Lateral System Analysis and Confirmation Design

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building

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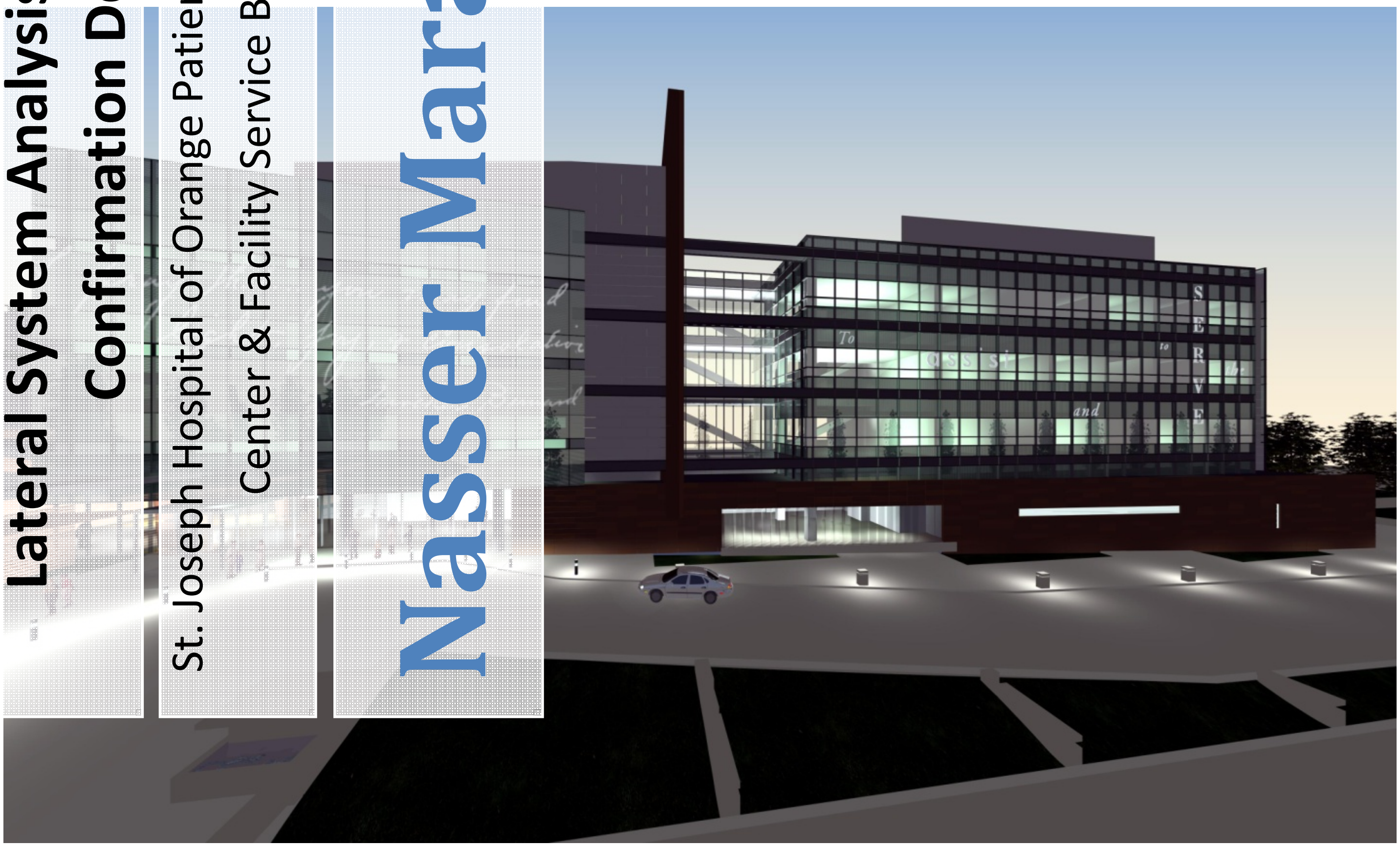

Technical Report 3

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STRUCTURAL OPTION
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Executive Summary

The following report analyzes the main lateral force resisting system and determines the controlling lateral forces which are seismic in this case. The analysis was done in ETABS and was also checked using hand calculations to ensure accuracy of the computer model. When checking the serviceability and strength capacity of the structural members, we find that all criteria are met according to ASCE 7-05 code. This report also concludes that the structure is more rigid than expected by ASCE 7-05 and is justified accordingly. Please refer to appendix for calculations, and any additional computer output and hand calculations are available upon request.

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Introduction

St. Joseph Hospital of Orange Patient Care Center & Facility Service Building is to be built within Saint Joseph Hospital Campus serving the healthcare needs of the Orange county community in Orange, CA. The Patient Care Center is linked to the main hospital through an underground tunnel and through a lobby to further serve the patients' needs. The building consists of four stories with basement that gives 252,712 square foot of additional hospital space. The buildings is approximately 285'-0" by 198'-0" on Level 1 and 2 and then the floor plan is reduced to 240'-0" by 198'-0" on Level 3, 4 and the roof.

The main entrance to the lobby is connected to the adjacent hospital reception area. The Patient Care Center consists of operating rooms to expand the surgical capacity of the main hospital. Operating rooms are equipped with latest innovative technology and medical equipment. To help further serve the main hospital, the Patient Care Center also has additional room for incoming patients and rooms for patients requiring intensive care.

The Patient Care Center has a central sterile plant located on the basement level with MEP equipment. The first level of the hospital consists of surgical rooms, administrative rooms and the lobby. The upper floors are separated by the central courtyard located on level 2. The west side consists of patient rooms and the east side consists of intensive care units. The remaining mechanical equipment is located on the roof level.

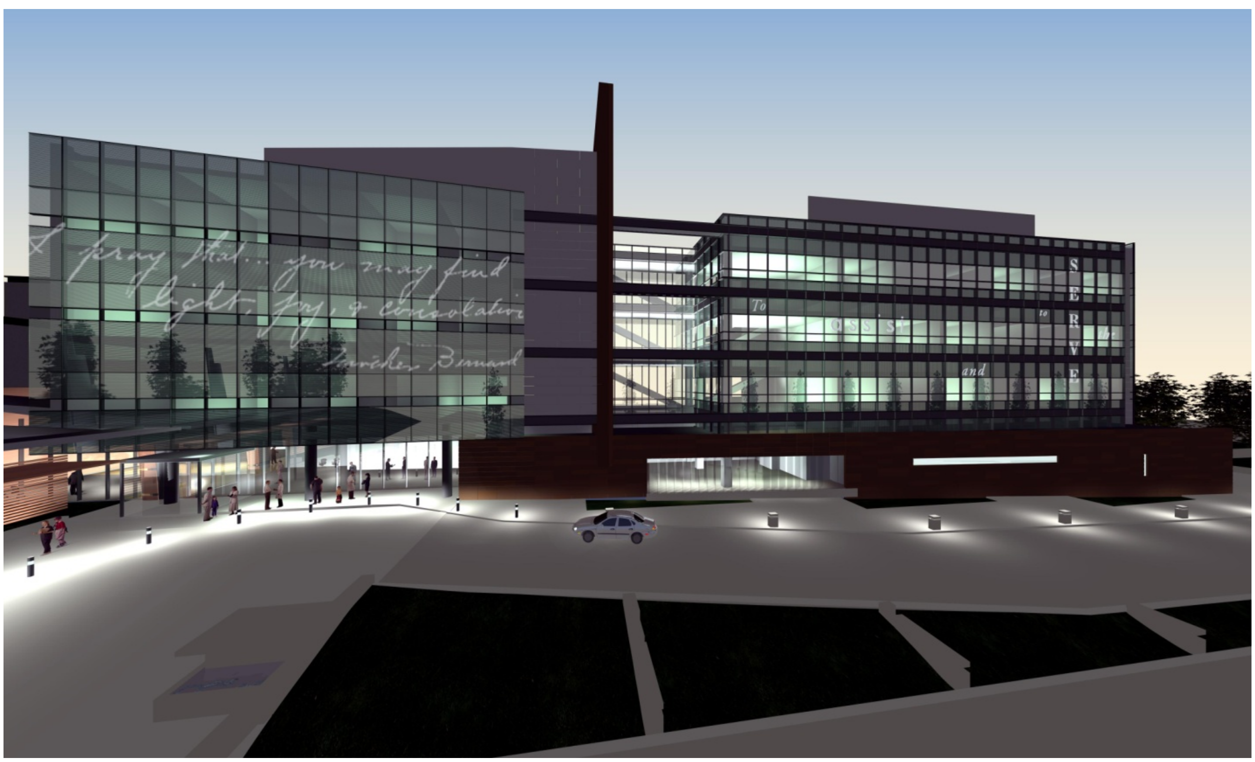

Figure 1. Computer rendering of Patient Care Center's North elevation.

Structural Systems

Lateral Resisting System

The lateral system consists of 6 sets of braced frames located both along the N-S and E-W planes. It ranges from 2 bays to as long as 6 bays framing vertically to the roof of the structure. These braces are supported by shear walls at basement level. The braced frames are typically X-bracing while a whole set running E-W is diagonally braced. Both configurations are considered concentrically braced frames. X-Braced frames are a TS shaped with a gusset plate slipped in and welded. The gusset plate is then welded onto the column and beam, allowing the brace to buckle out of plane to dissipate energy at time of an earthquake. While diagonally braced member consists of a W Shape section which has its web and flanges welded to a plate which are all then welded onto the gusset plate. The plates attached to the flange are slipped in the gusset plate and welded. The following two figures represent a connection detail to the braced frames.

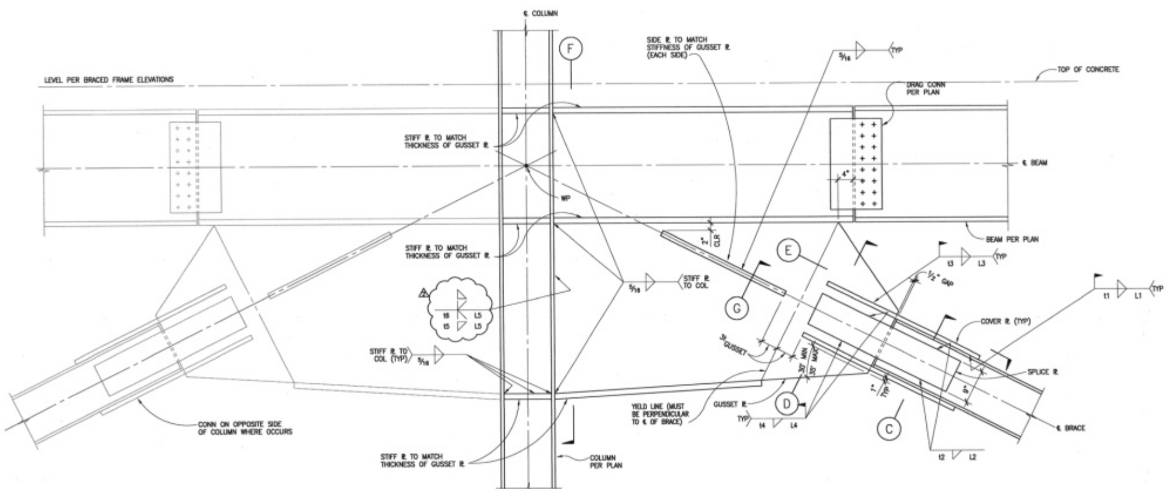

Fig3. Diagonal Brace Connection Detail.

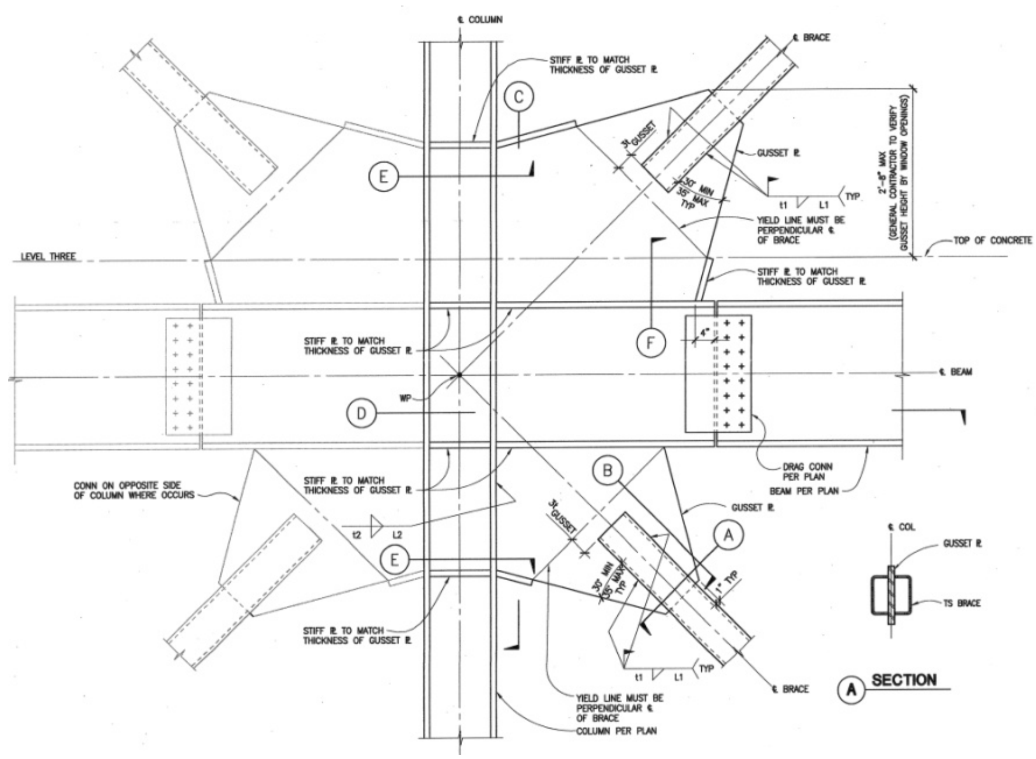

Fig4. X Brace Connection Detail

The following figure represents the lateral system labeled on level 1. The lateral system consists of concentrically braced frames. There are 6 groups of braced frames altogether, and two types, one consists of diagonal bracing while the others are all X braced frames.

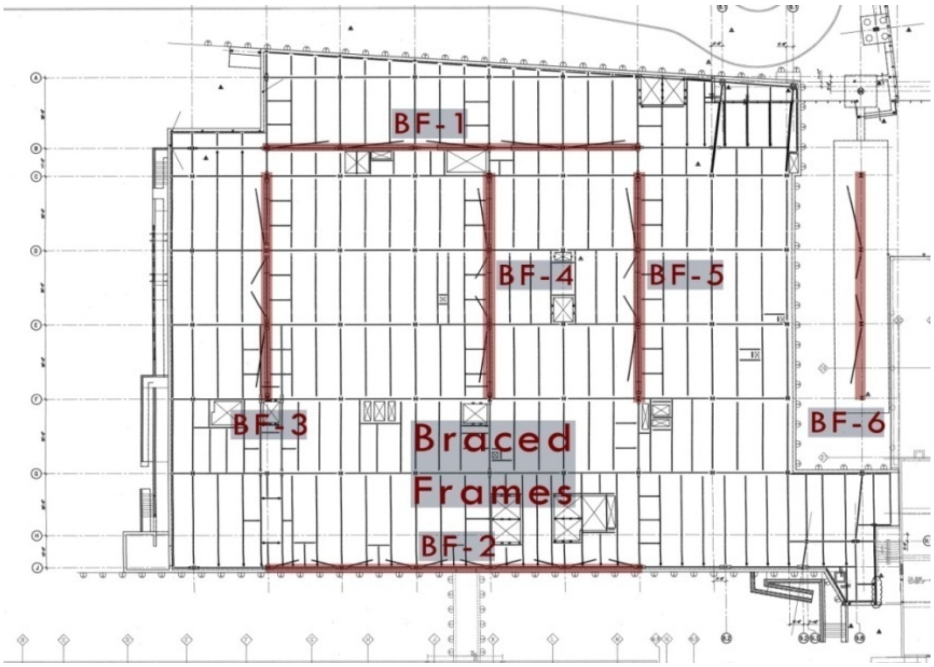

Fig5. 1st Floor plan labeling all braced frames

Consist of diagonal members, member's sizes range from W14x90 on level 4, to W14x211 on level 1. Braced frames are supported by shear walls located on the basement floor and tied into a 5'-0" continuous footing. The entire brace frame for BF-1 and BF-2 is 150'-0" wide.

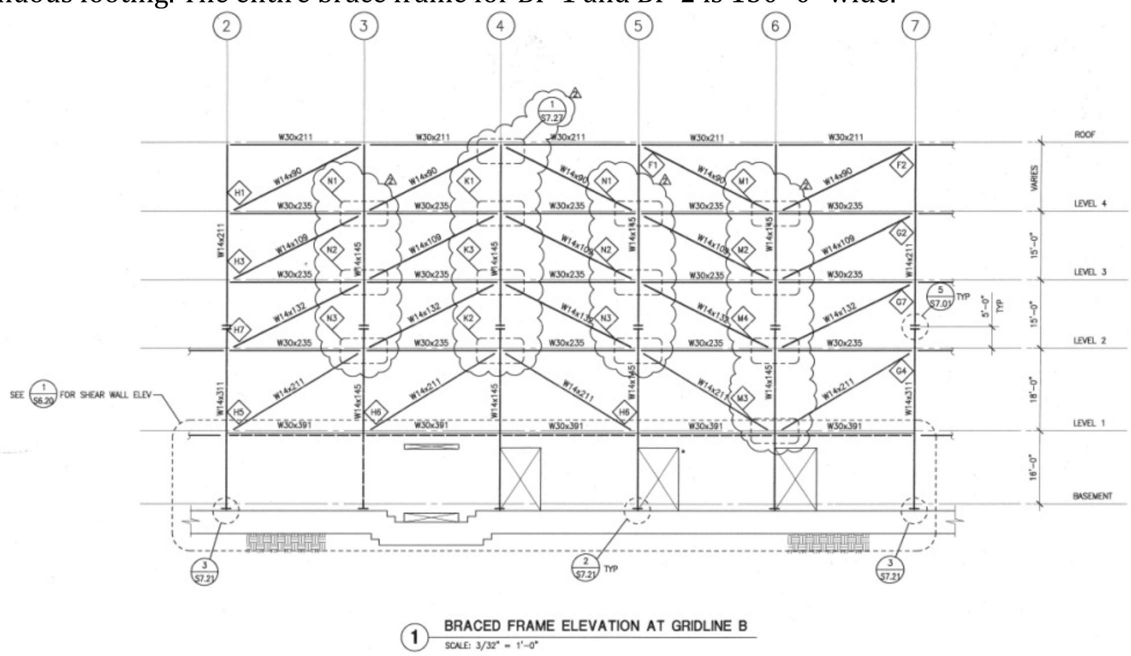

Fig6. BF-1 Braced Frame Elevation

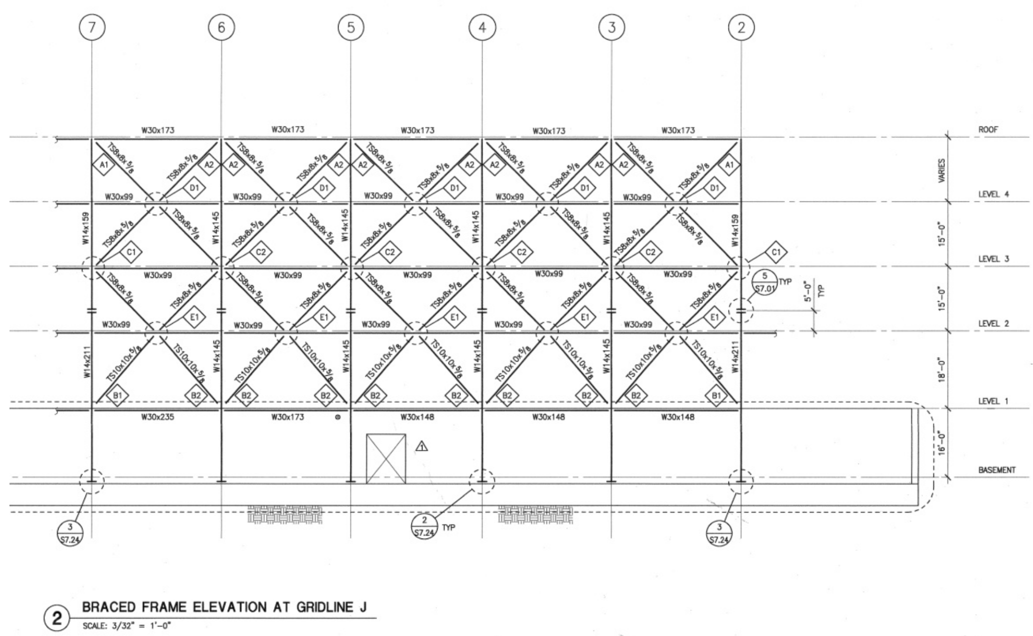

Fig7. BF-2 Braced Frame Elevation

BF-3 X-Braced Members

Consist of X-braced members; members are usually steel tubes. There are two W14x145 running as diagonal members on each end bay on level 1. All braced frames but BF-6 are supported by shear walls located on the basement floor and tied into a 5'-0" continuous footing. BF-6 is supported by a 5'-0" continuous footing located at level 1. The entire braced frame for BF-3, BF-4, BF-5, and BF-6 is 90'-0" wide.

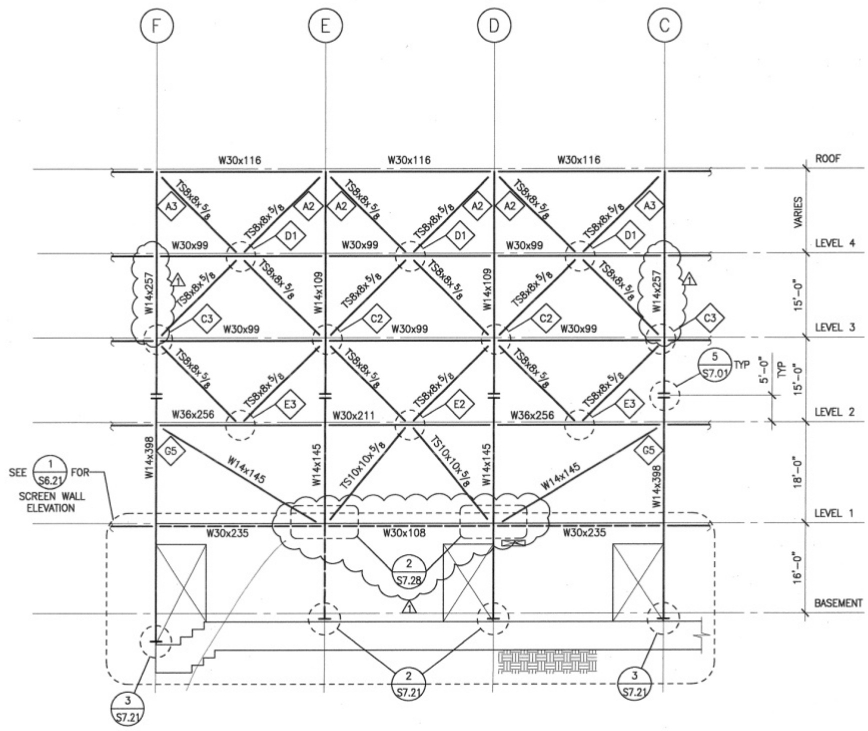

Fig8. BF-3 Braced Frame Elevation

Foundation system

Gravity columns at the basement level are supported by concrete footings. These footings range depth from 1'-6" to 3'-6" and their size ranges from 2'-0"x2'-0" to 16'-0"x16'-0". While the shear walls are supported by continuous deep footings typically 5'-0" deep and 7'-0" wide from each face of the wall. The majority of the foundation is considered shallow as advised by the geotechnical engineer. While the main entrance canopy is supported by piles caps each connected to 4 piles.

Identification of other structural elements

There are several areas in the building that were not discussed in depth in this report. These include the underground tunnel connecting to the adjacent hospital, the canopy at the building's main entrance. Other structural elements like checking the braced system connections and the foundation system where not discussed in this report but will be analyzed and justified in later reports.

Codes and Material Properties

Codes and Referenced Standards

The following table shows the codes that were adopted in this report and codes that were implemented by the designer.

Codes adopted by this report	Codes adopted by the designer
2007 California Building Code	Title 24, Part 1 2001 California Building Code
ASCE 7-05	1997 Uniform Building Code with California amendments
ACI 318-05	
13th Edition of the AISC Manual of Steel Construction	

Material Strength Requirements

These requirements correspond to the general structural notes on the plans.

Concrete	Strength	Density
Footings	4000 psi	150 pcf
Basement Walls	4000 psi	150 pcf
Composite Concrete Light Weight	3000 psi	110 pcf
Composite Concrete Normal Weight	4000 psi	150 pcf
Slab on Grade	4000 psi	150 pcf
Drilled Concrete Piles	4000 psi	150 pcf
Reinforcing (Steel)	ASTM706 Grade 60	

Steel Deck	I (in^4)	S (in^3)
3" x 18 GA Deck	1.203	.767

Structural Steel	ASTM	Fu (ksi)	Fy (ksi)
Wide-Flange Shapes (WF Shapes, W14 and larger)	A992	65	50
WF Shapes, W12x14, W10x12, W8x12 and smaller	A992	65	50
Plates	A572, Gr50	65	50
Connection Plates	A36	58	36
Pipe Columns	A53 Grade B	80	40
Tube Sections	A500 Grade B	58	40
Bolts	A325N, A490SC	$F_{nt} = 90$	$F_{nu} = 48$
Bolts in Concrete	A307, A3548C	$F_{nt} = 45$	$F_{nu} = 24$
Angles, Channel and WT Shapes	A36	58	36

Foundation	
Allowable Bearing (Gravity Loads)	4000 psf (Basement Footings) 2500 psf (Ground Floor Footings)
Equivalent Fluid Pressure	30 pcf (unrestrained walls) 23 pcf (unrestrained walls)
Passive Earth Pressure	300 pcf

Building Loads

Live Loads

Live loads are determined in accordance with ASCE 7-05.

Occupancy	Designer's Uniform Live load (psf)	2007 CBC Uniform Live loads (psf)
Roof	20	20
Patient Rooms	80[1]	40
Operating Rooms, Laboratories	80[1]	60
Corridors	80[1]	100
Storage	120	125
Computer Rooms	100	100
Elevator Machines Rooms	125[1]	
Public Areas, Assemblies	100	100
Mechanical Rooms	150[1]	50
Roof Gardens	100	100
Office	80[1]	50

[1] Designer's value used for simplicity reasons.

Dead Loads

Refer to Appendix for dead load calculations. Material weights are taken from the ASCE 7-05 Chapter C3.

	LVL1	LVL2	LVL3	LVL4	ROOF
Concrete Topping	75	44	44	44	94
Steel Deck (18 Gage)	3	3	3	3	3
Super Imposed	12	12	12	12	25
Partitions	20	20	20	20	
Total Dead Load	**110**	**79**	**79**	**79**	**122**

*Units in pounds per square foot

Level 2 Courtyard	PAVER	PLANTER W/ TREES	PLANTER
Concrete Topping	94	94	94
Steel Deck (18 Gage)	3	3	3
Super Imposed	22	552	342
Topping	80		
Total Dead Load	**200**	**649**	**439**

Wind Loads

Wind Load Criteria

Below is a list of assumption made for determining wind load calculations based on ASCE 7-05. Refer to appendix for calculations.

Wind Load Calculation

Locality Factors and Building Description

Basic Wind Speed	V	85	*h*	75	*Bldg. Height*
wind directionality factor	Kd	0.85	*N-S*		
Importance Factor	I	1.15	*B*	285	
Windward Wall	Cp	0.8	*L*	210	
Leeward Wall	Cp	-0.5	*h/L*	**0.3571**	
Side Wall	Cp	-0.7	*E-W*		
Topographic Factor	Kzt	1	*B*	210	
Period	T	0.47	*L*	285	
Internal pressure coeff.	GCpi ±	0.18	*h/L*	**0.2632**	

B Horiz. Dim. normal to wind dir.
L Parallel Dim. normal to wind dir.

The tables below summarize the pressures, loads and shears from the wind load calculations. Refer to Appendix for additional information.

Wind Pressures

Refer to page 13, for B and L in the direction specified. Refer to appendix for additional calculations.

				Pressure (psf)		
				N-S		
Level	Elevation	Trib Height	Height	Windward	Leeward	Sideward
Mech Room	238.5	6	75	18.54	-13.04	-16.71
Roof	226.5	13.5	63	18.02	-13.04	-16.71
Level 4	211.5	15	48	17.23	-13.04	-16.71
Level 3	196.5	15	33	16.22	-13.04	-16.71
Level 2	181.5	16.5	18	14.74	-13.04	-16.71

				Pressure (psf)		
				E-W		
Level	Elevation	Trib Height	Height	Windward	Leeward	Sideward
Mech Room	238.5	6	75	18.78	-13.19	-16.91
Roof	226.5	13.5	63	18.24	-13.19	-16.91
Level 4	211.5	15	48	17.44	-13.19	-16.91
Level 3	196.5	15	33	16.41	-13.19	-16.91
Level 2	181.5	16.5	18	14.91	-13.19	-16.91

Force due to Wind

				Force (kips)			
				N-S		E-W	
Level	Elevation	Trib Height	Height	N-S Dir	E-W Dir	N-S Dir	E-W Dir
Mech Room	238.5	6	75	54.02	-28.58	40.28	-21.31
Roof	226.5	13.5	63	119.50	-64.30	89.10	-47.95
Level 4	211.5	15	48	129.42	-71.44	96.48	-53.28
Level 3	196.5	15	33	125.09	-71.44	93.24	-53.28
Level 2	181.5	16.5	18	130.64	-78.58	97.36	-58.61

Shear Due to Wind

				Shear (kips)			
				N-S		E-W	
Level	Elevation	Trib Height	Height	N-S Dir	E-W Dir	N-S Dir	E-W Dir
Mech Room	238.5	6	75	0.00	1.00	2.00	3.00
Roof	226.5	13.5	63	54.02	-28.58	40.28	-21.31
Level 4	211.5	15	48	173.52	-92.87	129.37	-69.26
Level 3	196.5	15	33	302.94	-164.31	225.86	-122.54
Level 2	181.5	16.5	18	428.02	-235.75	319.10	-175.82
			Total	558.7	-314.3	416.5	-234.4

Over Turning Moment due to Wind Pressure

				Over Turning Moment (ft-K)			
				N-S		E-W	
Level	Elevation	Trib Height	Height	N-S Dir	E-W Dir	N-S Dir	E-W Dir
Mech Room	238.5	6	75	4051.22	-2143.18	3020.70	-1598.35
Roof	226.5	13.5	63	7528.66	-73.83	1849.80	-1032.73
Level 4	211.5	15	48	6212.04	-77.47	1891.62	-1147.48
Level 3	196.5	15	33	4127.89	-71.59	1689.42	-1147.48
Level 2	181.5	16.5	18	2351.61	-69.32	1552.74	-1262.23
			Total OTM	24271.4	-2435.4	10004.3	-6188.3

Please note that Over Turning Moment is computed at ground level.

Seismic Loads

Seismic Load Criteria

Below is a list of assumptions made for determining the buildings seismic loads based on ASCE7-05. Site class informational comes from the geotechnical report. Although the geotechnical report includes response spectra for maximum capable earthquakes intended to represent the 1000 year event; the spectrums were not used since they were intended to be used with UBC. While ASCE 7-05 uses 2% exceedence in 50 years for the 2500 year earthquake which was not specified in the geotechnical report. Therefore the building's longitude and latitude coordinates where used to determine the seismic design values with the USGS website.

Occupancy Category	IV
Importance Factor (I_E)	1.5
Mapped Spectral Response Accelerations	S_s = 1.378gr S_1 = .497
Site Class	D
Site Class Factors	F_a = 1 F_v = 1.5
$S_{MS} = F_a(S_s)$	1.378
$S_{M1}= F_v(S_1)$	.7455
$S_{DS}= 2/3(S_{MS})$	.92
$S_{D1}= 2/3(S_{M1})$	.497
Seismic Response Coefficient (C_t)	.02
Period Coefficient (x)	.75
Building Height (h_x)	63'-0"
Coefficient for upper limit (C_u)	1.4
Approx. Period $T = (C_u)(C_t)(h_x)^x$	.626
Period T_b from ETABS Model	.48
T_L Long Period	8
Seismic Design Category	D
Response Modification Factor (R)	6 (Special Steel Concentrically Braced Frames)
Seismic Response Coefficient (C_s)	.23
Over Strength Factor (Ω_o)	2

Refer to the lateral resisting system check for vertical distribution of seismic forces and base shear calculations on page XX.

Building Weight and Mass

Floor	W (kips)	Area (ft^2)	W (psf)	Mass/ft^2
Roof	5393.40	39400.00	136.888	4.25
Level 4	3566.00	39400.00	90.508	2.81
Level 3	3566.00	39400.00	90.508	2.81
Level 2	8089.20	53438.00	151.375	4.70
Level 1	6466.87	53438.00	121.016	3.76

Buildings Weight and Mass

Distribution of Seismic Loads

Vertical Distribution of Seismic Forces

Floor	hx (ft)	W (kips)	hxkWx	Cvx	Fx (kips)	Vx (kips)	Mx (ft-K)
Roof	63.00	5393.40	1896398	0.51	2407.26	0	151657.45
Level 4	48.00	3566.00	853371	0.23	1083.26	2407.26	51996.32
Level 3	33.00	3566.00	502202	0.13	637.49	3490.52	21037.12
Level 2	18.00	8089.20	483188	0.13	613.35	4128.01	11040.34
Level 1	0.00	6466.87	0	0.00	0.00	4741.36	0.00
	Total	20615	3735159	Base Shear	4741	Overturning Moment	235731

Load Combinations

The following load combinations were taken from ASCE 7-05 section 2.3.2.

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

Since the following statements are satisfied, the load combinations with seismic forces are only taken into consideration in the analysis.

1.6(Base Shear for Wind Loads) < (Base Shear for Seismic Loads)

1.6(Over Turning Moment for Wind Loads) < (Over Turning Moment for Seismic Loads)

Earthquake Load E were investigated based on ASCE 7-05 Section 12.4.2, and the following load combinations were used.

5. $(1.2 + 0.2S_{DS})D + \rho Qe + L + 0.2S$

7. $(.9 - .2S_{DS})D + \rho Qe + 1.5H$

Where $S_{DS} = .92$, computed on page 14. Redundancy Factor (ρ) was taken to be 1. See Page 22 for justification.

ASCE 7-05 section 12.5 determines the direction of the loadings for Seismic Design Category D to be taken 100% percent of the forces for one direction plus 30% in the perpendicular direction.

With all these provisions the following list of load combinations were taken account for in the analysis.

Load Combination Name	Load Combination
EQ1	1.38D + EQX + .3EQY + L
EQ2	1.38D - EQX + .3EQY + L
EQ3	1.38D + EQX - .3EQY + L
EQ4	1.38D - EQX - .3EQY + L
EQ5	1.38D + .3EQX + EQY + L
EQ6	1.38D - .3EQX + EQY + L
EQ7	1.38D + .3EQX - EQY + L
EQ8	1.38D - .3EQX - EQY + L
EQ9	.72D + EQX + .3EQY
EQ10	.72D - EQX + .3EQY
EQ11	.72D + EQX - .3EQY

EQ12	.72D - EQX - .3EQY
EQ13	.72D + .3EQX + EQY
EQ14	.72D - .3EQX + EQY
EQ15	.72D + .3EQX - EQY
EQ16	.72D - .3EQX - EQY

Load Combinations with over strength factor.

When checking axial forces in columns, the following load combinations were used according to ASCE 7-05 Section 12.4.3.2. With the over strength factor Ω_o equal to 2 according to Table 12.2-1

5. $(1.2 + 0.2S_{DS})D + \Omega_o Qe + L + 0.2S$

7. $(.9 - .2S_{DS})D + \Omega_o Qe + 1.5H$

The following two load combinations were taken account for, when checking for columns running in the E-W direction and N-S direction. EQ17 will control for columns running in the E-W direction, and EQ18 will control for columns running in the N-S direction, based on previous analysis results from load combinations EQ1-EQ16.

Load Combination Name	Load Combination
EQ17	1.38D + 2EQX + .6EQY + L
EQ18	1.38D + .6EQX + 2EQY + L

Lateral Analysis

The lateral analysis was done using ETABS. The following lists of items were modeled and lists of assumptions were made.

- The Main Lateral Resisting System was only modeled consisting of all the braced frame bays in the structure. The material properties and frame sections in those bays were modeled according to the structural drawings.
- A Rigid Diaphragm was modeled at every floor with the lateral load being assigned to the diaphragm.
- Lateral forces were applied to the center of mass with the appropriate eccentricity ratio. See torsion calculations on page 21 for more information.
- The mass of the structure was assigned to a Null Shell Property at each floor. This gives us an approximate period from the modal analysis. Please see appendix for the assigned mass at each level to the ETABS model.
- Tributary Dead and Live Loads were assigned to each beam frame.
- Proper Load Combinations were assigned, See Page 18 for reference.
- Infinite spring stiffness was assigned in the x and y translation, and z rotation on a 1st floor node, mimicking zero movement about the x and y and rotation about the z due to the existence of a basement.

The following figure represents a 3-D view of the ETABS model, with the null shell area, and lateral resisting frames shown.

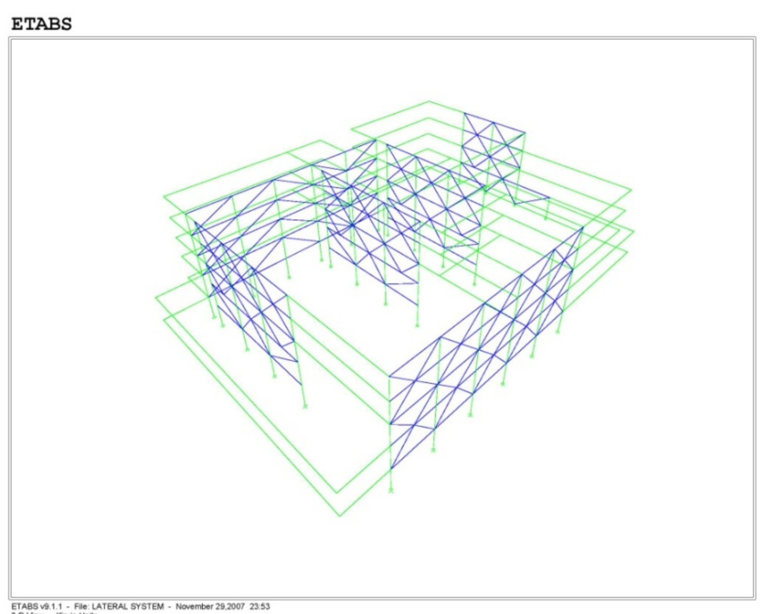

Fig9. 3-D ETAB Model

Torsion

According to ASCE 7-05 section 12.8.4.2, diaphragms that are non flexible are required to account for Inherent torsion M_t and Accidental Torsion M_{ta}.

Inherent Torsion

Since the Lateral forces are applied to the center of mass and the center of rigidity is calculated in the ETABS model, this will account for inherent torsion. A hand calculation for the center of rigidity and center of mass was done to verify the accuracy of the ETABS model. See Appendix for calculations.

Accidental Torsion

The analysis was first run with the seismic loads running in the x and y assigned to the center of mass + 5 percent accidental torsion. The following calculation was done to calculate the amplified accidental torsion according to ASCE 7-05 section 12.8.4.2.

From EQX, the displacements are;

$\Delta_a = .415''$, $\Delta_b = 1.3''$

Therefore $\Delta_{avg} = (1.3+.415)/2 = .858''$

$A_x =[\ 1.3/(1.2)(.858)\ ] = 1.59$ **Controls**

From EQY, the displacements are;

$\Delta_a = .629''$, $\Delta_b = 1.72''$

Therefore $\Delta_{avg} = (1.72+.629)/2 = 1.1745''$

$A_x =[\ 1.7/(1.2)(1.17)\ ] = 1.49$

5% (Accidental Torsion) (1.59) = **7.95%**

Therefore a 8% displacement eccentricity ratio to the center of mass was applied to each level where the lateral loads were applied. The analysis was then run again and the story drift from the controlling load combination was taken for comparison to the allowable story drift.

Redundancy Factor

After the ETABS analysis was performed, the lateral forces taken by each braced frame were calculated. The figure below represents the percentage contribution of the total applied seismic force in the x and y direction of each braced frame at level 1. Please refer to appendix for the hand calculations of relative stiffness for comparison with the ETABS model.

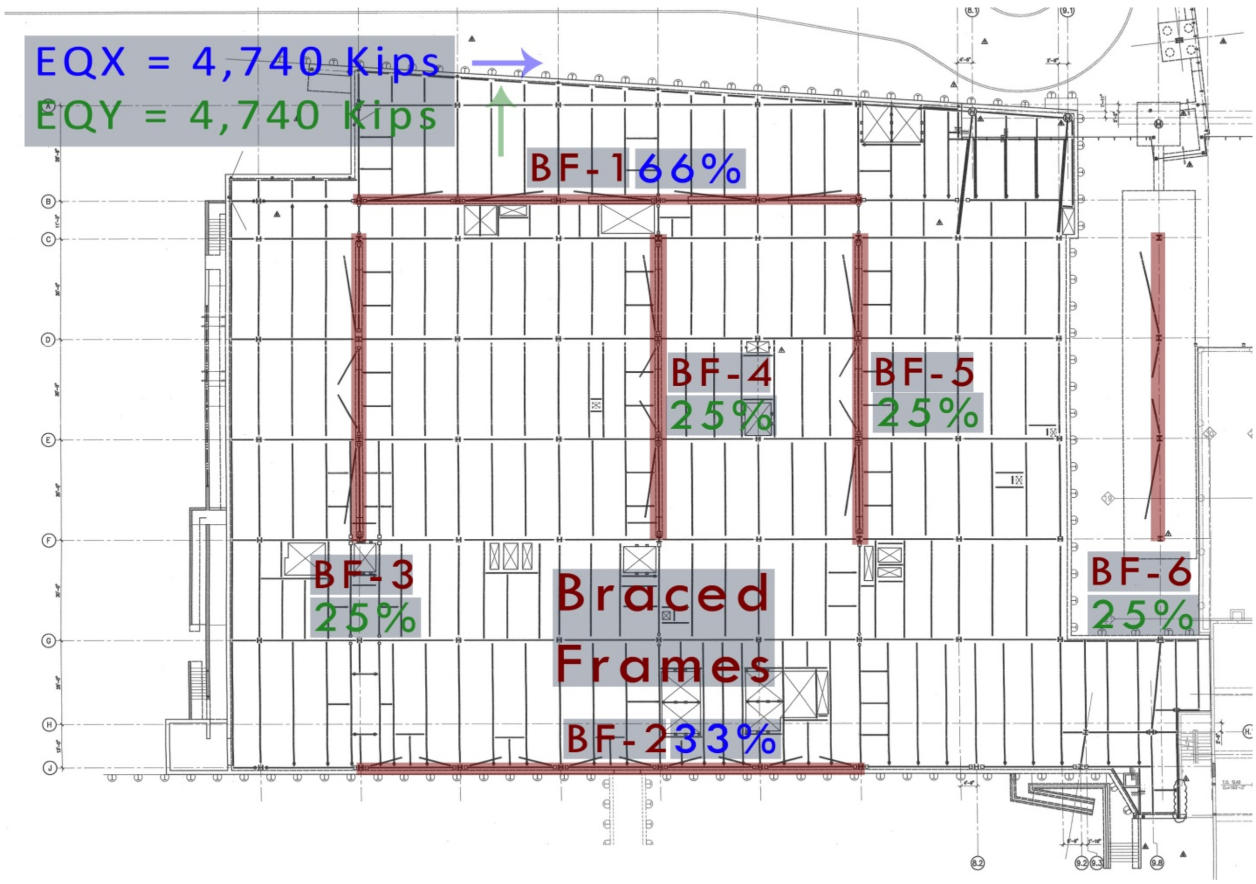

Fig10. Distribution of Lateral Forces among frames plan.

Since BF-1 is resisting more than 33% of the base shear, requirements per ASCE 7-05 12.3.4.2 must be met or else the redundancy factor must be taken to be 1.3 instead of 1.0. ASCE 7-05 section 12.3.4.2.a states that a removal of any brace in the frame shall not result in a 33% reduction in story strength. A quick spot check is done as follows:

BF-1 consists of 5 braces; removal of one would result in about 20% decrease. Since BF-1 is taking 66%, 20% of 66% = 13%. Therefore a 13% decrease would result in a removal of one braced frame which is less than 33%. This complies with Section 12.3.4.2.a therefore a redundancy factor of 1.0 can be used.

Story Drift

Story Drift was computed from the output of the ETABS model from the strength level design earthquake loads EQX and EQY, the following criteria was used to determine the allowable story drift.

Importance Factor (I_E)	1.5
Deflection Amplification (C_d)	5
Allowable Deflection	$\Delta_a = 0.01h_{sx}$
Story Drift	Δ = (Story Drift Ratio)(h_{sx})(C_d)/(I_E)

Story Drift

Story	hx (ft)	Allowable Drift (in)	Story Drift (in)	Story Drift Ratio (in/in)
Roof	15.00	2.70	0.73	0.001209
Level 4	15.00	2.70	0.87	0.001444
Level 3	15.00	2.70	0.86	0.001438
Level 2	18.00	3.24	0.75	0.001039

The strength level design earthquake force in the x direction controls, therefore its drift ratio was taken into consideration when comparing it to the allowable story drift. The following results show that the story displacements are all within the allowable displacement. The story drifts are very low compared with the allowable this is due to a stiff lateral resisting system.

Modal Period

The ETABS model's first period for the structure is at 0.4696 seconds. Compared to the calculated period based on ASCE 7-05 section 12.8.2.1 which is at .626 seconds; the ETABS model tells us that the building is stiffer than approximation by the ASCE code. This might be because the designer used the UBC code to design the building, or that the designer assumed a redundancy factor of 1.3 to be conservative when first starting the design, and therefore ended up with a stiff structure.

The following is a 3-D view of the first mode, which shows that the building is excited in the y direction.

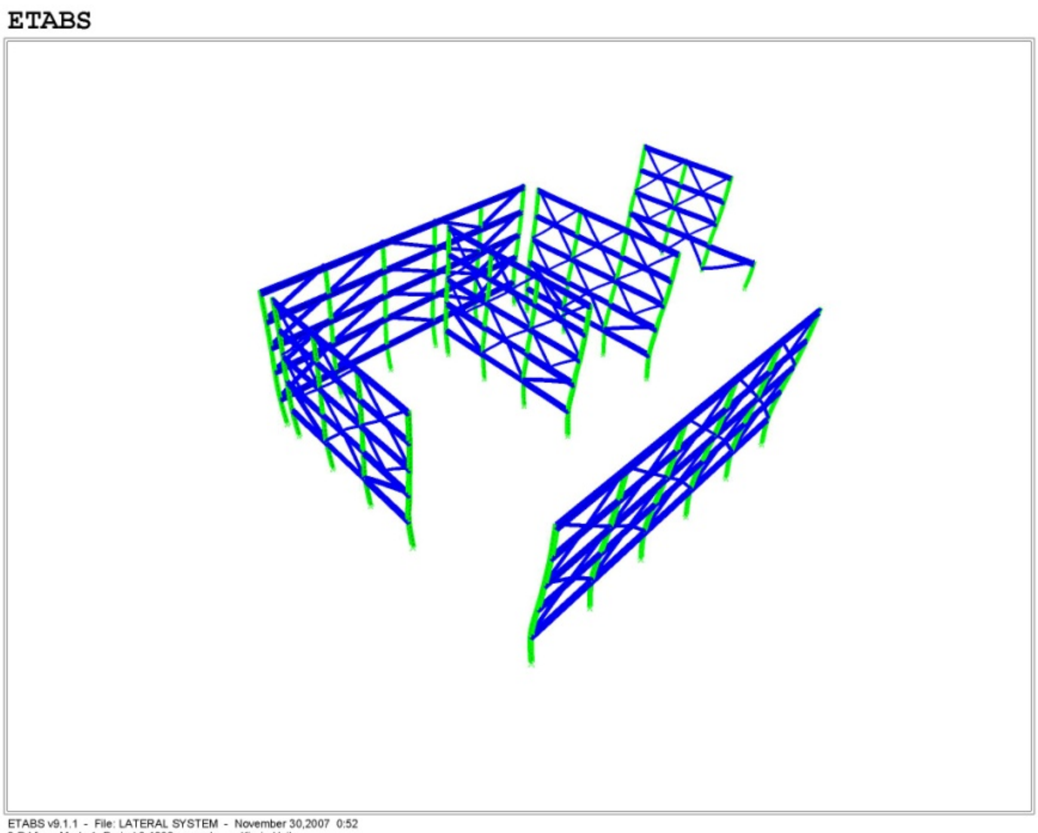

Fig11. 3-D Modal Period

Spot Checks of framing elements

Using AISC Steel Manual Table 4-1, the following Available Strength Axial Compression Capacities are computed to check for braces which are connected with the moments released in its local x and y.

Member	Type	KL	$\Phi_c P_n$	P_u	$P_u / \Phi_c P_n$
HSS10x10x5/8	Brace	23.4	603	343	.57
HSS8x8x5/8	Brace	21.2	420	296	.70
W14x109	Brace	33.5	617	584	.95
W14x132	Brace	33.5	757	700	.92
W14x211	Brace	35	1280	902	.70

The following checks for steel columns are done using the steel frame design in ETABS; a hand calculation was done to verify this answer. Refer to appendix for comparison. Please note that the load combinations for the column design are with over strength factor.

Member	Grid Location	Member Type	KL	Demand/Capacity Ratio
W14x145	B-3	Column	18’	.645
W14x311	B-7	Column	18’	.77
W14x398	C-2	Column	18’	.565

The following 3-D figure represents the Demand Capacity Ratio given by ETABS when doing the steel frame design using load combinations EQ1 to EQ16. Please note that those load combinations are for all beams and braces.

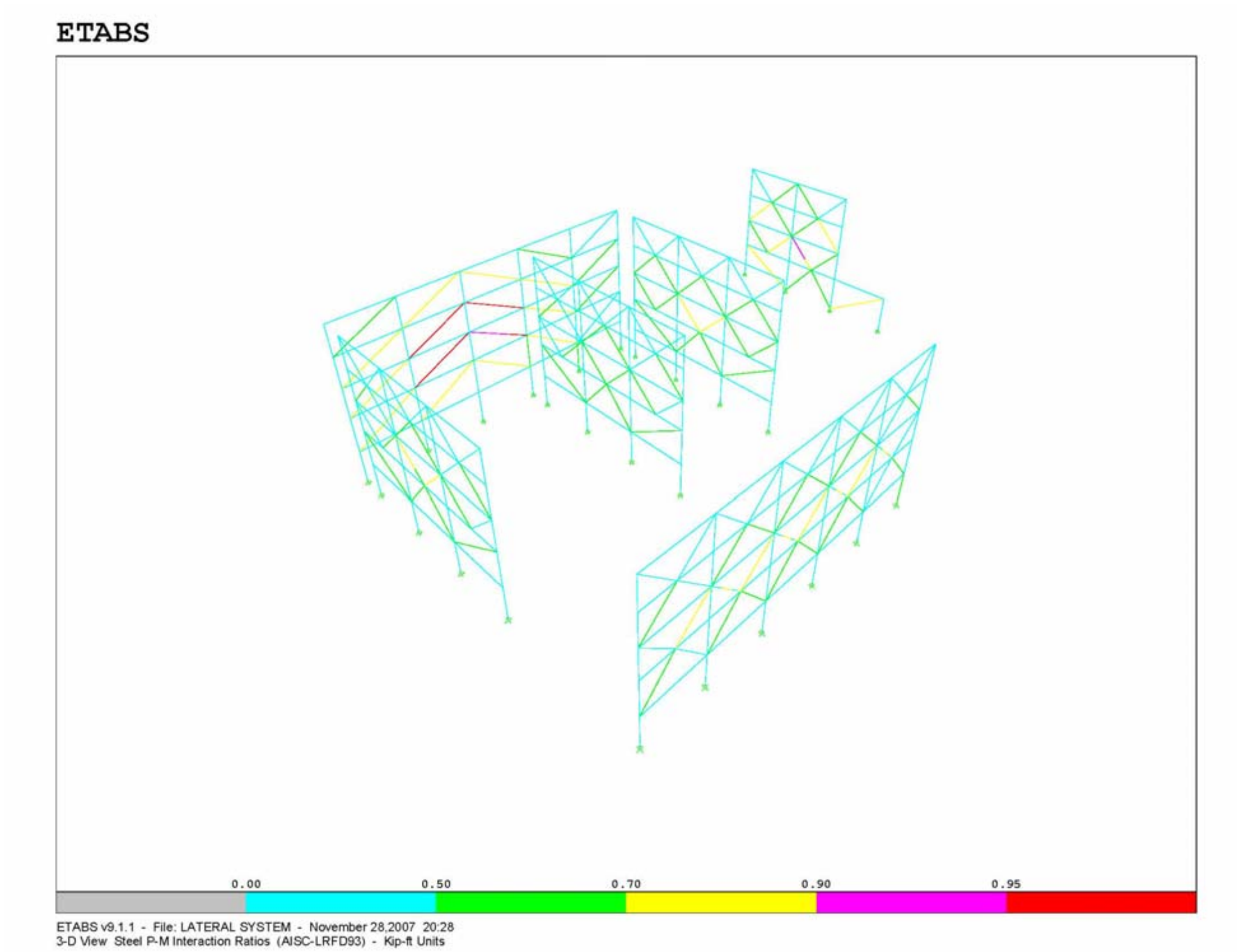

Fig12. 3-D Demand Capacity Ratio view.

The following figure represents the Demand Capacity Ratio given by ETABS when doing the steel frame design using load combinations EQ17 and EQ18. Please note that those load combinations are for all columns. Most columns are between .5 and .7 demand capacity ratio which means that the designer designed for a strong column with weak braces due to the over strength factor applied when designing columns.

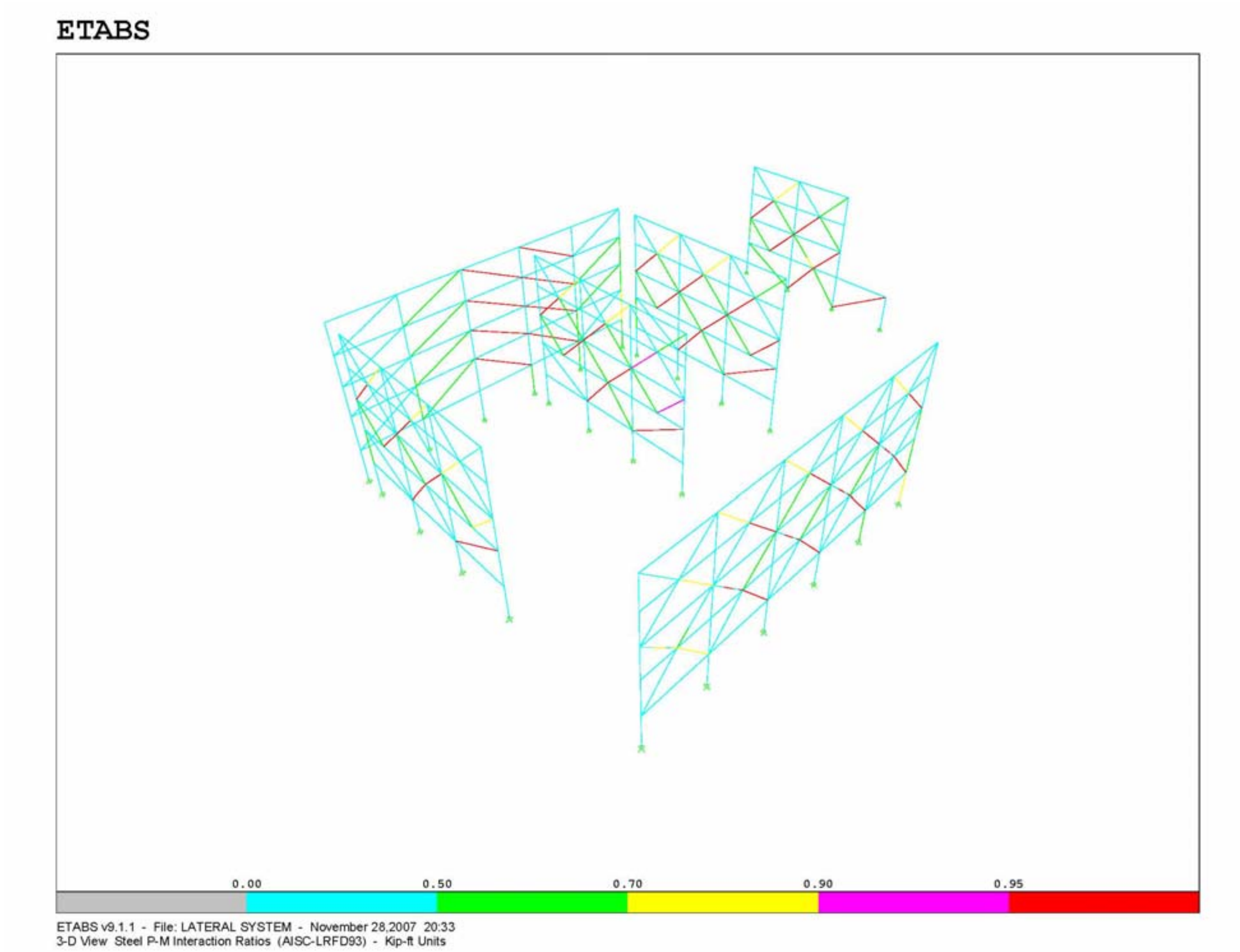

Fig13. 3-D Demand Capacity Ratio view.

Overturning Moment

The building's overall overturning is calculated by taking the shortest distance from the center of mass to the edge of the building in the closest direction and multiplying it with the weight of the whole structure. If the resultant is less than the overturning moment from seismic forces, then the building does not uplift. This is done in both the x and y directions, and the results show that no uplift will occur at the buildings edge due to overturning.

Although when overturning resisting moment is calculated based on the distance from the center of mass to the center of rigidity in the x and y direction overturning moment do control. Therefore uplift force will exist in the columns of the lateral resisting system. Please refer to Appendix for calculations.

When checking the ETABS model, some load combinations result in an uplift force on the edge columns of the braced frame. But when looking at the foundation of the lateral resisting system, the designer resolved this issue by implementing shear walls with a continuous footing at the basement level. Further checks can be done using SAFE to ensure that the overturning moment is addressed.

The following figure represents the axial forces in the members with the load combination EQ17. Please note when looking at uplift in the soil, a different load combinations would need to be used for allowable design, specified in ASCE 7-05 section 2.4.

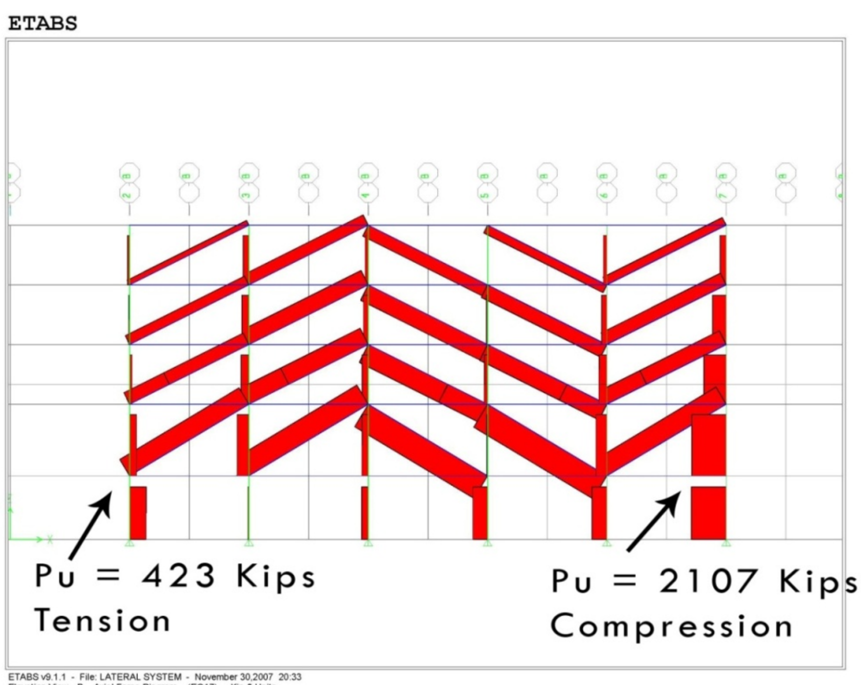

Fig14.Elevation of the Axial force diagram.

Conclusion

This report has provided a better understanding of the lateral resisting system and its behavior with seismic forces. Hand calculations were done to verify the accuracy of the ETABS model. Relative Stiffness, center of masses and center of rigidities were computed manually and compared. The ETABS model was then used to check the steel frames and compute demand capacity ratios, which were also verified with hand calculations.

This report revealed the importance of using a computer model to analyze and design the lateral system. The fast and effective analysis process compared to doing all hand calculation is a major advantage of using computer models. Computer models also generate 3-D animations of deflections and modal excitations of the structure which shows the buildings behavior and can be used to address torsional and deflections issues when checking the design.

Story drift is within the allowable limit according to the ASCE 7-05 code. The steel members are all within their capacities. It can be concluded that the steel braces have a higher demand capacity ratio compared to the columns therefore will buckle or yield before the columns. This tells us that the designer designed for a strong column weak brace concept due to the over strength factors specified by code. So that during a major earthquake the structure would yield the braces first and dissipate energy without causing major damage to the columns which may cause further catastrophic damages if failing.

The period of the structure in ETABS is way below the approximate period computed by ASCE, which tells us that the lateral resisting system is very stiff than expected. This may be because the structure was designed using the UBC code which may have taken different assumptions when computing seismic forces. The designer might have also used the redundancy factor of 1.3 when initiating design to be conservative. Further analysis would be required to determine if a more flexible structure can be used so that we increase drift but not pass the allowable limit while reduce the period hence reducing the seismic base shear coefficient.

When doing center of rigidity and center of mass calculation; the braced frames that resist lateral loads in the y direction, resist the loads equally hence have equal stiffness throughout. But when looking at the braced frames resisting the lateral loads in the x direction we get that one resist 66% and the other resist 33% of the total load. This unequal relative rigidity results in additional torsion to the structure, which would add torsional shear to the structure. This issue can be addressed by doing comparison to the architectural drawings first; seeing if it would possible to use HSS X braces instead of W Shapes or reduce the sizes of the W shapes hence reduce stiffness.

Appendix

Dead load Calculations

Lightweight Concrete (3.25" top, 3" Deck)	44 psf
Normal Weight Concrete (6" top, 3" Deck)	94 psf
Normal Weight Concrete (4.5" top, 3" Deck)	75 psf
Steel Deck (18 Gage)	3 psf
Glazing	12 psf
Partitions	20 psf
Metal Panels	14 sf
Precast Panels	68 psf
Super Imposed:	
Indoors	12 psf
Roof	25 psf
Courtyard Planter	350 psf*
Courtyard Tree+Planter	350 psf*

*Designer's value used instead.

Wind Calculations

Wind Load Calculation Spread Sheet

Locality Factors and Building Description

Basic Wind Speed	V	85	*h*	75	*Bldg. Height*
Wind Directionality Factor	Kd	0.85	*N-S*		
Importance Factor	I	1.15	*B*	285	
Windward Wall	Cp	0.8	*L*	210	
Leeward Wall	Cp	-0.5	*h/L*	0.3571	
Side Wall	Cp	-0.7	*E-W*		
Topographic Factor	Kzt	1	*B*	210	
Period	T	.47	*L*	285	
Internal pressure coeff.	GCpi ±	0.18	*h/L*	0.2632	

B Horiz. Dim. normal to wind dir.

L Parallel Dim. normal to wind dir.

Gust Effect Factor - Flexible Structure

Table 6-2

Exposure C	alpha	9.5		
	Zg	900		
	â	0.10526		
	Bhat	1		
	alpha line	0.15385		
	b line	0.65		
	c	0.2		
	l	500		
	Є line	0.2		
	Zmin	15	45	or .6h *Calc. assumes .6h always controls

Gust Effect Factor Calculations

Vz	84.99366595
Lz	531.9976564
n1	1
N1	6.259262387

	N-S			E-W			
Rh	0.21602	n	4.059126	Rh	0.216	n	4.0591
RB	0.06273	n	15.42468	RB	0.0841	n	11.366
RL	0.02594	n	38.04989	RL	0.0192	n	51.639
Beta	0.01						
Rn	0.043611365						

R	0.179002936	R	0.206672765
gr	4.189475724		
Iz	0.189924168		
Q	0.818520081	Q	0.837655817
gq &gv	3.4		
Gf	0.851287088	**Gf**	0.864740748

Level	Elevation	Trib Height	Height	Kz	qz	qh
Mech Room	238.5	6	75	1.19	21.54	21.54
Roof	226.5	13.5	63	1.15	20.76	21.54
Level 4	211.5	15	48	1.08	19.61	21.54
Level 3	196.5	15	33	1.00	18.12	21.54
Level 2	181.5	16.5	18	0.88	15.95	21.54

For wind pressures, forces, story shears, and overturning moments. See page 14.

Seismic Calculations

Vertical Distribution of Seismic Forces

Floor	hx (ft)	W (kips)	hxkWx	Cvx	Fx (kips)	Vx (kips)	Mx (ft-K)
Roof	63.00	5393.40	1896398	0.51	2407.26		151657.45
Level 4	48.00	3566.00	853371	0.23	1083.26	2407.26	51996.32
Level 3	33.00	3566.00	502202	0.13	637.49	3490.52	21037.12
Level 2	18.00	8089.20	483188	0.13	613.35	4128.01	11040.34
Level 1	0.00	6466.87	0	0.00	0.00	4741.36	0.00
	Total	**20615**	**3735159**	**Base Shear**	**4741**	**Overturning Moment**	**235731**

k	1.415	*I*	1.5
Cs	0.23	*R*	6
*V = Cs*W*	**4741.3**	*Sds*	0.92
		Sd1	0.497
		CuTa	0.626
		T	0.4696
		TL	8
		Cs	0.23

Story Drift

Story	hx (ft)	Allowable Drift (in)	Story Drift (in)	Story Drift Ratio (in/in)
Roof	15.00	2.70	0.73	0.001209
Level 4	15.00	2.70	0.87	0.001444
Level 3	15.00	2.70	0.86	0.001438
Level 2	18.00	3.24	0.75	0.001039

Importance Factor	1.5
Cd	5

Buildings Weight and Mass

Floor	W (kips)	Area (ft^2)	W (psf)	Mass/ft^2
Roof	5393.40	39400.00	136.888	4.25
Level 4	3566.00	39400.00	90.508	2.81
Level 3	3566.00	39400.00	90.508	2.81
Level 2	8089.20	53438.00	151.375	4.70
Level 1	6466.87	53438.00	121.016	3.76

Building Weight Calculations

Weight of Building

Floor	Component	Weight (psf)	Area	Weight (Kips)
		LVL 1		
	Beams			231
	Girders			168
	Columns			132
	Curtain Wall			218
	Composite Deck	75	53438	4007.85
	Paritions	20	53438	1068.76
	Super Imposed	12	53438	641.256
			Total Weight	6467
		LVL 2		
	Beams			231
	Girders			168
	Columns			132
	Curtain Wall			218
	Composite Deck	47	37000	1739
	Paritions	20	37000	740
	Super Imposed	12	37000	444
Courtyard	*Composite Deck*	94	7200	676.8
	Super Imposed	22	6500	143
	SI Tree	552	110	60.72
	SI Planters	342	271	92.682
Lvl 2 Roof	*Composite Deck*	47	12000	564
	Super Imposed	222	12000	2664
	Topping	18	12000	216
			Total Weight	8089
		LVL 3		
	Beams			151
	Girders			94
	Columns			180
	Curtain Wall			218
	Composite Deck	47	37000	1739
	Paritions	20	37000	740

	Super Imposed	12	37000	444
			Total Weight	3566
LVL 4				
	Beams			151
	Girders			94
	Columns			180
	Curtain Wall			218
	Composite Deck	47	37000	1739
	Paritions	20	37000	740
	Super Imposed	12	37000	444
			Total Weight	3566
ROOF				
	Beams			234
	Girders			116
	Composite Deck	97	39000	3783
	Super Imposed	25	39000	975
	AHU	25	7200	180
				0
Pent House	*Pent House*	170	620	105.4
			Total Weight	5393

Please note that additional weight calculations are available upon request.

ETABS Modeling

Relative Stiffness Calculations

These calculations are for comparison with the ETABS model, to verify that the relative stiffness calculated match the ETABS model.

ETABS output for earthquake load in the X, states that BF-1 takes 66% while BF-2 takes 33% of the total shear @ Level 1.

BF-1 @ Level 1 Consists of:

- (5) W14x211 Braces (A = 62 in^2)
- Length of Brace = 35'
- At an angle of 31 degrees from the floor

BF-2 @ Level 1 Consist of:

- (10) HSS 10x10x5/8 (A = 21 in^2)
- Length of Brace 23.4'
- At an angle of 50.2 degree from the floor

Note that stiffness that is contributed from the columns is neglected.

Using $K = \sum \frac{AE}{L}$

For BF-1

(5 Braces)[62 $(\cos(30.96)^2)$]/35 = 6.5, therefore 6.5/(3.67+6.5) = **64%** relative stiffness

For BF-2

(10 Braces)[21 $(\cos(50.2)^2)$]/23.4 = 3.67, therefore 3.67/(3.67+6.5) = **36%** relative stiffness

The relative stiffness percentages are close to the ETABS output, therefore it is safe to use the ETABS output relative stiffness to compute the center of rigidity.

Center of Rigidity Calculations

Relative rigidity is to be taken as the shear force resisted by the brace frame and is proportional to the stiffness.

Center of Rigidity @ Level 2

Frame	Distance from Zero Reference		Relative Rigidity		(Rx)y	(Ry)x
	E-W (x) (ft)	N-S (y) (ft)	Rx	Ry		
BF 1	0.00	169.00	3154.00	0.00	533026.00	0.00
BF 2	0.00	0.00	1572.00	0.00	0.00	0.00
BF 3	30.00	0.00	0.00	1225.00	0.00	36750.00
BF 4	120.00	0.00	0.00	1178.00	0.00	141360.00
BF 5	180.00	0.00	0.00	1147.00	0.00	206460.00
BF 6	270.00	0.00	0.00	1177.00	0.00	317790.00
Total			**4726.00**	**4727.00**	**533026.00**	**702360.00**

Yr	112.76	ft
Xr	148.62	ft

Center of Mass Calculations

Center of Mass @ Level 2

Element	Length	Width	Unit Weight (k/sf)	Weight (kips)	Dist. from zero reference		Wx (ft-k)	Wy (ft-k)
					x (ft)	y (ft)		
Area A	270	198	0.151	8072	135	99	1089782	799174
Area B	29	30	-0.151	-131	15	184	-1971	-24106
Area C	29	30	-0.151	-131	175	184	-22990	-24106
Total				**7809.72**			**1064822**	**750961**

xm =	136.35	ft
ym =	96.16	ft

Comparison with ETABS output

The following results are approximately equal to the computed center of mass and center of rigidity.

Story	Diaphragm	XCCM	YCCM	XCR	YCR
LEVEL1	D1	136.963	97.574		
LEVEL2	**D2**	**136.497**	**96.144**	**149.156**	**105.116**
LEVEL3	D3	141.226	98.754	144.822	92.595
LEVEL4	D4	141.226	98.754	142.841	89.972
ROOF	DROOF	141.226	98.754	141.636	86.933

Assigned Building Mass

The following table shows the assigned building mass to the null shell property in the ETABS model.

Assigned Buildings Mass

Floor	W (kips)	Area (ft^2)	W (psf)	Mass/ft^2
Roof	5393	33656	160	4.98
Level 4	3566	33656	106	3.29
Level 3	3566	33656	106	3.29
Level 2	8089	51708	156	4.86
Level 1	6467	52578	123	3.82

Steel Frame Member Spot Checking

The following represent the ETABS steel frame spot checking, this will be compared with hand calculations to ensure correctness.

ETABS Steel Design

Engineer__________
Project__________
Subject__________

```
AISC-LRFD93 STEEL SECTION CHECK    Units: Kip-ft  (Summary for Combo and Station)
Level: LEVEL2  Element: C27  Station Loc: 0.000  Section ID: W14X311
Element Type:  Moment Resisting Frame   Classification:  Compact

L=18.000
A=0.635  i22=0.078  i33=0.209  z22=0.176  z33=0.349
s22=0.115  s33=0.293  r22=0.350  r33=0.574
E=4176000.000  fy=7200.000
RLLF=1.000

P-M33-M22 Demand/Capacity Ratio is   0.768 = 0.617 + 0.137 + 0.014

STRESS CHECK FORCES & MOMENTS
                         P          M33          M22          V2          V3
    Combo    EQ17  -2080.111     348.143      17.385      34.876       1.194

AXIAL FORCE & BIAXIAL MOMENT DESIGN    (H1-1a)
                        Pu      phi*Pnc      phi*Pnt
                      Load     Strength     Strength
    Axial         2080.111     3371.611     4113.000

                        Mu      phi*Mn       Cm       B1       B2        K        L       Cb
                    Moment    Capacity   Factor   Factor   Factor   Factor   Factor   Factor
    Major Bending  348.143    2261.250    0.850    1.000    1.000    1.328    0.855    2.182
    Minor Bending   17.385    1118.056    0.850    1.005    1.000    1.000    0.855

SHEAR DESIGN
                        Vu      Phi*Vn       Stress
                     Force    Strength        Ratio
    Major Shear     34.876     650.997        0.054
    Minor Shear      1.194    1647.540        0.001
```

ETABS v9.1.1 - File:LATERAL SYSTEM - Kip-ft Units   November 30,2007 14:53

Steel Frame Hand Calculation

Column Located at grid B-7.
W14x311, L = 18'

ETABS analysis states that the load combination EQ17 Control. With the following loading:
P = 2080 kips
M_x = 348 ft-k
M_y = 17 ft-f

From AISC Steel Manual Table 6-1
p = .295 x E-3
b_x = .398 x E-3
b_y = .70 x E-3

From AISC Steel Manual Table 4-1
$\Phi_c P_n$ = 3390 kips

Since Pr/Pc = 2080/3390 = .61 > .2 therefore use equation H1-1a.

$pP_r + b_x M_x + b_y M_y < 1.0$

(.295e-3)(2080) + (.398e-3)(348) + (.70e-3)(17) = .76, approximately equal to ETABS which is .77, therefore ok to use steel design analysis in ETABS.

Over Turning Moment

OTM from seismic forces = 235,731 ft-k
Building Weight = 20,615 Kips
Center of Mass in the X direction = 136' from (0,y)
Center of Mass in the Y direction = 96' from (x,0)
Center of Rigidity in the X direction = 149' from (0,y)
Center of Rigidity in the Y direction = 105' from (x,0)
Bx = 270'
By = 198'

Checking Overturning in the X Direction @ shortest distance to Edge of Building

235,731 < (270-136)(20,615) = 2,762,410 ft-k, Therefore OK

Checking Overturning in the Y Direction @ shortest distance to Edge of Building

235,731 < (96)(20,615) = 1,979,040 ft-k, Therefore OK

Therefore no uplift occurs at the building edge.

Checking Overturning in the X Direction @ center of rigidity

235,731 < (149-136)(20,615) = 267,995 ft-k, Therefore OK

Checking Overturning in the Y Direction @ center of rigidity

235,731 > (105-96)(20,615) = 185,535 ft-k, Therefore not OK

The overturning moment will cause some columns to uplift at the lateral resisting system.