Lateral System (Depth Topic II)

As mentioned earlier in the report, the lateral force resisting system in the original design 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 20: Location of shear walls in the building
(shear walls highlighted in red)

Notice that the shear walls are located near the center of mass in the first two floors while being shifted away as you approach the residential floors. This relocation of center of rigidity causes a torsional moment on the building as discussed in previous technical reports. A new layout of shear design would have to reduce the distance between the center of rigidity and center of pressure from the loads.

As a result of relocating the building to Houston, Texas, the wind average speed increased from 75 mph to 120 mph. This change in wind speed doubled the story forces on the building. Refer to the loads section for a wind diagram and Appendix A for more calculations on the wind Loads.

The new system was designed using ETABS 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.

The new layout of shear walls shown below was designed to relocate the center of rigidity closer to the center of mass. All diaphragms where modeled as rigid members in order to make sure that all forces are transferred to the shear walls correctly while ensuring that only the shear walls are resisting the lateral loads.

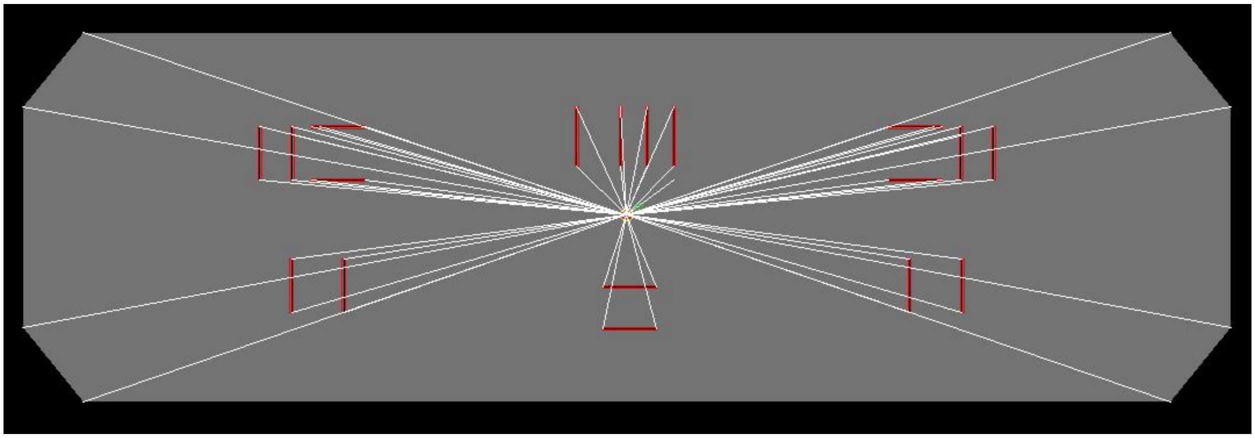

Figure 21: New shear wall layout

The new wall design includes (18) 140in shear walls with a thickness of 8in. Reinforcement consists of # 8's in the vertical direction and #4's in the horizontal direction. The walls checked out as adequate in both flexure strength and shear. Figure 21 shows the ETABS 3-D model with the new shear walls:

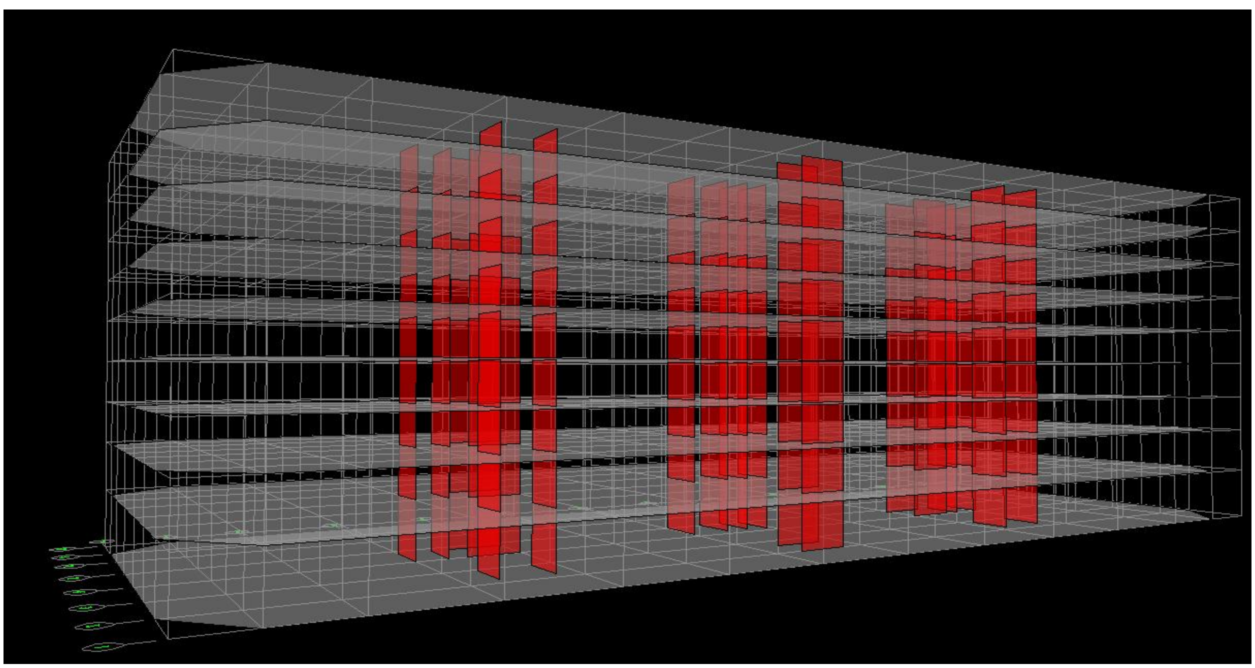

Figure 22: 3-D model from ETABS

The results obtained from ETABS were compared with hand calculations in order to check the designs capability. ETABS generated smaller rebar sizes for the vertical reinforcement hence the final design included the larger bar sizes from the hand calculations to ensure the design is conservative. Calculations are as follows:

Check Reinf.		
ρ_t	0.004167	>0.0025 o.k
Max Spacing	18	
ρ_l	0.004167	
h_w/l_w	8.22	> 3
Reinf. Ok		
Check Moment Strength		
$M_{(base)}$	4464	
$M_{u(base)}$	7142.4	
N_D	99.7	
N_u	89.73	
ω	0.05	
α	0.016023	
c	11.85032	
d	112	
ϕ	0.9	
A_{st}	4.666667	
T (kips)	256.2994	
M_n (Kip-ft)	1974.199	
ϕM_n (kip-ft)	1776.779	
No good		
Try # 8's for vertical Reinf.		
ρ_l	0.011	
A_{st}	18.43333	
T (kips)	1106	
M_n (Kip-ft)	7946.746	
ϕM_n (kip-ft)	7152.072	
Check Shear		
V_u	129.6	

l/2	5.8	
h/2	48	
Story height	14	
Critical Section	5.8	
$M_{u,critical\ section}$ (Kip-ft)	6390.72	
M_u/V_u	49.31111	
V_c	292.8787	
ϕVc	219.659	

Hence Shear Wall adequate in Flexure and shear

Check if Boundary Elements needed		
P_u (k)	100	
M_u	4464	
A_g	7.733333	
I_g	49152	
f_c (k/in^2)	0.093457	<0.2$f_{c'}$

Hence no boundary element needed

Table 6: Shear wall hand calculations

A hand calculation was used to determine the relative stiffness's 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. The thickness of the wall was 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 table summarizes the calculated stiffness factors in the N-S direction:

Table 7: wall stiffness calculations

WALL	FORCE	HEIGHT	WIDTH	THICNKESS	Δ FLEXURE	Δ SHEAR	Δ TOTAL	R_I	RELATIVE STIFFNESS
1	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
2	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
3	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
4	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
5	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
6	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
7	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
8	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
9	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
10	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
11	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083
12	1000	1152	140	8	0.069	4.9261E-06	0.069	14.47	0.083

Torsion, Deflection and Story Drifts

Due to the balanced layout of the shear walls, the torsional moment on the building decreased significantly compared to the original design. The center of mass and rigidity of the building were calculated using ETABS and the torsional moments were calculated manually. A maximum torsional moment was induced in the first floor since the floor plan is greater than the residential floors. The moment then drops in the residential floors since the new shear walls are designed around its core. A study of the effects of the walls on the architecture will be covered later in the report in a breadth topic analysis. For further details on the torsional moments see Appendix A.

The deflections caused by the different wind loads studied were compared to the L/400 requirement. At the roof level, the maximum wall deflection was 1.178in which passed the L/400 limit which is 2.88in. Story drifts caused by the wind loads were also compared to L/400 which is limited at 0.3in. The table on the right summarizes the story drifts due to wind.

	Disp-x	Drift-x
Roof	1.18	0.159
8	1.02	0.158
7	0.86	0.156
6	0.70	0.152
5	0.55	0.144
4	0.41	0.131
3	0.28	0.114
2	0.16	0.104
G	0.06	0.060

Table 8: Story Drifts caused by Wind Loads

Deflections resulting from seismic loads were compared to the allowable drift of 0.025h.

At 8th floor: $0.02\ h_{sx} = 0.02(10'\text{x}12) = 2.4 > 0.26$ √ Okay

At 2nd floor: $0.02\ h_{sx} = 0.02(14'\text{x}12) = 3.36 > 0.16$ √ Okay

	Disp-x	Drift-x
Roof	1.87	0.260
8	1.61	0.260
7	1.35	0.260
6	1.09	0.250
5	0.84	0.220
4	0.62	0.210
3	0.41	0.170
2	0.24	0.160
G	0.08	0.080

Table 9: Story Drifts caused by Seismic Loads

Impact on Foundation

A soil report from an arbitrary site in Houston was obtained in order to examine if a new foundation design would be needed to withstand the loads from the new structural system. The recommended bearing capacity of a spread/pad footing in the site in Houston is around 5000 psi. However, the allowable bearing capacity for the same footing in the site in Muscat is 5221psi.

Since the weight of the building was significantly reduced by removing the beams and using fewer columns, it is safe to assume that existing foundation would withstand the loads from the new system. A more efficient foundation system should not be considered since there is a significant amount of overturning moment from the relatively slender shear walls that would require a mat foundation to resist the moments.

Depth Summary

The post-tensioning slab design did not reduce the thickness of the building, but greater spans were achieved while eliminating the beams. A finished ceiling is also an advantage since it would create a better space aesthetically for the residential floors. Therefore a two-way post-tensioned slab design would be recommended as an alternative flooring system to the G.Muttrah Commercial & Residential Complex.

A new column layout was proposed to complement the new post-tensioned system. Fewer columns were used while also using smaller size since the weight of the building decreased. This new layout would be greatly appreciated in both retail and office spaces.

In order to rebuild the G.Muttrah complex in Houston, Texas, 8 more shear walls would be needed in an arrangement that balances the center of rigidity of the building. The increased wind speed in a hurricane prone area would require these 8 additional shear walls to provide adequate strength and resistance.