Temecula Medical Center
Temecula, CA

Technical Assignment #3

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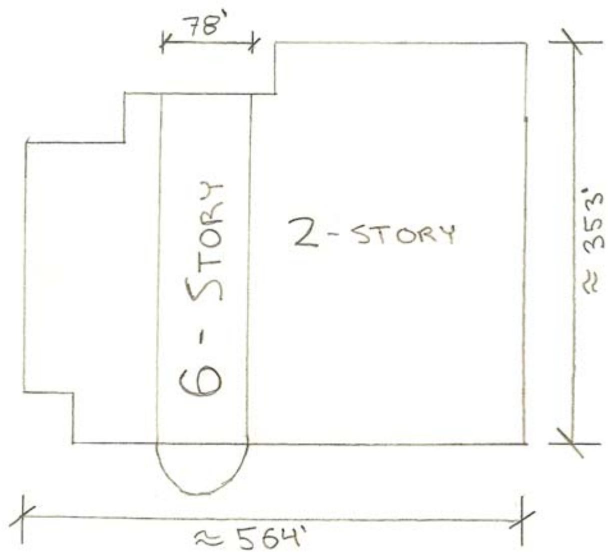
Executive Summary

The purpose of this report is to perform a detailed analysis on the lateral force resisting system in the Temecula Medical Center. The Temecula Medical Center is a 6-story hospital which features a 2-story Drug and Therapy center (D&T) as well as a 6-story bed tower. The engineers decided to resist the heavy West Coast lateral forces with concrete shear walls placed systematically throughout the plan. By using this approach, along with a concrete floor system, money was saved while still providing more than adequate lateral force resisting systems. Hospital designs require additional safety factors which had to be taken into consideration throughout the design of the structural system. The plan view of the medical center is shown.



Only strategically placed concrete shear walls are used in the building to resist the lateral loads. Since the 2-story D&T resists very little of the lateral load compared to the 6-story bed tower, only the tower will be analyzed in this report. The layout of each shear wall can be seen in the structural system overview.

In order to analyze the shear wall system, many methods and criteria has to be taken into consideration. A model was built in ETABS in order to get many of the critical drift values. From the model, it was determined that the governing load combination is $1.2D + 1.0E + L + 0.2S$. This did not come as a surprise primarily because of the heavy seismic loads presented in the Southwest region of the United States.

The lateral forces inserted into the ETABS model were determined from ASCE 7-05. Most of the forces used were copied from Technical Assignment #1 with minor adjustments. The wind forces were calculated from the Analytical Procedure in Section 6.5 and the seismic forces were determined from the Equivalent Lateral Force Procedure, per Section 12.8. The main discrepancy between the as-designed values and calculated was the R-value for seismic forces. While the calculations came out with a conservative R-value of 4, the building was designed using a value of 5.5 which is more aggressive and could have been assigned by the 2001 California Building Code.

The lateral force distribution was obtained through ETABS with the analysis consisting of each floor given a force applied at the center of mass. From this, deflection values were determined from the output tables. The largest displacement value was 4.51” which was higher than the max allowed of H/400. This result can be somewhat overlooked since columns were not included in the ETABS model and would provide, although minimal, additional stiffness.

Effects of inherent Torsion caused by eccentric loads as well as accidental torsion were analyzed using Chapter 12 of ASCE 7-05. Torsion had a substantial impact in the East-West direction due to the large tributary area.

Overturning moments were calculated using both wind and seismic loads which were calculated in Technical Report #1. Even though seismic controls, wind was considered primarily because wind pressures increase with the building height while seismic shears directly correlate to each floor weight. Nonetheless, calculations proved that seismic overturning controlled.

Lastly, a spot check was performed on a centrally located shear wall. The shear wall was loaded with an equivalent load and reinforcement was designed. The results were very similar to those designed and can be found in the Appendix.

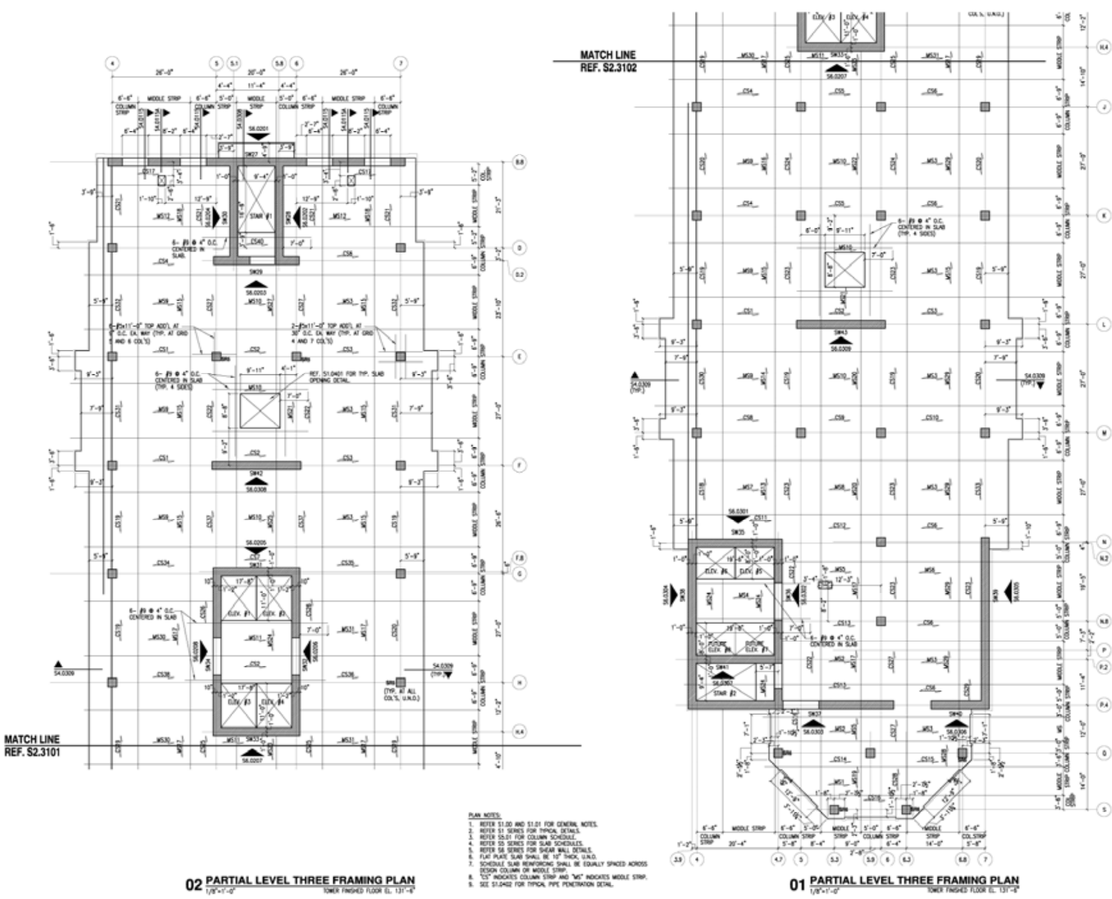
Structural System Overview

Lateral System

The lateral forces are resisted predominantly by concrete shear walls placed throughout the plan. The elevator shafts serve as the main component of the lateral resistance system. Shear walls are typically 27′-9″ long, and 2′ thick with varying reinforcement sizing and spacing. Each wall is built with a minimum 28-day compressive strength of 7000 psi. Specifically labeled walls have a compressive strength of 9000 psi. The shear walls are anchored to the supporting soil by footings, typically 6′ deep and reinforced with #9 at 9″ o.c. See Chart and Figure below for additional details on the existing system. The bold shapes represent the shear walls placed throughout the floor plan.

Concrete Strengths

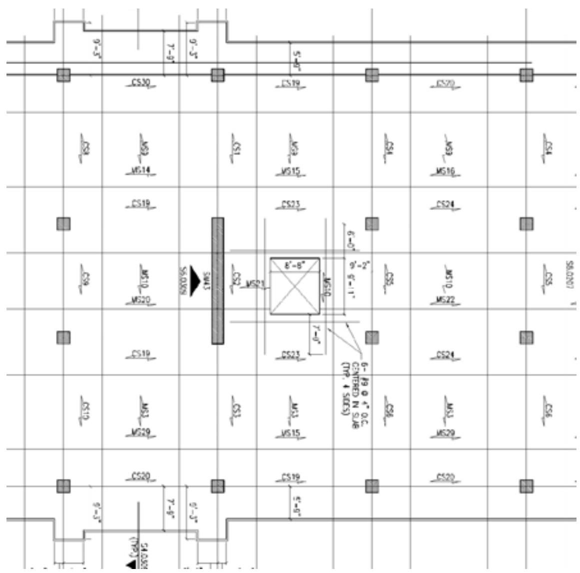
	F'c (psi)	Conc. Type	Max. Agg. Size
Typical Shear Walls	7000 (56-day)	N.W.C .	3/8"
Shear Walls (where noted)	9000 (90-day)	N.W.C.	3/8"



Floor System

The floor system of the first floor consists of a 5" slab-on-grade while the remaining floors of the Drug and Therapy Center (D&T) are supported by various sized precast, prestressed double-tees. The 6-story bed tower consists of two-way, 10" reinforced concrete flat slabs. Slab reinforcement ranges from #4 bars to #6 bars, spaced from 6" to 9" on center.

Topping slabs of the double tees in the D&T consists of 6" normal weight concrete, typically reinforced with #4 at 9" o.c. Typical spans between tee's is 6'-0 but vary on location. Two-way flat slab reinforcement sizes for the 6-story bed tower vary but are placed equally across designed column and middle strips. A typical floor layout is shown below.



Roof System

The lower roof over the 6-story bed tower is composite slab with 4 ½" normal weight concrete over 2", 16 gage composite metal deck (galvanized), reinforced with #3 at 9" o.c. each way. Supporting the 1 ½", 20 gage metal deck on the high roof are rolled steel W-shapes, typically W10x17, 33, or 45. The roof system over the 2-story D&T is very similar and consists of a 1 ½", 20 gage metal deck held up by rolled steel W-shapes, varying in size from W8 to W18.

Foundation

The foundation is a combination of spread footings and drilled piers with concrete pier caps. The spread footings vary in size from 5'x5' to 18'x18', depending on location, and are labeled F5-F18 accordingly. The reinforcement for these footings goes from 16 #5 each way in the F5 to 18 #9 each way in the F18.

Foundations for the shear walls feature footings anchored to the supporting soil by drilled piers, typically being 42″ in diameter. Each pier is spirally reinforced, varying in size while the pier caps are typically reinforced with #9 - #11 at 9″ o.c.

Columns

Vertical supports for the first level consist of 26″ x 26″ cast-in-place columns as well as 20″ x 20″ precast columns, however the upper floors (2-6) have only the 26″ x 26″ cast-in-place columns. A typical bay size is 54′ x 27′, although they vary depending on location and demand.

The cast-in-place columns typically run from spread footing through each floor while being reinforced with 12 #9's vertically and #4 at 6″ o.c. horizontally. Pre-cast columns are reinforced with 4 #9's vertically and #4 at 5″ o.c. horizontally. The compressive strength for the C.I.P. columns is 5000 psi and the strength of the PL columns is 6000 psi.

Gravity Loads

Gravity Loads

Live loads were found in ASCE 7-05 in table 4-1 under the Hospital category. The design loads are those used in the original design.

Live Loads		
Occupancy	ASCE 7-05 Load	Design Loads
Patient Rooms	40 psf	40 psf
Corridors	80 psf	100 psf
Light Storage Areas	125 psf	125 psf
Kitchens	150 psf	150 psf
Roof	20 psf	20 psf

Dead Loads		
Material/Occupancy	Load	Reference
Normal Weight Concrete	150 pcf	ACI 318
Steel	Per Shape	AISC 13th Ed.
Steel Deck	2 psf	USD
Plaster on Concrete	5 psf	ASCE 7
Miscellaneous	10 psf	
Exterior Wall	45 psf	ASCE 7

[1]United Steel Deck

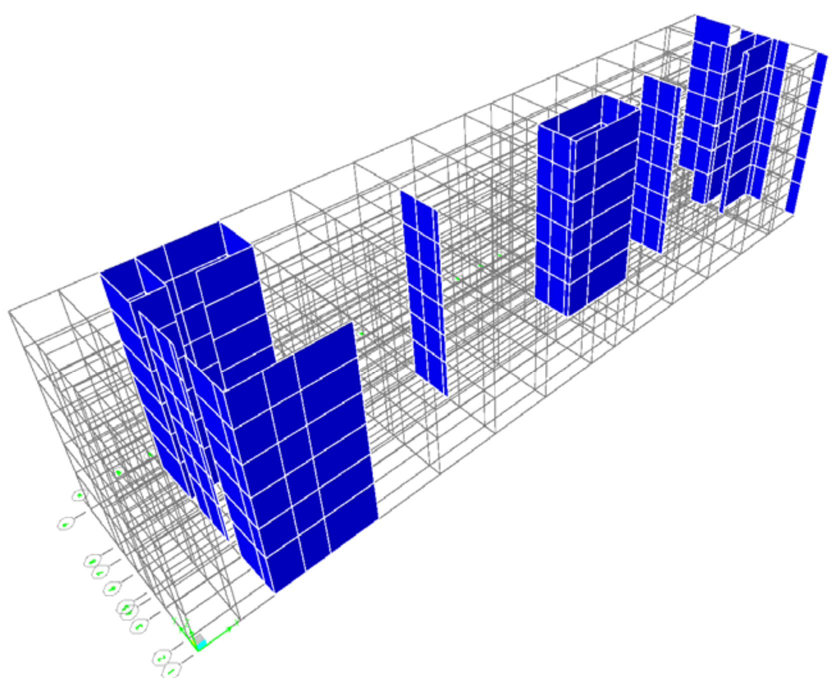
[2]Includes building components such as duct work, lighting, telecommunications, etc.

ETABS Model

For the simplicity reasons, only the lateral frame elements were modeled using ETABS. All shear wall thicknesses and sizes were modeled to match the actual design of the building. The floor systems as well as column sizes were also modeled as they were designed but were given zero stiffness.

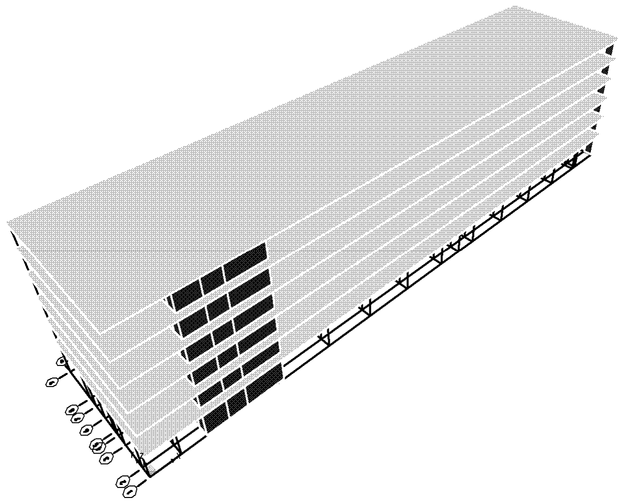
Since the floor and column system provided insignificant lateral force resistance, only the shear walls are modeled to handle the lateral loads. The concrete shear walls were also modeled such that they primarily resist in-plane shear. Material properties that contribute to out-of-plane bending resistance were manually reduced.

Uniform loading (dead and live) was assumed for the gravity loads and all lateral story shears were applied at the centroid of floor diaphragms. The figure below shows the ETABS model of the Temecula Medical Center.

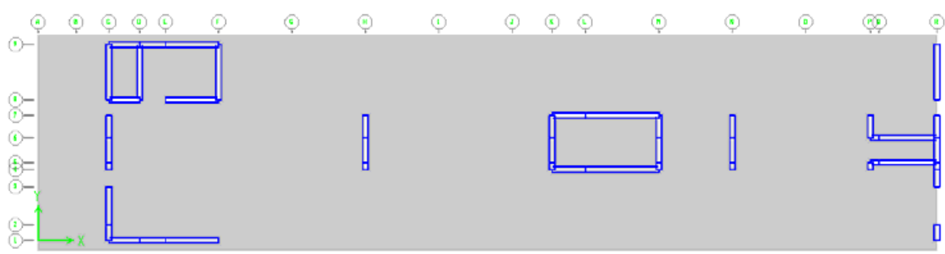


ETABS Model

In order to provide rigid support, a layer was added for each floor. While this played no part in lateral resistance, it gave a good point to apply seismic loads at each level. Shown below is how each layer was modeled.



While the exact form of the floor level was not followed, it was assumed that only the shear walls would resist lateral load and therefore a rectangle floor layout would give the same results. Shown below is the floor layout for each level. The objects in blue are shear walls and the grey area is the applied floor system.



Load Considerations

Load Path

Lateral forces in each direction (North-South and East-West) are resisted by the multiple shear walls placed throughout the structure. The forces are transferred from the shear walls to the rigid floor diaphragms and then brought down through the columns to the foundation. Once in the foundation, the forces are dissipated by the soil.

Load Combinations

The list below shows the seven load combinations that were considered in the design; each was found in ASCE 7-05 Section 2.3.2:

1. 1.4(D + F)
2. 1.2(D + F + T) + 1.6(L + H) + 0.5(L_r or S or R)
3. 1.2D + 1.6(L_r or S or R) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(L_r or S or R)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W + 1.6H
7. 0.9D + 1.0E + 1.6H

Each combination was used in the analysis of the Temecula Medical Center's lateral force-resistance system by means of ETABS. The controlling load combination is 1.2D + 1.0E + L + 0.2S. This is not surprising given that the building is located in California and therefore has very high shear force values. Of the two load combinations that include seismic, combination 5 provides a higher shear force value. Also, since the medical center is entirely above grade, combination 7 applies fewer loads to the structure since the "H" term is eliminated.

Lateral Force Analysis

Wind Analysis

A wind analysis of done using the Analytical Procedure as described in ASCE 7-05, Section 6.5. The East-West direction presents the most wind loads due to the large building face and therefore needs the most attention when it comes to lateral force resistance. Wind pressures were assumed to be evenly distributed across each story to estimate the triangular pressure distribution. When floor heights fell between two values, a conservative number for K_Z was used. The pressures were multiplied by the corresponding façade area to find the story shears. Wind did not control in any of the cases which is not unusual for a building located in the Southwestern region. This result, however, does agree with the calculations performed by the design engineer.

Seismic Analysis

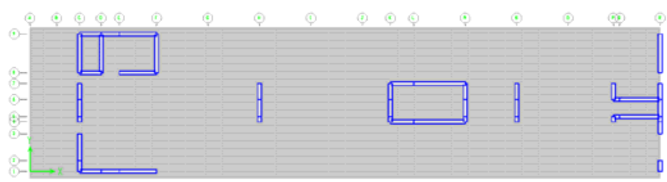
Seismic forces were calculated by using the Equivalent Lateral Force Procedure as outlined in ASCE 7-05, Section 12.8. The variables used in the modeling were those specified in the drawings although the calculated values were very similar and are included in the Appendix. The United States Geological Survey's software (which operates under NEHRP design provisions) was used to compute exact values for S_S and S_1. While the calculations proved a conservative R value of 4, the design engineer chose to use a more aggressive value of 5.5.

Once the base shear was found, Eq. 12.14-12 was used to determine the seismic shear contribution of each floor. Due to the region, importance factor, and R-value the seismic loads controlled in all cases.

Lateral Force Distribution

As mentioned previously in the report, all lateral forces were assumed to be resisted by the multiple shear walls. While the column and floor system provide minimal resistance, they were ignored to simplify the analysis. The lateral loads are resisted based on arrangement of each 24" thick concrete shear walls. Approximations made in Technical Assignment #1 for the lateral resistance system were used in the analysis.

While there is no methodological set up of the shear walls, they are placed spanning North-South and East-West to handle lateral loads from the respectable directions. It can be seen in the shear wall layout that more emphasis is put on the East-West direction. This is due to the larger tributary area along those sides. The East-West direction has nearly 4 times the area of the North-South direction which will force more support needed. If the unity method was taken into affect, each shear wall would provide the same lateral stiffness simply because each contains the same reinforcement (as mentioned in the structural overview) as well as the same thickness, 24".



By isolating each floor and applying a unit load to the centroid of each level, the deflections can be computed. The inverse of the deflection results is the stiffness for the particular floor. Relative stiffness was taken into affect in the ETABS model and a spot check is performed in the Appendix for one of the shear walls.

Drift

Drift is a serviceability consideration and should be limited as much as possible. The drift of a building is reversely proportionate to the total stiffness of the lateral force resistant structure. Unless the building requires a special deflection consideration, the maximum building deflection is often limited to 1/400th of the total building height. In the case of the Temecula Medical Center, the maximum allowable deflection would be:

$$\Delta_{MAX} = (107' \times 12''/')/400 = 3.21''$$

The deflection values were taken from ETABS at the center of mass of each floor, per Section 12.8.6 of ASCE 7-05. A summary of the results can be seen in the table below. The displacements are due to seismic loads since they controlled over the wind forces. Each level displacement can be compared to the Δ_{MAX} which is significantly lower than the displacement from floor 6. While there is a difference of 1.3”, it is most likely due to the large displacement on the first floor. Due to the fact that columns were assumed to resist no lateral load, the displacement on the first level is significantly higher than it would be if columns provided stiffness.

The displacement is larger in the East-West direction than the North-South direction which was expected since the building orientation provides more lateral support in the N-S. After examining the tables, it is clear that a fairly steady increase of deflection exists from level 1 to level 6.

Maximum Story Displacements		
Floor	East-West	North-South
	(in)	(in)
1	2.36	0.006
2	3.14	0.008
3	3.72	0.009
4	4.19	0.010
5	4.48	0.110
6	4.51	0.120
	Total = 4.51"	

Torsion

Along with the story shears, torsion is an important factor that should be analyzed while considering the lateral forces. A torsional analysis was performed on the Temecula Medical Center for lateral forces acting on the two principle axes of the building. This torsion was found from Section 12.8.4.1 of ASCE 7-05, which states:

"For diaphragms that are not flexible, the distribution of lateral forces at each level shall consider the effect of the inherent torsional moment, M_t , resulting from eccentricity between the locations of the center of mass and the center of rigidity."

The inherent torsional moment, M_t, is the story shear multiplied by the distance between the center of rigidity and the center of mass. Also included in the same section is the addition of M_{ta} and M_t. The accidental torsional moment, M_{ta}, is found my multiplying the shear by 5% of the building width (in each direction) at that level.

Overall Building Torsion						
	North-South Torsion			East-West Torsion		
Story	M_t	M_{ta}	M_{total}	M_t	M_{ta}	M_{total}
	(ft-kips)	(ft-kips)	(ft-kips)	(ft-kips)	(ft-kips)	(ft-kips)
1	0	0	0	0	0	0
2	351	547	898	935	2454	3389
3	531	828	1359	1416	3717	5133
4	924	1441	2365	2464	6468	8932
5	1370	2137	3507	3653	9588	13241
6	1869	2916	4785	4985	13085	18070
Roof L	5098	7952	13050	13593	35683	49276
Roof H	446	696	1142	1189	3122	4311
		Total	27106		Total	102352

Charts containing the data used to calculate M_t and M_{ta} are included in the Appendix.

Overturning

The overturning moment is being considered in this report because of its effect on various features of the structure. Most importantly is the effect on the foundation, which is responsible for carrying all of the loads to the soils surrounding the building. In addition to the foundation, the overturning moment is converted into axial loads by the lateral force resisting components.

The overturning moments caused by wind were calculated by multiplying the story forces by the mid-height of each story. The wind force resultants are assumed to act at mid-height of each level. While wind forces played a role in lateral resistance requirements, they are not considered in this report due to the fact that seismic controlled.

Overturning moments due to shear were also analyzed and calculated by multiplying the story shears by the story height in question. Each story force is assumed to act at the center of mass of each level.

Much like all other lateral computations, seismic controlled in the overturning analysis. The East-West wind overturning moment is significantly larger than the North-South direction because the tributary area is more than four times larger. The seismic overturning moment result is very large but expected for the region in which the Temecula Medical Center was built.

N-S Wind Forces					
Floor	Height	Tributary Height	Story Force	Story Shear	Overturning Moment
	(ft)	(ft)	(kips)	(kips)	(ft-kips)
1	0.0	0.0	0.0	504.3	0.0
2	18.0	18.0	279.0	504.3	2511.3
3	31.5	13.5	30.6	225.3	757.4
4	45.0	13.5	32.3	194.7	1235.5
5	58.5	13.5	33.6	162.4	1738.8
6	72.0	13.5	34.5	128.7	2251.1
roof	87.3	15.3	40.8	94.3	3249.7
ridge	107.0	19.7	53.5	53.5	5197.5
		Total	504.3		16941.3

Overturning

E-W Wind Forces					
Floor	Height	Tributary Height	Story Force	Story Shear	Overturning Moment
	(ft)	(ft)	(kips)	(kips)	(ft-kips)
1	0.0	0.0	0.0	1213.5	0.0
2	18.0	18.0	177.1	1213.5	1593.9
3	31.5	13.5	140.3	1036.4	3472.4
4	45.0	13.5	150.7	896.1	5764.3
5	58.5	13.5	154.5	745.4	7995.4
6	72.0	13.5	158.2	591.0	10322.6
roof	87.3	15.3	187.1	432.7	14902.5
ridge	107.0	19.7	245.6	245.6	23860.0
		Total	1213.5		67911.1

Seismic Forces					
Floor	Overall Height	Tributary Height	Cvx	Fx	Overturning Moment
	(ft)	(ft)		(kips)	(ft-kips)
1	0.0	0.0	0	0	0
2	18.0	18.0	0.033	140.18	1262
3	31.5	13.5	0.05	212.40	3133
4	45.0	13.5	0.087	369.58	14137
5	58.5	13.5	0.129	547.99	28354
6	72.0	13.5	0.176	747.65	48784
Roof Low	87.3	15.3	0.48	2039.04	162406
Roof High	107.0	19.7	0.042	178.42	17332
			Total	4235.26	275407

Conclusions

The following conclusions can be made after the analysis, included in this report, of the lateral force resisting structure of the Temecula Medical Center:

- The controlling load combination is combination 5 from ASCE 7-05 Section 2.3.2: $1.2D+1.0E+L+0.2S$
- Very small differences exist in the calculated seismic values due to the differences in codes used. The building was designed using the 2001 California Building Code while this report analyzed the building using ASCE 7-05.
- Although seismic controls in all directions, winds need to be considered for overturning moment. Base shear is not a good indicator of the overall overturning moment due to the distribution of story shears. Seismic base shear was 75% larger than the wind overturning moment in the North-South direction.
- The Inherent Torsion in the East-West direction had a large impact on the lateral structure of the Temecula Medical Center due to the large tributary area. Accidental Torsion was also significant in both directions, partially due to the large eccentricities.
- The shear wall spot check yielded aggressive but expected results. The design drawings show #6 @ 8″ horizontally (typical) and #6 @ 6″ vertically (typical). Calculations included in the Appendix yielded 2 #5 @ 9″ horizontally and #5 @ 9″ vertically. The horizontal results are a bit conservative although the vertical reinforcement is aggressive and could be due to the adjusted stiffness factor.
- Being a hospital in the Southwestern region of the United States, the Temecula Medical Center requires special attention. Most of this attention is present in this report, dealing with the lateral resistant systems. While the floor system is simple, the numerous shear walls provided are thick at 24″ and heavy reinforced with 6″ spacing. Even though the floor and column system provides minimal lateral resistance, these specifications are needed since these walls are the only primary system used to resist the heavy lateral loads.

Appendix

Included in the Appendix are Wind calculations, Torsional calculations, Seismic values, as well as a Shear wall spot check.

Wind Design

Wind forces were calculated using the design criteria in ASCE 7-05, Section 6.5. The wind pressures are distributed evenly across each level. The sum of these values produced the base shear due to wind.

K_d = 0.85
I = 1.15
K_{zt} = 1.29
V = 85 mph

Kz & qz		
z(ft)	Kz	qz
0-15	0.85	19.82
20	0.90	20.99
25	0.94	21.92
30	0.98	22.86
40	1.04	24.26
50	1.09	25.42
60	1.13	26.35
70	1.17	27.29
80	1.21	28.22
90	1.24	28.92
100	1.26	29.39
107	1.28	29.85

Gust Effect Coefficients										
Rigid Building										
I_z	Q_{N-S}	Q_{E-W}	B_{N-S} (ft)	B_{E-W} (ft)	g_Q	h (ft)	c	Є bar	z (ft)	g_V
0.179	0.769	0.805	564	353	3.4	97	0.2	1/5.0	64.2	3.4

Pressure				
Wind From N-S				
Windward		Leeward		Total
h (ft)	p (psf)	h (ft)	p (psf)	P (psf)
0-15	16.51	0-15	-10.00	26.51
20.00	17.48	20.00	-10.00	27.49
25.00	18.26	25.00	-10.00	28.26
30.00	19.03	30.00	-10.00	29.04
40.00	20.20	40.00	-10.00	30.20
50.00	21.17	50.00	-10.00	31.18
60.00	21.95	60.00	-10.00	31.95
70.00	22.73	70.00	-10.00	32.73
80.00	23.50	80.00	-10.00	33.51
90.00	24.09	90.00	-10.00	34.09
100.00	24.47	100.00	-10.00	34.48
107.00	24.86	97.00	-10.00	34.87

Pressure				
Wind From E-W				
Windward		Leeward		Total
h (ft)	p (psf)	h (ft)	p (psf)	P (psf)
0-15	16.78	0-15	-10.10	26.88
20.00	17.77	20.00	-10.10	27.87
25.00	18.56	25.00	-10.10	28.66
30.00	19.35	30.00	-10.10	29.45
40.00	20.53	40.00	-10.10	30.63
50.00	21.52	50.00	-10.10	31.62
60.00	22.31	60.00	-10.10	32.41
70.00	23.10	70.00	-10.10	33.20
80.00	23.89	80.00	-10.10	33.99
90.00	24.48	90.00	-10.10	34.58
100.00	24.87	100.00	-10.10	34.98
107.00	25.27	97.00	-10.10	35.37

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

Windward

$p_z = q_z G_f C_p - q_h(GC_{pi})$

Leeward

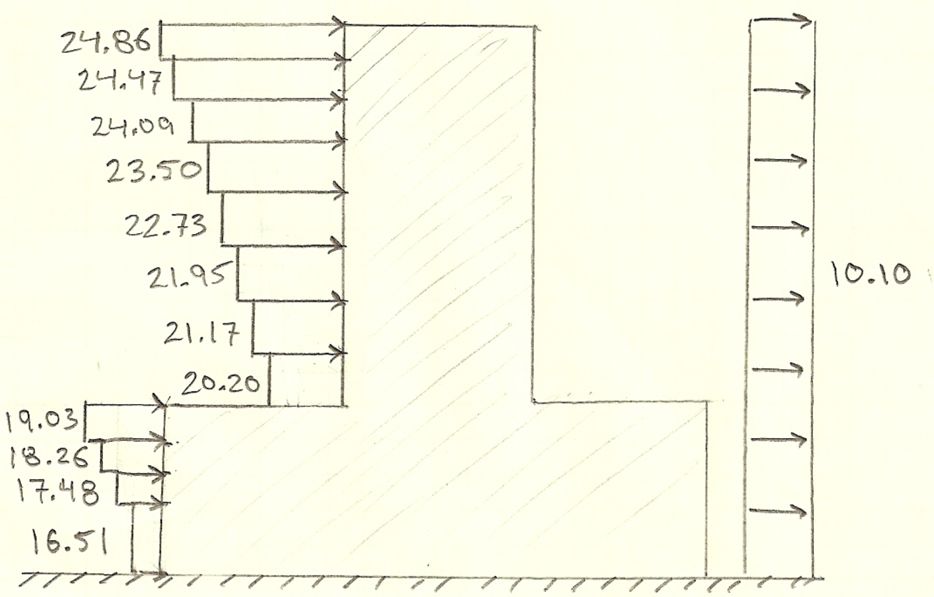
$p_h = q_h GC_p - q_h(GC_{pi})$

WIND FROM N-S

WIND LOAD

WINDWARD (PSF)

LEE-WARD (PSF)



Torsional Calculations

North-South Torsional Moment, M_{ta}				
Floor	Structural Width	5% Width	F_x	Torsion
	(ft)	(ft)	(kips)	(ft-kips)
1	78	3.9	0.0	0
2	78	3.9	140.2	547
3	78	3.9	212.4	828
4	78	3.9	369.6	1441
5	78	3.9	547.9	2137
6	78	3.9	747.7	2916
Roof Low	78	3.9	2039.0	7952
Roof High	78	3.9	178.4	696
			Total	16517.28

East-West Torsional Moment, M_{ta}				
Floor	Structural Width	5% Width	F_x	Torsion
	(ft)	(ft)	(kips)	(ft-kips)
1	350	17.5	0.0	0
2	350	17.5	140.2	2454
3	350	17.5	212.4	3717
4	350	17.5	369.6	6468
5	350	17.5	547.9	9588
6	350	17.5	747.7	13085
Roof Low	350	17.5	2039.0	35683
Roof High	350	17.5	178.4	3122
			Total	74116

North-South Torsional Moment, M_t			
Floor	Eccentricity	F_x	Torsion
	(in)	(kips)	(ft-kips)
1	30	0.0	0
2	30	140.2	351
3	30	212.4	531
4	30	369.6	924
5	30	547.9	1370
6	30	747.7	1869
Roof Low	30	2039.0	5098
Roof High	30	178.4	446
		Total	10588

East-West Torsional Moment, M_t			
Floor	Eccentricity	F_x	Torsion
	(in)	(kips)	(ft-kips)
1	80	0.0	0
2	80	140.2	935
3	80	212.4	1416
4	80	369.6	2464
5	80	547.9	3653
6	80	747.7	4985
Roof Low	80	2039.0	13593
Roof High	80	178.4	1189
		Total	28235

Seismic Calculations

- Building Location

zip code = 92592

- Using USGS software (2003 NEHRP Seismic Provisions)

  $\rightarrow S_s = 2.026g \rightarrow$ maximum

  $\rightarrow S_1 = 0.761\ g$

  Site Class – B

- IBC 2006

  - Occupancy Category – IV

    $S_1 \geq 0.75$

    $\downarrow$

    SDC F

- Permitted Analytical Procedures

  - No Damping systems on site

  $S_{ms} = F_a S_s$ $\quad F_v = 1.0$

  $S_{m1} = F_v S_1$ $\quad F_a = 1.0$ } ASCE TABLE 11.4

  $\rightarrow S_{ms} = 2.026\ g$

  $\rightarrow S_{m1} = 0.761\ g$

  $\rightarrow S_{DS} = 2S_{ms}/3 = \frac{2(2.026)}{3} = 1.351$

  $\rightarrow S_{D1} = 2S_{m1}/3 = \frac{2(.761)}{3} = 0.507$

- Equivalent Lateral Force Procedure

  Ordinary Reinf. Concrete Shear Walls:

  $R = 4$

  Importance Factor

  $I = 1.5$

- $T_a = C_t h_n^x$

$C_t = 0.016$, $x = 0.9$ } Concrete Frame

$T_a = .016(107)^{0.9}$

$h_n = 107'$

$\rightarrow T_a = 1.073$

$T = C_u T_a$ $\qquad C_u = 1.4$

$\rightarrow T = 1.4(1.073) = 1.502$

From Figure 22-15 in ASCE

$T_L = 8$

$T < T_L$

$$C_s = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} = \frac{0.507}{1.502\left(\frac{4}{1.5}\right)}$$

$\rightarrow C_s = 0.127$

$$C_s < \frac{0.5 S_1}{\left(\frac{R}{I}\right)} = \frac{0.5(.761)}{\left(\frac{4}{1.5}\right)} = 0.143$$

$\rightarrow C_s = 0.143$

EFFECTIVE SEISMIC WEIGHT

Area Calcs

Level 1 = 163,000 sq ft
Level 2 = 26,500 sq ft
Level 3-6 = 26,500 sq ft

Level 1:

5" slab on grade

$= \left(\frac{5}{12}\right)(163{,}000)(150) = 10188 \times 10^3$ lb

39 cols @ 26x26 @ 9'

$= \left(\frac{26 \times 26}{12}\right)(9')(150) = 76050^{lb} \times 39$

$= 2965 \times 10^3$ lb

Level cont'd

118 cols @ 20x20 @ 9'

$118 \times \frac{20 \times 20}{12} \times 9 \times 150 = 5310 \times 10^3$ lb

Level 2

$6''/12 \times 26{,}500 \times 150 = 1987.5 \times 10^3$ lb

39 cols @ 26×26 @ $9' + (\frac{13.5'}{2}) = 15.5$

$39 \times \frac{26 \times 26}{12} \times 15.5' \times 150 = 4715 \times 10^3$ lb

Double Tees

200 @ $6 \times 4 \times 1'$ (assumed conc. thickness)

$200 \times 6 \times 4 \times 1 = 4.8 \times 10^3$ lb

Level 3 – 6

$6/12 \times 26{,}500 \times 150 = 1987.5 \times 10^3$ lb

39 cols @ 26×26 @ 13.5'

$= 4449 \times 10^3$ lb

Roof Low

10" SLAB

$10''/12 \times 26500 \times 150 = 3312.5 \times 10^3$ lb

Cols

39 cols @ 26×26 @ $\frac{17.3}{2} + \frac{19.7}{2} \times 150$

$= 5767.13 \times 10^3$ lb

Roof High

$20\ psf + 2\ psf\ (Deck) = 22\ psf$

$22\ psf \times 26500 = 583 \times 10^3\ lb$

ADD 10 psf MISC TO EACH FLOOR

$26500\ sqF \times 6 \times 10\ psf = 1590 \times 10^3\ lb$

$\rightarrow \underline{W_{TOTAL}} = 29706.3^K$

BASE SHEAR

$\rightarrow \quad V = C_s W = 0.143 W$

$= 4248^K$

$k = 0.75 + 0.5\ (1.502)$

$\rightarrow \quad k = 1.501$

Seismic Forces at Each Level

$$C_x = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$$C_1 = 0$$

$$C_2 = \frac{(6707)(18)^{1.5}}{(6707)(18)^{1.5} + (4449)(31.5)^{1.5} + (4449)(45)^{1.5} + (4449)(58.5)^{1.5} + (4449)(72)^{1.5} + (9079)(87.3)^{1.5} + (583)(107)^{1.5}}$$

$$C_2 = 0.033$$

$$C_3 = \frac{(4449)(31.5)^{1.5}}{1.54 \times 10^7}$$

$$C_3 = 0.05$$

$$C_4 = \frac{(4449)(45)^{1.5}}{1.54 \times 10^7}$$

$$C_4 = 0.087$$

$$C_5 = \frac{(4449)(58.5)^{1.5}}{''}$$

$$C_5 = 0.129$$

$$C_6 = \frac{(4449)(72)^{1.5}}{''}$$

$$C_6 = 0.176$$

$$C_{RL} = \frac{(9079 \times 87.3^{1.5})}{''}$$

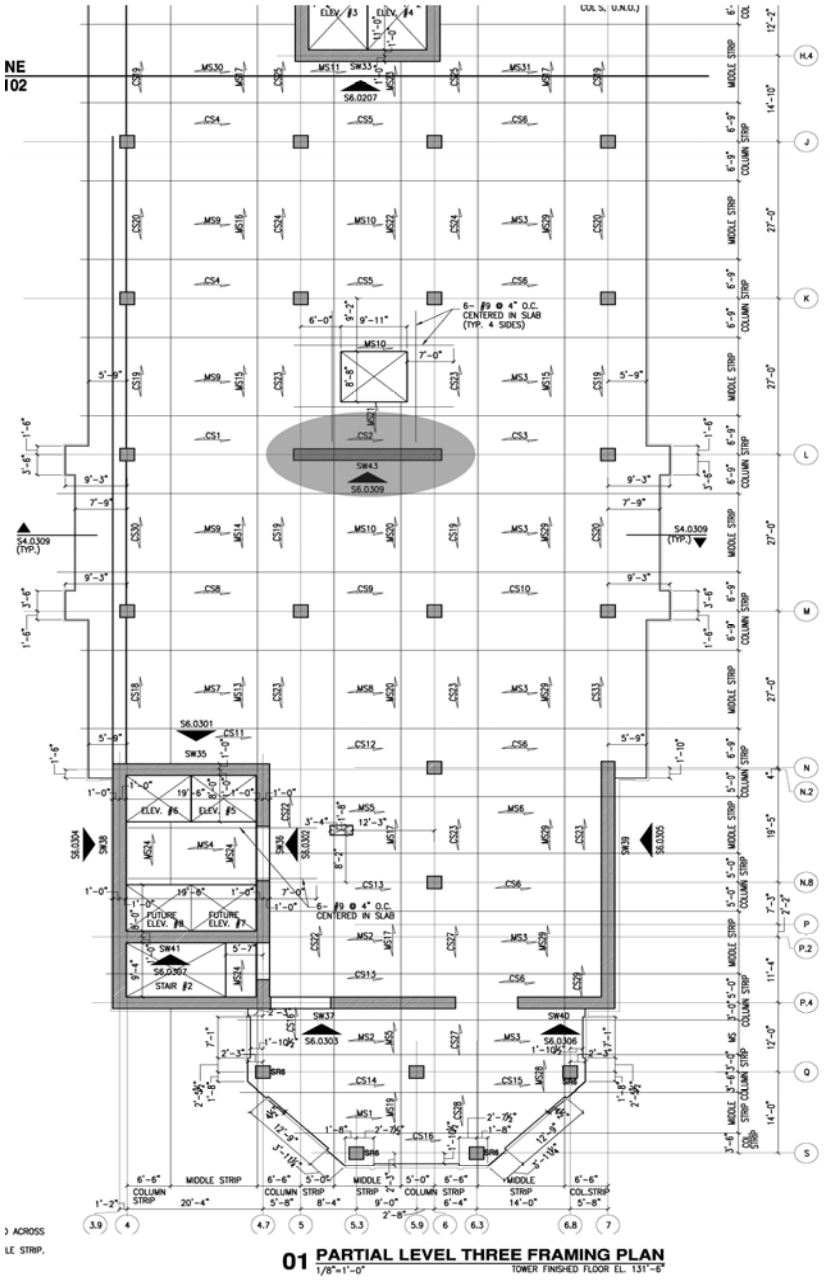
$$C_{RL} = 0.48$$

$$C_{RH} = \frac{(583 \times 107^{1.5})}{''}$$

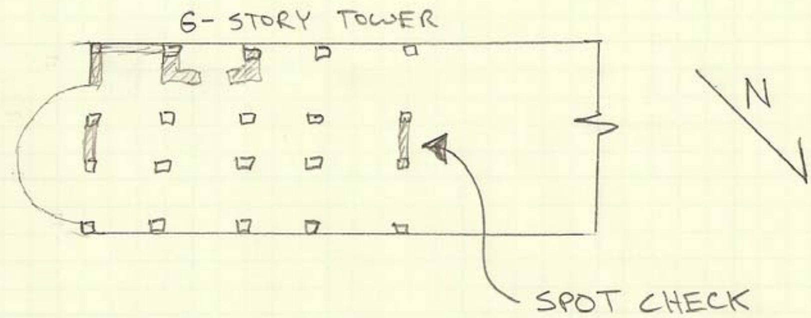
$$C_{RH} = 0.042$$

Spot Check – Shear Wall

Highlighted in grey is the spot checked shear wall. See calculations below for additional details.



SPOT CHECK | SHEAR WALL | 1 of 3



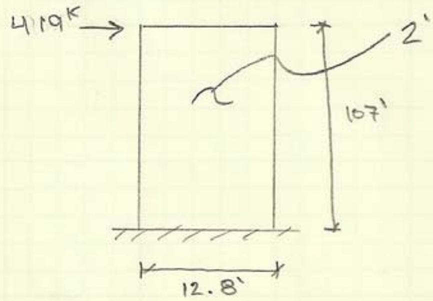
Overturning Moment ≈ 67,911.1 ft-K

≈ 13 shear walls (y-direction)

$M_u = 67{,}911.1/13 = 5223.9$ ft-K

$V_u = 1213.5^K + 4235.3 = 5448/13 = 419^K$

$f'_c = 7000$ psi
$F_y = 60000$ psi
$N_u = 0$



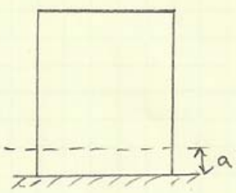
Step 1:

$$V_u \le \phi V_n = \phi\, 10\sqrt{f'_c}\cdot h \cdot d$$

$$d = 0.8\,\ell_w = 0.8(12.8)(12) = 122.88'$$

$$\phi V_n = 0.75\,(10)\frac{\sqrt{7000}}{1000}\cdot 107' \cdot 122.88 = 8250^k > V_u$$

Step 2:



$$\le \frac{\ell_w}{2} = \frac{12.8}{2} = 6.4'$$

$$\le \frac{h_w}{2} = \frac{107}{2} = 53.5'$$

$$V_c = 3.3\sqrt{f'_c}\cdot h \cdot d + \frac{N_u \cdot d}{4\ell_w} = 3.3\frac{\sqrt{7000}\cdot 2 \times 12 \cdot 122.9}{1000}$$

$$V_c = 814.4^k$$

$$M_u = V_u \cdot a = 419 \times 53.5' = 22416.5 \text{ ft-k}$$

Step 3: Horizontal Reinf.

$$V_u > \tfrac{1}{2}\phi V_c = \tfrac{1}{2}(.75)(814.4) = 305.4$$

$$V_u > 305.4 \qquad \text{* NEED } V_s$$

$$\phi V_n = \phi(V_c + V_s)$$

$$419 = 0.75(814.4 + V_s)$$

$$V_s = 255.7$$

$$V_s = \frac{A_v \cdot f_y \cdot d}{s}$$

$$\frac{A_v}{s} = \frac{255.7}{(60)(122.9)} = 0.0347$$

$$\#5 \rightarrow s = \frac{A_v}{.0347} = \frac{.31}{.0347} = 8.9''$$

Check min./max required s/ρ_t

#5 @ 9" $\rho_t = \frac{A_v}{s \cdot h} = \frac{.31}{9(24")} = .00144 < .0025$ ∴ NO GOOD

2 #5 @ 9" $\rho_t = \frac{2(.31)}{9(24")} = .00287 > .0025$ ∴ OK

$$s \begin{cases} \le \frac{l_w}{5} = 30.72" \\ \le 3h = 3852" \\ \le 18" \ * \text{ controls} \end{cases}$$

$\boxed{\text{USE 2 \#5 @ 9" FOR HORIZ. REINF.}}$

Step 4: Vertical Reinf.

$$\rho_l = \frac{A_v}{s \cdot h} \ge .0025 + 0.5(2.5 - \frac{h_w}{l_w})(\rho_t - .0025)$$

$$= .0025 + 0.5(2.5 - \frac{107}{12.8})(.0029 - .0025)$$

$$= .00133^*$$

$$s \le \begin{cases} \frac{l_w}{3} = 51.2" \\ 3h = * \\ 18" = 18" \quad * \text{CONTROLS} \end{cases}$$

$$\rho_l = \frac{A_v}{s \cdot h} = .00133$$

$$S_{req} = \frac{A_v}{.00133h}$$

TRY #5 $S_{req} = \frac{.31}{.00133(24)} = 9.7$

$\boxed{\text{USE \#5 @ 9" FOR VERT REINF}}$