OFFICE BUILDING
Ontario, CA

Structural Technical Report 1
October 5, 2006
Structural Concepts / Structural Existing Conditions

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Executive Summary:

The Office Building is a six story structure located in Ontario, CA. Steel is used as the main structural component to frame the building and support the floors. The simple building shape and steel frame allowed the architect to design for a free, open floor plan with the bathrooms, elevators and staircases centrally located on each floor. The design is simple, yet very functional for an office setting. The main entrance is a modern, inviting space highlighted by large glass doors and glass walls on all sides. Glass is also used as one of the main elements used to create the building's façade, alternating with architectural brick.

This first report presents an in-depth analysis of the existing structural system including the foundation, floor system, columns and lateral bracing system. A series of spot checks were performed on members and systems to compare designs with those done by the structural engineer.

Structural Design Code:

California Building Code

Design Calculations:

The fact that the structural engineer used the California Building Code s opposed to the ASCE-7 – 05 , which I used , is the reason for the differences in member sizes in which I designed compared to those of the structural engineer since the CBC presents different loadings and force distributions. Another reason may be because the analysis of seismic forces in southern California is more complex than the methods that I am familiar with.

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Structural System

Foundation:

The foundation is a 5" thick slab on grade with #9 bars placed at 15" O.C., and has a strength of 4000 psi. The slab is supported by grade beams and spred footings. The grade beams are typically 48" x 48" and 4'-0" deep and require a minimum of 18" of finished floor above the beam. The spread footings which also have a strength of f'c = 4000 psi , range in size from 10'-6" SQ and a thickness of 24" to 19'-6" SQ with a thickness of 50".

Floor System:

The second floor through the penthouse are typical with wide flange steel beams supporting a VERCO W3 Formlock 20 gauge metal deck. The metal deck is filled with 3-1/4" light weight concrete fill with #3 bars at 18" O.C. each way. The beams are designed to support a maximum span of 11'-0", and typical bays are 38' x 30' and 33'-10" x 34'-9".

Columns:

Wide flange ASTM A992 GR 50 steel columns support the structure and span 30.5'-0", supporting about 2 stories and are connected by 4'-0" splices between floors. Most columns at the ground level are supported by the grade beams. All columns have base plates made of ASTM A572 GR. 50 steel , and are also part of the braced frame lateral system.

Lateral System:

The lateral system is a network of concentrically braced frames that span the entire height of the building. The W12 shape members that make up the braced frame distribute the lateral forces through triangles, much like a truss. The fact that this lateral system frames into the grade beams is to counteract the large uplift force associated with braced frame lateral systems.

Codes and Code Requirements

Building Code:

California Building Code 2001 – Based on UBC 1997

Structural Steel:

"Manual of Steel Construction, LRFD Third Edition"
American Institute of Steel Construction

Wind and Seismic Calculations:

ASCE - 7 2005

Loads

Floors

Dead Loads:		Live Load:	
Slab	48 PSF		80 PSF
Ceiling and finish	5 PSF		
Mech. & Misc.	7 PSF		
Partitions	20 PSF		
Steel Framing	15 PSF		

Main Roof

Dead Loads:		Live Load:	
Slab	75 PSF		100 PSF
Ceiling			
Mech. & Misc.	10 PSF		
Roofing & Ins.	15PSF		
Steel Framing	15 PSF		

Penthouse Roof

Dead Loads:		Live Load:	
Slab	15 PSF		20 PSF
Ceiling ,			
Mech. & Misc.	10 PSF		
Roofing	5 PSF		
Steel Framing	15 PSF		

Lateral Loads:

Wind

Height	Windward		Leeward		Total	
	N-S	E-W	N-S	E-W	N-S	E-W
0-15	11.48	11.48	11.99	8.62	23.47	20.1
20	12.17	12.17	11.99	8.62	24.16	20.79
25	12.7	12.7	11.99	8.62	24.69	21.32
30	13.25	13.25	11.99	8.62	25.24	21.87
40	14.06	14.06	11.99	8.62	26.05	22.68
50	14.75	14.75	11.99	8.62	26.74	23.37
60	15.28	15.28	11.99	8.62	27.27	23.9
70	15.82	15.82	11.99	8.62	27.81	24.44
80	16.34	16.34	11.99	8.62	28.33	24.96
90	16.76	16.76	11.99	8.62	28.75	25.38
100	17.04	17.04	11.99	8.62	29.03	25.66

Seismic

Floor	Height	Area(ft^2)	Weight	Wxhx^k	Cvx	Fx	Vx
1	0	28,246	2683.37 k	0	0	0	1112.04
2	16	28,246	2683.37 k	50705	0.04	44.48	1112.04
3	30	28,246	2683.37 k	98725	0.078	86.74	1067.56
4	44	28,246	2683.37 k	148163	0.117	130.1	980.82
5	58	28,246	2683.37 k	198570	0.156	173.48	850.72
6	72	28,246	2683.37 k	249720	0.196	217.96	677.24
Roof	86	28,246	3248.29 k	364941	0.287	319.16	459.28
Penthouse	96	28,246	1272.07 k	160464	0.126	140.12	140.12

Spot Checks

Reasons for the differences is member sizes and loading conditions are listed above.

Wind:

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WIND CALCULATIONS (MOMEN FRAMES) ASCE 7-05

Period (T) = 0.1(N)
= (0.1)(6)
T = 0.6 ⟹ Rigid, use simplified method

assume K_{zt} = 1.0
Ht = 96'-0"
Exposure C
WIND SPEED = 85 MPH
Category II, I = 1.0G
K_d = 0.85
K_z = 0.85

$q = 0.00256 K_z K_{zt} K_d V^2 I$

$q_{0\text{-}15} = 0.00256(0.85)(1.0)(0.85)(85)^2(1.0)$
= 13.36 PSF

K_z : 20 = 0.9, 25 = 0.94, 30 = 0.98, 40 = 1.04, 50 = 1.09, 60 = 1.13, 70 = 1.17, 80 = 1.21, 90 = 1.24, 100 = 1.26

q_{20} = 14.15 PSF
q_{25} = 14.77 PSF
q_{30} = 15.41 PSF
q_{40} = 16.35 PSF
q_{50} = 17.14 PSF
q_{60} = 17.77 PSF
q_{70} = 18.40 PSF
q_{80} = 19.0 PSF
q_{90} = 19.49 PSF
q_{100} = 19.81 PSF

FOR RIGID STRUCTURE, G = 0.85
FOR ENCLOSED GC_{pi} = ± 0.18

C_p windward = 0.8
C_p leeward = -0.3

L/B = 230.2' / 122.7' = 1.88

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$P = q(GCp - GCpi)$

WINDWARD

0-15	P =	13.36(.85)(.8) - 13.36(±.18)	=	9.08 ± 2.40
20	P =	14.15(.85)(.8) - 14.15(±.18)	=	9.62 ± 2.55
25	P =	14.77(.85)(.8) - 14.77(±.18)	=	10.04 ± 2.66
30	P =	15.41(.85)(.8) - 15.41(±.18)	=	10.48 ± 2.77
40	P =	16.35(.85)(.8) - 16.35(±.18)	=	11.12 ± 2.94
50	P =	17.14(.85)(.8) - 17.14(±.18)	=	11.66 ± 3.09
60	P =	17.77(.85)(.8) - 17.77(±.18)	=	12.08 ± 3.20
70	P =	18.40(.85)(.8) - 18.40(±.18)	=	12.51 ± 3.31
80	P =	19.0(.85)(.8) - 19.0(±.18)	=	12.92 ± 3.42
90	P =	19.49(.85)(.8) - 19.49(±.18)	=	13.25 ± 3.51
100	P =	19.81(.85)(.8) - 19.81(±.18)	=	13.47 ± 3.57

LEEWARD ⊥ 230' $C_p = -0.5$
⊥ 122' $C_p = -0.3$

⊥ 230 P = 19.81(.85)(-0.5) - 19.81(±.18) = -8.42 ± 3.57
⊥ 122 P = 19.81(.85)(-0.3) - 19.81(±.18) = -5.05 ± 3.57

Seismic:

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SEISMIC CALCULATIONS

FLOOR	Height	Area	Weight (K)
1	0	28,246	2683.37
2	16	28,246	2683.37
3	30	28,246	2683.37
4	44	28,246	2683.37
5	58	28,246	2683.37
6	72	28,246	2683.37
roof	86	28,246	3248.29
penthouse	96	28,246	1271.07

– 1st not included for seismic

Total Wt. = 17,936.21 kips

$S_1 = 0.097$ $S_s = 0.249$ $T_L = 12$

Assume Site Class D

$F_v = 2.2$

$F_a = 1.6$

$S_{ms} = F_a S_s = (1.6)(0.249) = 0.3984$

$S_{m1} = F_v S_1 = (2.2)(0.097) = 0.213$

$S_{DS} = \frac{2}{3} S_{ms} = 0.2656 \Rightarrow B$

$S_{D1} = \frac{2}{3} S_{m1} = 0.142 \Rightarrow C$ ∴ SEISMIC CAT. C

$R = 3.75$ $C_d = 3.75$

$C_s = \frac{S_{DS}}{\left(\frac{R}{I}\right)} = \frac{0.2656}{\left(\frac{3.75}{1}\right)} = 0.071$

$C_s \le \frac{S_{D1}}{T(R/I)}$

$\le \frac{0.142}{(.613)(3.75)}$

$C_s \le 0.062$

$T = C_t h_n^x$ $C_t = 0.02$ (table 12.8-2)

$x = 0.75$

$T = (0.02)(96)^{.75} = 0.613$

$V = 0.062(17936.21\text{K})$

$= 1112.05\text{ K}$

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$K = (1/2)(.113) + 1 = 1.06$

$C_{vx} = \frac{W_x h_x^K}{\Sigma W_x h_x^K}$ $F_x = C_{vx} \cdot V$

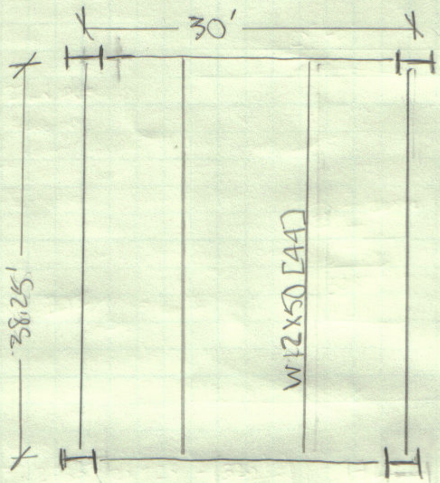
	$W_x h_x^K$	C_{vx}	F_x	V_x	Overturning Mo
1	0	0	0	1112.04	0
2	50705	.040	44.48	1112.04	17792.6 1K
3	98725	.078	86.74	1067.56	32026.8 1K
4	148163	.117	130.10	980.82	43156.1 1K
5	198570	.156	173.48	850.72	49341.8 1K
6	249720	.196	217.96	677.24	48761.3 1K
roof	364941	.287	319.16	459.28	39498.1 1K
P.H.	160464	.126	140.12	140.12	13451.5 1K

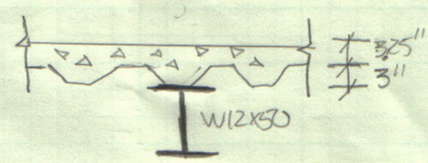
$\Sigma = 1271288$

Base Shear $= C_s \cdot W_t$

$= (1793621 K)(0.062)$
$= 111204.5 K$

Beam Check:





$F_y = 50$ ksi
$A = 14.6\,in^2$
$f'c = 3$ ksi
$d = 12.2''$
$b = 8.08''$

$b_{eff} \le 1/4$ span $= 7.5' = 90''$
$b_{eff} \le$ spcing $= 10'$

$C_c = 0.85 f'c\,(b_{eff})(t_{slab})$
$= 0.85(3)(90'')(3.25'')$
$= 745.88$ K

$T_s = A_s F_y$
$= (14.6\,in^2)(50\,ksi)$
$= 730$ K

$a = \frac{A_s F_y}{.85 f'c\, b_{eff}}$
$= \frac{730^K}{(.85)(3)(90)} = 3.18''$

$M_n = (730^K)\left(6.25'' - \frac{3.18''}{2}\right) + 730^K\left(\frac{12.2''}{2}\right) = 7854.8\,'K$

$\phi M_n = 0.9(7854.8\,'K) = 7069.32\,'K$

$\Sigma Q_n = 22(17.2) = 378.4 < C_c \therefore$ controls

$a = \frac{378.4}{(.85)(3)(90)} = 1.65''$

$\frac{730 - 378.4}{(2)(50)(8.08)} = 0.44 < 0.64 = t_f$

4453   77.22

$M_n = 378.4\left(6.25 - \frac{1.65}{2}\right) + 730\left(\frac{12.2}{2}\right) - 351\left(\frac{.44}{2}\right)$

2053

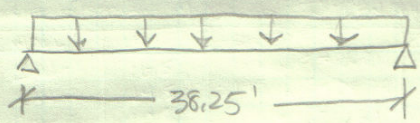
$= 6429 = 535.75$

$\phi M_n = (0.9)(535.75)$
$= 482.18\,'K$

DL = 48 psf (slab)
5 psf (ceiling & finish)
7 psf (mechanical)
20 psf (partitions)
15 psf (steel framing)
95 psf

LL = 80 psf

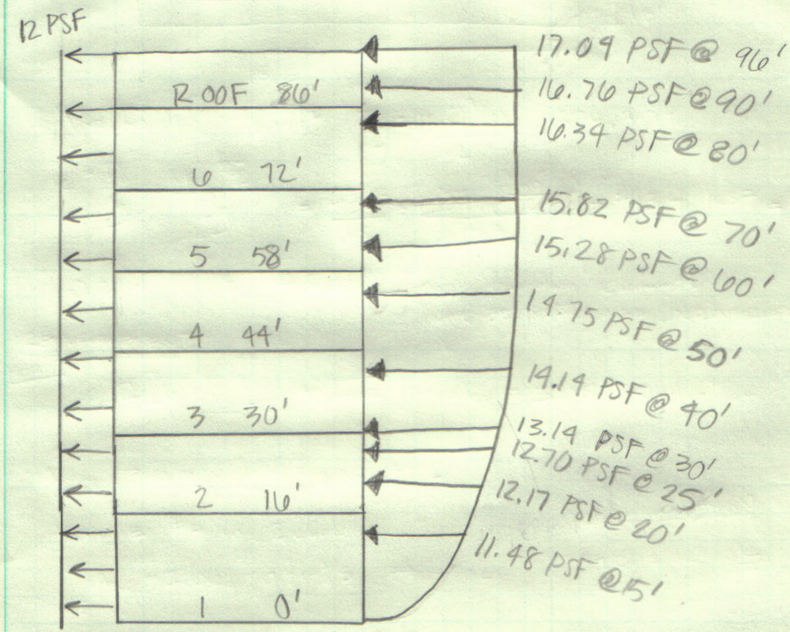
$$W = 1.2D + 1.6L$$
$$= 1.2(95) + 1.6(80)$$
$$= 242 \text{ psf}$$



trib. area = (10')(38.25')

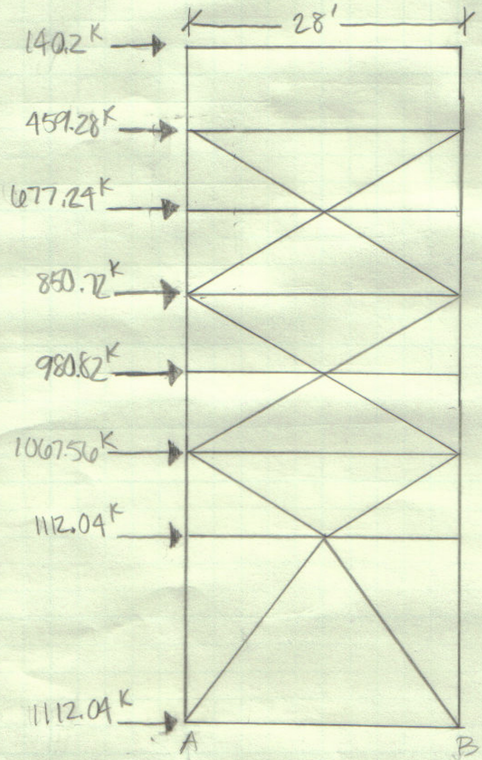
$$M = \frac{wl^2}{8} = \frac{(242)(38.25)^2(10)}{8} = 442.58'K$$

Wind Loading Diagram:



Lateral Bracing System:

BRACED FRAME



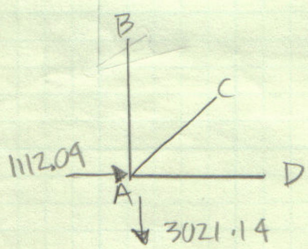
Check member

$\Sigma M_A = 0 \quad 5287(16) - B_y(28) = 0$

$28 B_y = 84592$

$B_y = 3021.14$

$A = -3021.14$



$\Sigma F_y = (-3021.14) + (\sin 45) AC = 0$

$0.7 AC = 3021.14$

$AC = 4315.91 K (T)$

$\phi = 0.9 = \frac{AC}{} \frac{Pa(K)}{F_y}$ = req'd Area

$\phi = 0.9$

$P_u = \frac{P}{\phi} = \frac{4315.91}{0.9} = \frac{4795.46K}{4 \text{ Braced Frames}}$

$= 1198.87$

$\frac{P_u}{F_y} = \text{area}$

$\frac{1198.87}{50 \text{ ksi}} = 24 \text{ in}^2$

use W12x96

· Engineer Designed for W12x152