0.75)(297.4)$
$= 111.54^K$ so Sect 11.10.9

Since $0.75 V_c = 222.8 > V_u$
no shear reinf needed.

• min reinf

$0.0025 = \frac{A_v}{sh}$

$A_v = 0.0025(12)(10)$
$= 0.3$ in

∴ use #5 bar each way

1/4

Tech 3

Flex Reinf

$M_n = A_s f_y j d$
$A_s f_y\, 0.9 d$

$$M_u = \phi M_n$$

$$\frac{4260(12000)}{2} = 0.9 A_s (60{,}000)(0.9)(192)$$

$A_s = 2.74\ in^2$

use #5's
Try (9) #5 Area = 2.79

• check tension Controlled

$$a = \frac{2.79(60)}{0.85(6)(10)} = 3.28$$

$$\beta = 0.85 - 0.05\ \frac{6000-4000}{1000} = 0.75$$

$$c = \frac{3.28}{0.75} = 4.37$$

$$\varepsilon_t = \frac{0.003}{4.37}(192 - 4.37) = 0.129 > 0.005 \therefore \phi = 0.9$$

so good, the design is the same as the structural engineer's.

2/4

Technical Report 3

• Flexural Reinf

$M_n = A_s f_y jd$ $\qquad jd \approx 0.9d$

$= A_s f_y \, 0.9\, d$

Mn

$\frac{4732(12000)}{2} = 0.9 A_s (60{,}000)(0.9)(192)$

$A_s = 3.04 \text{ in}^2$

Use (10) #5 bars $\quad A_s = 3.1 \text{ in}^2$

• Check tension controlled

$a = \frac{3.1(60)}{0.85(6)(10)} = 3.65$ $\qquad \beta = 0.85 - 0.05\frac{6000-4000}{1000} = 0.75$

$c = \frac{3.65}{0.75} = 4.87$

$\varepsilon_t = \frac{0.003}{4.87}\left(192 - 4.87\right) = 0.115 > 0.005 \quad \therefore \phi = 0.9$

So good design is the same as the structural engineers.

4/4