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STRUCTURAL OPTION
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AE 481W
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555 12TH STREET
OAKLAND, CALIFORNIA

Technical Report #1:
Structural Concepts & Existing Conditions Report

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. It is one of several buildings that make up what is known as the Oakland City Center. Its use of vision and spandrel glass on the façade, mixed with precast concrete panels, compliments the surrounding landscape and architecture perfectly.

The purpose of this report is to describe the existing structural system of the building and its components. Calculations of all relevant loadings conditions, including gravity, wind, and seismic will be found. With these calculations, spot checks of various structural elements will be carried out and compared to the original design.

The original design used the Uniform Building Code to get its design loads. As I am not familiar with using that code, I used the IBC 2003 and ASCE7 to determine my loading. The gravity calculations were close to the actual designed members, but slightly higher in some. This difference can be explained by the use of a different building code, or using greater loading. The lateral system will need to be addressed further, to verify that the correct spectral response numbers were used, and to gain a better understanding of how loads are distributed to the system. Loads on the moment frame we're too small compared to what they were designed for.

STRUCTURAL SYSTEM DESCRIPTION

Foundation:

The foundation was designed based on soil reports by URS Greiner Woodward Clyde, dated April 13, 2000. The soil bearing capacity was found for three different load combinations. For dead load, dead + live load, and dead + live + earthquake, the capacities are 5000, 7500, and 10000 psf respectively. A surcharge load at street side was calculated as 150 psf. All concrete for the foundation has a 28-day strength of f'c = 4000 psi. The reinforcing steel is ASTM A615 GR 60 deformed bars.

Over 650 truckloads of concrete – 24 Million lbs.- were required to pour the mat foundation. The foundation has a 5' thickness near the exterior walls, and transitions to 7' thick as it approaches the interior core. The entire mat is reinforced with #9 @ 8" Top EW and #10 @ 8" Bottom EW.

Spread and continuous footings are used to support the structural elements of the parking garage directly under the building on the mat. Spread footings, 3' thick and reinforced with #5 @ 12" Top EW and #9 @ 8" Bot. bars EW are used to support the interior columns of the parking garage. There sizes range from 10' to 20' for both length and width. Typical exterior and interior wall footings are continuous and 2'-6" thick. They are reinforced with #6 @ 14" bars T&B EW, unless otherwise noted.

Slab on Grade:

Slabs were poured over a max distance of 60' per pour in both directions. Control joints were cut into the slab as 20'x20' squares. There are two types of SOG, both with their top elevation at +17'-0", and both the floor for the second underground parking level. A 4" SOG reinforced with #4 @ 18" EW is placed over a layer of class 2 aggregate fill, over the mat footing. When not over the mat footing, the SOG is 6" thick, reinforced with #4 @ 12" EW typically. All concrete for the SOG has a strength of f'c = 4000 psi.

One-way Slab System:

Floors for level P1 and the 1st floor are cast-in-place one way slab systems, supported by precast concrete members. The thickness ranges from 6-12", depending on location, and reinforcing varies from #4-#7 bars @ 12" T&B. The precast beams supporting the slab were specified by the subcontractor, and the sizes are not given in the contract documents

Elevated Slab:

The elevated floors, starting from level 2, are composite metal deck systems. The 2nd floor is 3" 18 gage composite decking with 4" of normal weight concrete cover. It is reinforced with #4 @ 16" EW. 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 mechanical room on the 22nd floor, along with its mezzanine level, uses a variety of composite decking. There is either 3"-16 or 18 gage composite deck with up to 7" of normal weight concrete over it. Reinforcement is typically #4 @ 12" unless otherwise noted. The roof uses 3"-18 gage composite decking with 2 ½" of lightweight fill. It is reinforced with #3 @ 16" EW.

Structural Framing:

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. The elevated slabs are supported by wide flange beams with varying lengths, the longest being about 44', because of the curved exterior wall. They are typically W18x35 up to W18x55, unless otherwise noted.

The girders, which are not part of the moment frames, are sized from W24x55 up to W27x84 and span at the greatest, 35'. Smaller W-shapes are used on the interior core area to support the slabs. The 22nd floor-mechanical floor has the same location of beams and girders, but different sizes. The typical beam is a W24x55 up to a W24x94. The typical girder is slightly larger, being a W27x84 on the exterior wall, and W30x124 on the interior core.

The roof uses W12x22 up to W21x44 for its beams and girders, along with TS shapes for exterior beams, sized as TS10x8.

Columns:

Most of the columns in the building help make up the lateral resisting system. They will be described in that section. They attach to base plates and anchor to the top of concrete columns that run from the foundation up to the first floor. The EBF frame concrete column is 4'x4' reinforced with (40) 1 3/8" diameter DYWIDAG treaded bar(ASTM722) and #5 @ 3" Ties Baugrid. The SMRF frame concrete columns vary in size from 3' to 3'-9" square. They larger columns are reinforced with (20) 1" diameter DYWIDAG with #5 ties @ 3". The 3' columns are reinforced with (20) #11 vertical and #5 ties @ 3".
TS 8x8x3/8 are used typically as columns for the 21st floor up to the roof for the mechanical floor. All frame concrete columns are required to have a 56 day strength of f'c = 7500psi.

All non-frame steel columns range from W14x109 at the 21st floor, up to W14x500 at ground level on Marks C1 and C2. The canopy columns at the 1st floor are W14x53 and W14x48 on marks C3 and C4 respectively. The base plates are 30"x30"x3" with (4) 1 ½" Anchor bolts with 24" embedment.

Lateral System:

The lateral system of 555 City Center is a dual system, with several mechanisms for distributing the lateral load. Here is a list of the various parts of the system:

- **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. EBF's along gridlines 3.2, 6.1, and 7.8 are similar, while EBF's along gridlines 3.9, 4.9, and 7.1 are similar. 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. Refer to appendix for drawings of the frames.

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

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.

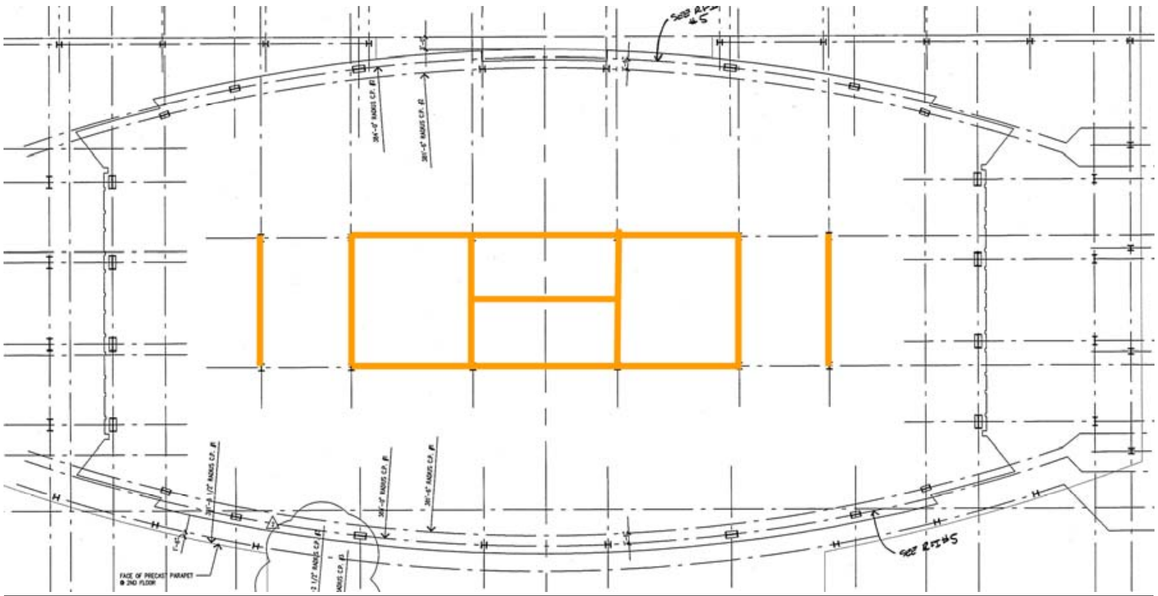
East-West Lateral System: Red = EBF, Green = SMRF

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

These frames are located on the exterior 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 W36's.

- **Shear Walls**

The shear walls are located directly under the EBF frames, and occupy the same gridlines. They run from the SOG 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.



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

<u>Codes and Code Requirements:</u>

Building Code:	Uniform Building Code (UBC 1997)
	California Building Code (CBC 1998)
Design References:	American Society of Civil Engineers (ASCE 7)
	American Concrete Institute (ACI 318)
	ASTM Standards

My preliminary analysis of the building will use the IBC 2003 and ASCE 7-02 as design references.

Material Strengths

Concrete:

Shall have following 28-day stengths(minimum f'c)
Hardrock Concrete 145PCF unless otherwise noted.

- Foundation: Mat and Spread Footing: 4000 psi
- SOG, structural slab, basement retaining wall and bearing wall: 4000 psi
- Core shear wall: 5000 psi
- Concrete fill metal deck: 3000 psi
- Roof concrete fill metal deck(light weight): 3000 psi
- Precast concrete column and beam: as required per design
- Frame Concrete Column: 7500 psi(56 days)

Reinforcing Steel:

- To be ASTM A615, GR 60 deformed bars, grade 40 for #3 UON.
- WWF to be ASTM A185.
- ASTM 706 at shear wall boundary element(trim bars), lateral frame element, and where welding is required.
- High strength frame column: DYWIDAG threadbar ASTM A722 Grade 150

Structural Steel:

- Non-Frame Members: ASTM A572, Gr. 50
- Frame Members: ASTM A992 or ASTM A913 Grade 50

Design Loads:

Live Loads: Taken from table 1607.1 from IBC 2003

- Roof: No snow load
- Office Floor: 80 psf – Assume any spot could be a corridor at some point
- Partitions: 20 psf – Assume 10 psf for seismic calculations
- Plaza, Lobby, Corridors, Stairs: 100 psf
- Parking: 50 psf
- Loading Dock/Court: 250 psf
- Storage: 125 psf for light storage
- Mechanical Floor: assume 150 psf for equipment

Snow Load: p_g = ZERO for Oakland, California (Figure 1608.2 of IBC 2003)

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: Found using ASCE7-02 and IBC 2003

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 = 288', East and West face = 125
- Parapet at roof was ignored, and made into the top of the roof @ 306'

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	III	Table 1-1
Importance Factor: Iw	1.15	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)	288'	
Length Perpendicular to wind: B (feet)	125'	

	L/B 1.9	L/B 0.4
External Pressure (Cp)	E-W	N-S
Windward	0.8	0.8
Leeward	-0.3	-0.5

Velocity Pressure, q_z was calculated = $0.00256*k_z*k_{zt}*k_d*(V^2)*I$
Final Pressure, P was calculated = $q_z(GC_p)-q_i(GC_{pi})$

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	15.4	-12.5	15.6	28.1	-17.4	15.6	33.0
20	0.9	16.3	-12.5	16.2	28.7	-17.4	16.2	33.6
25	0.94	17.0	-12.5	16.7	29.2	-17.4	16.7	34.1
30	0.98	17.7	-12.5	17.2	29.7	-17.4	17.2	34.6
40	1.04	18.8	-12.5	18.0	30.5	-17.4	18.0	35.4
50	1.09	19.7	-12.5	18.6	31.1	-17.4	18.6	36.0
60	1.13	20.4	-12.5	19.1	31.6	-17.4	19.1	36.5
70	1.17	21.2	-12.5	19.6	32.1	-17.4	19.6	37.0
80	1.21	21.9	-12.5	20.0	32.5	-17.4	20.0	37.4
90	1.24	22.4	-12.5	20.4	32.9	-17.4	20.4	37.8
100	1.26	22.8	-12.5	20.7	33.2	-17.4	20.7	38.1
120	1.31	23.7	-12.5	21.3	33.8	-17.4	21.3	38.7
140	1.36	24.6	-12.5	21.9	34.4	-17.4	21.9	39.3
160	1.39	25.1	-12.5	22.3	34.8	-17.4	22.3	39.7
180	1.43	25.9	-12.5	22.7	35.2	-17.4	22.7	40.1
200	1.46	26.4	-12.5	23.1	35.6	-17.4	23.1	40.5
250	1.53	27.7	-12.5	24.0	36.5	-17.4	24.0	41.4
300	1.59	28.7	-12.5	24.7	37.2	-17.4	24.7	42.1
306	1.59	28.7	-12.5	24.7	37.2	-17.4	24.7	42.1

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

East/West: Base shear = V = 1446 kips
Overturning Moment = 237,323 ft-kips

North/South: Base Shear = V = 2921 kips
Overturning Moment = 481,381 ft-kips

See Appendix for Calculations

Seismic Loads:

The site of the building is in a very high seismic area. Looking at the ASCE tables for long and short period response, it was hard to determine exactly what S_s and S_l were. It was assumed that the building fell just on the outside of a major fault line. A better method of determining the spectral response would provide more accurate data.

The lateral system is very complex, and involves dual systems of eccentrically braced frames, moment frames, and special moment frames. 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

- S_s = 1.75 or 175%
- S_l = 0.75 or 75%
- Site Class: Assumed C
- Seismic Design Category: D
- Response Modification Factor: R=8 for dual system
- Cs = 0.183
- K = 1.48
- Total Load (W) = 56888 kips
- Seismic Base Shear (V) = 10411 kips
- Overturning Moment = 2220781 ft-kips

See Appendix for Calculations

When more is known about the distribution of shear forces, a more accurate calculation and comparison can occur.

<u>Results of Spot Checks</u>

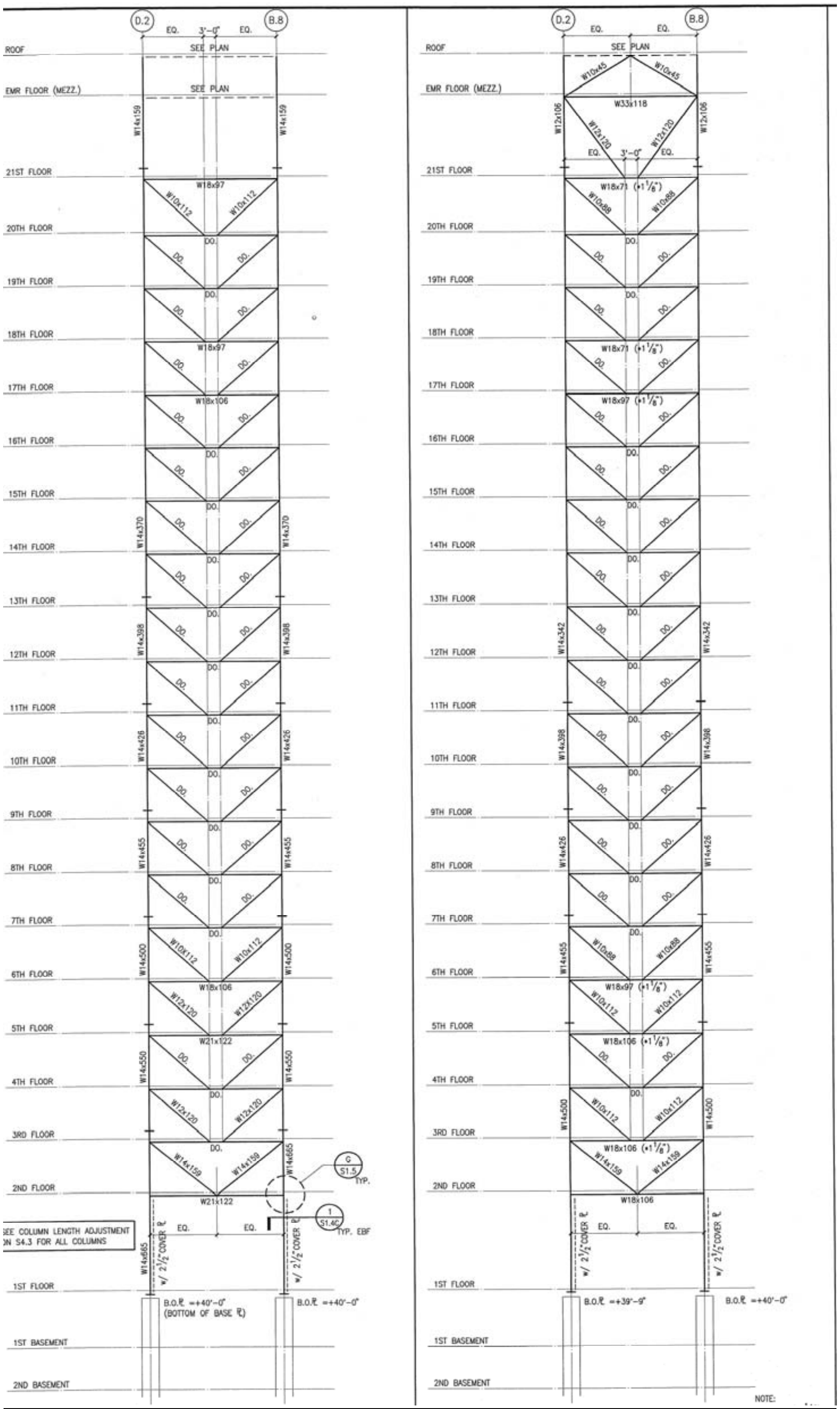
I conducted spot checks of a bay on the north face of the building, checking a composite beam, girder, and column. I also identified moments in the lateral SMRF on line 10. The purpose was to compare my loads on the building with what the designers used. I obtained the same size members for the exterior column on line 5, and composite beam that was located in the same bay. I obtained a slightly larger weight girder for the one checked, and slightly less moment in the moment frame than what was designed. The variations in size of the girder could be due to using different design references. They would provide different design dead and live loads, which could affect the data. The moment frame that was analyzed had small loads compared to the designed. The best explanation is that the incorrect percentage of shear was used. I used 8%, based on tributary area in the N-S direction.

<u>Looking into the future</u>

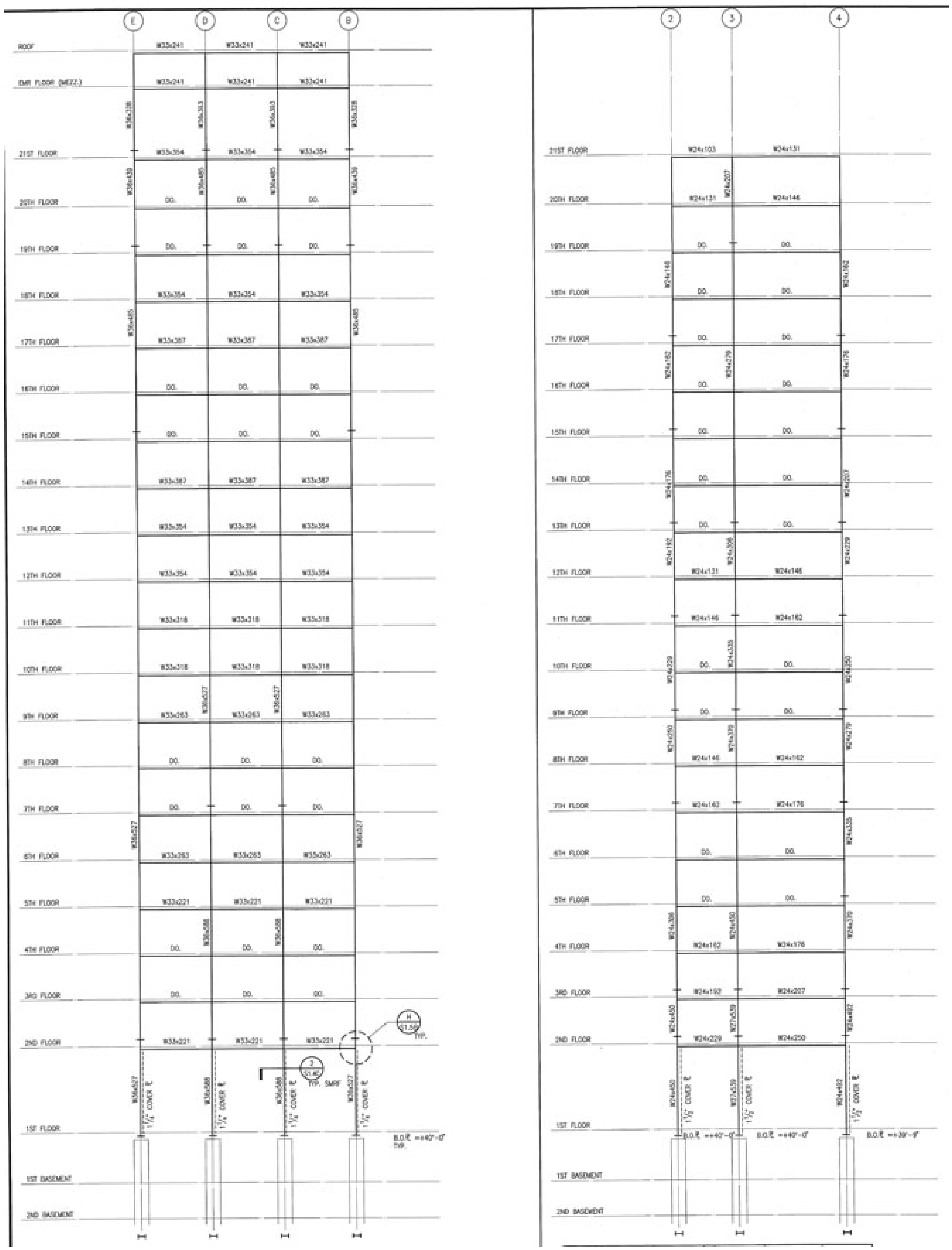
It is understood that other elements of the building will need to be looked at. Wind pressures have been calculated for walls, but will eventually need to be done for roofs. Basement walls and foundations will need to be checked for bearing capacity and soil uplift. The canopy on the North face of the building will need to be analyzed structurally. The façade support system will also need to be looked at. Lastly, the lateral system of this building will need to be looked at very closely to determine the exact distribution of loads to each element.

Appendix A: Lateral Resisting Frames

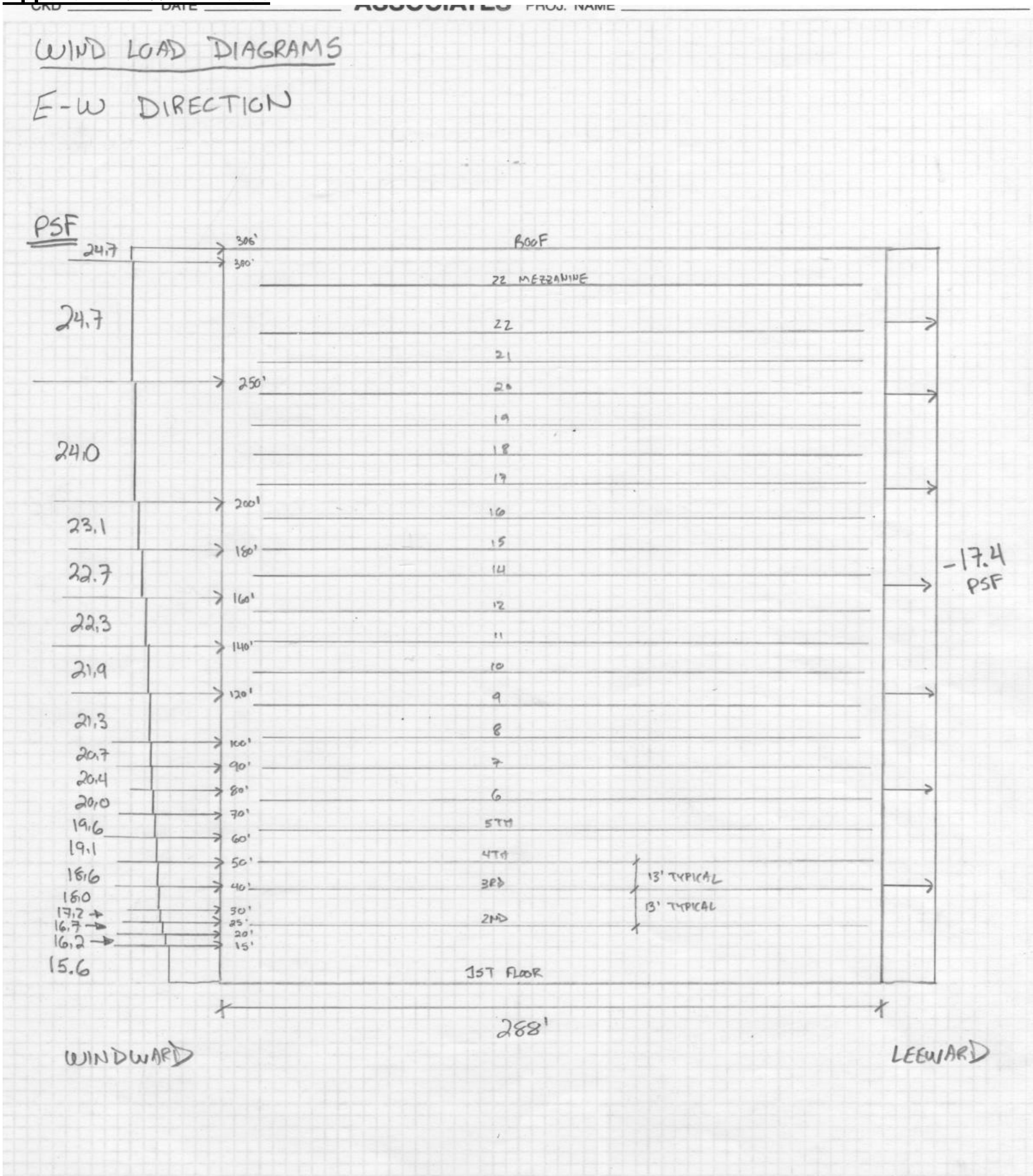
EBF Frames on interior core



SMRF Frames on Exterior Walls

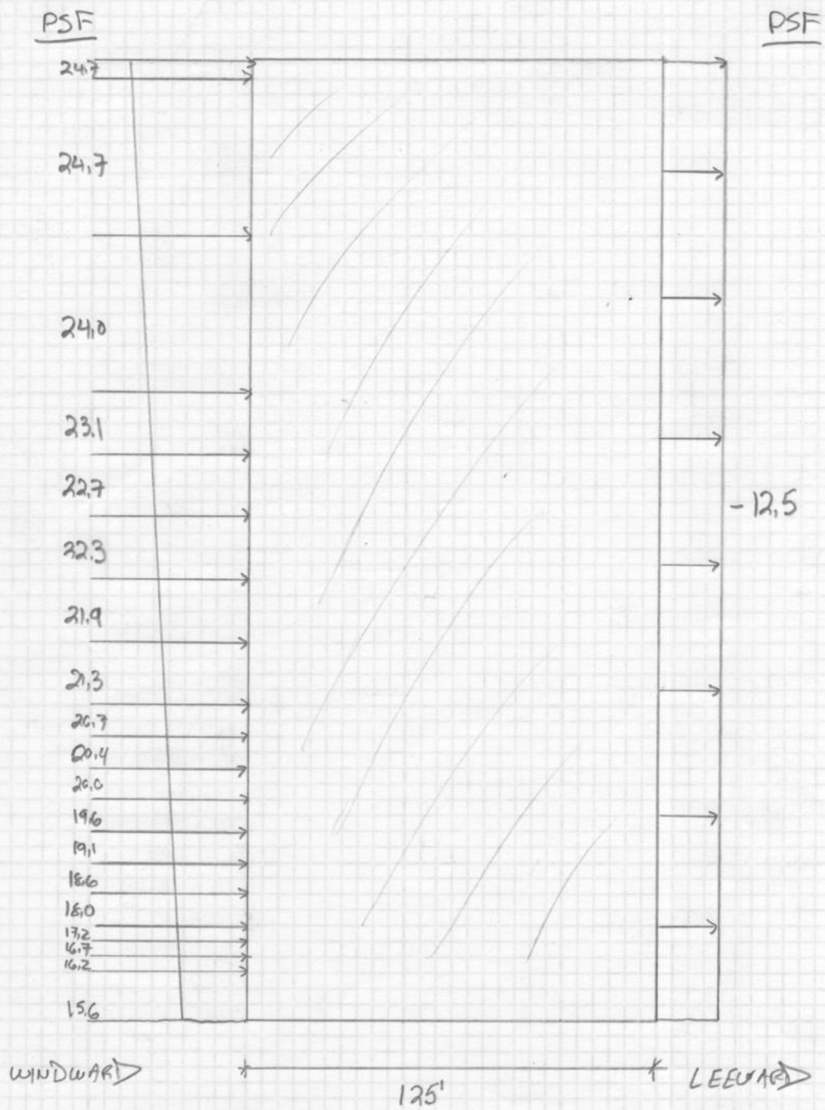


Appendix B: Wind Loads



WIND LOADING: NORTH-SOUTH DIRECTION

• PSF CONSTANT IN INTERVALS,



WIND LOAD FORCES AT EACH FLOOR (E-W)

- TRIBUTARY WIDTH AT EACH FLOOR × PSF WIND LOAD

FLOOR	HT		
2ND	24'	[5'(16.7 PSF) + 5'(16.2 PSF) + 3'(15.6 PSF) + 5'(17.2 PSF) + 0.5'(18 PSF)](125')	= 38.3^k
3RD	37	[6.5'(18.0 PSF) + 3'(18 PSF) + 3.5'(18.6 PSF)](125') =	= 30.0^k
4TH	50	[6.5'(18.6 PSF) + 6.5'(19.1 PSF)](125') =	= 30.6^k
5TH	63	[3.5'(19.1 PSF) + 3.0'(19.6) + 6.5'(19.6)](125') =	= 31.6^k
6TH	76	[0.5(19.6) + 10'(20.0 PSF) + 2.5'(20.4 PSF)](125') =	= 32.6^k
7TH	89	[7.5'(20.4 PSF) + 5.5'(20.7 PSF)](125') =	= 33.4^k
8TH	102	[4.5'(20.7 PSF) + 8.5'(21.3 PSF)](125') =	= 34.3^k
9TH	115	[11.5'(21.3 PSF) + 1.5'(21.9 PSF)](125') =	= 34.7^k
10TH	128	[13'(21.9 PSF)](125') =	= 35.6^k
11TH	141	[5.5'(21.9 PSF) + 7.5'(22.3 PSF)](125') =	= 36.0^k
12TH	154	[12.5'(22.3 PSF) + 0.5(22.7 PSF)](125') =	= 36.3^k
14TH	167	[13'(22.7 PSF)](125') =	= 36.9^k
15TH	180	[6.5'(22.7 PSF) + 6.5(23.1 PSF)](125') =	= 37.2^k
16TH	193	[13'(23.1 PSF)](125') =	= 37.5^k
17TH	206	[0.5'(23.1 PSF) + 12.5'(24.0 PSF)](125') =	= 38.9^k
18TH	219	[13'(24.0 PSF)](125') =	= 39.0^k
19TH	232	" " " =	= 39.0^k
20TH	245	[11.5'(24.0 PSF) + 1.5(24.7 PSF)](125') =	= 39.1^k
21ST	258	[13.5'(24.7 PSF)](125') =	= 41.7^k
22ND	272	[17'(24.7 PSF)](125') =	= 52.5^k
MEZZANINE	292	[17'(24.7 PSF)](125') =	= 52.5^k
ROOF	306	[7'(24.7 PSF)](125') =	= 21.6^k

TOTAL BASE SHEAR = $\boxed{756.8^k}$
FROM WINDWARD

CKD ______ DATE ____________ ASSOCIATES PROJ. NAME ______________________

WIND LOAD FORCES AT EACH FLOOR (N-S)

- SAME AS EAST-WEST FORCES, BUT DIFFERENT BUILDING WIDTH.

USE 288' INSTEAD OF 125' : $\frac{288}{125} = 2.304$

FLOOR	
2	$38.3^k\left(\frac{288'}{125'}\right) = 88.2^k$
3	$30.0^k(2.304) = 69.1^k$
4	$30.6^k(2.304) = 70.5^k$
5	$31.6\ (2.304) = 72.8^k$
6	$32.6\ (2.304) = 75.1^k$
7	$33.4\ (2.304) = 77.0^k$
8	$34.3\ (2.304) = 79.0^k$
9	$34.7\ (2.304) = 79.9^k$
10	$35.6\ (2.304) = 82.0^k$
11	$36.0\ (2.304) = 82.9^k$
12	$36.3\ (2.304) = 83.6^k$
14	$36.9\ (2.304) = 85.0^k$
15	$37.2\ (2.304) = 85.7^k$
16	$37.5\ (2.304) = 86.4^k$
17	$38.9\ (2.304) = 89.6^k$
18	$39.0\ (2.304) = 89.9^k$
19	$39.0\ (2.304) = 89.9^k$
20	$39.1\ (2.304) = 90.1^k$
21	$41.7\ (2.304) = 96.1^k$
22	$52.5\ (2.304) = 121.0^k$
MEZZANINE	$52.5\ (2.304) = 121.0^k$
ROOF	$21.6\ (2.304) = 49.8^k$

TOTAL BASE SHEAR = $\boxed{1864.6^k}$

BASE SHEAR FROM LEEWARD PRESSURES

E-W DIRECTION

FLOOR	
2	(12'+6.5')(-17.4 PSF)(125') = 40237.5 = 40.2^k = 40.2^k
3-21	(13')(-17.4 PSF)(125') = 28275 = 28.2^k (18) = 507.6^k
22, MEZ,	(17')(-17.4 PSF)(125') = 36975 = 37.0^k (2) = 74^k
ROOF	(7')(-17.4 PSF)(125') = 15225 = 15.2^k = 15.2^k

BASE SHEAR FROM LEEWARD $\boxed{637^k = V}$

N-S DIRECTION

FLOOR	
2	(18.5')(-12.5 PSF)(288') = 66600 = 66.6^k
3-21	(13')(-12.5 PSF)(288') = 46800 (18) = 842.4^k
22, MEZ,	(17')(-12.5 PSF)(288') = 61200 (2) = 122.4^k
ROOF	(7')(-12.5 PSF)(288') = 25200 = 25.2^k

BASE SHEAR FROM LEEWARD $\boxed{1057^k = V}$

- COMBINE WINDWARD AND LEEWARD FOR EACH DIRECTION TO GET TOTAL SHEAR AT EACH FLOOR, AND TOTAL BASE SHEAR

East - West Floor	Story Height	windward Shear	leeward shear	Total Shear	Overturning Moment
2	24	38.3	40.2	78.5	1884
3	37	30	28.2	58.2	2153.4
4	50	30.6	28.2	58.8	2940
5	63	31.6	28.2	59.8	3767.4
6	76	32.6	28.2	60.8	4620.8
7	89	33.4	28.2	61.6	5482.4
8	102	34.3	28.2	62.5	6375
9	115	34.7	28.2	62.9	7233.5
10	128	35.6	28.2	63.8	8166.4
11	141	36	28.2	64.2	9052.2
12	154	36.3	28.2	64.5	9933
14	167	36.9	28.2	65.1	10871.7
15	180	37.2	28.2	65.4	11772
16	193	37.5	28.2	65.7	12680.1
17	206	38.9	28.2	67.1	13822.6
18	219	39	28.2	67.2	14716.8
19	232	39	28.2	67.2	15590.4
20	245	39.1	28.2	67.3	16488.5
21	258	41.7	28.2	69.9	18034.2
22	272	52.5	37	89.5	24344
mezzanine	292	52.5	37	89.5	26134
roof	306	21.6	15.2	36.8	11260.8
TOTALS		809.3	637	1446.3	237323.2

North/South Floor	Story Height	windward Shear	leeward shear	total shear	Overturning Moment
2	24	88.2	66.6	154.8	3715.2
3	37	69.1	46.8	115.9	4288.3
4	50	70.5	46.8	117.3	5865
5	63	72.8	46.8	119.6	7534.8
6	76	75.1	46.8	121.9	9264.4
7	89	77	46.8	123.8	11018.2
8	102	79	46.8	125.8	12831.6
9	115	79.9	46.8	126.7	14570.5
10	128	82	46.8	128.8	16486.4
11	141	82.9	46.8	129.7	18287.7
12	154	83.6	46.8	130.4	20081.6
14	167	85	46.8	131.8	22010.6
15	180	85.7	46.8	132.5	23850
16	193	86.4	46.8	133.2	25707.6
17	206	89.6	46.8	136.4	28098.4
18	219	89.9	46.8	136.7	29937.3
19	232	89.9	46.8	136.7	31714.4
20	245	90.1	46.8	136.9	33540.5
21	258	96.1	46.8	142.9	36868.2
22	272	121	61.2	182.2	49558.4
mezzanine	292	121	61.2	182.2	53202.4
roof	306	49.8	25.2	75	22950
TOTALS		1864.6	1056.6	2921.2	481381.5

Appendix C: Seismic Calculations

SEISMIC ANALYSIS

• USE IBC 2003 AND ASCE 7-02 - SECTION 8

OAKLAND, CALIFORNIA

S_S = 0.2s SPECTRAL RESPONSE = ≈175% = 1.75

S_1 = 1.0s SPECTRAL RESPONSE = ≈ 75% = 0.75

→ VALUES FROM MAPS IN ASCE 7-02 ; ASSUMED BUILDING LAY JUST OFF FAULT LINES.

BUILDING SITE CLASS: C

BUILDING OCCUPANCY: OFFICE ⇒ TYPE III ⇒ SEISMIC USE GROUP II

$I = 1.25$ 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 $S_1 \geq 0.5$

$S_{MS} = F_a S_S = 1.0(1.75) = 1.75 \Rightarrow S_{DS} = \frac{2}{3}(1.75) = 1.17$

$S_{M1} = F_v S_1 = 1.3(0.75) = 0.975 \Rightarrow S_{D1} = \frac{2}{3}(0.975) = 0.65$

SEISMIC DESIGN CATEGORY FROM $S_{DS} \Rightarrow$ D

$S_{D1} \Rightarrow$ D

RESPONSE MODIFICATION FACTORS

N-S: DUAL SYSTEM: SPECIAL MOMENT FRAMES AND ECCENTRIC BRACED FRAMES

E-W: DUAL SYSTEM $\underline{\underline{R=8}}$, $\Omega_0 = 2.5$, $C_d = 4.0$

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

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

100.8PSF ≈ 100 PSF DEAD LOAD

TYPICAL FLOOR DEAD LOADS

3" METAL DECK w/ 2½" CONCRETE = 68.8 PSF
MEP = 10 PSF
DECK = 2 PSF
STRUCTURAL = 15 PSF
5 PSF

100.8 PSF ≈ **100 PSF**

2ND FLOOR

3" METAL 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" METAL DECK w/ 6" CONCRETE ⇒ 150(6/12) = 75 PSF
10
2
15
5

107 ≈ **110 PSF**

22ND FLOOR - MECHANICAL

3" METAL DECK w/ 6" CONCRETE ⇒ 150(6/12) = 75 PSF
6" CONCRETE PADS FOR MECHANICAL = 75 PSF × AREA OF 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 OF ASCE 7-02)

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FIND WEIGHT OF FLOORS

ROOF: GROSS AREA = 13500 ft^2 > 13500(100) = 1350000 lb = $\underline{1350^k}$

LOADING = 100 PSF

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

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

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

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

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

TOTAL LOAD = 2ND + TYPICAL 3-20 (EXCLUD. 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 = $V = C_s W$

$$C_s = \frac{S_{DS}}{R/I} = \frac{1.17}{8/1.25} = 0.183 \Rightarrow V = .183(56888^k) = \boxed{10411^k = V}$$

VERTICAL DISTRIBUTION OF FORCES

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

$h_n = 306'$

$$T_a = .02(306)^{.75} = 1.46s$$

$0.5 < T_a = 1.46 < 2.5 \Rightarrow$ INTERPOLATE FOR k (BETWEEN 1 AND 2)

$\underline{k = k = 1.48}$

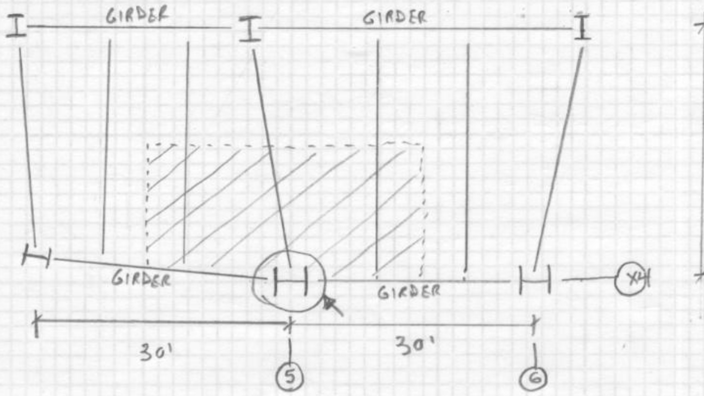
Vertical Distribution of Seismic Forces

Level	w_x	h_x	$w_x h_x^k$	C_{vx}	F_x (kips)	M_x (ft-kips)
roof	1350	306	6444682	0.0586	610.0	186648
21	1822	292	8115495	0.0738	768.1	224284
20	2971	272	11914193	0.1083	1127.6	306714
19	2872	258	10650785	0.0968	1008.0	260076
18	2633	245	9045169	0.0822	856.1	209741
17	2633	232	8343977	0.0759	789.7	183215
16	2633	219	7661400	0.0696	725.1	158801
15	2633	206	6998007	0.0636	662.3	136440
14	2633	193	6354416	0.0578	601.4	116073
13	2633	180	5731314	0.0521	542.4	97640
12	2633	167	5129455	0.0466	485.5	81075
11	2633	154	4549687	0.0414	430.6	66314
10	2633	141	3992962	0.0363	377.9	53286
9	2633	128	3460363	0.0315	327.5	41921
8	2633	115	2953137	0.0268	279.5	32143
7	2633	102	2472744	0.0225	234.0	23871
6	2633	89	2020915	0.0184	191.3	17023
5	2633	76	1599761	0.0145	151.4	11507
4	2633	63	1211923	0.0110	114.7	7226
3	2633	50	860848	0.0078	81.5	4074
2	2633	37	551301.8	0.0050	52.2	1931
1	3112	24	343362	0.0031	32.5	780
SUMS	56888		1.1E+08	1.0	10449.4	2220781

Appendix D: Spot Checks

SPOT CHECK: COLUMN (X4, 5) ON NORTH FACE, 1ST FLOOR

AS DESIGNED; W14x426 - EXTERIOR GRAVITY COLUMN



TRIBUTARY AREA:

$30' \times 43\frac{1}{2}$

$30' \times 21.5' = 645\ ft^2 = A_T$

TOTAL AREA OF ALL FLOORS ABOVE (20 FLOORS)

$K_{LL} = 4$ FOR COLUMN

LOADING: 2ND FLOOR: $LL = 80 + 20 = 100$ PSF

$DL =$ CALCULATED PREVIOUSLY $= 120$ PSF

3RD–20TH: $LL = 100$ PSF

$DL = 100$ PSF - CALCULATED PREVIOUSLY

21ST: $LL = 100$ PSF

$DL = 110$ PSF - CALCULATED PREVIOUSLY

22ND - MECH: $LL = 150$ PSF

$DL = 110$ PSF

TOTAL AREA: $A_T = 645\ ft^2 (20\ \text{floors}) = 12900\ ft^2$

$A_I = 4(12900) = 51600\ ft^2$

$LL = L_o\left(0.25 + \frac{15}{\sqrt{A_I}}\right) \Rightarrow L_o\left(0.25 + \frac{15}{\sqrt{51600}}\right) = 0.316 < 0.4$ FOR COLUMNS

USE $0.4 L_o$

DL @ 1ST FLOOR $= (110\ PSF \times 645\ ft^2 (2)) + (645\ ft^2 \times 120\ PSF) + (10965\ ft^2 \times 100\ PSF)$

$DL = 1315800\ lb = 1316^k$

$LL = 0.4(100\ PSF)(12255) + 0.4(150\ PSF)(645\ ft^2) = 528900 = 529^k$

LOAD COMBINATIONS

$1.2D + 1.6L = 1.2(1316) + 1.6(529) = 2426^k$ TOTAL LOAD WILL CONTROL

$1.4D = 1.4(1316) = 1842^k$

SPOT CHECK CONTINUED FOR COLUMN

FOR LOAD OF $P = 2426^k$, GOTO LRFD MANUAL, TABLE 4-2

ASSUME $KL = 37'$, WITH $K = 1.0$

A W14x426 WITH EFFECTIVE LENGTH $= 37'$, HAS A DESIGN AXIAL STRENGTH OF 2470^k.

$$\phi P_n = 2470^k > 2426^k \therefore \text{OK.}$$

- THIS IS THE SAME SIZE COLUMN THAT WAS DESIGNED FOR THE BUILDING, W14x426.

SPOT CHECK OF COMPOSITE BEAM : FLOOR BEAM AT LEVEL 4
BETWEEN (5+6) ON DRAWINGS (NORTH FACE)

- ORIGINALLY CALLED OUT AS A W16×55, SPANNING 43'-0", SPACED 10' O.C.

LOADS

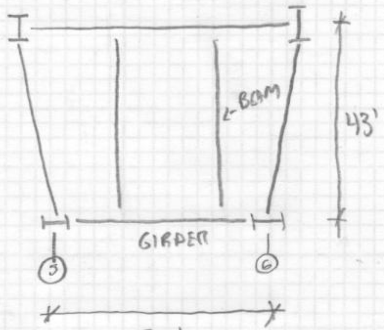
DL = 100PSF ON 4TH FLOOR

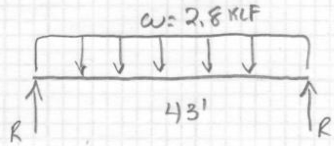
LL = 100PSF OFFICE

LOAD COMBINATIONS

$1.2D + 1.6L = 1.2(100PSF)(10') + 1.6(100PSF)(10')$

$= 1200 + 1600 = 2800$ PLF $= 2.8$ KLF $= w_u$





$R = \frac{2.8(43')}{2} = 60.2^k$ $\quad M_{max} = \frac{2.8(43)^2}{8} = \underline{\underline{647'k}}$

USE LRFD + AISC COMPOSITE STEEL MANUAL

ASSUME THAT $a = 1'' \Rightarrow Y2 = 5.5 - \frac{1}{2} = 5.0''$

TRY W18×55 w/ $\phi M_n = 665'k$ @ BFL , $\Sigma Q_n = 336$

$b_{eff} = 1/4$ SPAN $= 1/4(43') = 10.75'(12) = 129''$

$=$ spacing $= 10'(12) = 120''$ ⇐ CONTROLS

$336 = 0.85(3)(120)a \Rightarrow a = 1.09 > 1.0 \Rightarrow$ UNCONSERVATIVE

TRY $a = 2'' \Rightarrow Y2 = 5.5 - 1 = 4.5''$

TRY W18×55, $\phi M_n = 652'k$, @ BFL 2 , $\Sigma Q_n = 336^k$

$336 = 0.85(3)(120)a \Rightarrow a = 1.09 < 2.0$ ∴ $\underline{\underline{OK}}$ CONSERVATIVE

Assume $Q_n = 21^k$/STUD $\Rightarrow \frac{336}{21} = 16 \times 2 = 32$ SHEAR STUDS NEEDED
3/4" Φ

A W18×55 w/ 32 3/4" Φ SHEAR STUDS WORKS. IT IS COMPARABLE TO THE DESIGNED W18×55 w/ 36 shear studs.

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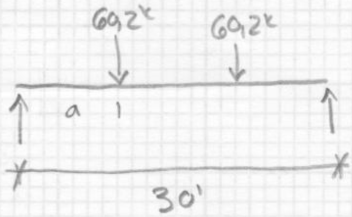
CHECK GIRDER ON WHICH COMPOSITE BEAM SITS.

- SAME DIAGRAM AS PREVIOUS
- GIRDER IS DESIGNED AS A W24x55 CARRYING TWO POINT LOADS FROM BEAMS.



$M_{max} = Pa = 60.2^k(10') = \underline{602'k}$

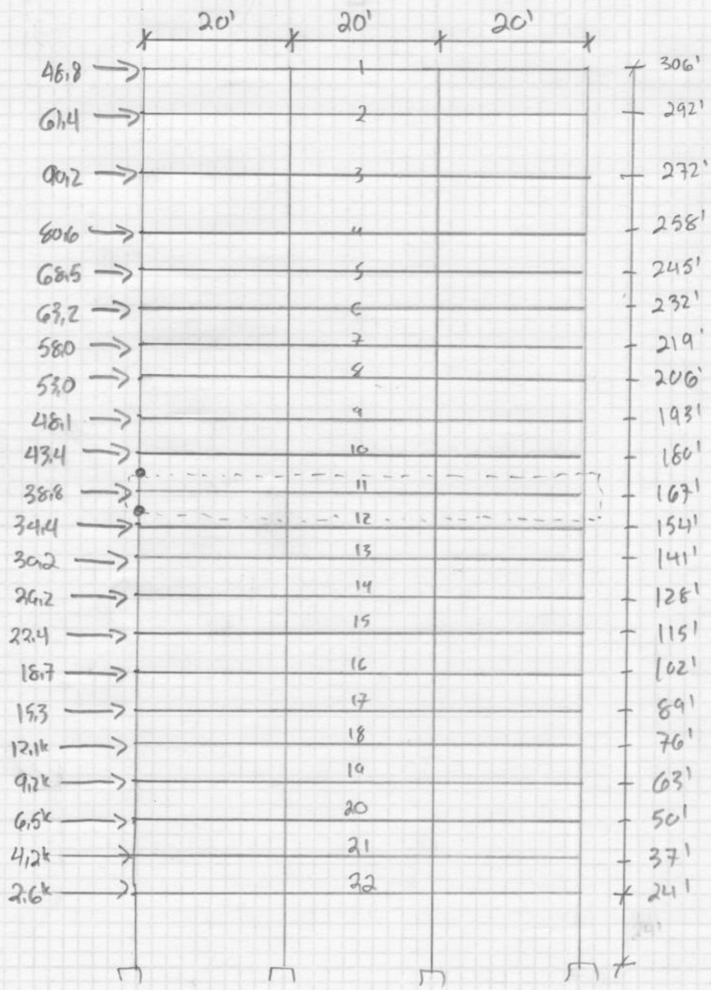
TRY $\underline{W24\times68}$ w/ $\phi M_n = \underline{664'k}$

THIS GIRDER IS SLIGHTLY LARGER THAN THE DESIGNED. POSSIBLE THAT LIVE LOAD REDUCTIONS NEED TO BE TAKEN INTO ACCOUNT.

SPOT CHECK LATERAL SYSTEM: SMRF ON LINE 10 (N-S)

SEISMIC CONTROLS: TRY PORTAL METHOD

ASSUME BAYS ARE ALL 20' (SOME ARE 19.5')

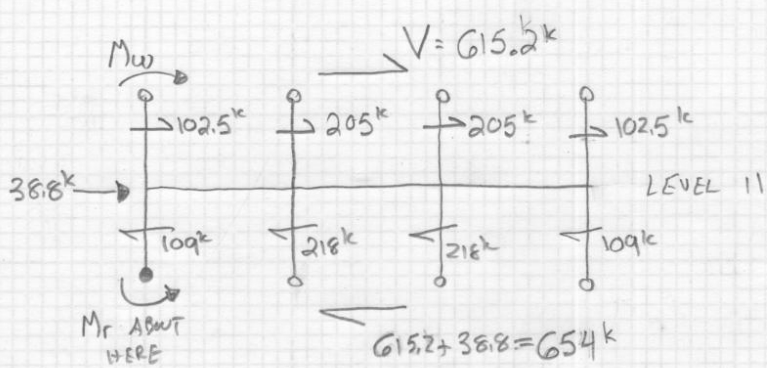


CHECK LEVEL 11

ASSUME FRAME TAKES

$$\left(\frac{18'}{220'}\right) = .08 = 8\% \text{ LATERAL LOAD}$$

- ALL LOADS FROM SEISMIC CALC ARE NOW 8% ONTO THIS MOMENT FRAME, AND ARE DRAWN TO CORRESPONDING FLOORS ON THE LEFT



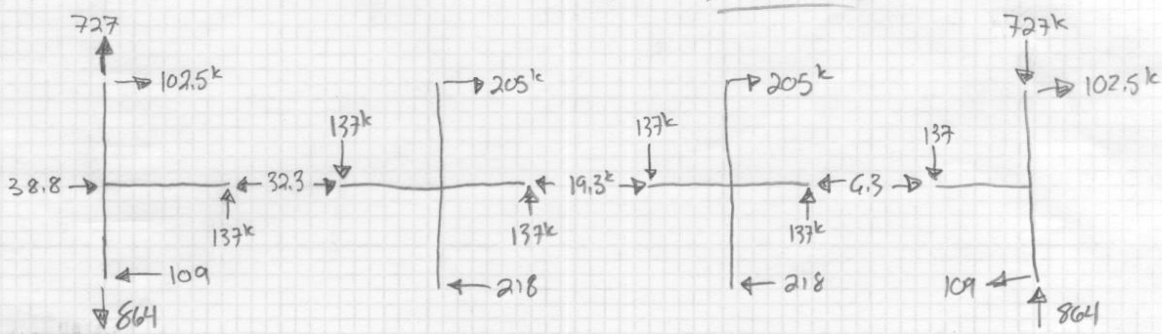
$$V = 48.8 + 61.4 + 90.2 + 80.6 + 68.5 + 63.2 + 58.0 + 53.0 + 48.1 + 43.4 = 615.2^k$$

OVERTURNING MOMENTS

$$M_w = 43.4\left(\tfrac{13}{2}\right) + 48.1(6.5+13) + 53.0(19.5+13) + 58.0(32.5+13) + 63.2(45.5+13) +$$
$$+\ 68.5(71.5') + 80.6(84.5') + 90.2(98.5') + 61.4(118.5') + 48.8(132.5')$$

$$M_w = \underline{43613.8\ 'k}$$

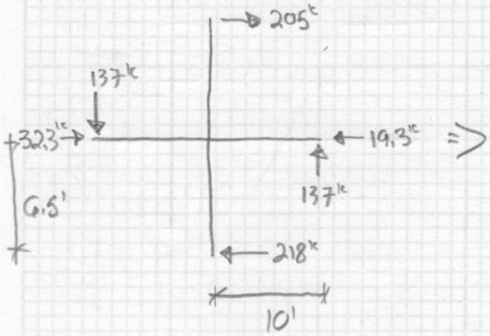
$$M_r = 43613.8 + 38.8(6.5) + 615.2(13') = \underline{51863.6\ 'k}$$

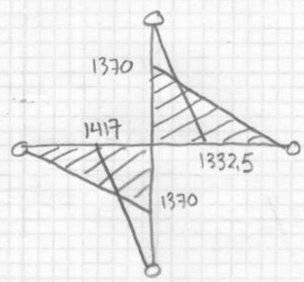


$$\frac{43613.8'k}{20'(3)} = \underline{727^k} \text{ (FROM } M_w)$$

$$\frac{51863.6'k}{20'(3)} = \underline{864^k} \text{ (FROM } M_r)$$

CHECK MOMENT FOR 2ND COLUMN IN





MOMENT DIAGRAM

137k(10') = 1370'k
205k(6.5') = 1332.5'k
218k(6.5') = 1417'k

BEAMS DESIGNED AS W33x354 AT LEVEL 11 ϕM_n = 5330'k
COLUMNS DESIGNED AS W36x527 AT LEVEL 11 ϕM_n = 8550'k

MY DESIGN ONLY REQUIRES 1370'k FOR BEAM, AND 1417'k FOR COLUMN,
THIS WOULD ONLY GIVE ME A W33x118 FOR BEAM