G.Muttrah Commercial & Residential Complex

Muscat, Sultanate of Oman

Technical Report III

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Structural Option

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Executive Summary

The lateral system of the G.Muttrah Commercial and Residential complex consists of ten shear walls at the core of the building. Three of the shear walls are connected and the majority run along the North-South direction.

In this technical report, the lateral system of the G.Muttrah Complex will be analyzed using ETABS, a three-dimensional structural building design and analysis software. Hand calculations will also be performed to check the results from ETABS in order to have a more accurate analysis.

Since the British Standards direct the design, the metric unit was used in the original design of the G.Muttrah building. This report will however analyze the building using United States Customary System (English units). The conversions will be accurately approximated and also increased or decreased depending on the calculation in order to obtain a conservative result. Values will hence be reported in English units.

The results obtained from ETABS were close to the hand calculations, thus proving that they are correct. The difference in the values is caused by the over simplification of the hand methods. The deflection values of the building under the lateral loads calculated were checked and satisfied the requirements. The story drifts and shear checks of the members also satisfied the requirements, while a torsional shear and moment was discovered to drive the design due to the layout of the shear walls. The Overturning moment was also large which explained the mat foundation used by the engineer.

The ETABS model turned out to be an efficient way of modeling the later system, but a more complete model design of the building would yield more accurate results since other building members such as columns can add to the stiffness and resistance of torional forces. Further details of the analysis are provided in the report.

Introduction

The G.Muttrah Commercial & Residential Complex is a mixed use building in a commercially developing region in the city of Muscat, Sultanate of Oman. Covering an area of approximately 280,000 square feet, the reinforced concrete building will consist of eight floors excluding the parking at the basement level. Retail space will occupy the ground floor, offices in the second floor and 96 apartments in the rest of the 6 floors. A set back of about 35 feet from the north side starts from the fourth floor onwards. The parking garage in the basement will serve 115 slots for the tenants due to the limited parking spaces in the area. More parking spaces will be available around the perimeter of the building which will only provide space for 63 cars.

The typical floor height is 10 ft for the basement level, 14 ft for the retail, 12 ft for the offices and 10 ft on the rest of the residential floors. A flat roof is used to place all the HVAC equipment. The plot has a slope of about 10 ft from the northwest corner to the southeast corner. This slope is used to incorporate the basement level as a parking garage. The ground level is set at 2.6 ft cm below grade while the basement level floor is constructed at 12 ft below grade (Figure 1). Like a typical parking garage, the concrete reinforced columns are placed in a rectangular grid in order to accommodate all the spaces and for ease of transportation.

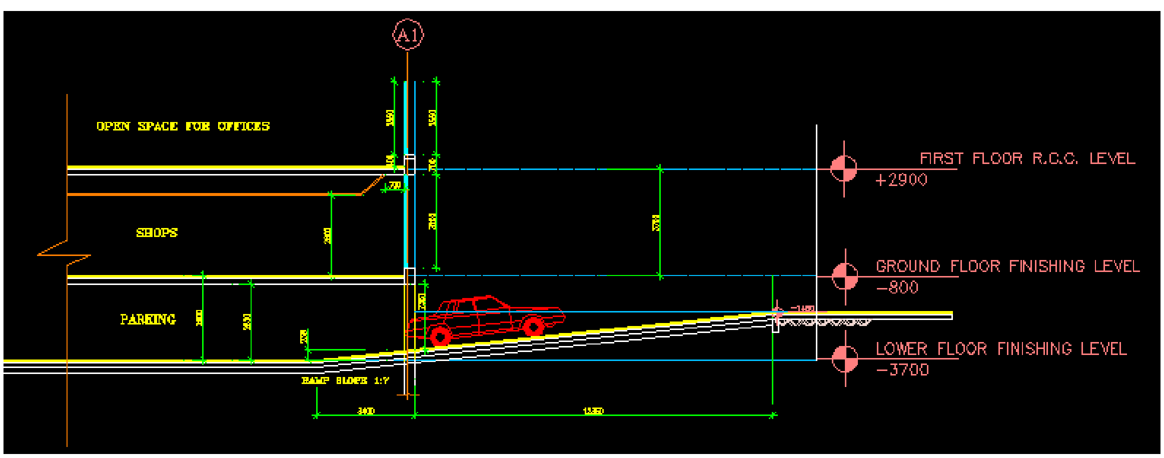


Figure 1: A section showing the entrance of the garage level

Structural System Overview

Summary

The G.Muttrah Commercial & Residential Complex is a reinforced concrete frame building with shear walls. The flooring system consists of a combination of reinforced concrete flat plate slabs on some floors, and typical two way slabs on beam frame system on the others. The dimensions of the building plan are about 300ft by 132ft. The typical roofing/floor system span is between 10ft and 30ft. The material strength used is approximately 5,500 psi strength concrete and 60,000 psi steel strength. Finally, the roof of the building is a 6 in thick slab that only has to carry the loads from the mechanical equipment on the rooftop. There are no snow loads for this building since the weather statistics show that the chances of snow in Oman are slim to none.

Floor Slabs & Beams

The second and third floor of the G.Muttrah complex consists of a flat plate slab system with drop panels. The floors have 2 varying slab thickness; one at 10in slab thickness with a drop panel of 14in and reinforcement of # 3's and #4's in U.S standard. The second is at 14in slab thickness with a drop panel of 22in and reinforcement of #5's (see figure 2). The rest of the floors have a typical two-way slab system with slabs thickness varying from 6in to 8in. The slabs are supported by the usual rectangular beams that range from 6in x 20in to 32in x 20in.

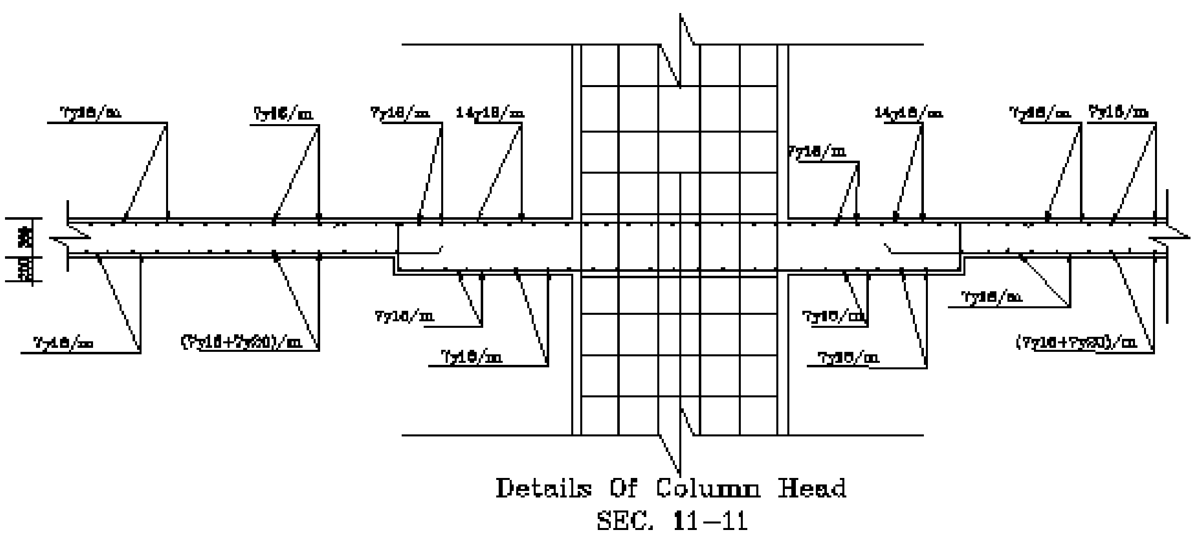


Figure 2: Flat plate slab and column on the second floor

Foundation & Columns

As for the foundation, a 4 ft thick mat slab is used to carry the loads from the different columns. The mat slab is reinforced with 2 layers of #20's and 2 layers of # 10's mesh running both ways. Gravity loads from the building are carried down through reinforced concrete columns that are aligned together in a simple grid, with the majority running throughout the entire building. The columns have a base at the foundation slab level (see figure 2) and range between 14in x 21in to 28in x 47in.

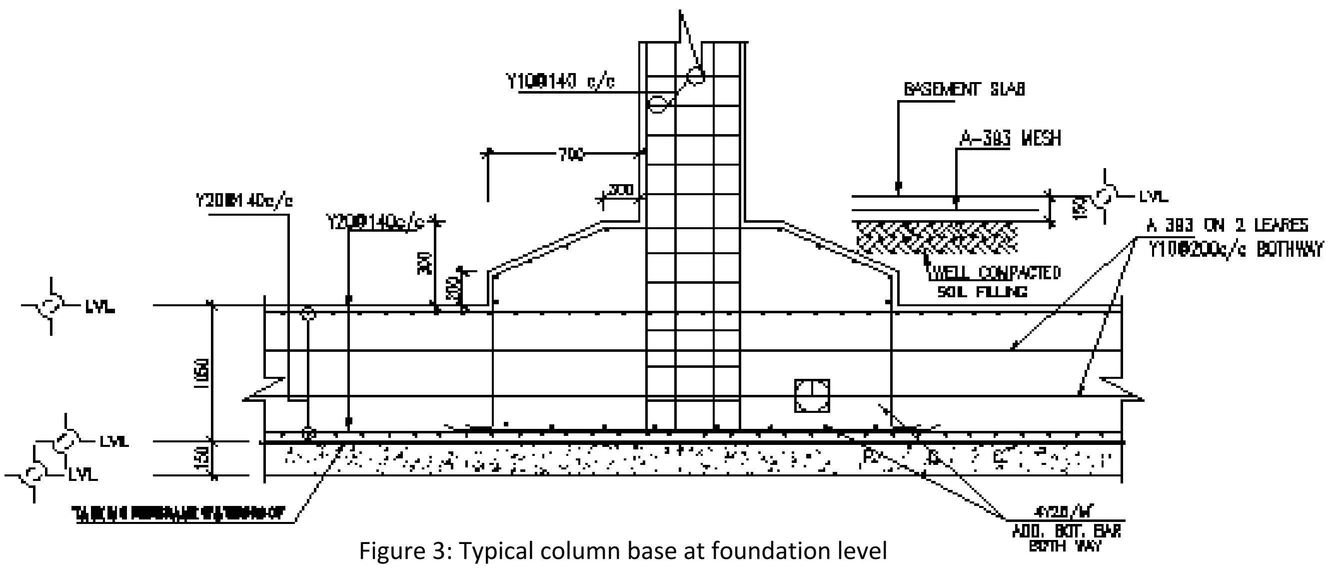


Figure 3: Typical column base at foundation level

Lateral System

Shear walls are used to resist the lateral force in the G.Muttrah complex. The shear walls are located in the core of the building and start at a thickness of 14in at the basement and decrease to 8in as they reach the roof. These walls run in the North-South direction which is expected since that is the weaker axis due to the wind direction and exposure to a larger surface area. There is only one shear which runs in the East-West direction.

Design Loads

The codes for the original design of the building are from The British Standards (BS8110). The codes used by the engineer are currently unavailable for comparison; however, below is a list of the loads from ASCE 7-05 which were used in this analysis of this report.

Live Loads:

Table-1

Occupancy	Load (psf)
Parking	40
Entry	100
Office	50
Retail	100
Residential	40
Corridor	100
Restrooms	100
Roof	20
Stairs	100
Ramps (vehicle)	250
Sidewalk	250
Exterior	100

Dead Loads

Table-2

Material/Occupancy	Load (psf)
Normal Weight Concrete	150 pcf
Floor Superimposed	15 psf
Roof Superimposed	30 psf
Facade	30 psf

Loads and Load Cases

The loads used in this report are from the wind and seismic analysis from Tech report 1. As mentioned in Tech report 1, the wind speed in the sultanate of Oman averages around 75mph which results in loads significantly lower than those expected in the United States. The seismic loads were also estimated based on the lowest loads allowed by code since the Sultanate of Oman is located in a seismic safe zone. More calculations and details for lateral loads can be found in Appendix A. The following is a summary of the loads:

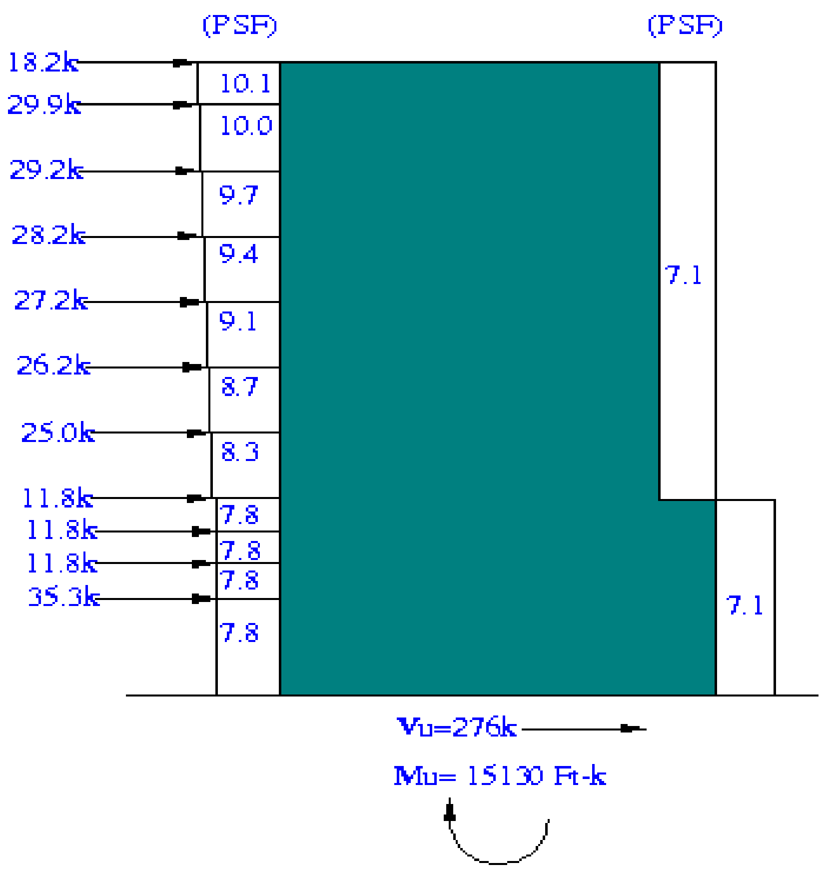


Figure 4: Wind Loads on North-South Face

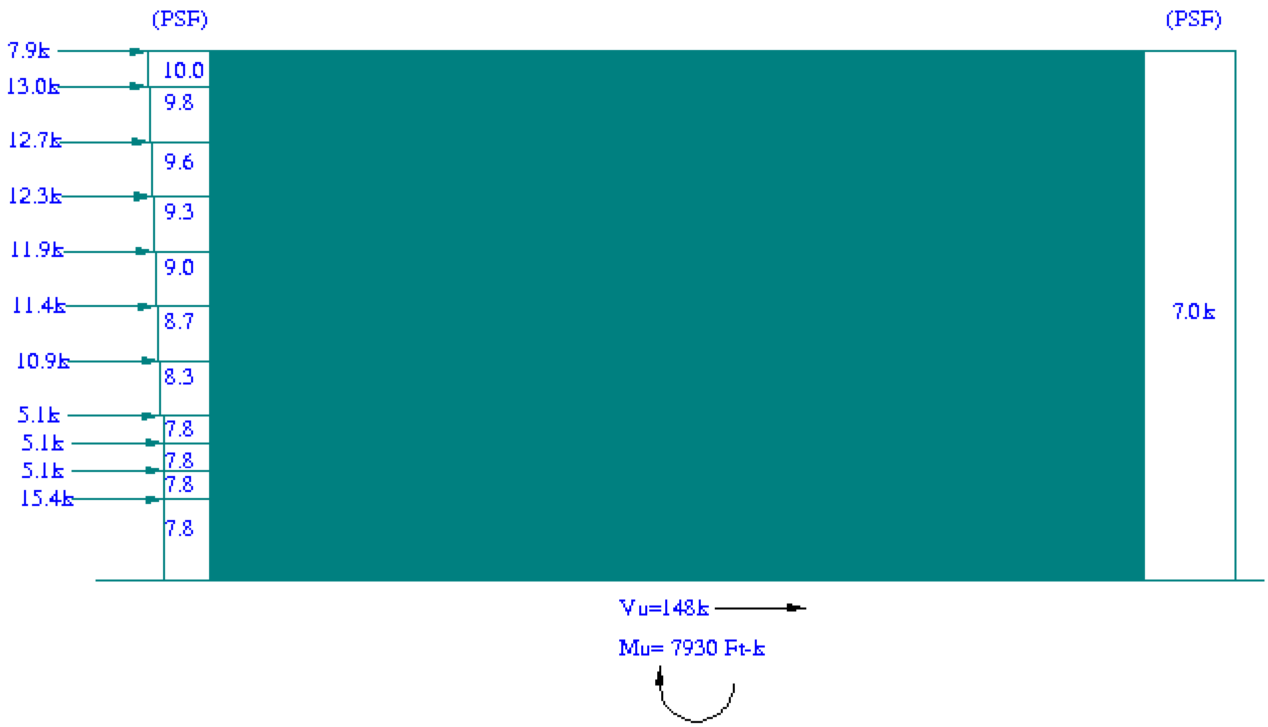


Figure 5 Wind Loads on East-West Face

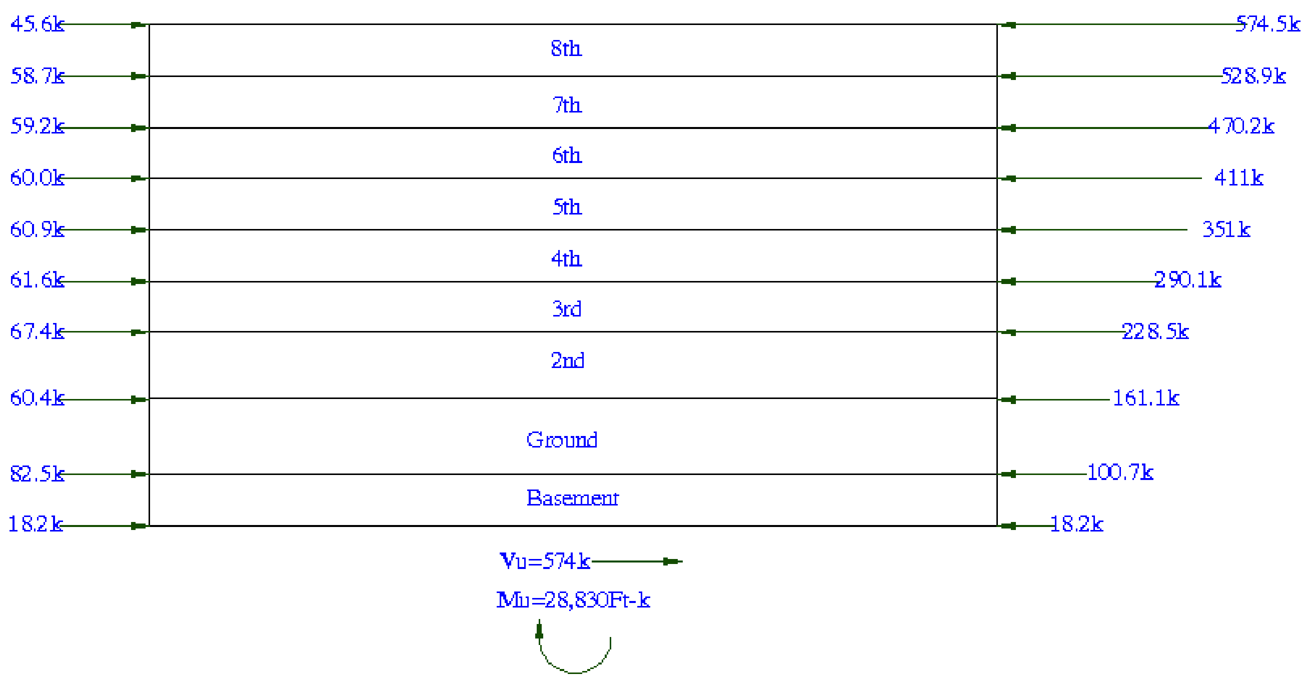


Figure 6: Seismic Loads on East-West face

The load combinations examined are from the ASCE 7-05 which resulted in controlling load combination of **1.2 D + 1.6 L + 0.8W**. This load combination also satisfies many of the assumptions made for analysis such as low seismic force due to the building being located in a seismic safe zone. Other factors such as rain and snow load were not taken into consideration.

Different wind load cases from ASCE 7-05 were also studied. The following figure shows how the different cases were applied to the building for analysis:

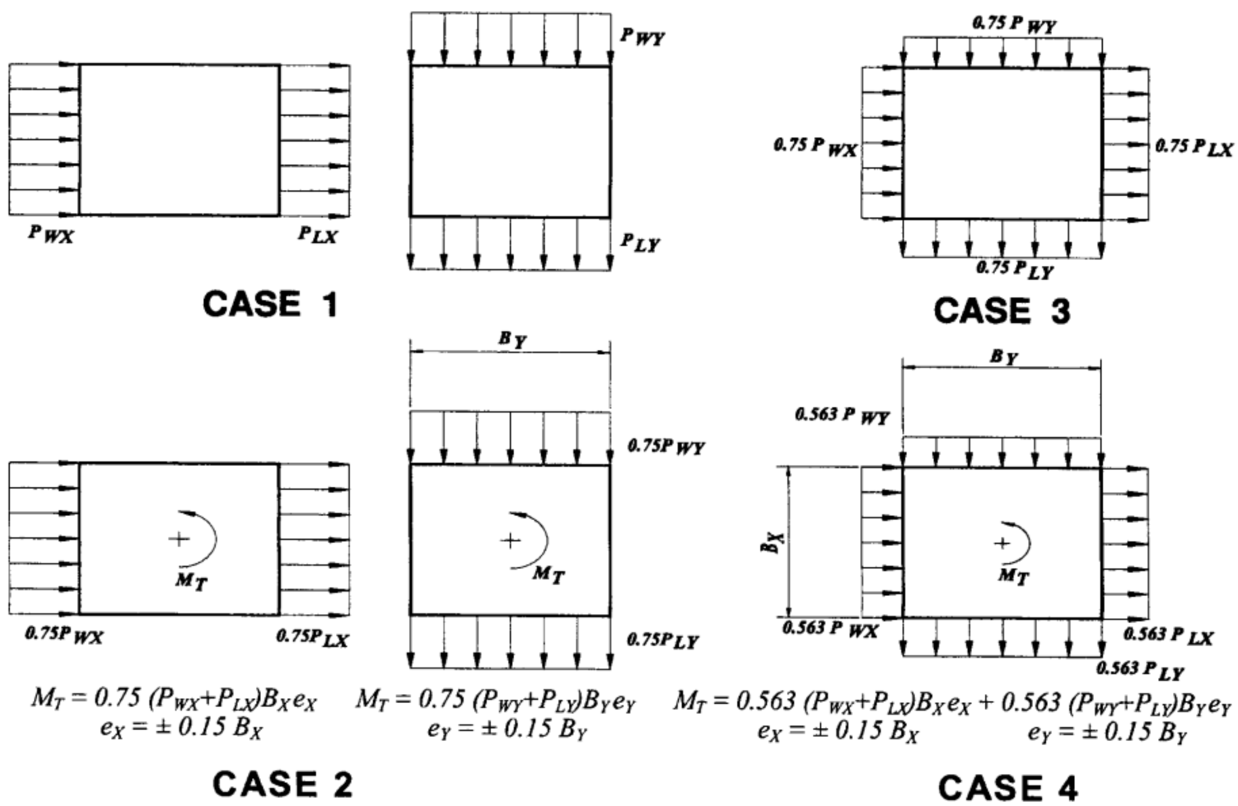


Figure 7: Wind Load Cases

Lateral System Analysis

As mentioned earlier in the report, the lateral force resisting systems consists of shear walls within the core of the building. Most of the shear walls are spread out about the East-West direction running along the North-South axis. The following plan highlights the location of the shear walls in the building.

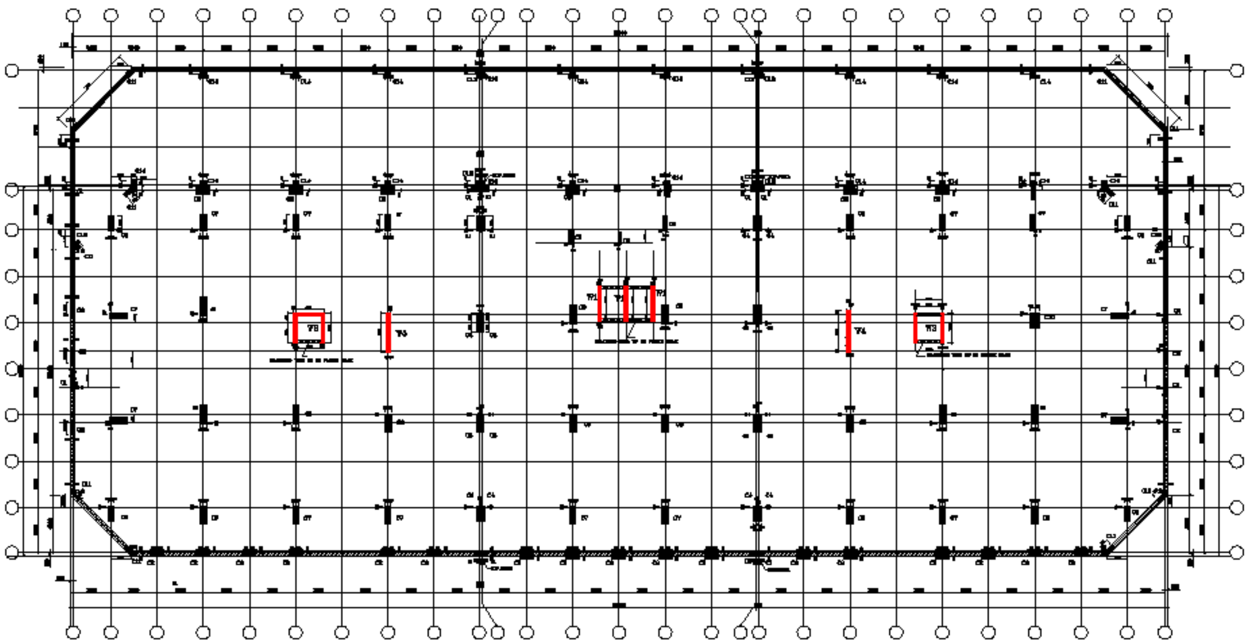


Figure 8: Location of shear walls in the building
(shear walls highlighted in red)

The loads resisted (wind/seismic) are transferred through the floors which act as diaphragms. Each shear wall receives a load based on its relative stiffness. The walls then behave like cantilevered beams and carry the loads down to the foundation.

A hand calculation was used to determine the relative stiffnesses of the shear walls. A load of 1000 kips was applied at the top of the wall and the following equation was used to calculate the deflection of the wall:

$$\Delta = \frac{Ph^3}{3EI} + \frac{2.78Ph}{AE} \qquad \text{Eqn. 1}$$

The stiffness of the walls was then calculated by taking the reciprocal of the deflection. Keep in mind that this hand calculated method is only an approximation of the real stiffness. In reality, the shear walls act differently since three of them are connected together. The calculated stiffness assumes that the walls function separately. The thickness of the wall was also assumed to be uniform throughout the entire height of the building for simplification. In addition, the calculated stiffness value was assumed to be the same for each floor. The difference in K values is small between floors and can be ignored. For the purpose of this report, the calculated values are close enough to reality for analysis.

The following figure shows the shear walls with their labels while the table provides the relative stiffness of each wall.

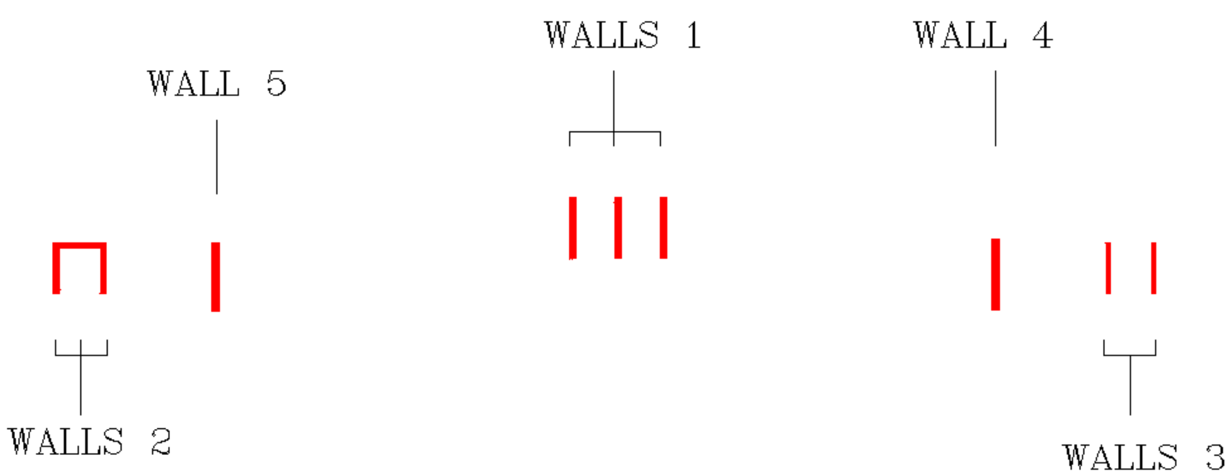


Figure 9: Labels of shear walls

WALL	FORCE	HEIGHT	WIDTH	THICNKESS	Δ FLEXURE	Δ SHEAR	Δ TOTAL	R_I	RELATIVE STIFFNESS
1-1	1000	1152	118	10	0.088	6.40E-04	0.0886	11.282	0.116
1-2	1000	1152	118	10	0.088	6.40E-04	0.0886	11.282	0.116
1-3	1000	1152	118	10	0.088	6.40E-04	0.0886	11.282	0.116
2-1	1000	1152	98	10	0.154	7.73E-04	0.1548	6.461	0.066
2-3	1000	1152	98	10	0.154	7.73E-04	0.1548	6.461	0.066
3-1	1000	1152	98	10	0.154	7.73E-04	0.1548	6.461	0.066
3-2	1000	1152	98	10	0.154	7.73E-04	0.1548	6.461	0.066
4	1000	1152	134	12	0.05	4.71E-04	0.0505	19.813	0.203
5	1000	1152	130	12	0.055	4.85E-04	0.0555	18.023	0.185
							Total	97.525	

Table 3: Stiffness Factors of Shear Walls

ETABS Model

The lateral system of the building was designed with the aid of ETABS, a three-dimensional structural building design and analysis software. In order to simplify the design and get a better understanding, the lateral system was designed independent of the remainder of the building. Only the shear walls and diaphragms were included in the model for analysis. Other minor assumptions include ignoring the angled corners and small openings in the diaphragm.

The ETABS model was used to calculate the center of rigidity, center of mass, deflections and story drifts. Hand calculations were used to check the deflections and story drifts while the center of mass and rigidity were assumed to be correct since the building has a simple rectangular shape. More details on the results from ETABS are found in the Appendix. The following figures show the model created for analysis:

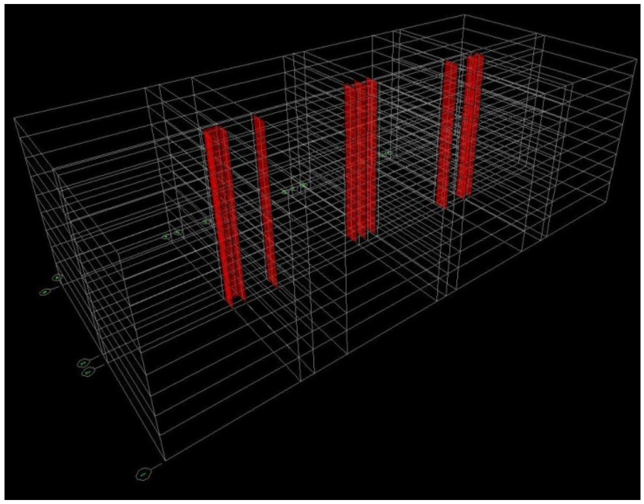


Figure 10: View of shear walls only

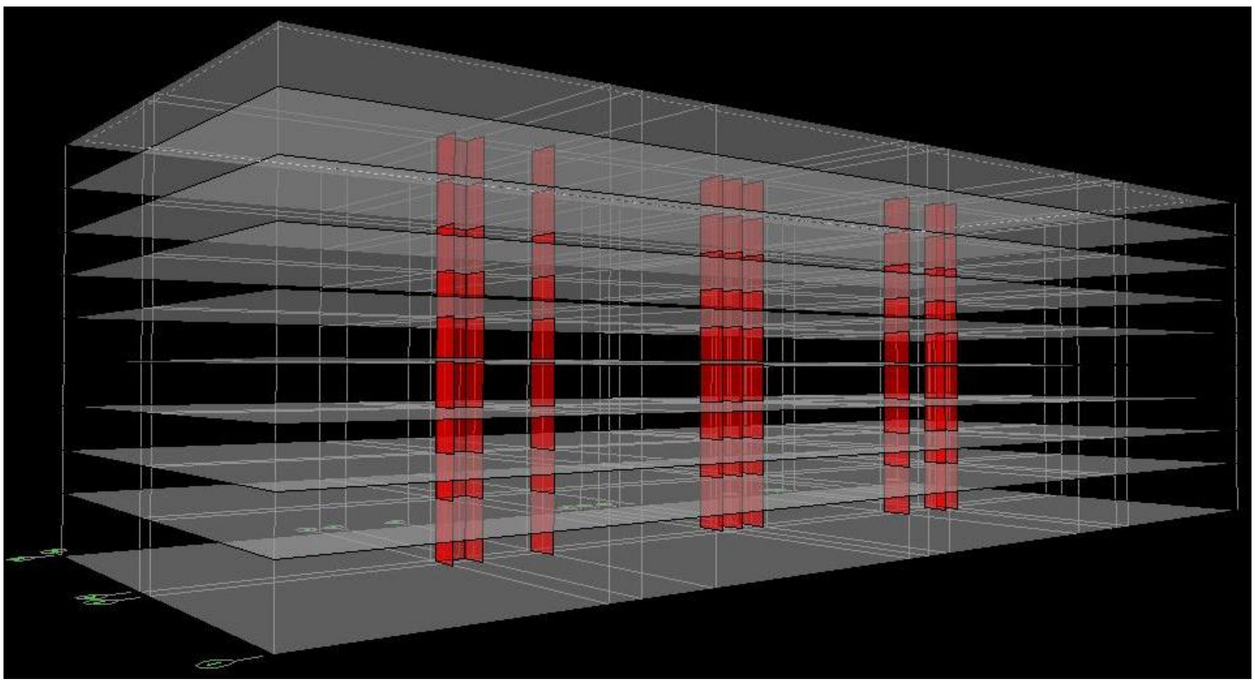


Figure 11: View of shear walls with diaphragms

Torsion

Using ETABS, it was revealed that the center of pressure does not act on the center of mass which results in a torsional moment that has to be taken into account in the design process. Hand calculations were used to determine the torsional moments caused by the wind loads on the buildings. Table 4 shows the location of the center of mass and center of rigidity calculated from ETABS while Table 5 summarizes the torsional moments on the building.

Story	Diaphragm	MassX	MassY	XCM	YCM	XCCM	YCCM	XCR	YCR
ROOF	D1	7.6	7.6	1758.6	784.9	1758.6	784.9	1665.7	808.8
8TH	D1	7.7	7.7	1757.5	784.8	1758.0	784.9	1667.4	809.1
7TH	D1	7.7	7.7	1757.7	784.8	1757.9	784.8	1669.5	809.7
6TH	D1	7.7	7.7	1757.7	784.8	1757.9	784.8	1672.3	810.5
5TH	D1	7.7	7.7	1757.8	784.9	1757.9	784.8	1676.0	811.6
4TH	D1	7.7	7.7	1757.1	784.8	1757.8	784.8	1680.7	813.3
3RD	D1	44.8	44.8	1759.5	785.0	1758.6	784.9	1685.2	818.0
2ND	D1	12.7	12.7	1758.5	784.8	1758.6	784.9	1690.2	825.5
GROUND	D1	10.3	10.3	1758.1	784.7	1758.5	784.9	1707.4	825.6
					Average	1758.20	784.87	1679.38	814.68

Table 4: Center of mass and rigidity from ETABS

The torsional moment in the N-S direction is greater than the E-W direction due to the larger difference between the center of mass and center of rigidity of the building. The shear walls are aligned closer together in the E-W direction while also being positioned closer to the center of mass. This formation would however result in a greater torsional shear which can be seen in table 6 and 7. The shear walls are concentrated in the core of the building causing a greater torsional shear in the E-W direction which must also be taken into account while design the walls.

	M_{tor} N-S	M_{tor} E-W
	134.0658	123.1178
	333.2412	227.9197
	303.2237	162.5544
	298.7816	128.0949
	325.709	124.6382
	347.5225	122.6689
	366.1412	121.7796
	382.5448	121.6931
	198.5227	60.87556
Total	2689.752	1193.342

Table 5: Calculated torsional moments

N-S Direction								
Wall	Stiffness	Shear	Loc. of Wall	Dist. From COR	COM to COR	Torsional	Direct	Total Shear
1-1	0.108	377.0	1700	20.6	78.8	0.131	40.900	41.031
1-2	0.108	377.0	1780	100.6	78.8	0.640	40.900	41.540
1-3	0.108	377.0	1870	190.6	78.8	1.213	40.900	42.113
2-1	0.062	377.0	720.5	958.9	78.8	3.494	23.423	26.917
2-2	E-W	E-W	763.8	915.6	78.8	E-W	E-W	E-W
2-3	0.062	377.0	807	872.4	78.8	3.179	23.423	26.602
3-1	0.062	377.0	2710	1030.6	78.8	3.756	23.423	27.178
3-2	0.062	377.0	2780	1100.6	78.8	4.011	23.423	27.433
4	0.191	377.0	2500	820.6	78.8	9.171	71.832	81.003
5	0.173	377.0	1020	659.4	78.8	6.703	65.340	72.044

Table 6: Torsional shear at 2nd story in N-W direction

E-W Direction								
Wall	Stiffness	Shear	Loc. of Wall	Dist. From COR	COM to COR	Torsional	Direct	Total Shear
1-1	N-S	N-S	748	66.7	29.8	N-S	N-S	N-S
1-2	N-S	N-S	748	66.7	29.8	N-S	N-S	N-S
1-3	N-S	N-S	748	66.7	29.8	N-S	N-S	N-S
2-1	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S
2-2	0.062	373.7	787.5	27.2	29.8	11.86	23.22	35.08
2-3	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S
3-1	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S
3-2	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S
4	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S
5	N-S	N-S	827	12.3	29.8	N-S	N-S	N-S

Table 7: Torsional shear at 2nd story in E-W direction

The following equation was used to determine the torsional shear:

$$Torsional\ shear = \frac{Story\ shear\ X\ COM\ to\ COR\ X\ Dist.to\ COR\ X\ Stiffness}{Torsional\ moment\ of\ inertia}$$

The total shear will be used to check if the members are adequate later on in the report.

Deflection and Story Drift

The deflections caused by the different wind loads studied were compared to the L/400 requirement. Both ETABS output results and hand calculations passed the check of L/400 which is 2.88”. The following tables summarize the deflections calculated.

Notice that the deflections in the E-W directions are great than the N-S direction. This can be explained by the one shear wall only resisting the loads in the E-W direction. A possible hypothesis for the shear wall design in the E-W direction is that the engineer might have used a different wind load distribution that resulted in less wind being exposed to the E-W face of the building. Surrounding buildings might also disturb the flow of the wind since the front and the back of the building are in the South and North face respectively.

		Deflection	Max Drift
Wind Case 1	N-S	1.728	0.002371
	E-W	2.101	0.003087
Wind Case 2	N-S	1.77	0.002996
	E-W	1.712	0.00248
Wind Case 3	N-S	1.263	0.001833
	E-W	1.464	0.002435
Wind Case 4	N-S	0.558	0.002025
	E-W	1.212	0.003047
Seismic	N-S	1.648	0.003286
	E-W	2.77	0.00568

Table 8: Deflections of different load cases from ETABS

The hand calculated deflections are lower than the ETABS value due to the simplification in the method used. Nonetheless, the approximated values from the hand calculation confirm that the ETABS are correct. Hence deflection check passes for the wind and seismic loads. An example of the hand calculation can be found in Appendix A.

Walls	Story								Total
	2	3	4	5	6	7	8	Roof	
1-1	0.01	0.02	0.03	0.06	0.10	0.16	0.24	0.17	0.79
1-2	0.01	0.02	0.03	0.06	0.10	0.16	0.24	0.17	0.79
1-3	0.01	0.02	0.03	0.06	0.10	0.16	0.24	0.17	0.79
2-1	0.00	0.01	0.02	0.04	0.07	0.11	0.17	0.12	0.55
2-3	0.00	0.01	0.02	0.04	0.07	0.11	0.17	0.12	0.55
3-1	0.00	0.01	0.02	0.04	0.07	0.11	0.17	0.12	0.55
3-2	0.00	0.01	0.02	0.04	0.07	0.11	0.17	0.12	0.55
4	0.01	0.03	0.05	0.10	0.16	0.25	0.37	0.26	1.22
5	0.01	0.02	0.05	0.09	0.15	0.24	0.35	0.24	1.15

Table 9: Deflections of Wind case 1 hand calculated

Although records from the engineer state that seismic loads should not control, story drifts for the lowest allowable seismic loads were still checked. The seismic load was applied at the center of mass of the model. The drifts are compared to the $0.02h_{sx}$ requirement from ASCE7-05. The following tables provide the story drifts from ETABS and hand calculations:

		Drift
Story	2	0.00100
	3	0.00141
	4	0.00174
	5	0.00197
	6	0.00213
	7	0.00223
	8	0.00229
	Roof	0.00230

Table 10: Seismic Wall 5 Story Drift from ETABS

At 8th floor: $0.02\ h_{sx} = 0.02(10'x12) = 2.4 > 0.00229$ √ Okay

At 2nd floor: $0.02\ h_{sx} = 0.02(14'x12) = 3.36 > 0.001$ √ Okay

		Drift
Story	2	0.00898
	3	0.02482
	4	0.03070
	5	0.05050
	6	0.07100
	7	0.09300
	8	0.12300
	Roof	0.03200

Table 10: Seismic Wall 5 Story Drift from Hand Calc.

At 8th floor: $0.02\ h_{sx} = 0.02(10'x12) = 2.4 > 0.00229$ √ Okay

At 2nd floor: $0.02\ h_{sx} = 0.02(14'x12) = 3.36 > 0.001$ √ Okay

Again the values from the hand calculations differ from the ETABS value but with a higher value this time around. This is also due to the simplification of the hand method and also ETABS calculates the weights of the masses more accurately. Drift does not control the design since the building is more likely to twist in torsion.

Shear Check and Overturning

A hand calculation spot check was carried to check the shear capacity and overturning moments of the members. The overturning moment calculation for wall 1 in the North-South direction resulted in a moment of 2756 k-ft caused by the wind loads. The resisting moment due to self weight was about 360 k-ft. This problem is solved by the large mat foundation used by the engineer to counter the overturning moment. Detailed calculations are provided in Appendix A.

The critical member checked for shear was also wall 1 in the North-South direction due to wind load. The point checked was on the 2nd story where the total shear, including torsional, on the wall is 41k. The (2) #5's @ 6" horizontally and (2) #6's @ 6" vertically were sufficient to carry the applied load. Refer to appendix A for the detailed calculation.

Conclusion

The ETABS software demonstrated a great ability in analyzing the structural system of a building. Results from the lateral analysis of the G.Muttrah Complex turned out to be close to the approximated hand calculation methods. A more vigorous analysis with a complete building model would be needed in order to get more accurate results. This is due to the fact that the different structural members of a building would change the stiffness of the building and hence affect the lateral system.

The controlling load combination after analysis proved to be **1.2 D + 1.6 L + 0.8W.** In addition, four wind load cases were examined which resulted in more deflection in the E-W direction than the N-S direction. This increase in deflection can be caused by different load combination used by the engineer, or other surrounding conditions that might affect the wind distribution onto the building.

The deflections calculated from ETABS and hand calculations both satisfied deflection requirements. Story drifts and shear check also passed their respective requirements while a tosional shear and moment resulted from the layout of shear walls in the building. The increase in torsional moment in the E-W axis was due to the fact that the shear walls were concentrated in the middle core leading to a greater shear force on the building. An overturning moment also resulted from the wind load which was counteracted by the large mat foundation that holds the shear walls in place.

The results from this technical report can be used to create a basic model for the thesis proposal. A more precise analysis can be executed with more details in order to create a model similar to the original design.

Appendix A: Calculations

Wind

Table A-1

Mean Velocity(mph)	75	Provided by engineer
Occupancy Category IBC	II	IBC
Exposure Category	B	
Directionality Factor K_d*	0.85	ASCE 7-05
Importance Factor. I	1	ASCE 7-05
Topographic Factor K_{zt}	1	ASCE 7-05
Velocity Factor $q_z=0.00256K_zk_{zt}k_dv^2I$	Table	
Velocity Coefficient K_z	Table	
α	7	
Z_g	1200	
ε	1/3.0	
ℓ	320	
c	0.3	
β	1 (Assumed)	
b	0.45	
Building Frequency η_1	0.980	Structure is flexible
Peak Factors g_q	3.4	
Peak Factors g_v	3.4	
Peak Factors g_R	4.18	
Turbulence Factor Z	57.6	$>z_{min}= 30'$
Intensity of Turbulence I_z	0.273	
Integral Length L_z	385	
Background Response Q	0.83	
Mean Wind Speed V	56.8	
Reduced Frequency N_1	6.64	
R_n	0.042	
R_h	0.123	for η=7.62
R_b	0.091	for η=10.5
R_L	0.12	for η=79.7
Resonant Response	0.0166	(N-S)
Resonant Response	0.0188	
Gust Effect Factor	0.83	(N-S)
Gust Effect Factor	0.82	

Building is Enclosed		
k_p	0.99	
q_p	12.12	
GC_{pn}	1.5 Windward	
GC_{pn}	(-1.0) Leeward	
P_p	18.18	windward
Pp	-12.12	Leeward
High-Rise building		
GC_{pi}	0.18 or -0.18	
External Pressure Coefficient		
Windward C_p	0.8	
Leeward (N-S) C_p	-0.5	L/B=.44
Leeward (E-W) C_p	-0.5	L/B=2.27
Sidewall C_p	-0.7	

North-South

Height= 96 ft
B= 300
L= 132

Table A-2

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	M_o (ft-kips)
Windward	0-15	0.7	8.568	7.848	35.317	529.757
	20	0.7	8.568	7.848	11.772	235.448
	25	0.7	8.568	7.848	11.772	294.309
	30	0.7	8.568	7.848	11.772	353.171
	40	0.76	9.302	8.336	25.008	1000.307
	50	0.81	9.914	8.742	26.227	1311.339
	60	0.85	10.404	9.067	27.202	1632.124
	70	0.89	10.894	9.392	28.177	1972.415
	80	0.93	11.383	9.718	29.153	2332.211
	90	0.96	11.750	9.961	29.884	2689.569
	96	0.98	11.995	10.124	18.223	1749.412
Leeward	ALL	0.98	11.995	-7.137	-21.400	-1027.0

East-West

Height= 96 ft
B= 132
L= 300

Table A-3

Location	Height (Ft)	K_z	q_z	P_z (psf)	P_z (Kips)	M_o (ft-kips)
Windward	0-15	0.7	8.568	7.780	15.404	231.0573
	20	0.7	8.568	7.780	5.135	102.6921
	25	0.7	8.568	7.780	5.135	128.3652
	30	0.7	8.568	7.780	5.135	154.0382
	40	0.76	9.302	8.261	10.905	436.2058
	50	0.81	9.914	8.663	11.435	571.7545
	60	0.85	10.404	8.984	11.859	711.5426
	70	0.89	10.894	9.305	12.283	859.8099
	80	0.93	11.383	9.626	12.707	1016.5562
	90	0.96	11.750	9.867	13.025	1172.2427
	96	0.98	11.995	10.028	7.942	762.4452
Leeward	ALL	0.98	11.995	-7.077	-37.200	-1785.6

Seismic

Table A-4

Concrete	150pcf
Floor superimposed	10psf
Roof superimposed	30psf
Façade	30psf

Design Category	A
Cs	0.01

Table A-5

Slab	Area(sqft)	Slab thickness(ft)	Weight
Ground	37500	0.67	3768750
2nd	37500	0.67	3768750
3rd	37500	0.67	3768750
4th	37500	0.5	2812500
5th	37500	0.5	2812500
6th	37500	0.5	2812500
7th	37500	0.5	2812500
8th	37500	0.5	2812500

Roof	37500	0.5	2812500
		Total=	28181250

Table A-6

Column	Weight
C1	10725
C2	107712
C3	94248
C4	120912
C5	408672
C6	176484
C7	100848
C8	30162
C9	154044
C10	20196
C11	80784
C12	165132
C13	252996
C14	525393
Total	2248308

Table A-7

Beam	Quantity	Span(ft)	Area(sq-ft)	Weight
B110	23	24	2.72	225216
B107	11	24	2.72	107712
B104	8	24	2.22	63936
B106	2	24	2.72	19584
B109	16	24	2.72	156672
B111	14	5	2.72	28560
B114	2	4	2.67	3204
B203	13	24	2.72	127296
B113	12	24	2.72	117504
B112	8	24	2.72	78336
B30	2	24	2.72	19584
B29	11	24	2.72	107712
B201	24	24	2.72	235008
B202	11	30	2.72	134640
B205	12	30	2.72	146880
B101	44	12	1.56	123552
B102	16	12	1.95	56160
			Total	1751556

Table A-8

Wall	Thickness	Area	Weight
B	0.67	8640	868320
1	0.67	12096	1397088
2	0.67	10368	1197504
3	0.67	8640	997920
4	0.67	8640	997920
5	0.67	8640	997920
6	0.67	8640	997920
7	0.67	8640	997920
8	0.67	8640	997920
		Total =	9450432

Table A-9

		Weight					
		Column	**Slab(w/ superimposed)**	**Beams**	**Walls&Façade**	**Total**	**Shear (K)**
Floor	**B**	949440	0	0	868320	1817760	18.1776
	1	1329216	3768750	1751556	1397088	8246610	82.4661
	2	1075230	3768750	0	1197504	6041484	60.41484
	3	1971255	3768750	0	997920	6737925	67.37925
	4	597909	2812500	1751556	997920	6159885	61.59885
	5	530934	2812500	1751556	997920	6092910	60.9291
	6	433899	2812500	1751556	997920	5995875	59.95875
	7	354654	2812500	1751556	997920	5916630	59.1663
	8	304959	2812500	1751556	997920	5866935	58.66935
	R	0	2812500	1751556	0	4564056	45.64056
	Total	7547496	28181250	12260892	9450432	5744007 0	574.4007

Lateral Calculations

Table A-10

Floor	Floor Height (ft)	Forces (k)		Story Shear (k)	
		N/S	E/W	N/S	E/W
1	14	31.469	31.2	408.462	404.882
2	12	58.442	57.942	376.993	373.682
3	10	49.597	49.172	318.551	315.74
4	10	46.541	46.134	268.954	266.568
5	10	47.734	47.316	222.413	220.434
6	10	48.709	48.279	174.679	173.118
7	10	49.685	49.242	125.97	124.839
8	10	50.638	50.181	76.285	75.597
Roof	0	25.647	25.416	25.647	25.416

ETABS and Hand Calculations

Table A-11

N-S Accidental Torsional Moment, M_{ta}				
Story	Structural Width (in)	5% width (ft)	Story Force (k)	Torsion (ft-k)
1	3600	15	31.469	472.035
2	3600	15	58.442	876.63
3	3600	15	49.597	743.955
4	3600	15	46.541	698.115
5	3600	15	47.734	716.01
6	3600	15	48.709	730.635
7	3600	15	49.685	745.275
8	3600	15	50.638	759.57
Roof	3600	15	25.647	384.705

Table A-12

E-W Accidental Torsional Moment, M_{ta}				
Story	Structural Width (in)	5% width (ft)	Story Force (k)	Torsion (ft-k)
1	132	0.55	31.2	17.16
2	132	0.55	57.942	31.8681
3	132	0.55	49.172	27.0446
4	97	0.404	46.134	18.638136
5	97	0.404	47.316	19.115664
6	97	0.404	48.279	19.504716
7	97	0.404	49.242	19.893768
8	97	0.404	50.181	20.273124
Roof	97	0.404	25.416	10.268064

Table A-13

N-S Inherent Torsional Moment, M_t					
Story	Center of Mass	Center of Rigidity	Difference (ft)	Story Force (k)	Torsion (ft-k)
1	1758.535	1707.412	4.26025	31.469	134.0658073
2	1758.579	1690.154	5.702083333	58.442	333.2411542
3	1758.59	1685.225	6.11375	49.597	303.2236588
4	1757.753	1680.716	6.41975	46.541	298.7815848
5	1757.876	1675.995	6.823416667	47.734	325.7089712
6	1757.894	1672.278	7.134666667	48.709	347.5224787
7	1757.944	1669.513	7.36925	49.685	366.1411863
8	1758.046	1667.392	7.5545	50.638	382.544771
Roof	1758.602	1665.715	7.740583333	25.647	198.5227408

Table A-14

E-W Inherent Torsional Moment, M_t					
Story	Center of Mass	Center of Rigidity	Difference (ft)	Story Force (k)	Torsion (ft-k)
1	784.867	825.62	3.396083333	31.2	105.9578
2	784.885	825.488	3.383583333	57.942	196.0515855
3	784.894	817.964	2.755833333	49.172	135.5098367
4	784.84	813.311	2.372583333	46.134	109.4567595
5	784.84	811.602	2.230166667	47.316	105.522566
6	784.83	810.472	2.136833333	48.279	103.1641765
7	784.848	809.677	2.069083333	49.242	101.8858015
8	784.885	809.138	2.021083333	50.181	101.4199828
Roof	784.941	808.835	1.991166667	25.416	50.607492

Table A-15

WALL	FORCE	HEIGHT	WIDTH	THICNKESS	Δ FLEXURE	Δ SHEAR	Δ TOTAL	RELATIVE STIFFNESS
1-1	5.49	1152	118	10	0.312	0.0000035	0.312	0.108
1-2	5.49	1152	118	10	0.312	0.0000035	0.312	0.108
1-3	5.49	1152	118	10	0.312	0.0000035	0.312	0.108
2-1	3.15	1152	98	10	0.215	0.0000024	0.215	0.062
2-3	3.15	1152	98	10	0.215	0.0000024	0.215	0.062
3-1	3.15	1152	98	10	0.215	0.0000024	0.215	0.062
3-2	3.15	1152	98	10	0.215	0.0000024	0.215	0.062
4	9.65	1152	134	12	0.000	0.0000045	0.000	0.191
5	8.78	1152	130	12	0.000	0.0000043	0.000	0.173

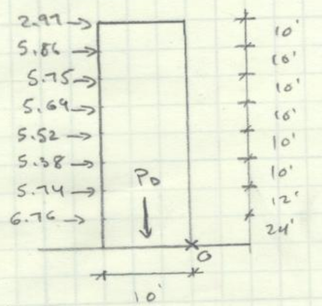
Note: Deflection on roof in N-S with story shear = 45.6

Overturning Moment Calculations

TECH III SAMIR AL-AZRI 2nd Dec '09

OVERTURNING MOMENT WALL 1 N-S

STORY	STIFFNESS FACTOR	STORY FORCE (k)	RESULTANT FORCE (k)
2	0.1157	58.44	6.76
3	0.1157	49.60	5.74
4	0.1157	46.54	5.38
5	0.1157	47.73	5.52
6	0.1157	48.71	5.64
7	0.1157	49.69	5.75
8	0.1157	50.64	5.86
Roof	0.1157	25.65	2.97



M_{over} = 6.76 (24) + 5.74 (36) + 5.38 (46) + 5.52 (56) + 5.64 (66) + 5.75 (76)

+ 5.86 (86) + 2.97 (96)

= 2756 k-FT

WALL SELF WEIGHT = (96)(10')(½')(150 PCF) = 72 K

M_{RESIST} = 72 (5) = 360 K-FT

M_{over} = 2756 FT-K > M_{RESIST} = 360 K-FT

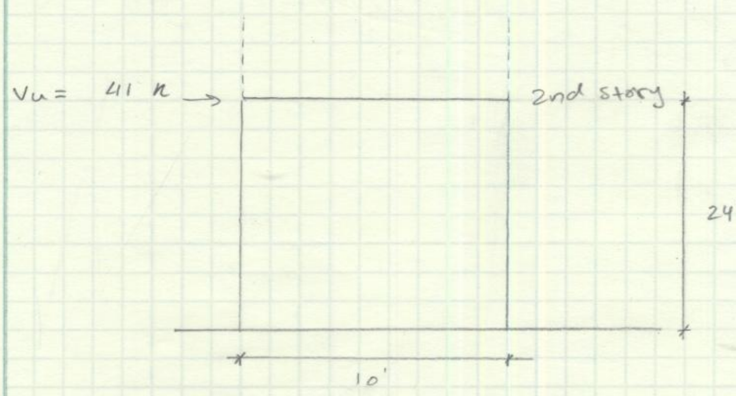
⇒ OVERTURNS

⇒ NEED CONSIDERATION IN FOUNDATION

Shear check

TECH III SAMIR AL-AZRI 2nd DEC '09

MEMBER CHECK WALL 1 N-S



$F_c' = 5{,}500$ psi

$F_y = 60{,}000$ psi

$h = 12$ in

HORZ REINF = 2 #5 @ 6"
VERT REINF = 2 #6 @ 6"
6 #6 @ EACH END

SHEAR

$V_u \le (\phi V_n)_{max} = \phi\, 10 \sqrt{f_c'}\, hd$

$d = 0.8 L_w = 0.8\,(10 \times 12) = 96"$

$(\phi V_n)_{max} = 0.75\,(10)\sqrt{5{,}500}\,(12)\dfrac{(96)}{1000} = 640\,k > 41\,k$ ✓ ok

$a \le \dfrac{L_w}{2} = \dfrac{10}{2} = 5'$ ⇒ governs.

$\le \dfrac{h_w}{2} = \dfrac{24}{2} = 12'$

$$V_c \le \left[0.6\sqrt{f_c'} + \frac{L_w\left(1.25\sqrt{f_c'} + 0.2\,\frac{N_u}{L_w h}\right)}{\frac{M_u}{V_u} - \frac{L_w}{2}}\right] hd$$

$M_u = V_u \times 19' = 228\, V_u$

$$V_c \le \left[0.6\sqrt{5{,}500} + \frac{120\left(1.25\sqrt{5{,}500} + 0.2\,\frac{N_u}{L_w h}\right)}{228 - 60}\right](12)\frac{(96)}{1000}$$

$N_u \approx 0$

$= 127.5\,k$

$\tfrac{1}{2}\phi V_c = 0.5(0.75)(127.5) = 47.8 > V_u = 41k$ ⇒ DESIGN AS PER CH 14 ACI 318-08

Vertical Reinf.

$\rho_{min} = 0.0015$

$\rho = \frac{48(0.44)}{114(12)} = .015$ ✓ ok

HORIZONTAL REINF

$\rho_{min} = 0.0025$

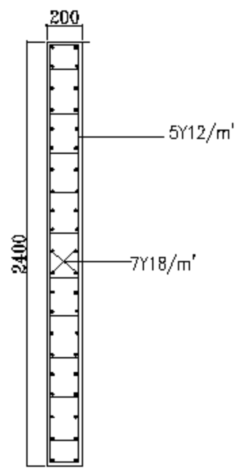
$\rho = \frac{48(0.44)}{288(12)} = 0.0061$ ✓ ok

Spacing $= 6'' < 3(h) = 3(12) = 36$

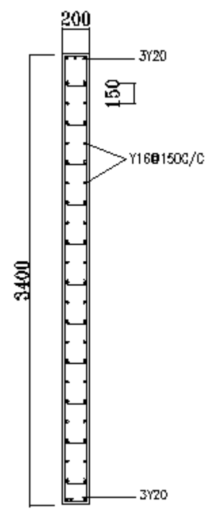
< 18 ✓ ok

Appendix B: Shear Wall Detail

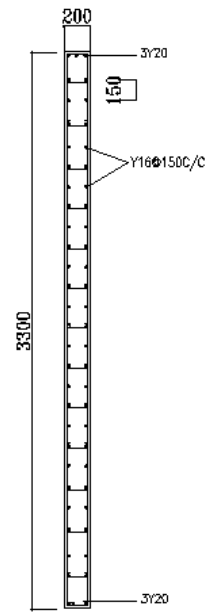
Plan B-1



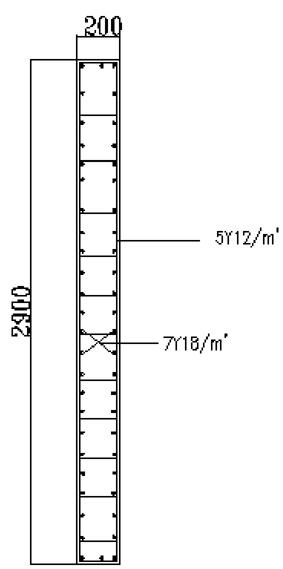
DET. OF W3



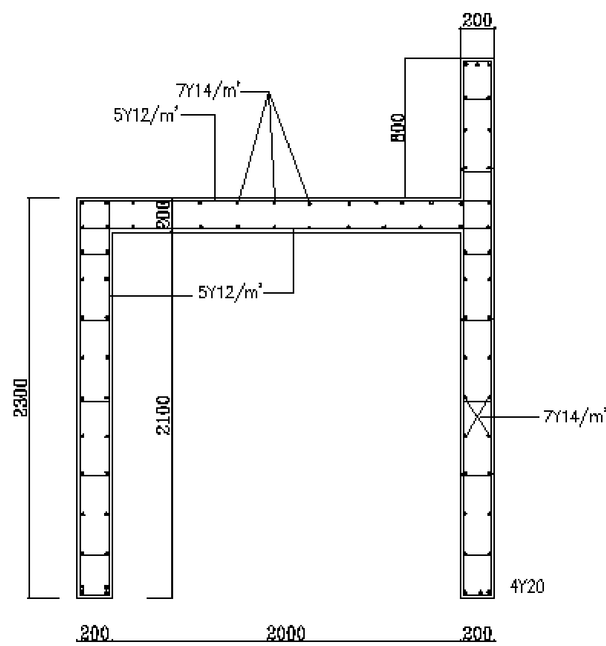
DET. OF W4



DET. OF W5



DET. OF W1



DET. OF W2