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555 12TH STREET
OAKLAND, CALIFORNIA

Technical Report #3:
Lateral System Analysis

Executive Summary

555 12th Street is a 21 story, 487,000 square foot complex that features Class-A office space, retail space, and dining in one covenant location. Located in the heart of downtown Oakland, California, the building provides great views of the San Francisco Bay, as well as the East Bay Hills.

The purpose of this report is to analyze the lateral force resisting system of the building. There is a dual system acting in both directions of the building, comprised of eccentrically braced frames at the core, and special moment resisting frames on the perimeter. The composite decking acts as a rigid diaphragm to transfer loads to braces.

Seismic and wind forces were refined from technical report one using ASCE7-02, to come up with new base shears. Lateral loads were distributed to each story based on relative stiffnesses, calculated using RAM Advanse 6.0. They were also compared to the IBC code requirement that 25% of lateral loads in a dual system, must be distributed to the moment frames. Overturning moment from wind and seismic were calculated in both directions as well from story shears. Spot checks were performed to verify correct load distribution.

Ram Advanse yielded about the same distribution in each EBF frame, which supports the typicality of each frame compared to the other. However, suspicious percent distributions to the frames near the base arose because of the analysis method. The IBC required ratio was then used instead of these numbers for spot checks. Members were found to be slightly smaller than those designed for the building. Overturning was checked on one EBF frame, and it passed. The moment frames in the North South direction appear to be controlled by drift, not strength.

An ETABS model was created, but not able to be used in this report. It will be helpful in the future redesign and determination of overall building drift.

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INTRODUCTION

555 12th Street is a 21 story, 487,000 square foot complex that features Class-A office space, retail space, and dining in one covenant location. Located in the heart of downtown Oakland, California, the building provides great views of the San Francisco Bay, as well as the East Bay Hills. Completed in May 2002, it is one of several buildings that make up what is known as the Oakland City Center. It was designed using the Uniform Building Code, but I have analyzed it with the International Building Code 2003 with references to ASCE7-02.

EXISTING GRAVITY SYSTEM

The gravity force resisting system for all elevated floors is comprised of composite slab on metal deck and steel beams. These beams/girders then transfer load to steel columns, to concrete piers, to the mat foundation. The ground floor and first underground stories are one way concrete slab on precast concrete beams and columns.

Typical floors 3-21 are 3" 18 gage composite deck with 2 ½" of normal weight concrete cover. The slabs are reinforced by either #6 @ 13" EW or WWF6x6 W1.9. The majority of the structural system is steel framing. All structural framing steel is designated as ASTM A992, Gr 50, unless otherwise noted. The building takes advantage of two lines of symmetry, one in the N-S direction, and the other in the E-W direction. The typical floors, 2-21, have the same framing, unless otherwise noted.

On a typical floor, beams are sized as W18x55 and W18x60 and span 43'-6". The girders are sized W18 – W27, and span 30 – 35', depending on location in the bay analyzed. The overall max depth of the floor system is 26.7" for a W27, plus an additional 5.5" for the composite deck, for a total of 32.2".

EXISTING LATERAL SYSTEM

The lateral system of 555 City Center is considered a dual system in the N/S and E/W directions. Dual systems are systems with shear walls and/or braced frames and moment frames working in parallel to resist lateral forces. The building has a steel braced frame core and Special Moment Resistant Frames (SMRF) at the perimeter. From the basement to the 2nd floor, a concrete shear wall core was utilized to help stiffen the structure at the first floor, which has a high floor-to-floor height of 24 feet. A steel braced frame was used from level 2 through the roof. The steel braced frame "jamb" columns extended into the concrete shear wall. A more detailed description of each component of the lateral system is provided below.

• **Eccentric Braced Frames (EBF)**

North-South direction

These frames are fairly typical, and run from just below the first floor, all the way up to the roof. They occupy one bay width, 31’-4”, from B.8-D.2, and there are six of them. The following frames are similar; EBF 1 and 6, EBF 2 and 5, and EBF 3 and 4. The heaviest column members are located at the bottom, and are a robust W14x665. They progressively get smaller as they reach the roof, where they have fell to W14x106 or W12x159, depending on the gridline. The beams spanning the brace also depend on which EBF it is, but range from W18x71 to W21x122, from top to bottom. Lastly is the knee bracing, which makes it an eccentric braced frame. These members form an upside down trapezoid with the columns and beams. Their sizes range from W10x88 up to W14x159 at the bottom. On all EBF’s, a distance of 3’ in the middle creates the eccentricity. This 3 foot section allows for energy absorption due to cyclical loading from lateral forces. Refer to appendix for drawings of the frames.

East-West Direction

There are four of these frames in the E-W direction. They are all similar, and use the same sized members. Two of them are located between 3.3 and 4.9 on gridlines B8 and D2, and the other two are located between 6.1 and 7.8 on gridlines B8 and D2. Columns for these frames are shared with the EBF’s in the N-S direction. Beam sizes range from W16x57 to W18x97, and brace sizes range from W8x58 to W14x159. These braces form right-side up trapezoids between columns, the opposite as the N-S. The collector portions of the frames are 2’-6” and 4’ and allow for energy absorption from cyclical loading. This eccentricity also allows for doorway and elevator openings in the walls.

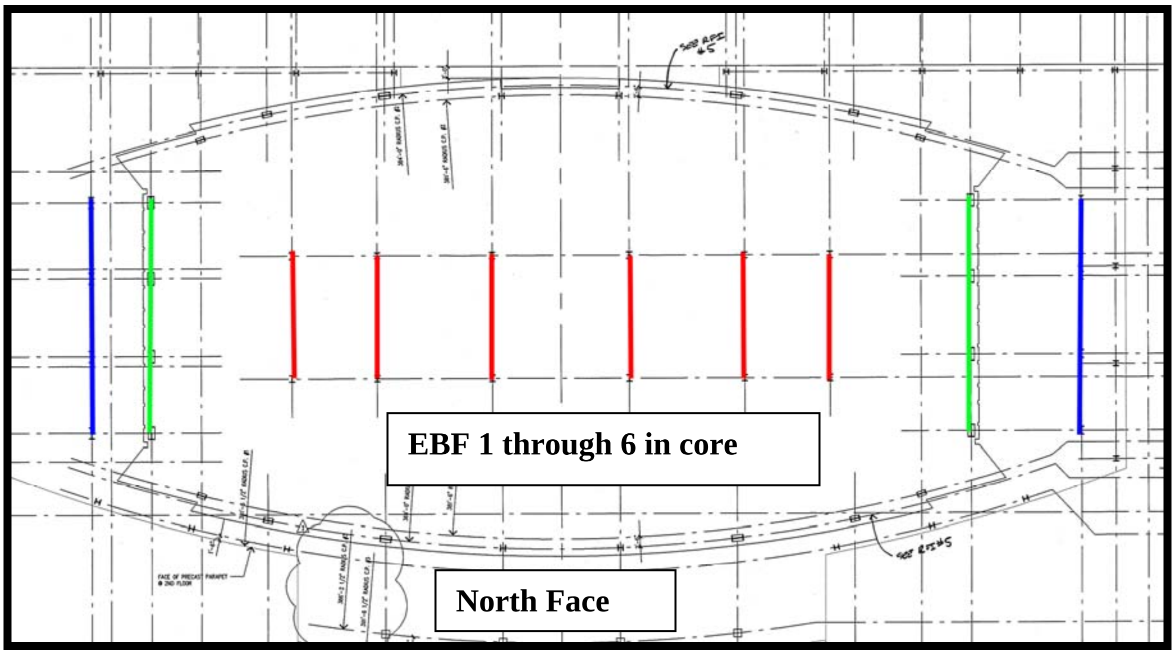
• **Special Moment Resisting Frames (SMRF)**

Moment frames have good ductility and are more flexible than braced frames. All connections within the frames are moment connections. These frames are located on the perimeter walls of the building. Four of these frames are located on the curved portions of the North and South faces, and the other four are on the East and West faces. Two on the E-W faces only go from the first to second floor, as represented by the larger first floor footprint, compared to the upper levels. The other two go all the way to the roof. These frames use only W shapes for beams and columns. Beams for the N-S faces are W24’s and the columns range from W24-W33. On the E-W faces, for the frames that reach the roof, there beams are W33’s and columns are sized W36’s.

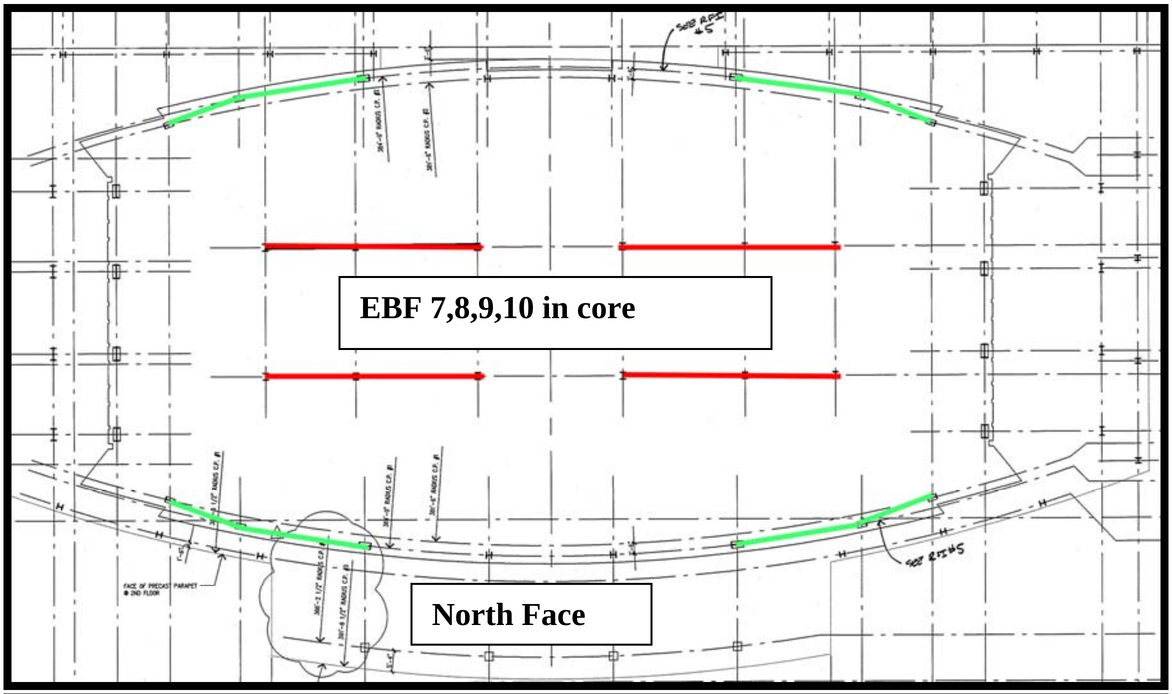
• **Shear Walls**

The shear walls provide stiffness to the eccentric frames of the core. The shear walls are located directly under the EBF frames, and occupy the same gridlines. They run from the mat foundation up to the second floor, where they meet the beams of the frames. They are typically 24” thick and reinforced with #6 @ 12” each face each way, unless otherwise noted. 25” thick walls exist on the grid lines D2 and B8. All core shear walls are required to have a f’c = 5000 psi

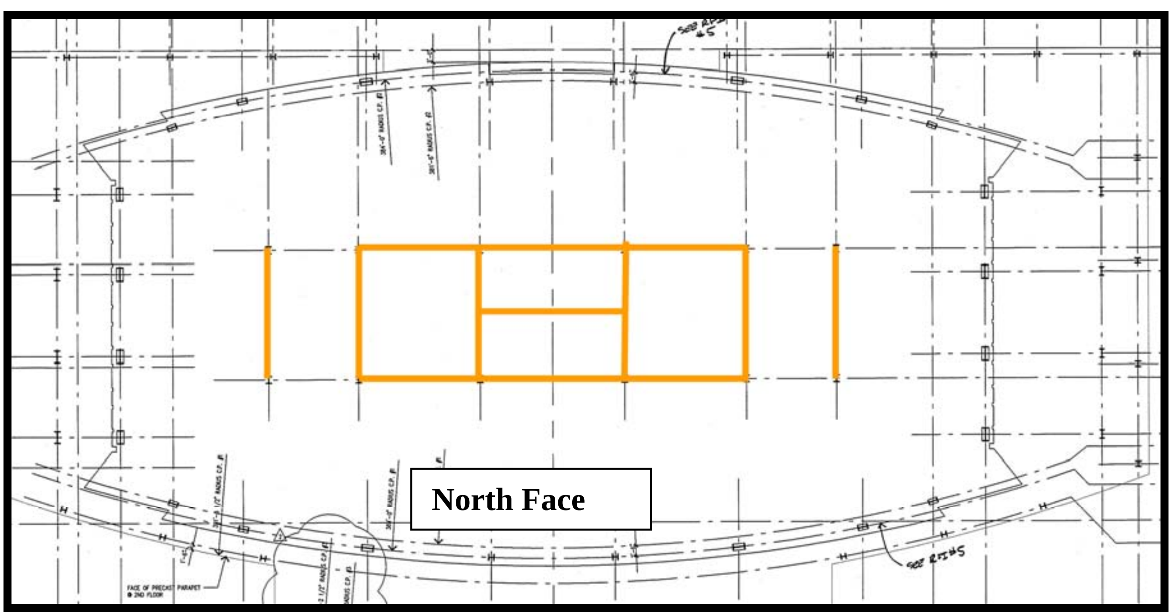
North-South Lateral System: Red = EBF in Core , Green = SMRF on Perimeter, Blue = SMRF(1 story)



East-West Lateral System: EBF in Core, SMRF on Perimeter



Orange = Shear Walls in N/S and E/W – From foundation to 2nd floor



GRAVITY LOADS

Live Loads: Taken from table 1607.1 from IBC 2003

- Office Floor: 80 psf – Assume any spot could be a corridor at some point
- Partitions: 20 psf – Assume 10 psf for seismic calculations

Dead Loads: (Assumed)

- Metal Deck: 2 psf
- Reinforced Concrete: 150 pcf (includes reinforcing)
- Steel Structural Members: 15 psf
- MEP: 10 psf
- Collateral: 5 psf

LATERAL LOADS

To evaluate the lateral forces on the building, I reused ASCE7-02, like in Technical Report 1. This time, corrections were made to the previous analysis to determine more accurate loads. Among them were Importance factors, spectral response coefficients, and building dimensions. The building was changed from a category III to a category II which changed the importance factor from I = 1.15 to a value of 1.0. The basis for the change was from an addendum to the IBC which stated that in order for a building to be a category III, the main space of congregation, not the entire building, must have greater than 300 people. This does not occur on my building.

WIND LOADS

Loads were calculated in both the North/South and East/West directions. 555 12th Street is an irregular building, so several assumptions were made in determining base shear:

Assumptions:

- Assume building does not have curved façade, that is is rectangular.
- Assume no canopy and 1st floor is same footprint as all floors
- Height is 306 feet, North and South face = 227’, East and West face = 125’
- Parapet at roof was ignored, and made into the top of the roof @ 306’

Velocity Pressure, q_z was calculated = $0.00256 * k_z * k_{zt} * k_d * (V^2) * I$

Final Pressure, P was calculated = $q_z(GC_p)-q_i(GC_{pi})$

Because the building is in Oakland, high seismic region, wind will not govern.

General Building Information		
Building:	555 12th Street	Reference
Location:	Oakland, CA	
Basic Wind Speed(mph): V	85	Fig. 6-1
Exposure Category:	C	6.5.6.3
Enclosure Class	Enclosed	Sect. 6.2
Building Category	II	Table 1-1
Importance Factor: Iw	1.0	Table 6-1
Topography Factor: Kzt	1	Sect. 6.5.7
Directionality Factor: Kd	0.85	Table 6-4
Internal Pressure Coefficient: Gcpi	0.18	Fig. 6-5
Gust Factor- assume rigid G	0.85	6.5.8.1
Building Height: h (feet)	306	
Length Parallel to wind: L (feet)	227'	
Length Perpendicular to wind: B (feet)	125'	

	L/B	L/B
	1.9	0.5
External Pressure (Cp)	E-W	N-S
Windward	0.8	0.8
Leeward	-0.3	-0.5

Results			North-South Wind Loading			East-West Wind Loading		
Height	Kz	qz	P leeward	P windward	P total	P leeward	P windward	P total
0-15	0.85	13.4	-15.1	13.6	**28.7**	-10.9	13.6	**24.5**
20	0.9	14.1	-15.1	14.1	**29.2**	-10.9	14.1	**25.0**
25	0.94	14.8	-15.1	14.5	**29.6**	-10.9	14.5	**25.4**
30	0.98	15.4	-15.1	15.0	**30.1**	-10.9	15.0	**25.9**
40	1.04	16.4	-15.1	15.6	**30.7**	-10.9	15.6	**26.5**
50	1.09	17.1	-15.1	16.2	**31.3**	-10.9	16.2	**27.1**
60	1.13	17.8	-15.1	16.6	**31.7**	-10.9	16.6	**27.5**
70	1.17	18.4	-15.1	17.0	**32.1**	-10.9	17.0	**27.9**
80	1.21	19.0	-15.1	17.4	**32.5**	-10.9	17.4	**28.3**
90	1.24	19.5	-15.1	17.8	**32.9**	-10.9	17.8	**28.7**
100	1.26	19.8	-15.1	18.0	**33.1**	-10.9	18.0	**28.9**
120	1.31	20.6	-15.1	18.5	**33.6**	-10.9	18.5	**29.4**
140	1.36	21.4	-15.1	19.0	**34.1**	-10.9	19.0	**29.9**
160	1.39	21.9	-15.1	19.4	**34.5**	-10.9	19.4	**30.3**
180	1.43	22.5	-15.1	19.8	**34.9**	-10.9	19.8	**30.7**
200	1.46	23.0	-15.1	20.1	**35.2**	-10.9	20.1	**31.0**
250	1.53	24.1	-15.1	20.9	**36.0**	-10.9	20.9	**31.8**
300	1.59	25.0	-15.1	21.5	**36.6**	-10.9	21.5	**32.4**
306	1.59	25.0	-15.1	21.5	**36.6**	-10.9	21.5	**32.4**

From the total pressures, shear at story heights, base shear, and overturning moment were found.

East/West: Base shear = V = 1105 kips
Overturning Moment = 182,196 ft-kips
North/South: Base Shear = V = 2286 kips
Overturning Moment = 375,334 ft-kips

See Appendix for Calculations

SEISMIC LOADS

The site of the building is in a high seismic area, located directly near a major fault line on the west coast. Looking at the ASCE tables for long and short period response, it was hard to determine exactly what S_s and S_l were. To determine a more accurate response values, I consulted the USGS website with an exact latitude and longitude of the building. After talking to the structural engineer it was found that a site specific spectral analysis was performed. However, this data has yet to be acquired. Corrections to loads will be made in future reports if found to be different. For simplicity, the building was assumed to be rectangular, like in the wind calculations. A vertical redistribution of forces was done with the assumed weight of the structure, and a base shear and overturning moment were calculated. It was assumed that the same type systems acted in both N/S and E/W directions.

Building Information: Latitude: 37.804603, Longitude: -122.275486

S_s = 240.95% or 0.24095
S_l = 94.05% or 0.9405
Site Class: C (from structural drawings)
Building: Office Category II
Seismic Use Group: I
Importance Factor: I = 1.0
S_{DS} = 1.6063
S_{DL} = 0.815
Seismic Design Category: D
Response Modification Factor: R=8 for dual system
Cs = 0.0707
K = 1.48
T = 2.05 s

Total Load (W) = 56888 kips
Seismic Base Shear (V) = 4022 kips
Overturning Moment = 857,937 ft-kips

See Appendix for Calculations

Distribution of Lateral Forces

Lateral loads developed on the structure from both wind and seismic forces act at each story level. They are transferred from the building perimeter to the building core through the rigid diaphragm action of the composite slab, deck, and steel members. According to IBC code, moment frames are required to take at least 25% of the lateral loads in a given direction, and the rest be distributed to the braced frames of the core. However, a two dimensional analysis was used to determine to actual percentage taken by the frames.

RAM Advanse 6.0 was used to determine the distribution of lateral forces to each frame, in the north-south and east-west directions. Each frame was assembled with the as-built beams, braces and columns. After finished building, a 100 kip point load was placed the top joint of each frame at the top story. This load is an arbitrary load, and could be any value as long as it is the same for every frame.

An analysis was run to determine the deflection from this load at each story level for each frame. For each story, the relative stiffness of each frame was determined by the equation k = 1 / deflection. This number was then divided by the sum of all 'k' values acting in that direction, to give us the percent distribution of lateral forces at each story.

The table below presents the average percent distribution to each frame from every story. On the EBF frames, the beam members decreased in size as story level increased. However, on the SMRF 1 and 2, the beam members actually become larger as story level increases. The frames resisting load in the north-south direction took about the same percent load per floor, but those in the east-west direction varied with story. The EBF frames took an average of 7% per frame at the 2nd story, and 16% at the top story. The SMRF frames took 18% per frame at the 2nd story and dropped to 9% at the top story.

This analysis is effective for finding the percent distributed at the top floor, but not good for determining the bottom floors. The SMRF most likely do not take percent of load as indicated at the base. To get a more accurate stiffness, loads could be placed at the story level in question, not taken from the top floor. For now, I will assume that there is a 25/75 split throughout the building. Spot checks will performed under theses assumptions. Future reports will yield more accurate modeling.

These tables provides relative stiffness' for the top and bottom stories of each frame, and then a percent distribution for these floors and an overall average for each frame.

East – West

K values (1 / Deflection)

STORY	EBF 7	EBF 8	EBF 9	EBF 10	SMRF 3	SMRF 4	SMRF 5	SMRF 6	TOTAL
21	0.19	0.19	0.19	0.19	0.10	0.10	0.10	0.10	1.19
2	0.32	0.32	0.32	0.32	0.75	0.75	0.75	0.75	4.25

Percent Distribution (%)

STORY	EBF 7	EBF 8	EBF 9	EBF 10	SMRF 3	SMRF 4	SMRF 5	SMRF 6	TOTAL
21	16	16	16	16	9	9	9	9	100
2	7	7	7	7	18	18	18	18	100
Avg.	13	13	13	13	12	12	12	12	100

North – South

K values (1 / Deflection)

STORY	EBF 1	EBF 2	EBF 3	EBF 4	SMRF 5	SMRF 6	SMRF 1	SMRF 2	TOTAL
21	0.16	0.13	0.15	0.15	0.13	0.16	0.46	0.46	1.81
2	0.60	0.40	0.58	0.58	0.40	0.60	2.78	2.78	8.71

Percent Distribution (%)

STORY	EBF 1	EBF 2	EBF 3	EBF 4	SMRF 5	SMRF 6	SMRF 1	SMRF 2	TOTAL
21	9	7	9	9	7	9	26	26	100
2	7	5	7	7	5	7	32	32	100
Avg.	9	7	8	8	7	9	26	26	100

If ASCE requirement is used for dual systems, the distribution to each EBF in the North-South, will be 75 percent of the total base shear, divided by 6 frames, yielding 12.5% for each. Moment frames in this direction will take 25% / 2 frames, yielding 12.5% as well.

In the East-West direction, the same applies, but there are 4 similar EBF's and 4 similar SMRF's. Each EBF will get 18.75% of the lateral load, and each SMRF will receive 6.25% of the load.

TORSION and DRIFT

555 12th street has two lines of symmetry running the N/S and E/W directions. This creates 4 typical sections of the building which are composed of the same sized members. Because of this symmetry, the center of mass and center of stiffness are located in the geometric center of the building, or very close to. For simplicity Torsional effects on the structure will be ignored for this report. Review lateral system layout and floor plans in appendix for verification of symmetry.

Drift will be calculated when the correct distribution of loads is determined. The ETABS model did not provide useful information as to story drift at this time. It will be used to compare story drift to the allowable, when errors are fixed.

ETABS RESULTS

An ETABS model was set up to determine drifts for the entire building. At first, the model was designed with every beam, and floor system present. However, it was determined that a more simplified model could be used for this technical assignment. Only the braced frames and special moment frames were designed. A rigid diaphragm was used to connect the frames, and allow them to act as one system. This simplified model allows for easier application of wind and seismic loads, and less errors when the analysis is run. A picture of the model can be found in the appendix.

Only seismic was checked, as it will be the governing lateral forces. The assumption that the Special Moment frames take 25% of the load and the braced frames take the rest was used to distribute the forces on each story.

The model constructed did not run properly several times. Results could not be printed. The model will be saved and worked on to correct errors that occurred for next semester.

Load Cases:

The following load cases as obtained from ASCE7-02 chapter 2 were used in the Analysis.

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
8.) 1.0D + 1.0L + 1.0E

SPOT CHECKS

Spot checks were performed on the following members:

1. EBF column
2. EBF brace
3. SMRF column
4. Overturning of EBF frame/column

All calculations can be found in the Appendix.

Members calculated were found to be smaller than designed. Reasons would be incorrect distribution of forces, and not accounting for combined axial and shear effects on the columns. Overturning was not an issue.

CONCLUSIONS

Design seismic and wind forces were refined and calculated over again, and appear to be near design loads. Seismic base shear still governed the design of the building over wind, even after it was reduced from 10000 kip from tech one, to 4022 kips in this report. The main difference was the Cs value was changed to its correct value. Dead load and live loads remained unchanged, and also appear to be correct.

It was difficult to find a correct assumption for the distribution of lateral forces to each lateral frame. The relative stiffness method used with RAM Advanse was an approximation. The effects of the load placed at the top story give approximate relative stiffnesses at each floor. In reality, to get a more accurate relative stiffness for every floor, a load should be placed on the story in question. The IBC clearly states that the Moment frames of a dual system must take on 25% of the total base shear. This percentage was about twice as small as those calculated with the relative stiffness in RAM. The RAM Advanse model can be critiqued more to find each story stiffness more accurately in the near future. It is essential that the correct distribution be found, in order to determine what types of alternate lateral systems could be used.

The spot checks performed yielded smaller sized members than what was originally designed for. This can be accounted for by the uncertainty of lateral force distribution. Also, when looking at the moment frames in the North South orientation, it is likely that the columns are designed based on drift and not strength. The structural engineer will be contacted to clear up misinformation and to possibly gain access to the original ETABS model, or calculations. If I can find the distribution percentages to each frame, I can work backwards to see how they were calculated.

Looking ahead to the proposal, it appears a change to the lateral system will occur. When the new design occurs, the distribution can be controlled by myself, and should not be a problem.

Appendices

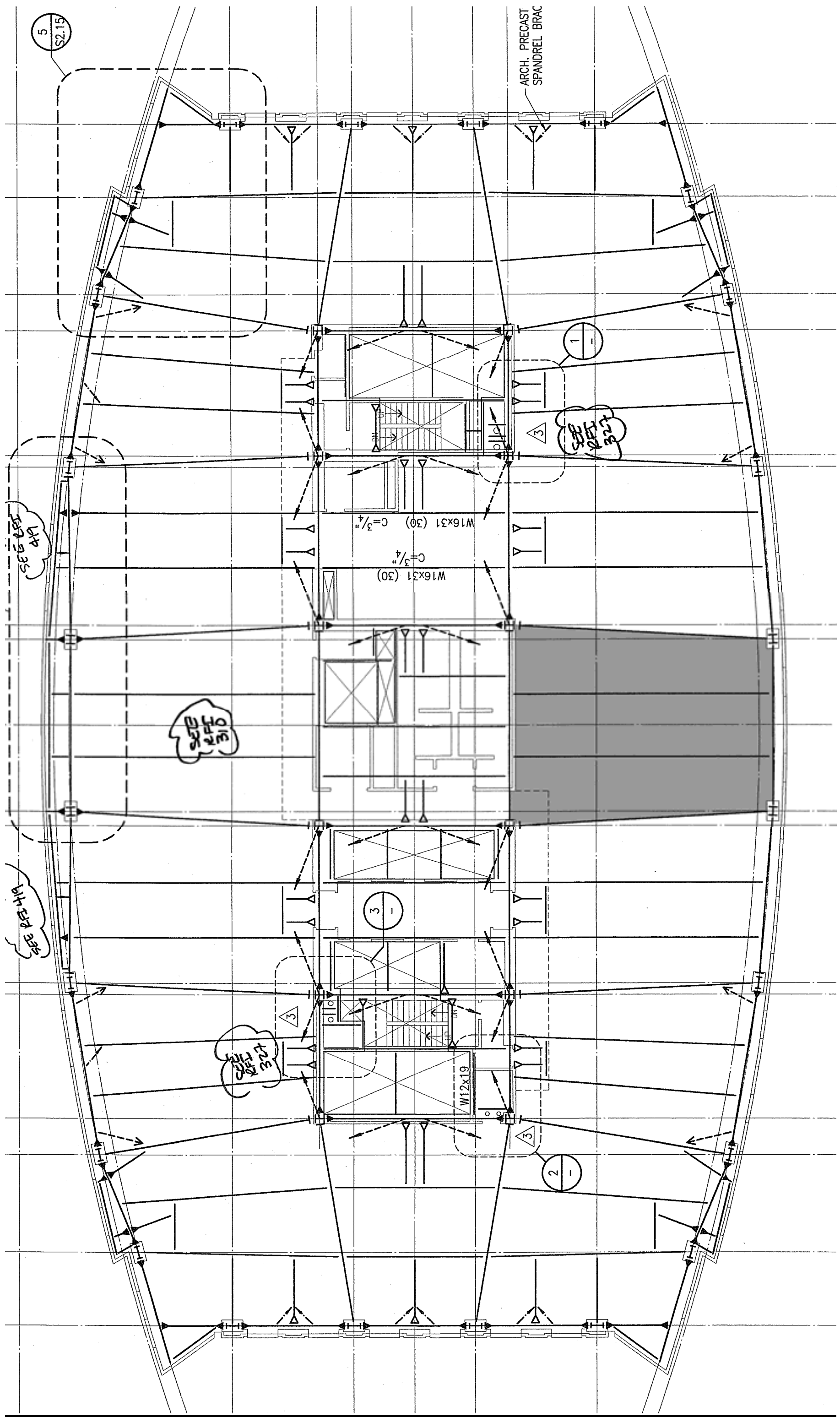
A. Floor Pan and Lateral Resisting Frames

B. Wind Load Diagrams and Calculations

C. Seismic Load Calculations

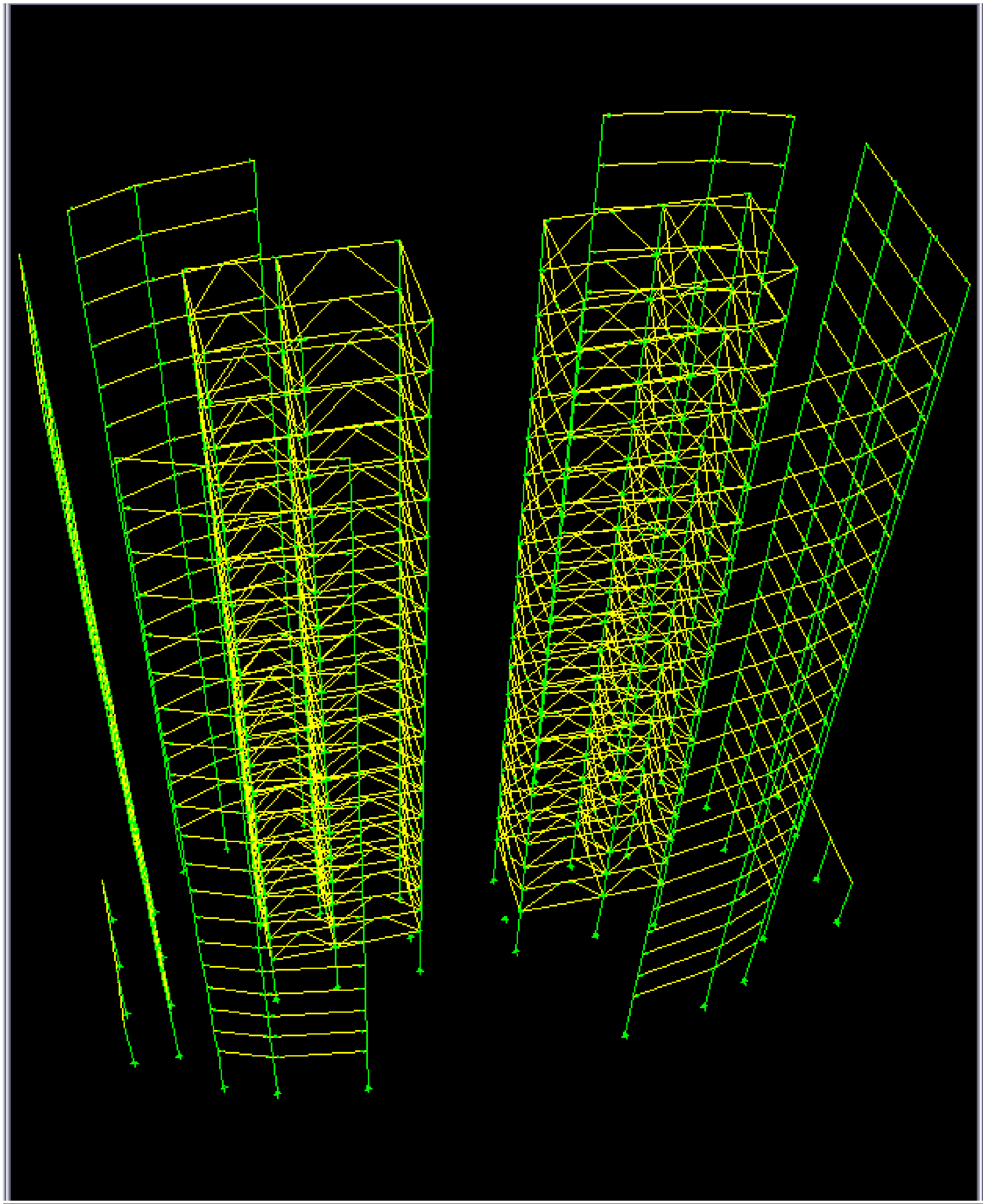
D. Spot Checks

APPENDIX A: Lateral Resisting Frames and Floor Plan

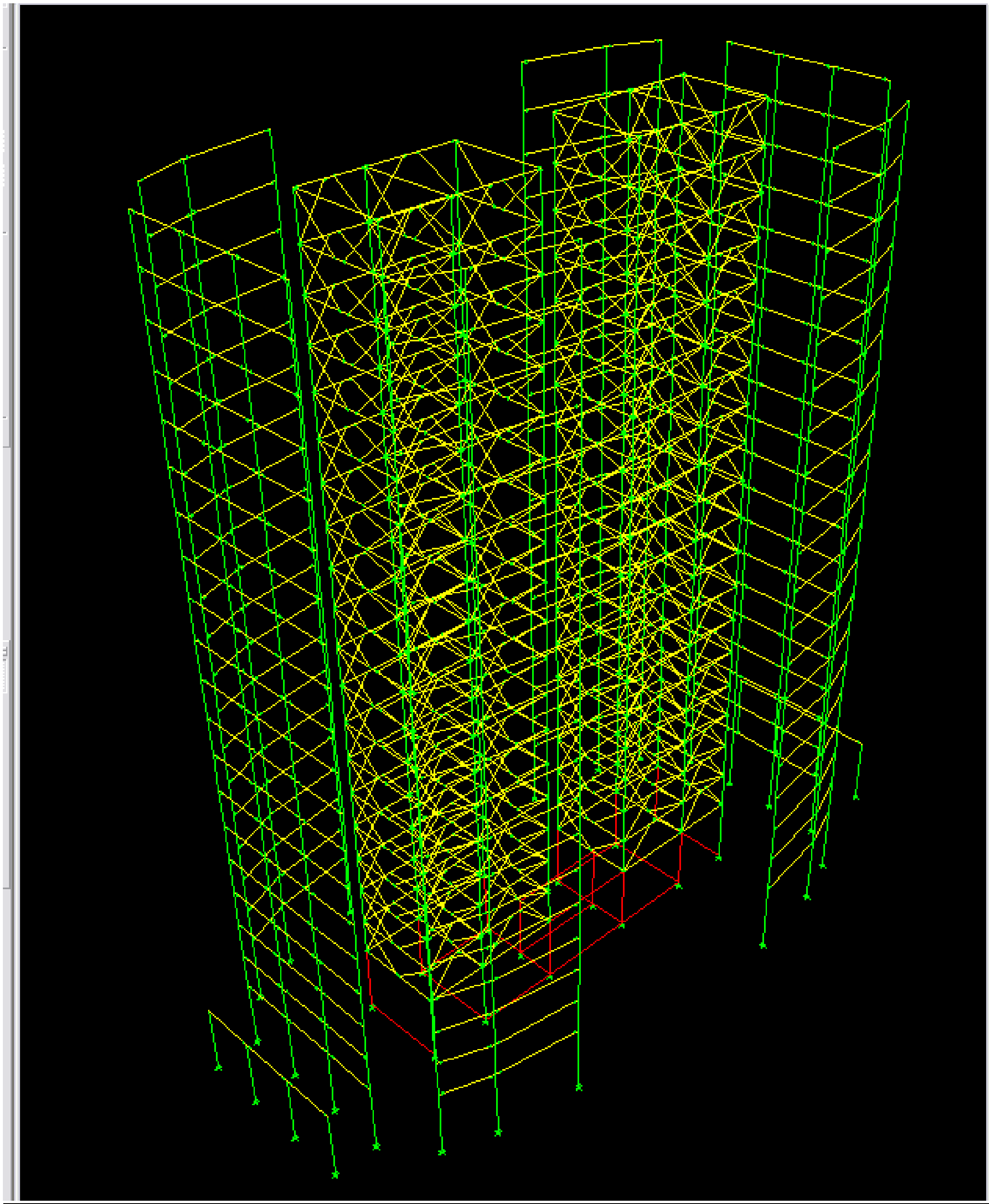


Typical floor pan

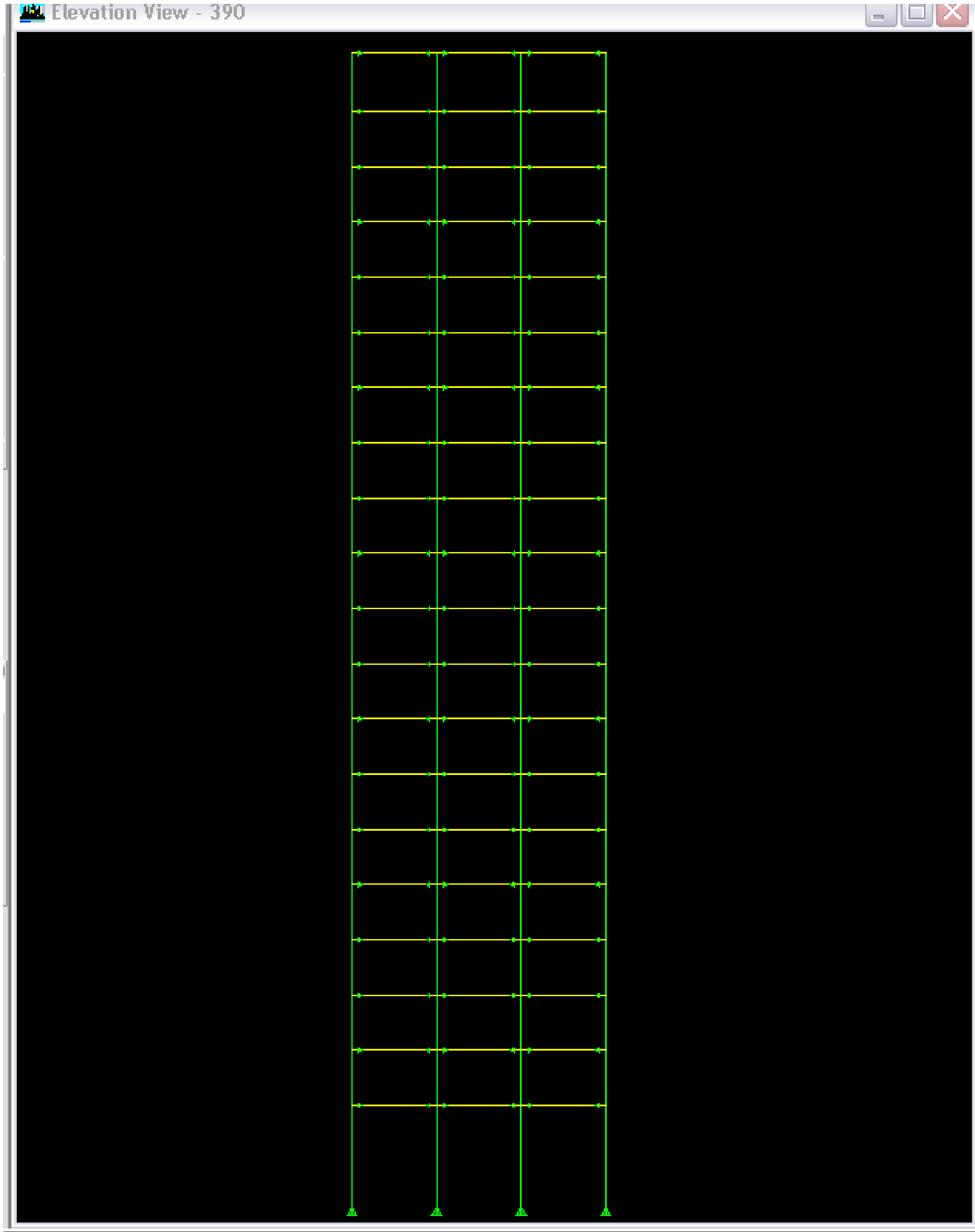
North Face 3D Elevation



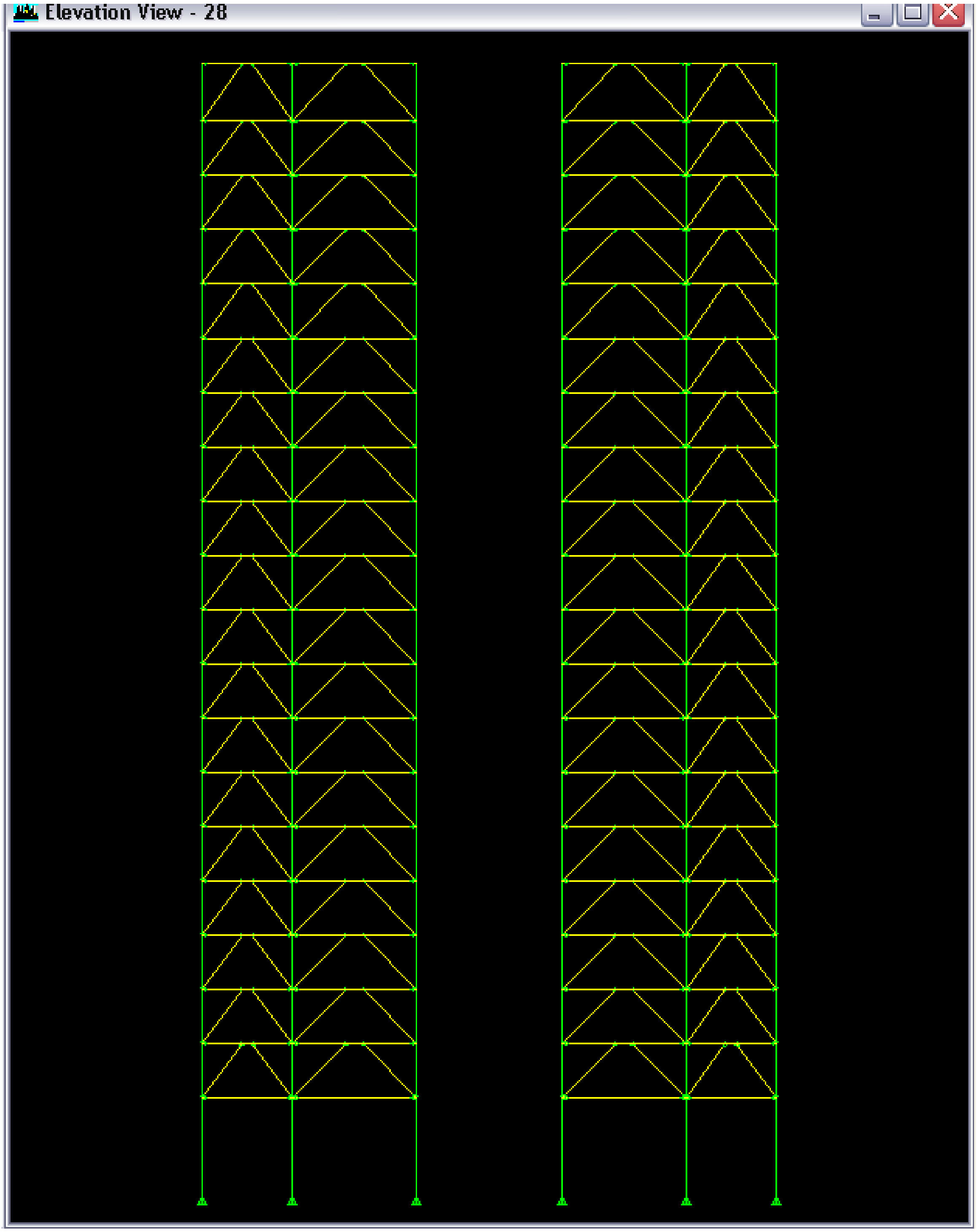
North – East 3D Elevation_from ETABS Model



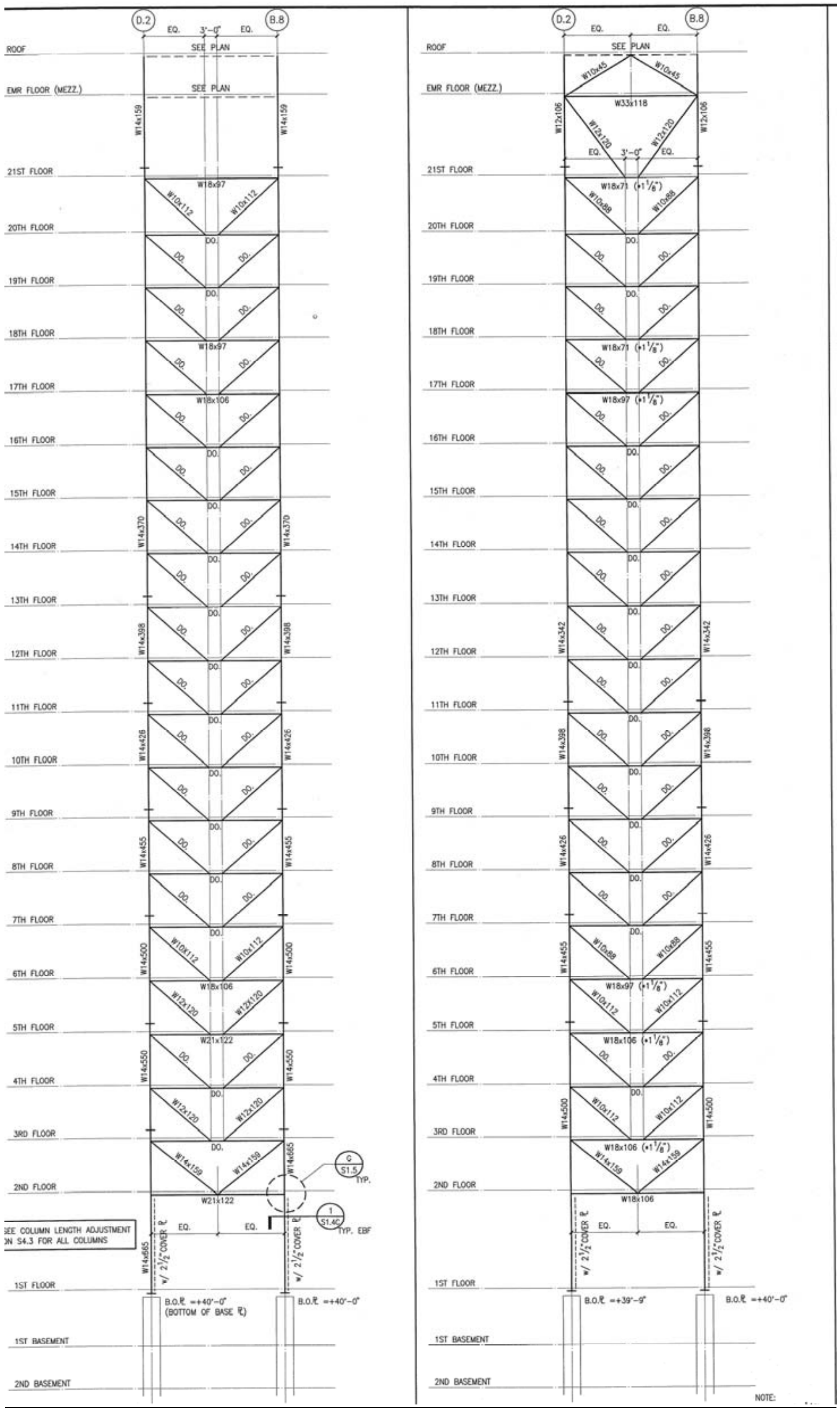
Special Moment Resisting Frames 1 and 2



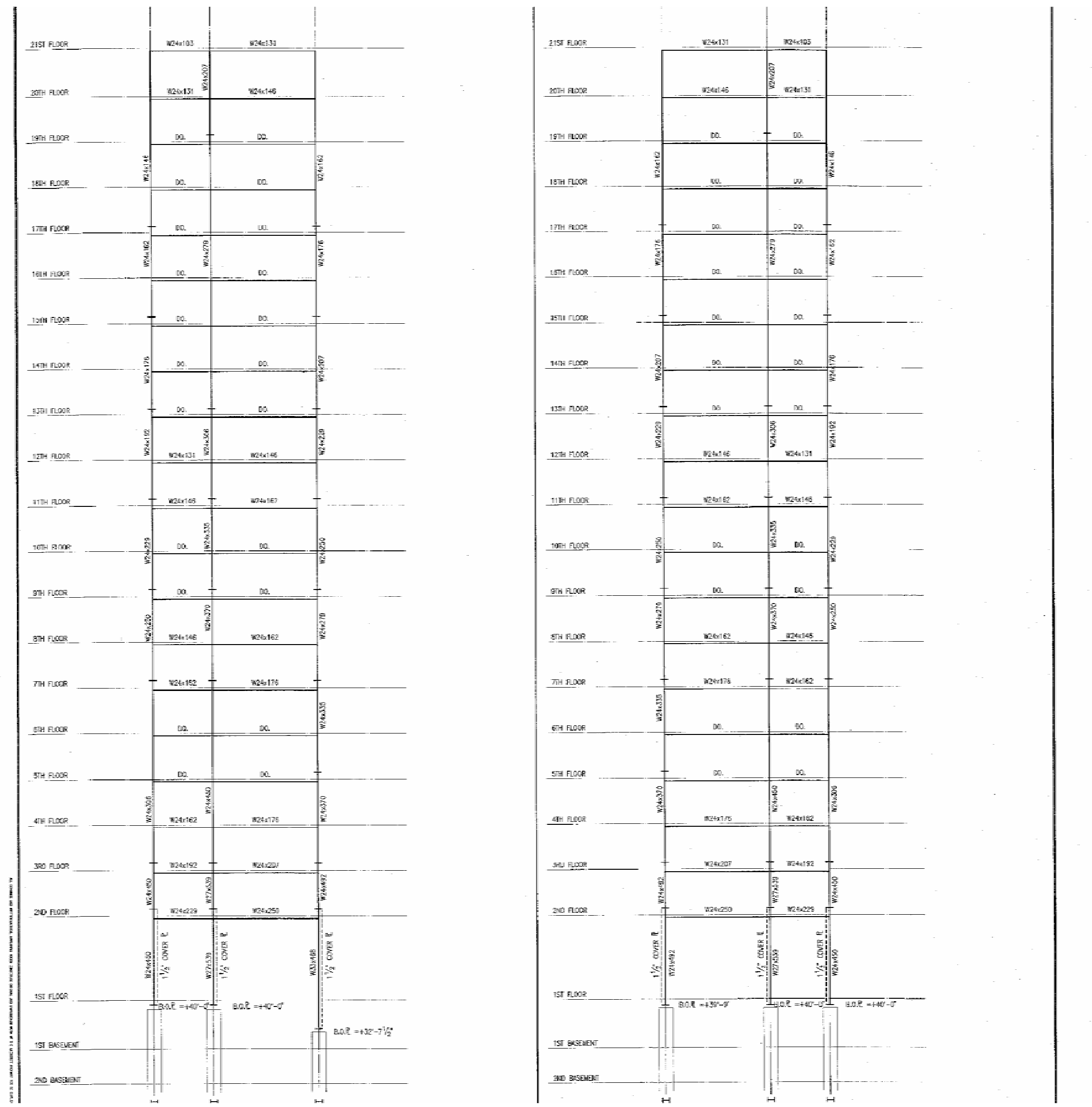
Eccentric Braced Frames 7,8,9,10



EBF Frames (1-6) on interior core



SMRF Frames (3,4,5,6) on Exterior Walls



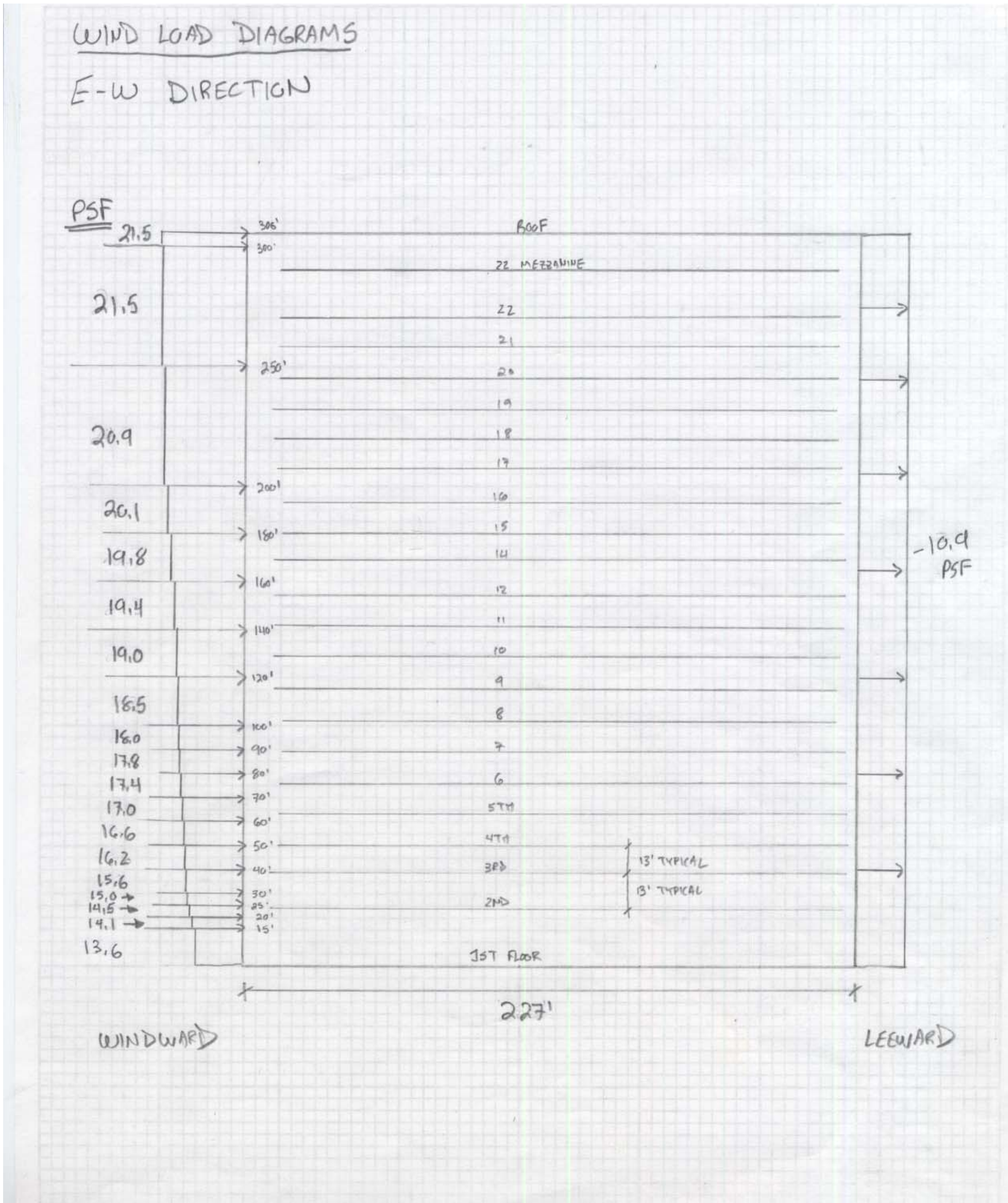
APPENDIX B: Wind Load Diagrams and Calculations

WIND LOAD FORCES AT EACH FLOOR (E-W)

- TRIBUTARY WIDTH AT EACH FLOOR x PSF WIND LOAD (FROM WINDWARD + LEEWARD)

FLOOR	HT (FEET)		
2	24	[5'(25.4 PSF) + 5'(25 PSF) + 3'(24.5 PSF) + 5'(25.9 PSF) + 0.5'(26.5 PSF)] (125') =	58.5^{k}
3	37	[9.5'(26.5) + 3.5(27.1)] (125) =	$= 43.3^{k}$
4	50	[6.5(27.1) + 6.5(27.5)] (125) =	$= 44.4^{k}$
5	63	[3.5(27.5) + 9.5(27.9)] (125) =	$= 45.2^{k}$
6	76	[0.5(27.9) + 10(28.3) + 2.5(28.7)] (125) =	$= 46.1^{k}$
7	89	[7.5(28.7) + 5.5(28.9)] (125) =	$= 46.8^{k}$
8	102	[4.5(28.9) + 8.5(29.4)] (125) =	$= 47.5^{k}$
9	115	[11.5(29.4) + 1.5(29.9)] (125) =	$= 47.9^{k}$
10	128	[13(29.9)] (125) =	$= 48.6^{k}$
11	141	[5.5(29.9) + 7.5(30.3)] (125) =	$= 49.0^{k}$
12	154	[12.5(30.3) + 0.5(30.7)] (125) =	$= 49.3^{k}$
14	167	[13(30.7)] (125) =	$= 49.9^{k}$
15	180	[6.5(30.7) + 6.5(31.0)] (125) =	$= 50.1^{k}$
16	193	[13'(31.0)] (125) =	$= 50.4^{k}$
17	206	[0.5(31.0) + 12.5(31.8)] (125) =	$= 51.6^{k}$
18	219	[13'(31.8)] (125) =	$= 51.7^{k}$
19	232	" " =	$= 51.7^{k}$
20	245	[11.5(31.8) + 1.5(32.4)] (125) =	$= 51.8^{k}$
21	258	[13.5(32.4)] (125) =	$= 54.7^{k}$
22	272	[17(32.4)] (125) =	$= 68.9^{k}$
MEZZ	292	[17(32.4)] (125) =	$= 68.9^{k}$
ROOF	306	[7(32.4)] (125) =	$= 28.4^{k}$

TOTAL BASE SHEAR $= \boxed{1104.7^{k}}$


WIND LOAD DIAGRAMS
E-W DIRECTION
PSF
21.5
21.5
20.9
20.1
19.8
19.4
19.0
18.5
18.0
17.8
17.4
17.0
16.6
16.2
15.6
15.0
14.5
14.1
13.6
306'
300'
250'
200'
180'
160'
140'
120'
100'
90'
80'
70'
60'
50'
40'
30'
25'
20'
15'
ROOF
22 MEZZANINE
22
21
20
19
18
17
16
15
14
12
11
10
9
8
7
6
5TH
4TH
3RD
2ND
1ST FLOOR
13' TYPICAL
13' TYPICAL
-10.9 PSF
227'
WINDWARD
LEEWARD

CKD ________ DATE ____________ ASSOCIATES PROJ. NAME ____________

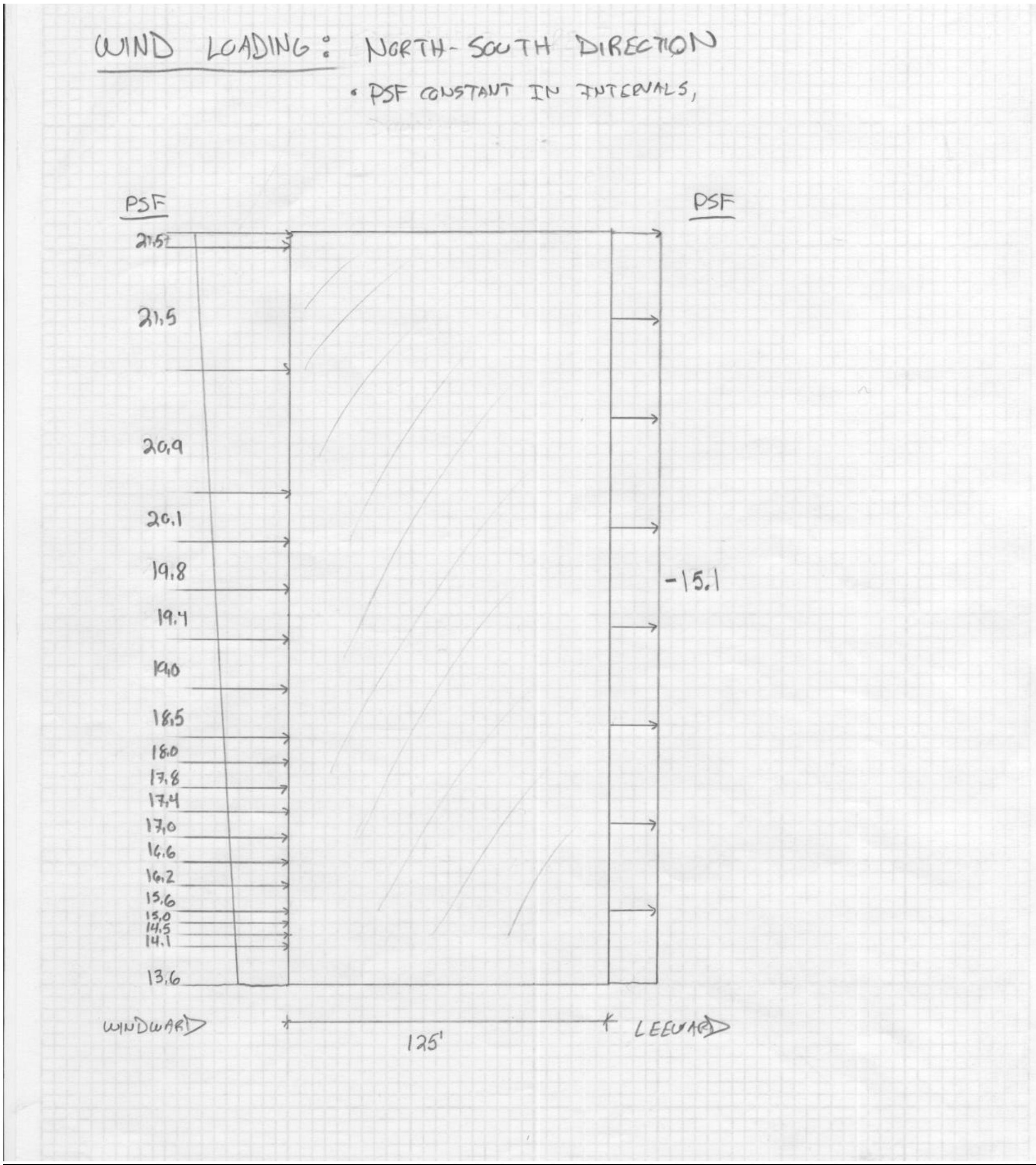
WIND LOAD FORCES AT EACH FLOOR (N-S)

→ TRIB. WIDTH AT EACH FLOOR × PSF WIND LOAD (WINDWARD + LEEWARD)

FLOOR			
2	[5'(29.6 PSF) + 5'(29.2 PSF) + 3'(28.7) + 5'(30.1 PSF) + 0.5'(30.7 PSF)](227')	=	123.9 k
3	[9.5(30.7) + 3.5(31.3)](227)	=	91.0 k
4	[6.5(31.3) + 6.5(31.7)](227)	=	92.9 k
5	[3.5(31.7) + 9.5(32.1)](227)	=	94.4 k
6	[0.5(32.1) + 10(32.5) + 2.5(32.9)](227)	=	96.1 k
7	[7.5(32.9) + 5.5(33.1)](227)	=	97.3 k
8	[4.5(33.1) + 8.5(33.6)](227)	=	98.7 k
9	[11.5(33.6) + 1.5(34.1)](227)	=	99.3 k
10	[13(34.1)](227)	=	100.7 k
11	[5.5(34.1) + 7.5(34.5)](227)	=	101.3 k
12	[12.5(34.5) + 0.5(34.9)](227)	=	101.8 k
14	[13(34.9)](227)	=	103.0 k
15	[6.5(34.9) + 6.5(35.2)](227)	=	103.4 k
16	[13(35.2)](227)	=	103.9 k
17	[0.5(35.2) + 12.5(36.0)](227)	=	106.2 k
18	[13'(36)](227)	=	106.2 k
19	" "	=	106.2 k
20	[11.5(36) + 1.5(36.6)](227)	=	106.4 k
21	[13.5(36.6)](227)	=	112.2 k
22	[17(36.6)](227)	=	141.2 k
MEZZ	[17(36.6)](227)	=	141.2 k
ROOF	[7(36.6)](227)	=	58.2 k

TOTAL BASE SHEAR = $\boxed{2286.0^k}$

TO COMPARE AGAINST SEISMIC ⇒ 1.6(2286) = 3658 k



WIND LOADING: NORTH-SOUTH DIRECTION
• PSF CONSTANT IN INTERVALS,
PSF
PSF
21.57
21.5
20.9
20.1
19.8
19.4
19.0
18.5
18.0
17.8
17.4
17.0
16.6
16.2
15.6
15.0
14.5
14.1
13.6
-15.1
WINDWARD
125'
LEEWARD

Shear Forces acting at each story in both directions

	EAST_WEST			NORTH_SOUTH		
Floor	**Story Height**	**Shear(Kip)**	**Over. Mom (ft-kip)**	**Story Height**	**Shear**	**Over. Mom**
2	24	58.5	1404	24	123.9	2973.6
3	37	43.3	1602.1	37	91	3367
4	50	44.4	2220	50	92.9	4645
5	63	45.2	2847.6	63	94.4	5947.2
6	76	46.1	3503.6	76	96.1	7303.6
7	89	46.8	4165.2	89	97.3	8659.7
8	102	47.5	4845	102	98.7	10067.4
9	115	47.9	5508.5	115	99.3	11419.5
10	128	48.6	6220.8	128	100.7	12889.6
11	141	49	6909	141	101.3	14283.3
12	154	49.3	7592.2	154	101.8	15677.2
14	167	49.9	8333.3	167	103	17201
15	180	50.1	9018	180	103.4	18612
16	193	50.4	9727.2	193	103.9	20052.7
17	206	51.6	10629.6	206	106.2	21877.2
18	219	51.7	11322.3	219	106.2	23257.8
19	232	51.7	11994.4	232	106.2	24638.4
20	245	51.8	12691	245	106.4	26068
21	258	54.7	14112.6	258	112.2	28947.6
22	272	68.9	18740.8	272	141.2	38406.4
mezzanine	292	68.9	20118.8	292	141.2	41230.4
roof	306	28.4	8690.4	306	58.2	17809.2
		1104.7	**182196.4**		**2285.5**	**375333.8**

APPENDIX C: Seismic Calculations

CKD ______ DATE ______ ASSOCIATES PROJ. NAME ______

SEISMIC ANALYSIS

USGS WEBSITE, OAKLAND, CA : LATITUDE: 37.804603 , LONGITUDE: − 122.275486

* S_S = 0.2s = SPECTRAL RESPONSE = 240.95

* S_1 = 1.0s = SPECTRAL RESPONSE = 94.05

* VALUES DIFFERENT FROM TECH REPORT ONE

BUILDING SITE CLASS: C

* BUILDING OCCUPANCY : OFFICE ⇒ TYPE II ⇒ SEISMIC USE GROUP I

$I = 1.0$ FROM TABLE 9.1.4

$F_a = 1.0$ FROM TABLE 9.4.1.2.4a FOR $S_S \geq 1.25$

$F_v = 1.3$ FROM TABLE 9.4.1.2.4b FOR $S_1 \geq 0.5$

$S_{MS} = F_a S_S = 1.0(2.4095) = 2.4095 \Rightarrow S_{DS} = 2/3(2.4095) = 1.6063$

$S_{M1} = F_v S_1 = 1.3(0.9405) = 1.2227 \Rightarrow S_{D1} = 2/3(1.2227) = 0.815$

SEISMIC DESIGN CATEGORY FROM S_{DS} = D

S_{D1} = D

RESPONSE MODIFICATION FACTOR: $R = 8$ FOR DUAL SYSTEM of SMRF AND EBF'S BOTH DIRECTIONS

FUNDAMENTAL PERIOD: $T_a = C_t h_n^x$ w/ $C_t = .02$, $x = 0.75$ (Table 9.5.5.3.2)

$T_a = .02(306)^{.75} = 1.463\,s$

C_u = w $S_{D1} \geq 0.4 \Rightarrow 1.4$

$C_u T_a = 2.05\,s = T$

SEISMIC RESPONSE COEFF. (C_s) =

$C_s = \frac{S_{DS}}{R/I} = \frac{1.6063}{8} = \underline{\underline{0.200}}$

$\leq \frac{S_{D1}}{T(R/I)} = \frac{0.815}{2.05(8/1)} = 0.0497$

$\geq \frac{0.5S_1}{R/I} = \frac{0.5(.9405)}{8} = 0.0588$ (IN DESIGN CATEGORY E OR F)

$\geq .044 S_{DS} I = \boxed{0.0707}$ ← GOVERNS

ROOF DEAD LOADS: 3" METAL DECK w/ 2½" CONCRETE

150 PCF (5.5/12) = 68.8 PSF
DECK : 2 PSF
MEP = 10 PSF
STRUCTURAL FRAME = 15 PSF
COLLATERAL = 5 PSF

100.8 PSF ≈ 100 PSF DEAD LOAD

TYPICAL FLOOR: 3" DECK w/ 2½" CONCRETE = 68.8 PSF
MEP = 10 PSF
DECK = 2 PSF
STRUCTURAL = 15 PSF
COLLATERAL = 5 PSF

100.8 ≈ 100 PSF

2ND FLOOR: 3" DECK w/ 4" CONCRETE = 150 PCF (7/12) = 87.5 PSF
MEP = 10
DECK = 2
STRUCTURAL = 15
COLLATERAL = 5

119.5 ≈ 120 PSF

21ST FLOOR: 3" DECK w/ 6" CONCRETE ⇒ 150(6/12) = 75 PSF
MEP 10
DECK 2
STRUCT. 15
COLLATERAL 5

107 ≈ 110 PSF

22ND FLOOR: (MECHANICAL) 3" DECK w/ 6" CONCRETE ⇒ 150(6/12) = 75 PSF
6" CONCRETE PADS FOR MECHANICAL : 75 PSF × AREA PADS
AREA ≈ 4500 ft^2

+10
2
5
15
75

107 ≈ 110 PSF + MECH

NOTE: ALLOW 10 PSF FOR PARTITIONS IN SEISMIC CALCULATION (9.5.3)

FIND WEIGHT OF FLOORS

ROOF: GROSS AREA = 13500 ft^2
LOADING = 100 PSF
= 1350000 lb = $\underline{1350^k}$

TYPICAL FLOOR: 23937 ft^2 (100+10 PSF) = 2633070 lb = $\underline{\underline{2633^k}}$

2ND FLOOR: 23937 ft^2 (120+10 PSF) = 3111810 lb = $\underline{\underline{3112^k}}$

21ST FLOOR: 23937 ft^2 (110+10) = 2872440 lb = $\underline{\underline{2872^k}}$

22ND FLOOR: 23937 ft^2 (110 PSF) + 4500 ft^2 (75 PSF) = 2970570 lb = $\underline{\underline{2971^k}}$

MEZZANINE: 18217 ft^2 (100 PSF) = 1821700 lb = $\underline{\underline{1822^k}}$

TOTAL LOAD = 2ND + TYPICAL 3-20 (EXCLUDING 13TH) + 21 + 22 + MEZZANINE + ROOF

$W_T = 3112 + 17(2633) + 2872 + 2971 + 1822 + 1350$

$W_T = 56,888^k$

USE EQUIVALENT LATERAL FORCE PROCEDURE

SEISMIC BASE SHEAR = $V = C_s W = 0.0707(56,888^k) = \boxed{4022^k = V}$

VERTICAL DISTRIBUTION OF FORCES

$T_a = 1.463s$, $\Rightarrow 0.5 < T_a = 1.463 < 2.5 \Rightarrow k =$ interpolated between 1 AND 2

$k = 1.48$

Vertical Distribution of Seismic Forces

Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)	Shear(kips)
roof	1350	306	6444682	0.0586	235.6	72106	235.6
21	1822	292	8115495	0.0738	296.7	86646	537.3
20	2971	272	11914193	0.1083	435.6	118490	972.9
19	2872	258	10650785	0.0968	389.4	100473	1362.3
18	2633	245	9045169	0.0822	330.7	81027	1693.0
17	2633	232	8343977	0.0759	305.1	70780	1998.1
16	2633	219	7661400	0.0696	280.1	61348	2278.2
15	2633	206	6998007	0.0636	255.9	52710	2534.1
14	2633	193	6354416	0.0578	232.3	44842	2766.4
13	2633	180	5731314	0.0521	209.6	37720	2976.0
12	2633	167	5129455	0.0466	187.6	31321	3163.6
11	2633	154	4549687	0.0414	166.4	25618	3330.0
10	2633	141	3992962	0.0363	146.0	20586	3476.0
9	2633	128	3460363	0.0315	126.5	16195	3602.5
8	2633	115	2953137	0.0268	108.0	12417	3710.5
7	2633	102	2472744	0.0225	90.4	9222	3800.9
6	2633	89	2020915	0.0184	73.9	6576	3874.8
5	2633	76	1599761	0.0145	58.5	4445	3933.3
4	2633	63	1211923	0.0110	44.3	2792	3977.6
3	2633	50	860848	0.0078	31.5	1574	4009.1
2	2633	37	551301.8	0.0050	20.2	746	4029.3
1	3112	24	343362	0.0031	12.6	301	4041.9
			1.1E+08	1.0	4036.8	857937	4042.0

APPENDIX D: Spot Checks of Lateral System and Members

CHECK CRITICAL MEMBERS OF SMRF ON LINE 10 (NORTH - SOUTH)

SEISMIC CONTROLS DESIGN

ASSUME SMRF IN DUAL SYSTEM TAKES 25% OF BASE SHEAR

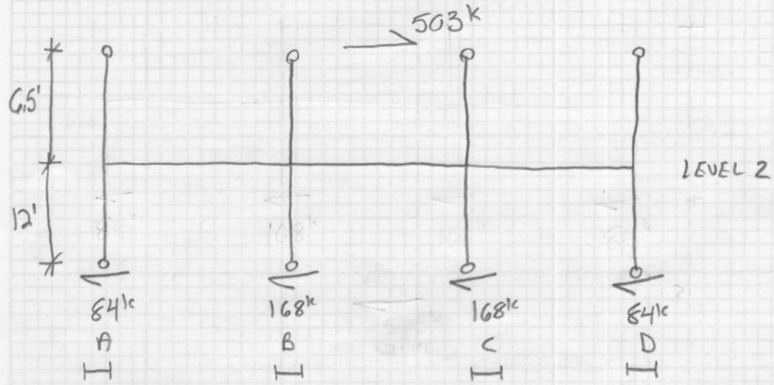
$0.25(4022^k) = 1005.5^k$ @ BASE

$\div 2$ FRAMES IN N-S DIRECTION

502.8^k

W36x527

36x588



$M_B = 168^k(12') = \underline{\underline{2016'k}} \Rightarrow Z_{req} = \frac{2016'k(12)}{0.9(50)} = 537.6\ in^3$

DESIGNED AS W36x588, MY CALCS REQUIRE A W36x160 w/ $Z_x = 624\ in^3$

* NEED TO ACCOUNT FOR ATIAL LOAD ON COLUMN

TRIBUTARY AREA: $20' \times 18' = 360\ ft^2 = A_T$, $K_{LL} = 4$

TOTAL AREA $= 360\ ft^2(20) = 7200\ ft^2$

$A_I = 4(7200) = 28800\ ft^2$

$LL = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) = L_o\left(0.25 + \frac{15}{\sqrt{28800}}\right) = .338 < 0.4$

USE $0.4L_o$

DL @ 7st Floor: $(110\ PSF \times 360(2)) + (360 \times 120\ PSF) + (6120\ ft^2 \times 100\ PSF)$

$DL = 734.4^k$

$LL = 0.4(100\ PSF)(6840) + 0.4(150\ PSF)(360) = 295.2^k$

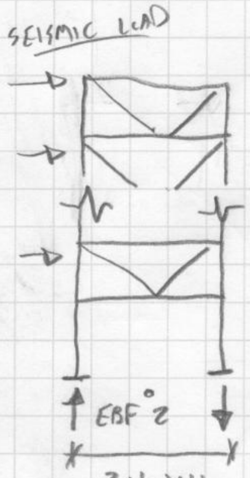
LOAD COMBINATION: $1.2D + 1.6L = 1353.6^k$ w/ $KL = 24'$

Should check for combined loading to verify exact size. W36 was not in LRFD manual Table 4-2 to check. Member most likely sized based on Drift controlling, not strength.

OVERTURNING CHECK ON EBF FRAME (N-S DIRECTION)

FRAME 2 — DUE TO WIND:

OVERTURNING MOMENT = $\frac{375333\ 'k}{2}$ IN N-S DIRECTION



187666 'k (COUPLE CREATED)

EBF TAKES 12.5%

OVERTURNING MOMENT – 23458 'k

TENSILE FORCE = $\frac{23458}{31.333}$ = 748^k (T)

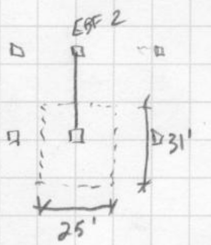
DEAD LOAD FROM FLOORS

TYPICAL = (110 PSF)(31.333 × 25) = 86165 lb = 86.16^k

20 FLOORS (86.16^k) = 1723^k (COMPRESSION)

FORCE IN COLUMN = 1723 − 748 = 975^k (C)

∴ OK FOR OVERTURNING

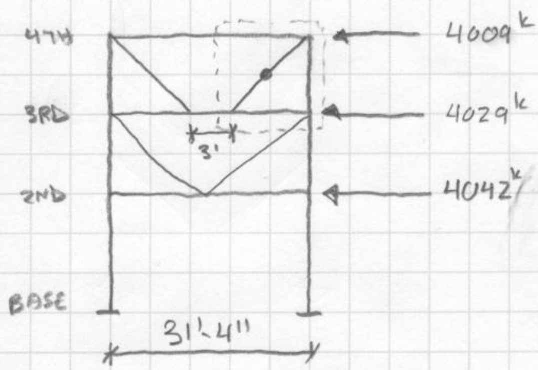


PLAN – TRIB AREA

SPOT CHECK: BRACING

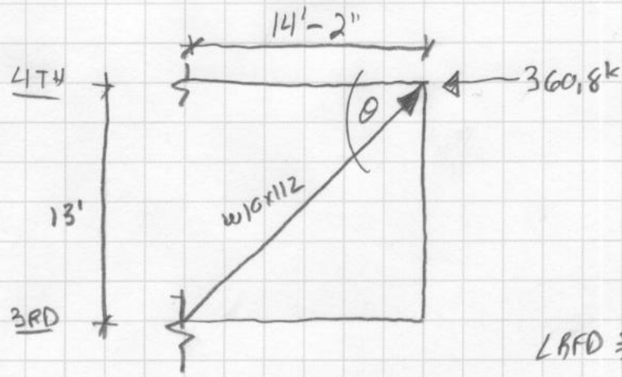
EBF TWO IN NORTH-SOUTH DIRECTIONS

- FROM RELATIVE STIFFNESS, THIS FRAME TAKES 9% OF TOTAL SEISMIC LATERAL LOAD



$V_{4TH} = 4009(0.09) = 360.8^k$

MEMBER DESIGNED AS W10×112



$\theta = \tan^{-1}\left(\frac{13}{14.167}\right) = 42.54°$

$360.8 - F\cos(42.54) = 0$

$F = 489.7^k$

Length = 19.22'

LRFD ⇒ USE W10×77 w/ $\phi P_n = 515^k$

IF 25/75 DISTRIBUTION $V_{4TH} = 4009\left(\frac{.75}{6}\right) = 501^k$

$50\cancel{6} = F\cos 42.54 = 0$

$F = 680^k$ w/ $KL = 19.22'$

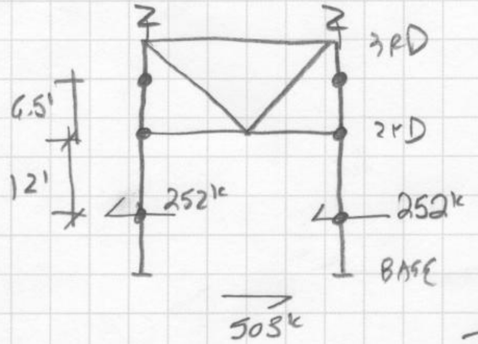
$\boxed{\text{LRFD} \Rightarrow \text{USE W10×100 w/ } \phi P_n = 686^k}$

CHECK BRACED FRAME COLUMN (1st – 2ND STORY COLUMN)

W14×665 USED ON EBF 1

FRAMES TAKE 75% OF LATERAL FORCE
6 EBF IN N-S DIRECTION

$.75/6\,(4022^k) = 503^k$ TO EACH FRAME (SIMILAR)



IGNORE AXIAL LOAD

$\text{Moment} = 252^k(12') = 3024\,'k$

$Z_{required} = \dfrac{3024(12)}{0.9(50)} = 806\,in^3$

TRY A W14×426 w/ $Z_x = 869\,in^3$

DESIGNER USED W14×665

* ADDING AXIAL LOAD WOULD BRING SIZE UP