TOWER 333

Technical Assignment 1

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Paul Parfitt

AE 481W - 5th Year Thesis
Pennsylvania State University

Faculty Advisor: Andres Lepage

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Executive Summary:

The purpose of this paper is to understand the existing conditions and design procedures for Tower 333 in Bellevue, Washington.

Building Description:

Tower 333 is an 18 story office building on 333 108th Avenue in Bellevue Washington. The engineers decided to use a performance based lateral system, this allowed them to utilize the existing foundation and core of a previous project that was abandoned. This decision saved considerable time and money in the excavation and foundation process. Having highly transparent 10 foot high windows allows maximum light penetration and a view of the Lake Washington framed in the Olympic Mountains. This, coupled with state of the art operation systems, column free open plan and drought resistant vegetation located in the ½ acre plaza qualifies it for LEED certification. The first floor will contain retail and professional services, while floors 2-18 are designated for office use. In addition to the 18 levels above grade, Tower 333 contains 8 levels of below grade parking with the entrance on the lower mezzanine level.

Figure 1.1 Tower 333 Satellite Imagery

Design Code:

IBC 2003 with reference to ASCE-7 ’02

I have used the current IBC 2005 which references ASCE-7 ’05 for my calculations and design of Tower 333. The discrepancies between these two codes could account for some differences in sizes and loads attained. I also have used the Thirteenth Edition of the AISC Steel Construction Manual which could also account for design discrepancies between myself and the engineer. It is also worthy to note that in this report I used a simplified approach to attaining loads and might not have made all the full assumptions that the engineer had, thus resulting in smaller design sizes. In no way does this report make the claim that any of the designer’s approaches, assumptions, calculations or resulting designs are incorrect or unsuitable.

Structural System:

Foundation:

Tower 333's foundation consists of a previous, abandoned building's existing foundation. Plans indicate that sub levels 8-5 were completed before the project was abandoned. The existing foundation consists of spread concrete footings. Where designated, these footings were either demolished or partially demolished and replaced or thickened to provide higher capacity. Where the footings are reinforced, rebar was drilled and grouted into the bottom of the footings. The foundation supporting the concrete core shear walls is a mat slab foundation with a new topping applied to the existing mat.

Floor System:

The sublevel floor systems from P8-P2 are a 7-1/2 inch, 2-way post-tensioned concrete slab with an f'c = 5,500psi. Parking Level 1 has a one way concrete slab varying in thickness from 10-12" with #5 bars in the bottom of the slab and #6 in the top with an f'c of 5,000psi. Supporting the one way slab are 48x27 concrete girders, f'c = 5,000psi, spanning 55'-10" in the N-S direction of a typical bay. The upper floors are a 2-1/2 inch concrete slab on a 3 inch deep metal composite deck with an f'c of 4,000psi and WWF 6x6 W3.5xW3.5 reinforcing. Supporting the slab are W18x40 beams which span 42 feet N-S in a typical bay. The beams frame into girders on the interior which are typically W18x97 spanning E-W while beams framing into girders on the exterior of the bays are either W18x40 or specialized moment frames.

Columns:

Columns on the sublevels are concrete with an f'c = 8,000 psi. A typical column size is 2'x2' with (12) # 8 bars tied with #5 at either 4" or 6" spacing. Columns on the north and south exterior are 3'x3' with (16) #18 bars and #5 ties spaced 4" and 6".

The columns beginning at the mezzanine level are rolled W14 shapes with an Fy of 50ksi and continue to the full height of 260 ft to the top of the building. The typical gravity columns range in size from a W14x53 to a W14x500. At the moment frame locations the columns range in size from W14x132 to a W14x730. Both gravity and moment frame columns are spliced every 28feet at mid floor. The maximum unbraced length is 13'-10" which is the typical floor to floor height of one floor.

Lateral Framing:

There is a dual, performance-based lateral system implemented in Tower 333 consisting of special moment frames at selected locations on the exterior walls and a centralized core of special shear walls. It is assumed that the moment frame is capable of taking at least 25 percent of the seismic lateral loads. The columns are welded with ¾ inch fillet welds to L4x4x5/8 angles on each side which are then welded to the base plates with ¾ inch fillet welds. Typical base plates are 3-1/2x26x32 for a three-bayed frame and 3-1/2x32x32 for a two-bayed frame. Kicker braces are also applied to the moment frame beams where they are not braced by incoming floor beams.

The core concrete shear walls are 2feet thick with a length of 40feet in the North-South direction and 32feet long with 5feet openings for elevator access in the E-W direction. The bearing capacity of the concrete is f'c = 9,000 psi with two curtains of #7 rebar at 12 inch spacing and #5 hoops and ties at 6 inch spacing.

Codes:

Building Code:

International Building Code (IBC), 2003 edition

Structural Concrete:

American Concrete Institute (ACI) 2003 edition

Steel Design:

American Institute of Steel Construction LRFD (AISC), 1999 edition

AISC Seismic Provisions 2002 edition

AISC Specification of Structural Joints 2000 edition

Building Design Loads:

American Society of Civil Engineers (ASCE-7) 2002 edition

Loads:

Dead Loads:

Metal Deck + Normal Weight Concrete	50 PSF	(Vulcraft Catalog)
Steel Beams	Varies	AISC

Superimposed Dead Loads: ASCE-7

Office:	
Mechanical/Electrical/Sprinkler	5 PSF
Partitions	20 PSF
Lobby/Circulation:	
Mechanical/Electrical/Sprinkler	15 PSF
Partitions:	20 PSF
Built-Up Slabs:	75 PSF (where applicable)
Pavers, Topping Slabs:	35 PSF (where applicable)
Retail/Restaurant:	
Mechanical/Electrical/Sprinkler	15 PSF
Plaza & Vegetation:	
Mechanical/Electrical/Sprinkler	15 PSF
Finishes/Waterproofing	15 PSF
Soil/Plantings	150 PSF (where applicable)
Parking:	
Mechanical/Electrical/Sprinkler	5 PSF
Roof:	
Mechanical/Electrical/Sprinkler	15 PSF

Live Loads ASCE-7 Chapter 4

Office:	50 PSF
Lobby:	100 PSF (NR)
Retail/Restaurant:	100 PSF (NR)
Plaza (Assembly):	100 PSF (NR)
Parking:	50 PSF
Roof (live load or snow):	25 PSF

Lateral Loads:

Wind: **In accordance with ASCE-7 Chapter 6**

The wind pressures and loads shown below are for a flexible building with an exposure category B.

Pressure				
Wind From N-S				
Windward		**Leeward**		**Total**
h (ft)	**P (psf)**	**h (ft)**	**P (psf)**	
0-15	9.70	0-15	-10.06	19.76
20	10.23	20	-10.06	20.28
25	10.65	25	-10.06	20.71
30	11.07	30	-10.06	21.13
40	11.71	40	-10.06	21.76
50	12.24	50	-10.06	22.29
60	12.66	60	-10.06	22.71
70	13.08	70	-10.06	23.14
80	13.50	80	-10.06	23.56
90	13.82	90	-10.06	23.88
100	14.14	100	-10.06	24.19
120	14.67	120	-10.06	24.72
140	15.19	140	-10.06	25.25
160	15.62	160	-10.06	25.67
180	16.04	180	-10.06	26.10
200	16.36	200	-10.06	26.41
250	17.20	250	-10.06	27.26
267	17.41	267	-10.06	27.47
Wind From E-W				
Windward		**Leeward**		**Total**
h (ft)	**P (psf)**	**h (ft)**	**P (psf)**	
0-15	9.77	0-15	-10.22	19.99
20	10.31	20	-10.22	20.53
25	10.73	25	-10.22	20.95
30	11.16	30	-10.22	21.38
40	11.80	40	-10.22	22.02
50	12.34	50	-10.22	22.56
60	12.77	60	-10.22	22.98
70	13.19	70	-10.22	23.41
80	13.62	80	-10.22	23.84
90	13.94	90	-10.22	24.16
100	14.26	100	-10.22	24.48
120	14.80	120	-10.22	25.02
140	15.33	140	-10.22	25.55
160	15.76	160	-10.22	25.98
180	16.19	180	-10.22	26.41
200	16.51	200	-10.22	26.73
250	17.36	250	-10.22	27.58
267	17.58	267	-10.22	27.80

Table 3.1 Wind Pressures

Wind cont.:

Total Base Shear:

N-S Direction: 1365 kips
E-W Direction: 807 kips

Total Overturning Moment:

N-S Direction: 195,908 ft-kips
E-W Direction: 115,039 ft-kips

Seismic Load:

Although this building is located in a seismic sensitive region, with a site class designation C and a seismic design category designation D, the simplified analysis was approached in this report to confirm that seismic loading will be the controlling load. Once this is confirmed, a further and more in-depth analysis will be done at a later time.

Seismic Loading						
	V= 1774					
K=1.4	Level	w_x	h_x	$w_x h_x^{1.4}$	C_{vx} (k)	Fx (k)
Roof	20	156	267.61	390506.1	0.008	14.57
Penthouse	19	2490	253.78	5786816.3	0.122	215.94
Office	18	2490	239.95	5350180.1	0.113	199.65
Office	17	2490	226.12	4923499.7	0.104	183.73
Office	16	2490	212.29	4507136.4	0.095	168.19
Office	15	2490	198.46	4101488.8	0.086	153.05
Office	14	2490	184.63	3706999.5	0.078	138.33
Office	13	2490	170.8	3324163.4	0.070	124.04
Office	12	2490	156.97	2953538.7	0.062	110.21
Office	11	2490	143.14	2595761.4	0.055	96.86
Office	10	2490	129.31	2251564.0	0.047	84.02
Office	9	2490	115.48	1921802.8	0.040	71.71
Office	8	2490	101.65	1607495.2	0.034	59.99
Office	7	2490	87.82	1309875.9	0.028	48.88
Office	6	2490	73.99	1030484.8	0.022	38.45
Office	5	2490	60.16	771312.4	0.016	28.78
Office	4	2490	46.33	535065.0	0.011	19.97
Office	3	2490	32.5	325713.7	0.007	12.15
Office	2	2432	18.67	146408.3	0.003	5.46
Lobby	1	3442	0	0	0	0

Table 3.2 Seismic Distribution of Forces

Total Base Shear: 1774 kips
*Total Overturning Moment:*327,640 ft-kips

Other Loads:

The foundation and walls of Tower 333 drop 93 feet below grade, with soil nailing of the exterior walls below grade. However, soil loads, lateral pressures created by the adjacent soil and snow loads (ASCE-7 Chapter 7) are not covered in the scope of this report.

Spot Checks:

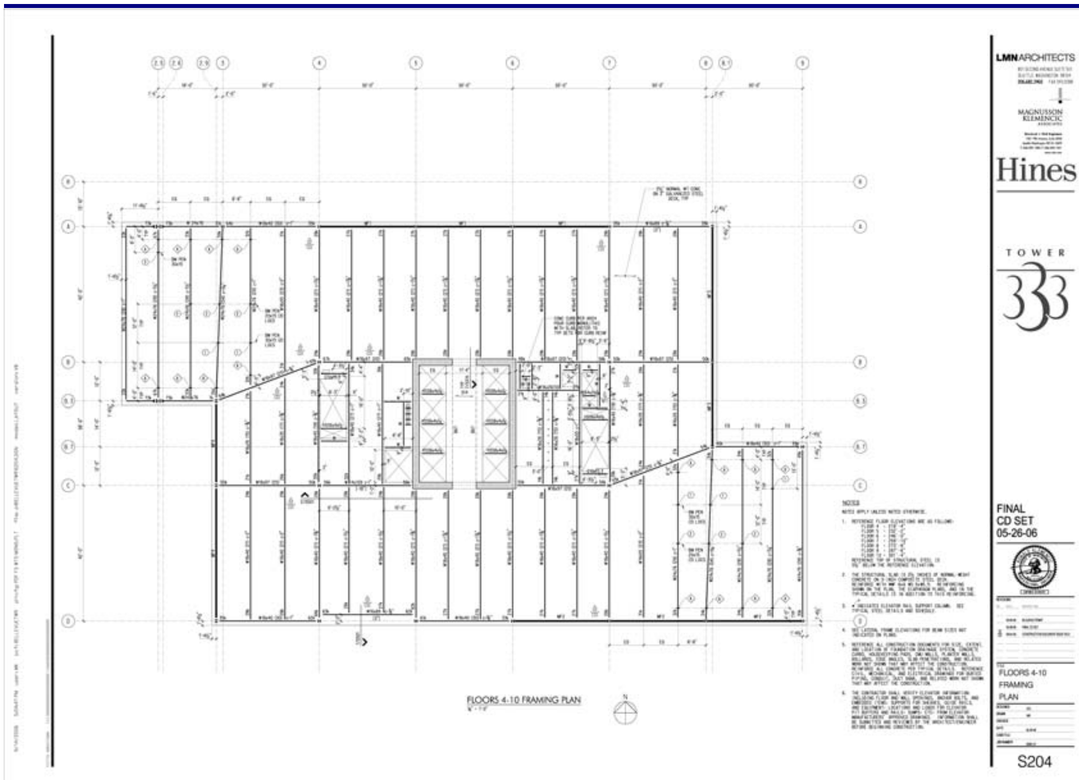
Upon conclusion of my calculations, I discovered a few discrepancies between the design values and sizes I computer and those of the design engineer's. There are several possibilities as to the cause of this.

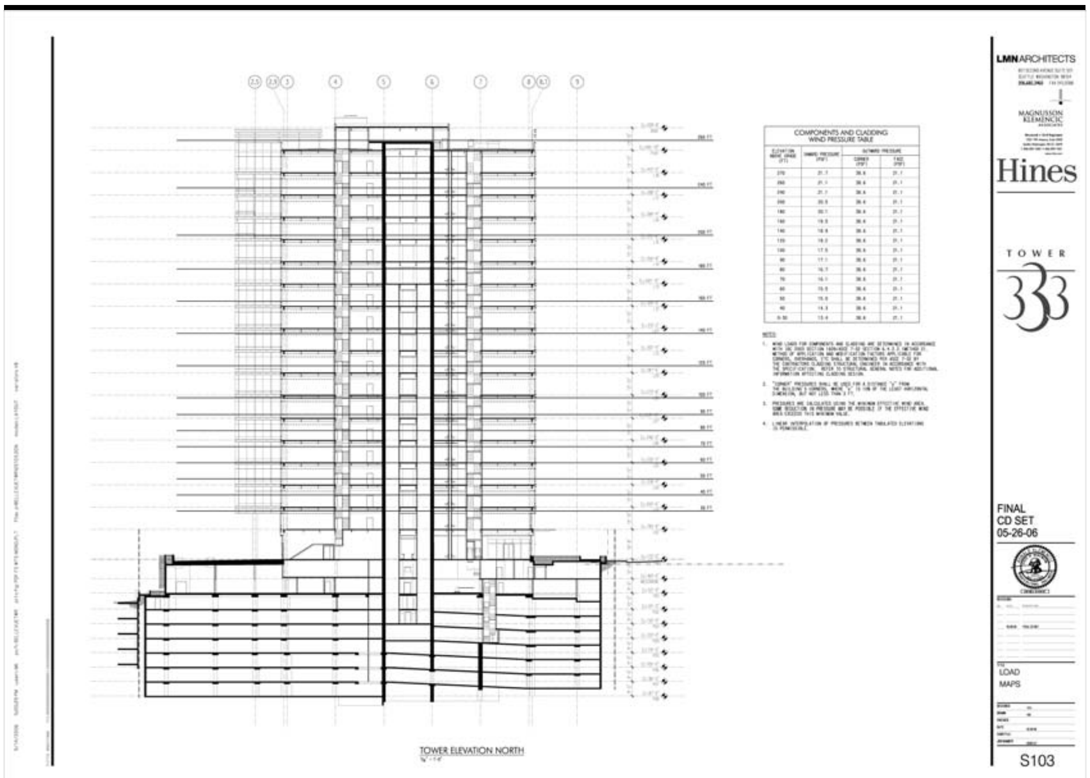
The first discrepancy I encountered was the value of the total seismic shear force. I produced a result of only 70% of what the total shear force as calculated by the engineer is. My first assumption as to why this occurred is that I did not account for all the dead load in the structure. With 8 levels of structure below grade, at least 2 of which are semi exposed due to the parking garage entrance and the fact that the core shear walls extend all the way to the mat foundation on grade it is possible that the engineer included the dead weight of the garage into his calculations. This is an assumption I did not make. With these 8 levels of parking included, that would add roughly an additional 40% of dead weight to the building. Making a rough estimate calculation and taking 140% of my resulting seismic shear value, I get a value that comes within 2% of the engineer's. Another reason could be the fact that I used a simplified approach for my initial seismic loads, whereas the engineer uses the modal response spectrum analysis (ASCE-7 '02 Section 9.5.6).

Another discrepancy discovered was the column size I achieved as compared to the engineers. Having calculated a smaller W shaper in my analysis, it is again possible my simplified approach did not account for all the loading and loading conditions that the engineer assumed. Differences in the codes and design manuals could also account for differences in sizes.

Other structural elements that I still need to check are a modal response spectrum analysis of my seismic forces, foundations and moment frame. Due to seismic being the controlling lateral force and the fact that I have a seismic category D, I will need to do a more detailed analysis to determine a more accurate shear force. Also, due to the deep level of excavation, I will need to check my foundation and external concrete walls for soil pressure capacity. In regards to the moment frames, once the percentage of lateral forces distributed to the moment frames is known, I will need to check member sizes.

APPENDIX


LMNARCHITECTS
MAGNUSSON KLEMENCIC
Hines
TOWER
333
FINAL
CD SET
05-26-06
FLOORS 4-10
FRAMING
PLAN
S204
FLOORS 4-10 FRAMING PLAN


LMNARCHITECTS
MAGNUSSON KLEMENCIC
Hines
TOWER
333
COMPONENTS AND CLADDING
WIND PRESSURE TABLE
FINAL
CD SET
05-26-06
LOAD
MAPS
S103
TOWER ELEVATION NORTH

PRESSURE FLEXIBLE $P = qG_fC_p - q_i(GC_{pi})$ (lb/ft²)

WINDWARD

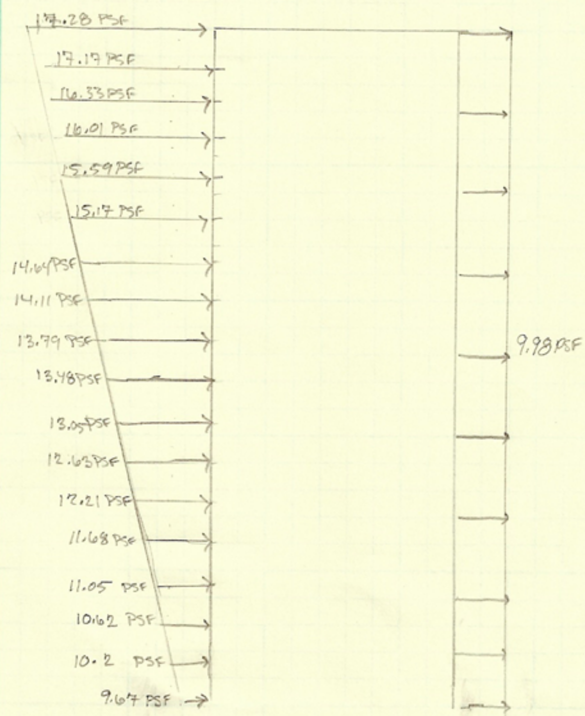
$P = q_z G_f C_p - q_h(-GC_{pi})$

LEEWARD

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

WIND FROM N-S

WINDWARD WIND LOAD LEEWARD



NOTE: DISTRIBUTION ON WINDWARD SIDE IS NOT LINEAR

Values obtained from ASCE-7 Chapter 6

Building Information	
Exposure:	B
V (mph)	85
Importance	II
I	1
Kd	0.85
Kzt	1
h (ft)	260
Enclosure:	Enclosed
α	7
Zg (ft)	1200
Zmin (ft)	30
c	0.3
â	0.143
b hat	0.84
α bar	0.25
l(ft)	320
€ bar	0.33
b bar	0.45

Flexible Building	
g_R	4.13
g_Q & g_v	3.40
R_n	0.049
R_h	0.097
n_1	0.77
η_h	9.80
V_z	94.0
ß	0.05
Wind from N-S	
η_B	8.06
η_L	15.9
R_B	0.12
R_L	0.06
Q	0.83
R	0.08
G_f	0.84
Wind from E-W	
η_B	4.75
η_L	27.00
R_B	0.19
R_L	0.04
Q	0.85
R	0.10
G_f	0.85

Pressure Coefficients:	
Internal	
Gcpi	0.18 -0.18
External	
Windward	
Cp	0.8
Leeward	
N-S---Cp	-0.5
E-W---Cp	-0.36
Period	
C_t	0.02
x	0.75
$h_{n\,(ft)}$	260
T_a	1.3
Nat. Freq: $n_{1\,(hz)}$	0.77

K_z & q_z		
Z (ft)	Kz	qz
0-15	0.57	8.96
20	0.62	9.75
25	0.66	10.38
30	0.70	11.01
40	0.76	11.95
50	0.81	12.73
60	0.85	13.36
70	0.89	13.99
80	0.93	14.62
90	0.96	15.09
100	0.99	15.56
120	1.04	16.35
140	1.09	17.14
160	1.13	17.77
180	1.17	18.39
200	1.20	18.87
250	1.28	20.12
267	1.30	20.44

Pressure				
Wind From N-S				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	9.70	0-15	-10.06	19.76
20	10.23	20	-10.06	20.28
25	10.65	25	-10.06	20.71
30	11.07	30	-10.06	21.13
40	11.71	40	-10.06	21.76
50	12.24	50	-10.06	22.29
60	12.66	60	-10.06	22.71
70	13.08	70	-10.06	23.14
80	13.50	80	-10.06	23.56
90	13.82	90	-10.06	23.88
100	14.14	100	-10.06	24.19
120	14.67	120	-10.06	24.72
140	15.19	140	-10.06	25.25
160	15.62	160	-10.06	25.67
180	16.04	180	-10.06	26.10
200	16.36	200	-10.06	26.41
250	17.20	250	-10.06	27.26
267	17.41	267	-10.06	27.47
Wind From E-W				
Windward		Leeward		Total
h (ft)	P (psf)	h (ft)	P (psf)	
0-15	9.77	0-15	-10.22	19.99
20	10.31	20	-10.22	20.53
25	10.73	25	-10.22	20.95
30	11.16	30	-10.22	21.38
40	11.80	40	-10.22	22.02
50	12.34	50	-10.22	22.56
60	12.77	60	-10.22	22.98
70	13.19	70	-10.22	23.41
80	13.62	80	-10.22	23.84
90	13.94	90	-10.22	24.16
100	14.26	100	-10.22	24.48
120	14.80	120	-10.22	25.02
140	15.33	140	-10.22	25.55
160	15.76	160	-10.22	25.98
180	16.19	180	-10.22	26.41
200	16.51	200	-10.22	26.73
250	17.36	250	-10.22	27.58
267	17.58	267	-10.22	27.80

Floor	Height (Ft.)	Trib. Height (Ft.)	Windward (PSF)	Leeward (PSF)	Total (PSF)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
					Wind from N-S			
1 (ground)	0	0	0	0	0	0	1365.32	195908.2
2	18.67	16.25	10.23	-10.06	20.29	70.56	1365.32	1317.3
3	32.5	13.83	11.71	-10.06	21.77	64.43	1294.76	2094.0
4	46.33	13.83	12.24	-10.06	22.3	66.00	1230.33	3057.8
5	60.167	13.83	12.66	-10.06	22.72	67.24	1164.33	4045.8
6	74	13.83	13.5	-10.06	23.56	69.73	1097.09	5159.9
7	87.83	13.83	13.82	-10.06	23.88	70.68	1027.36	6207.4
8	101.67	13.83	14.67	-10.06	24.73	73.19	956.68	7441.4
9	115.5	13.83	14.67	-10.06	24.73	73.19	883.49	8453.6
10	129.33	13.83	15.19	-10.06	25.25	74.73	810.30	9664.9
11	143.167	13.83	15.62	-10.06	25.68	76.00	735.57	10881.1
12	157	13.83	15.62	-10.06	25.68	76.00	659.57	11932.5
13	170.833	13.83	16.04	-10.06	26.1	77.25	583.56	13196.2
14	184.66	13.83	16.36	-10.06	26.42	78.19	506.32	14439.1
15	198.5	13.83	16.36	-10.06	26.42	78.19	428.13	15521.3
16	212.33	13.83	17.2	-10.06	27.26	80.68	349.93	17130.6
17	226.167	13.83	17.2	-10.06	27.26	80.68	269.25	18247.0
18	240	13.83	17.2	-10.06	27.26	80.68	188.57	19363.0
Pent	253.833	13.83	17.41	-10.06	27.47	81.30	107.89	20636.8
Roof	267.67	13.83	17.41	-10.06	27.47	26.59	26.59	7118.3

Floor	Height (Ft.)	Trib. Height (Ft.)	Windward (PSF)	Leeward (PSF)	Total (PSF)	Story Force (Kip)	Total Shear (Kip)	Overturning Moment (Ft.-Kip)
					Wind From E-W			
1 (ground)	0	0	0	0	0	0	807.11	115038.9
2	18.67	16.25	10.31	-10.22	20.53	42.04	807.11	784.8
3	32.5	13.83	11.8	-10.22	22.02	38.37	765.07	1247.1
4	46.33	13.83	12.34	-10.22	22.56	39.31	726.70	1821.4
5	60.167	13.83	12.77	-10.22	22.99	40.06	687.39	2410.4
6	74	13.83	13.62	-10.22	23.84	41.54	647.33	3074.2
7	87.83	13.83	13.94	-10.22	24.16	42.10	605.78	3697.7
8	101.67	13.83	14.8	-10.22	25.02	43.60	563.68	4432.7
9	115.5	13.83	14.8	-10.22	25.02	43.60	520.08	5035.7
10	129.33	13.83	15.33	-10.22	25.55	44.52	476.48	5758.1
11	143.167	13.83	15.76	-10.22	25.98	45.27	431.96	6481.5
12	157	13.83	15.76	-10.22	25.98	45.27	386.69	7107.7
13	170.833	13.83	16.19	-10.22	26.41	46.02	341.42	7862.0
14	184.66	13.83	16.51	-10.22	26.73	46.58	295.39	8601.3
15	198.5	13.83	16.51	-10.22	26.73	46.58	248.82	9246.0
16	212.33	13.83	17.36	-10.22	27.58	48.06	202.24	10204.7
17	226.167	13.83	17.36	-10.22	27.58	48.06	154.18	10869.7
18	240	13.83	17.36	-10.22	27.58	48.06	106.12	11534.5
Pent	253.833	13.83	17.58	-10.22	27.8	48.44	58.06	12296.6
Roof	267.67	13.83	17.58	-10.22	27.8	9.61	9.61	2572.8

Seismic Loading						
		V=	1774			
K=1.4	Level	w_x	h_x	$w_x h_x^{1.4}$	C_{vx} (k)	Fx (k)
Roof	20	156	267.61	390506.1	0.008	14.57
Penthouse	19	2490	253.78	5786816.3	0.122	215.94
Office	18	2490	239.95	5350180.1	0.113	199.65
Office	17	2490	226.12	4923499.7	0.104	183.73
Office	16	2490	212.29	4507136.4	0.095	168.19
Office	15	2490	198.46	4101488.8	0.086	153.05
Office	14	2490	184.63	3706999.5	0.078	138.33
Office	13	2490	170.8	3324163.4	0.070	124.04
Office	12	2490	156.97	2953538.7	0.062	110.21
Office	11	2490	143.14	2595761.4	0.055	96.86
Office	10	2490	129.31	2251564.0	0.047	84.02
Office	9	2490	115.48	1921802.8	0.040	71.71
Office	8	2490	101.65	1607495.2	0.034	59.99
Office	7	2490	87.82	1309875.9	0.028	48.88
Office	6	2490	73.99	1030484.8	0.022	38.45
Office	5	2490	60.16	771312.4	0.016	28.78
Office	4	2490	46.33	535065.0	0.011	19.97
Office	3	2490	32.5	325713.7	0.007	12.15
Office	2	2432	18.67	146408.3	0.003	5.46
Lobby	1	3442	0	0	0	0
Total		48360		47539813	1.000	1774.00

Total Base Shear	1774.00
Total Overturning Moment	327658

Seismic Design Values:

Location: Bellevue, Washington
Number of Floors: 18
Height: 260 feet
Site Class: C
Occupancy: II
Seismic Category: D
Importance Factor: 1.0

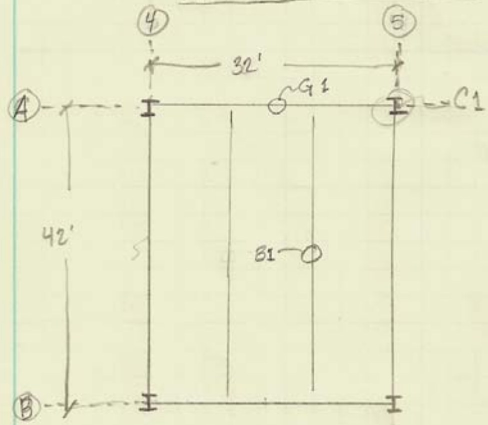
Seismic Design Values Cont..

R:	8	$\mathbf{S_s}$**:**	1.356
		$\mathbf{S_1}$**:**	.615
		$\mathbf{F_a}$**:**	1.0
K:	1.4	$\mathbf{C_S}$:	1.341
$\mathbf{T_L}$**:**	6	$\mathbf{S_{M1}}$**:**	
$\mathbf{T_a}$:	1.3	$\mathbf{S_{DS}}$**:**	.904
		$\mathbf{S_{D1}}$**:**	.410

Dead Loads:

	Slab & Deck	**Beams**	**SDL**	**Total**
Ground:				
section PL:	50 PSF	4-6 PSF	180 PSF	232 PSF
section P:	50 PSF	4-13 PSF	30 PSF	84-93 PSF
section D:	50 PSF	3-9 PSF	15 PSF	68-74 PSF
section E:	50 PSF	3-9 PSF	35 PSF	88-94 PSF
2:	50 PSF	4-8 PSF	25 PSF	79-83 PSF
3-18	50 PSF	4-8 PSF	25 PSF	79-83 PSF
Roof:	50 PSF	4-8 PSF	25 PSF	79-83 PSF

TYPICAL BAY FL. 3-18



BEAMS 10' 8" o.c.

A_T = 420 ft²

LOADING : OFFICE

LL = 50 PSF + 20 PSF (PARTITIONS
DL = 50 PSF — (2.5" CONC. ON 3" COMPOSITE DECK)
SDL = 5 PSF MECH. + 5 CARPET / MISC.

DL = 50 PSF + 5 PSF + 5 PSF
= 60 PSF

LL = 50 PSF + 20 PSF
= 70 PSF

LIVE LOAD REDUCTION B1

$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) = (70\,PSF)\left(.25 + \frac{15}{\sqrt{(2)(420 ft^2)}}\right)$ L = 54 PSF

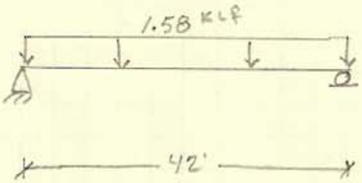
$K_{LL} = 2$ TBL. 4-2

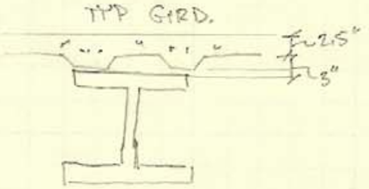
FACTORED LOAD: 1.2D + 1.6L = 1.2 (60 PSF) + 1.6 (54 PSF) = 158.4 PSF

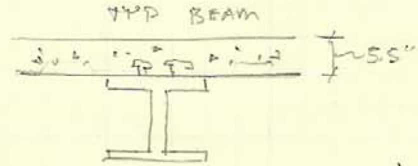
P_u = 158.4 PSF

B1 CONT.

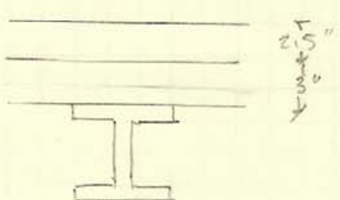
$P_u = 158.4\ PSF$ $\qquad W_u = (158.4\ PSF)(10') = 1584\ PLF$







BEAM B1



$W_u = 1.58\ KLF$

$M_u = \frac{wL^2}{8} = \frac{(1.58\ KLF)(42')^2}{8}$

$M_u = \underline{348.4'^K}$

$f_y = 50\ ksi$
$f'_c = 4000\ psi$

ASSUME $a = 1"$

$b_{eff} = 10'\text{-}8" = \underline{128"}$ or $\frac{42'(12)}{4} = \underline{\underline{126"}}$

$Y_2 = (5.5") - \frac{1"}{2} = 5"$

B1 CONT. LRFD TABLE 3-19

TRY W18×35

@ $Y_2 = 5$ & PNA @ TFL $\phi M_n = 535 > 348$ ✓ OK

$\Sigma Q_n = 515^k$

Check Assumption $a = 1.0''$

$$a = \frac{515^k}{.85(4000\,psi)(126'')} = 1.2 \quad \therefore \text{NO GOOD}$$

PNA @ BFL $\Rightarrow \Sigma Q_n = 260^k$ $\phi M_n = 435 > 348$ OK

$$a = \frac{260^k}{.85(4)(126'')} = .61 \quad \text{OK}$$

$$Y_2 = 5.5 - \frac{.64}{2} = 5.2'' \quad \text{ok}$$

ASSUME SHEAR STUD HOLD 21^k

$\Sigma Q_n = 260^k \rightarrow$ # of STUDS = 13 PER SIDE

TOT # OF. STUDS 2(13) = 26

USE W18×35 w/ 26 STUDS

BEAM SPECIFIED W18×40

CONCLUSION: LOAD ASSUMPTIONS YIELD RESULTS CLOSE & REASONABLE TO DESIGNER'S THERE FOR VALID.

COLUMN C1 LEVEL 8

LOADS OFFICE

LL = 50PSF + 20PSF (PART) = 70 PSF

DL = 50PSF + 5PSF (MECH.) + 5 (MISC.) + 4PSF (Beams) = 64PSF

LL REDUCTION $K_{LL} = 4$ TABLE 4-2 ASCE-7 $A_T = 1240 ft^2$

$$L = L_o\left(.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right) > .4L$$ — MULTI STORY BLD. LIMIT

$A_T = (31')(40') = 1240 ft^2$

FLOORS ABOVE: 10 $\Longrightarrow$ $1240 ft^2 (10) = 12,400$ A_T

$$\left(.25 + \frac{15}{\sqrt{4(12,400 ft^2)}}\right) = .32 < .4 \quad \therefore \quad .4L \text{ CONTROLS}$$

$L = .4(70 PSF) = 28 PSF$

LOAD COMB.

$1.4D = 1.4(64 psf) = 89.6^{psf}$

$1.2D + 1.6L = 1.2(64) + 1.6(28) = 125^{psf}$ ← CONTROLS
+2.5 S.W.

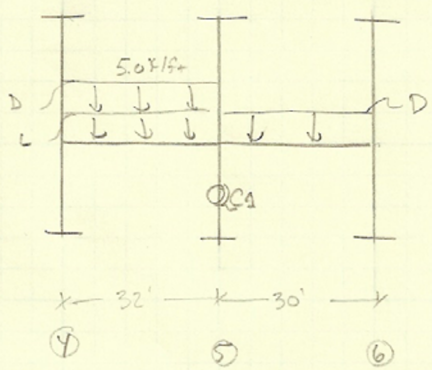
$P_{floor} = (125 psf)(12,400 ft^2) = 1550$ Kips $= P_u$

COLUMN C1 LEVEL 8 CONT.

$(125\ PSF)\left(\frac{42'}{2}+\frac{38'}{2}\right) = 5.0\ K/ft$

$+ .18\ K/ft$ (S.W.)

$= 5.2\ K/ft$



$FEM_{4\text{-}5} = \frac{(5.2\ K/ft)(32')^2}{12} = 444'K$

BEAM 5-6

TRIB WIDTH = 40' $A_T = (40')(30') = 1200\ ft^2$

DEAD LOAD ONLY:

$DL = 60\ PSF + 4\ PSF$ (S.W.) $= 64\ PSF$ $1.2(64\ PSF) = (76.8\ PSF)(40')$

$= 3.0\ K/ft$

$FEM_{5\text{-}6} = \frac{(3.0\ K/ft)(30')^2}{12} = 225'K$

$\Delta FEM = 444'K - 225'K = 220'K$

COLUMN C1 LEVEL 8 CONT.

$P_u = 1513^k$ $M_u = 197^{1k}$

TRY W14 SHAPE

$P_{eff} = P_u + \alpha M_u$

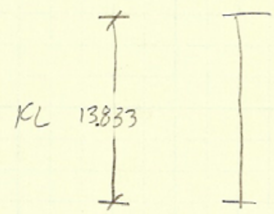
$\alpha = \frac{24}{d} = \frac{24}{14} = 1.71$

$P_{eff} = 1513 + 1.71(220^{1k}) = 1890^k$

TABLE 4-2

$P_{eff} = 1890$ @ KL = 14'

W14x176

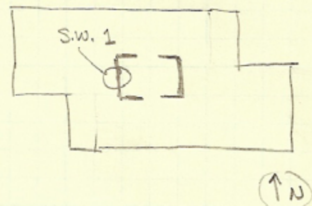


CONCLUSION:

BASED ON RESULTS IT COULD BE ASSUMED SINCE SIZES CALCED ARE SMALLER THAN DESIGNERS THAT THERE WERE SOME. UN ACCOUNTED FOR LOADS, POSSIBLY SNOW OR OTHER "DEAD" WT. SUCH AS MECH EQUIPMENT

CORE WALL S.W. 1

SEISMIC BASE SHEAR $V = 2{,}513^K$



SYMETRICAL WALL LAYOUT

DISTRIBUTION IN N-S $= \frac{1}{2}$

ASSUME MOMENT FRAMES TAKE AT LEAST 25% OF LATERAL
TABLE 12.2.1 ASCE-7 05

CORE SHEAR $= .75(2513^K) = 1884.75^K$

EACH WALL TAKES 50% $\Longrightarrow (1884.75^K)(.5) = \underline{942.4^K}$

$h = 24''$
$\ell_w = 40'$
$V = 942.4^K$

$f'_c = 9000$

DET. LONG & TRANSVERSE REINF.

ACI 21.7.2.2 2-CURTAINS REQ. IF $V_u > 2A_{cv}\sqrt{f'_c}$

$$2(24'')(40')(12'')\sqrt{9000} = 2185^K > 942.4^K$$

$\therefore$ NEED 2 CURTAINS

ACI 21.7.2.1

$$\rho_\ell,\ \rho_t \geq .0025$$

$$\therefore \frac{A_{s\ell}}{A_{cv}} \geq .0025$$

$A_{cv} = (24'')(12'') = 288\ in^2/ft$

$(288\ in^2/ft)(.0025)$

$= .72\ in^2/ft$ MINIMUM

ASSUME #5 IN 2 CURTAINS

$A_{s\ell} = \frac{.62\ in^2}{Space}$ $\quad \frac{.72\ in^2}{12''} = \frac{.62\ in^2}{S} \Longrightarrow S = 10.33''$ MAX.

S.W. 1 CONT.

TRY #5 @ 10" SPACING FOR BOTH DIRECT.

NOMINAL SHEAR: ACI 21.7.4.1

$$V_n = A_{cv}(\alpha_c \sqrt{f'_c} + \rho_t f_y)$$

$$\frac{h_w}{l_w} = \frac{260'}{40'} = 6.3 > 2.0 \therefore \alpha_c = 2.0$$

$$A_{cv} = (24")(40')(12") = 11{,}520 \text{ in}^2$$

$$\rho_t = \frac{2(.31 \text{ in}^2)}{(24")(12")} = .0022$$

$$V_u = (11{,}520 \text{ in}^2)\left[(2.0)\sqrt{9000 \text{ psi}} + .0022(60 \text{ksi})\right] = 3706^k$$

$$\phi V_n = .6(3706^k) = 2224^k > 942^k \therefore \text{ OK}$$

$(42+38) \times 15$ $\frac{1200 \text{ ft}^2}{22{,}500 \text{ ft}^2} = .053 \times$ TOT DEAD WEIGHT (SHEAR WALL TRIB; AREA FLOOR PLATE)

$= 2415318$ lbs

$P_u = 2415^K$

$M_u = 327{,}639^{1K}$ (FLOOR FORCE x HEIGHT)

$$C_u = \frac{P_u}{2} + \frac{M_u}{d} = \frac{2415^K}{2} + \frac{327{,}639^{1K}}{40'}$$

$= 9400^K \equiv P_u$ @ B.E.

$$A_g = (2')(40') = 80 \text{ ft}^2$$

$$I_g = \frac{(2')(40')^3}{12} = 10{,}667 \text{ ft}^4$$

M_u ON S.W. 1

$$\frac{(327{,}639)(.75)}{2} = 122{,}865^{1K}$$

S.W. 1 CONT.

$$f_c = \frac{P_u}{A_g} + \frac{M_u \cdot \frac{h_w}{2}}{I_g} = \frac{2415^k}{80ft^2} + \frac{(122{,}865^{1k})(\frac{40'}{2})}{19{,}667\,ft^4} = 261\,KSF$$

$$= 1.8\,KSi$$

$$.2f'_c = .2(9\,ksi) = 1.8\,ksi$$

$$f_c = 1.8\,ksi = 1.8\,ksi \quad \therefore \text{NO BOUNDARY NEEDED}$$

CONCLUSION:

THE RESULTS ATTAINED WERE SIMILAR & WITHIN REASON TO THE DESIGNERS THEREFORE CAN BE ASSUMED VALID.