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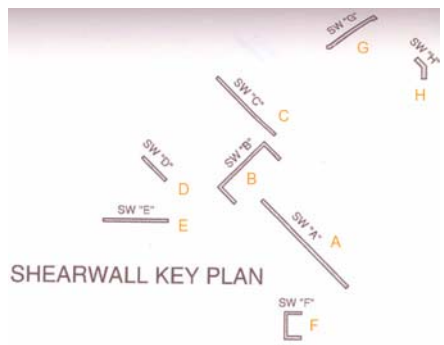
Structural Option
Metropolis at Dadeland
Adviser: Schneider
Tech 3 – Exec Summary Resubmittal
12/11/05

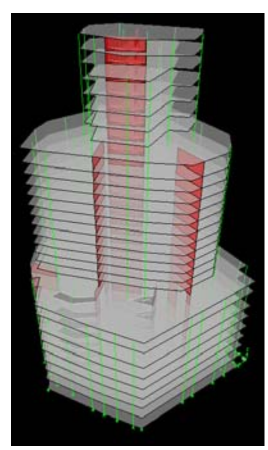
Technical Report 3 – Executive Summary
Lateral System Analysis and Confirmation Design

The structure of the Metropolis at Dadeland tower resists lateral loads by utilizing shear walls. There are eight different walls that run in four different directions within the building. The most unique part of the shear walls is that while most of the walls are solid on many levels there are openings within the walls both at the top of wall A and within the parking deck in walls A, C, and E. Walls D, F, G, and H end at the eighth floor, C and E reach the 22nd floor, and A and B reach the roof level. On the top floors wall A once again has an opening in it to allow for the use of the space as a connected living space.



The loads lateral loads on the structure were found using the method found in ASCE 7-02. As was expected due to the location of my building and confirmed in technical report 1, wind easily controls over seismic loading on the structure even though the building is rather massive. In order to calculate my deflection values and shear wall loads I modeled the structure using ETABS. The resulting drifts were reasonable, but noticeably larger than what should be allowed based on an H/400 limitation. The strength checks revealed that my structure is sufficiently designed for the loads that it sees. Both the columns and shear walls pass design checks at critical locations, whether calculated by hand or computer. This fact is amplified if the wind tunnel results were used to calculate lateral loads instead of the ASCE method's results.

After analyzing the results obtained from this report is it evident that most of the structure adequately carries the applied lateral loads. The one limiting part of the design is the excessive lateral deflections when lateral stiffness is attributed only to the shear walls.



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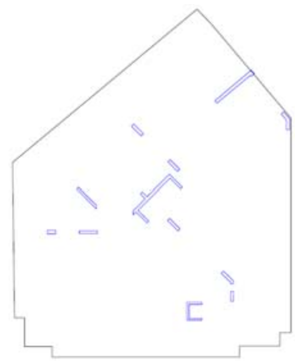
Lateral System Explanation

The lateral resisting system in the Metropolis at Dadeland structure is made up of an array of shear walls. There are eight (8) different walls that run from the base to varying heights within the building. There are also openings in many of the shear walls on the lower floor since to allow for traffic flow through the parking deck which runs up to the seventh floor. Walls A and B run the entire height of the building, walls C and E run to the 22 floor, and walls D, F, G, and H reach the eighth floor.



As was seen in technical report one (1), and further emphasized in appendix J, the wind clearly controls the lateral system design. Being in Miami, with a 150 mph mean wind velocity, was a clear give away even before my calculations justified the results, that seismic loads were not of any concern. There is even a stipulation in the Florida Building Code (FBC) that notes that seismic loads do not need to be checked in the area my structure was built.

The lateral loads within the structure are distributed to the shear walls based on rigidity. The concrete floors are substantial enough to be considered a rigid diaphragm. Since wind is the controlling lateral load on my structure, the direction at which the wind exerts force on the structure is important. I found that different load cases controlled for different shear walls within the building. These comparisons can be seen in appendix C. Loads applied to the shear walls are both from direct force on the walls and from torsional effects, since the center of rigidity is not at the geometric center of the structure. One concern related to the distribution of loads from the structure to the ground is the shear walls from level 1-8. Not only do they have the most loads applied to them as shears are accumulated through the building, but because the cross-sectional areas of the walls decrease to permit traffic flow within the parking deck. This is where I will focus some of my shear wall strength checks.

Drift

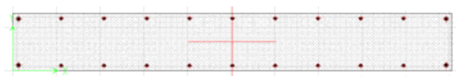
Due to the complexity and irregular geometry of my structure I utilized ETABS (appendix D) to calculate story drift and overall building drift of my structure. The results suggest that the drifts that I am seeing in the structure are slightly too high. The drift criterion I am using of H/400 for the entire structure would be a drift of 9.39". My model moved 14.7" in the x-direction and 15.1" in the y-direction. This can be seen in appendix E. I have not gotten a response back from the engineer of record, but I suspect that the most reasonable explanation for this excessive drift is that some lateral rigidity was attributed to the concrete frames created by the columns and slab.

Overturning

The overturning is competently mitigated by the foundation. The combination of a 5' thick concrete mat and approximately 250 piles are sufficient to resist the present lateral loads. An explanation can be found in appendix G. Assuming an elastic distribution of loading this creates a factor of safety of approximately 7.

Strength Check

I checked the shear walls both by hand and attempted to use the "design" feature in ETABS. My hand calculations yielded two results. Either I had the same design required as what is in the drawings, or the given design was much more than needed. For shear wall C on the ground floor I found that #5 @ 12" was exactly what was needed. For wall G I found that the concrete alone would resist the shear loads and just needed to be reinforced for minimum rho values. The ETABS designs the walls with less reinforcing than was calculated by the original designer. The wall actually in the building has 30 #9 bars, whereas the computer calculated wall has 24 #7 bars. This increase in area, $30"^2$ versus $14.4"^2$, may be attributed to the need for closer spacing for stirrups and an attempt to minimize the different sizes of bars being used in shear walls to limit possible confusion. This is much the same reason concrete gradations were distributed uniformly across floors, even though not all members would need the added strength at the level in which the concrete strength was adjusted. Similar results were found for other walls as well.



When checking the columns I got different results for different columns. I would have checked more columns by hand, but the many unique, varying geometries would have made this task very difficult. There for I checked some columns utilizing PCACOL and some with more basic rectangular geometries by hand. Theses results are looked at in appendixes H and I. All the results I achieved passed, but to widely varying degrees.

Conclusions

As has been apparent since completing technical report 1 the results I have calculated for strength and design of members has often been less than what is actually in the building. The lateral deflection is too large, but there are other design possibilities such as the stiffness of columns that would make up for this value. The total loads on my shear walls when calculated by hand were not unreasonably different from the loads found in ETABS. The combination of imperfect measuring of eccentricities and locations of members when drawn in ETABS should account for most of the inconsistency. After this analysis it is clear that while the geometry of the shear wall system present in this structure is grossly irregular, it is nearly adequate in carrying the needed loads down to the foundation and ultimately into the earth.

Appendix A

Direct Wind Loads

floor #	V(EW)	V(NS)
1	0.0	0.0
2	119.4	76.2
3	118.0	75.3
4	124.5	79.5
5	131.9	84.2
6	137.9	88.0
7	141.2	90.1
8	186.2	129.3
9	112.7	78.3
10	100.3	69.7
11	101.9	70.8
12	103.4	71.8
14	104.8	72.8
15	106.2	73.7
16	107.5	74.6
17	108.7	75.5
18	109.9	76.3
19	111.0	77.1
20	112.1	77.9
21	113.2	78.6
22	126.3	99.5
23	127.7	100.5
24	117.8	92.8
25	118.9	93.6
26	119.9	94.4
27	75.0	104.2
28	56.1	77.9
29	61.8	85.9
30	29.0	68.5

(Please refer to Technical Report 1 for calculations of these values)

level	wall	length	%load
roof	A	15	0.60
	B	10	0.40
28	A	15	0.43
	B	20	0.57
23-27	A	34.75	0.63
	B	20	0.37
9-22	A	51.58	0.42
	B	20	0.16
	C	31.65	0.26
	E	19.85	0.16
1-8	A	24.5	0.21
	B	20	0.17
	C	15.3	0.13
	D	15.8	0.13
	E	10.55	0.09
	F	18.92	0.16
	G	2.35	0.02
	H	10.34	0.09

Appendix B

Torsional Wind Loads

level	wall	length	rotation Θ	effective length X	effective length Y	distance X	distance Y
roof	A	15	45	10.6	10.6	88	36
	B	10	45	7.1	7.1	62	68
28	A	15	45	10.6	10.6	88	36
	B	20	45	14.14	14.14	62	68
23-27	A	34.75	45	24.6	24.6	88	36
	B	20	45	14.14	14.14	62	68
9-22	A	51.58	45	36.38	36.38	88	36
	B	20	45	14.14	14.14	62	68
	C	31.65	45	22.38	22.38	56	100
	E	19.85	0	0	19.85	0	46
1-8	A	24.5	45	17.32	17.32	108	60
	B	20	45	14.14	14.14	80	88
	C	15.3	45	10.82	10.82	80	120
	D	15.8	45	11.17	11.17	40	90
	E	10.55	0	0	10.55	0	68
	F	18.92	0	9.92	16.83	94	24
	G	2.35	45	1.66	1.66	128	156
	H	10.34	15	10	2.68	156	130

CR x dimension	CR y dimension	linear dimension of incidence	wind resultant	eccentricity
77.57	48.84	91.66	72	-19.66
73.14	54.29	91.09	72	-19.09
78.51	47.68	91.85	72	-19.85
73.13	58.46	93.63	72	-21.63
93.55	74.16	119.38	100	-19.38

R/Et	x	Rx^2	sums	Rx/sums
0.26	13	43.28		0.02549
0.11	28	87.31	130.58	0.02388
0.26	15	57.62		0.01374
0.42	23	222.04	279.66	0.03452
0.91	13	153.52		0.02447
0.42	28	329.07	482.59	0.02435
1.44	16	369.75		0.03127
0.42	20	167.89		0.01136
0.81	8	51.66		0.00874
0.41	19	149.72	739.02	0.01066
0.57	2	2.28		0.00022
0.42	18	135.99		0.00146
0.27	22	128.55		0.00113
0.28	25	175.99		0.00136
0.13	7	6.15		0.00017
0.38	100	3837.81		0.00741
0.00	44	4.33		0.00002
0.12	86	887.57	5178.66	0.00199

level	A	B	C	D	E	F	G	H
ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
1	0.51	3.38	2.61	3.15	0.39	17.15	0.04	4.61
2	0.50	3.34	2.58	3.11	0.39	16.95	0.04	4.56
3	0.53	3.52	2.73	3.28	0.41	17.89	0.05	4.81
4	0.56	3.73	2.88	3.47	0.43	18.94	0.05	5.09
5	0.59	3.90	3.02	3.63	0.45	19.81	0.05	5.33
6	0.60	3.99	3.09	3.72	0.46	20.28	0.05	5.45
7	0.80	5.26	4.07	4.91	0.61	26.74	0.07	7.19
8	0.54	3.56	2.75	3.32	0.41	18.07	0.05	4.86
9	67.83	24.64	18.95		23.13			
10	68.90	25.03	19.25		23.49			
11	69.92	25.40	19.54		23.84			
12	70.88	25.75	19.81		24.17			
14	71.80	26.08	20.06		24.48			
15	72.68	26.40	20.31		24.78			
16	73.52	26.71	20.54		25.07			
17	74.32	27.00	20.77		25.34			
18	75.09	27.28	20.98		25.61			
19	75.84	27.55	21.19		25.86			
20	756.55	27.81	21.39		26.10			
21	85.44	31.04	23.87		29.13			
22	79.25	28.79	22.14		27.02			
23	73.13	26.57	20.44		24.94			
24	57.74	57.46						
25	58.24	57.96						
26	35.02	34.86						
27	26.19	26.06						
28	16.70	41.97						
roof	14.54	13.62						
sum	1928.21	638.65	312.97	28.58	356.53	155.83	0.40	41.90

Appendix C

Wind Load Cases

Loading	level	A	B	C	D	E	F	G	H
Case 1	ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1	24.84	20.28	15.51	16.02	10.70	19.18	2.38	10.48
	2	24.55	20.04	15.33	15.83	10.57	18.96	2.35	10.36
	3	25.91	21.15	16.18	16.71	11.16	20.01	2.49	10.94
	4	27.43	22.39	17.13	17.69	11.81	21.19	2.63	11.58
	5	28.69	23.42	17.92	18.50	12.36	22.16	2.75	12.11
	6	29.38	23.98	18.35	18.95	12.65	22.69	2.82	12.40
	7	38.74	31.62	24.19	24.98	16.68	29.91	3.72	16.35
	8	47.25	18.32	28.99		18.18			
	9	42.02	16.29	25.79		16.17			
	10	42.69	16.55	26.20		16.43			
	11	43.32	16.80	26.58		16.67			
	12	43.92	17.03	26.95		16.90			
	14	44.49	17.25	27.30		17.12			
	15	45.03	17.46	27.63		17.33			
	16	45.55	17.66	27.95		17.53			
	17	46.05	17.86	28.26		17.72			
	18	46.53	18.04	28.55		17.91			
	19	46.99	18.22	28.83		18.08			
	20	47.43	18.39	29.10		18.25			
	21	52.94	20.53	32.48		20.37			
	22	53.50	20.75	32.83		20.59			
	23	49.38	19.15						
	24	75.45	43.42						
	25	76.10	43.80						
	26	47.59	27.39						
	27	35.58	20.48						
	28	26.50	35.34						
	roof	17.41	11.61						
	sum	1175.25	615.21	522.05	128.68	335.19	154.09	19.14	84.21

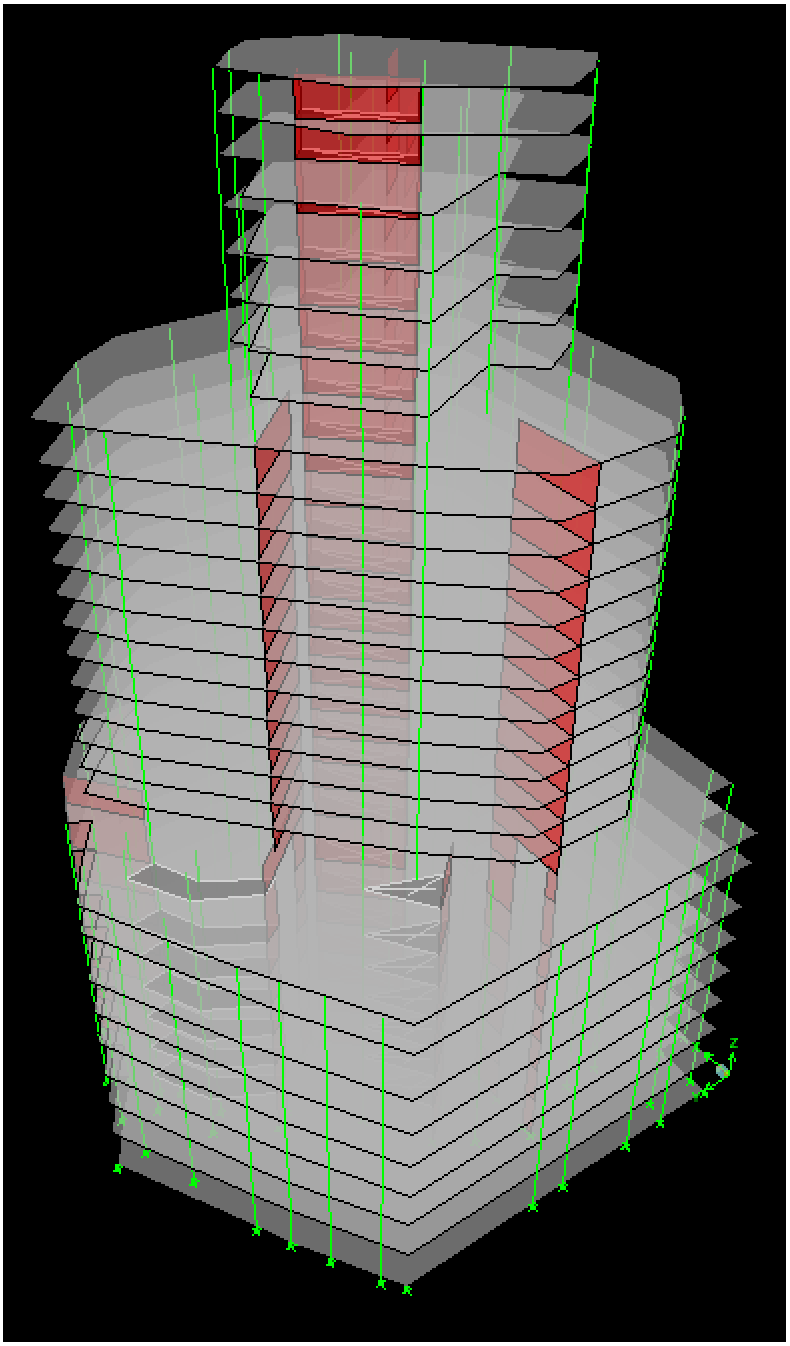
TOTAL	level	A	B	C	D	E	F	G	H
LOADING	ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Case 2	1	19.01	17.74	13.59	14.37	8.32	27.25	1.82	11.32
	2	18.79	17.53	13.43	14.20	8.22	26.93	1.80	11.19
	3	19.83	18.50	14.18	14.99	8.67	28.42	1.90	11.81
	4	21.00	19.59	15.01	15.87	9.18	30.09	2.01	12.50
	5	21.96	20.49	15.70	16.60	9.61	31.48	2.10	13.08
	6	22.49	20.98	16.08	17.00	9.84	32.23	2.15	13.39
	7	29.65	27.66	21.20	22.41	12.97	42.49	2.84	17.65
	8	35.84	16.41	23.81		13.95			
	9	82.39	30.70	33.55		29.48			
	10	83.69	31.19	34.08		29.94			
	11	84.93	31.65	34.59		30.38			
	12	86.10	32.08	35.07		30.80			
	14	87.22	32.50	35.52		31.20			
	15	88.28	32.90	35.96		31.58			
	16	89.30	33.28	36.37		31.95			
	17	90.28	33.64	36.77		32.30			
	18	91.21	33.99	37.15		32.64			
	19	92.12	34.33	37.52		32.96			
	20	602.99	34.65	37.87		33.27			
	21	103.78	38.67	42.27		37.13			
	22	99.57	37.15	41.23		35.71			
	23	91.88	34.29	15.33		18.71			
	24	99.89	75.66						
	25	100.75	76.32						
	26	61.95	46.69						
	27	46.33	34.90						
	28	32.40	57.98						
	roof	23.96	18.92						
	sum	2327.60	940.40	626.27	115.46	518.79	218.89	14.62	90.94

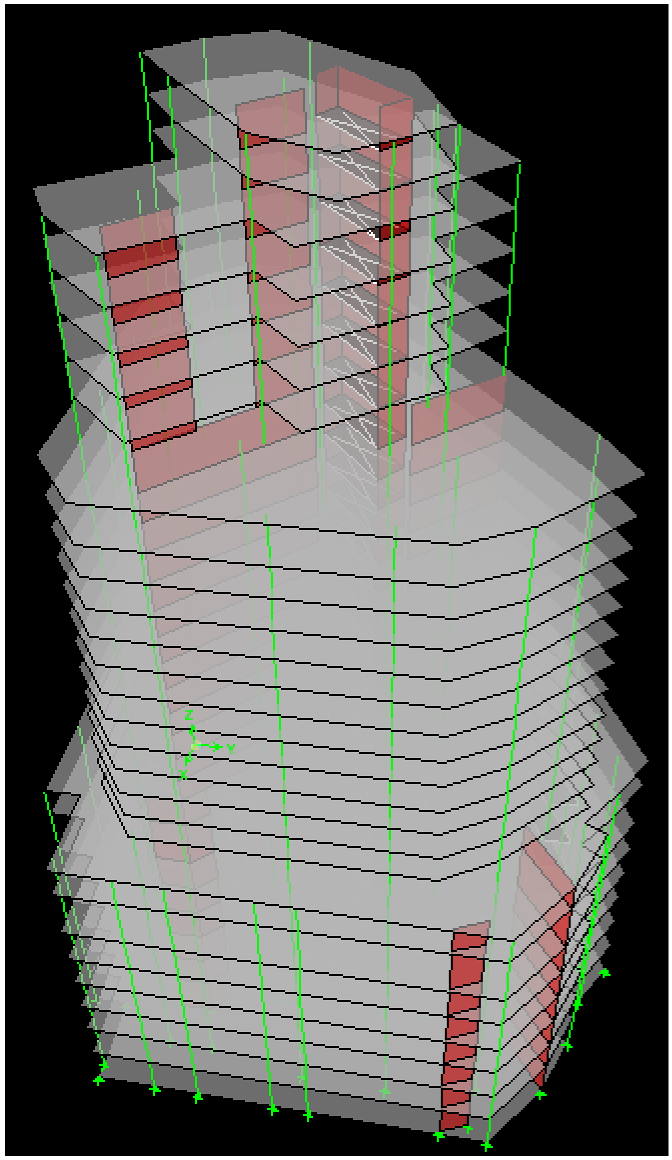
TOTAL	level	A	B	C	D	E	F	G	H
LOADING	ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Case 3	1	37.26	30.41	23.27	24.03	16.04	28.77	3.57	15.72
	2	36.82	30.06	22.99	23.75	15.86	28.43	3.53	15.54
	3	38.87	31.73	24.27	25.06	16.74	30.01	3.73	16.40
	4	41.15	33.59	25.70	26.54	17.72	31.78	3.95	17.37
	5	43.04	35.13	26.88	27.76	18.53	33.24	4.13	18.16
	6	44.07	35.97	27.52	28.42	18.98	34.03	4.23	18.60
	7	58.10	47.43	36.29	37.47	25.02	44.87	5.57	24.52
	8	53.03	28.10	32.73		21.21			
	9	47.17	24.99	29.11		18.87			
	10	47.91	25.39	29.57		19.17			
	11	48.62	25.77	30.01		19.45			
	12	49.29	26.12	30.42		19.72			
	14	49.93	26.46	30.82		19.97			
	15	50.54	26.78	31.19		20.22			
	16	51.12	27.09	31.55		20.45			
	17	51.68	27.39	31.90		20.67			
	18	52.22	27.67	32.23		20.89			
	19	52.74	27.95	32.55		21.10			
	20	53.23	28.21	32.86		21.30			
	21	59.41	31.49	36.67		23.77			
	22	60.05	31.82	37.06		24.02			
	23	55.42	29.37	0.00		0.00			
	24	75.14	47.71						
	25	75.78	48.12						
	26	47.39	30.09						
	27	35.43	22.50						
	28	29.53	34.38						
	roof	17.59	12.40						
	sum	1362.53	854.13	635.60	193.02	419.68	231.14	28.71	126.32

TOTAL	level	A	B	C	D	E	F	G	H
LOADING	ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
Case 4	1	28.25	24.73	18.93	19.81	12.26	31.25	2.71	14.40
	2	27.92	24.44	18.71	19.57	12.12	30.89	2.68	14.23
	3	29.47	25.80	19.76	20.66	12.79	32.60	2.82	15.02
	4	31.21	27.32	20.91	21.88	13.55	34.52	2.99	15.90
	5	32.64	28.57	21.87	22.88	14.17	36.10	3.13	16.63
	6	33.42	29.25	22.40	23.43	14.51	36.96	3.20	17.03
	7	44.06	38.57	29.53	30.89	19.13	48.74	4.22	22.46
	8	40.11	23.10	26.12		16.16			
	9	73.59	32.63	32.52		27.19			
	10	74.76	33.15	33.04		27.61			
	11	75.86	33.64	33.53		28.02			
	12	76.91	34.11	33.99		28.41			
	14	77.91	34.55	34.43		28.78			
	15	78.86	34.97	34.85		29.13			
	16	79.77	35.37	35.25		29.47			
	17	80.64	35.76	35.64		29.79			
	18	81.47	36.13	36.01		30.10			
	19	82.28	36.49	36.36		30.39			
	20	465.90	36.83	36.71		30.68			
	21	92.70	41.11	40.97		34.24			
	22	89.70	40.10	40.29		33.24			
	23	82.77	37.00	11.51		14.04			
	24	88.91	68.16						
	25	89.67	68.75						
	26	55.29	42.21						
	27	41.34	31.56						
	28	31.57	49.44						
	roof	21.39	16.98						
	sum	2108.39	1000.73	653.33	159.12	515.77	251.07	21.75	115.68

<u>**Appendix D**</u>

Computer Model





Appendix E

Drift Results

Story	Drift (in)	
	X	Y
1	0.12	0.10
2	0.29	0.24
3	0.40	0.34
4	0.47	0.41
5	0.50	0.46
6	0.49	0.47
7	0.44	0.45
8	0.38	0.45
9	0.38	0.69
10	0.51	0.80
11	0.59	0.83
12	0.64	0.83
13	0.69	0.84
14	0.73	0.83
15	0.75	0.82
16	0.78	0.81
17	0.80	0.80
18	0.82	0.80
19	0.84	0.79
20	0.86	0.79
21	0.87	0.79
22	0.56	0.51
23	0.26	0.20
24	0.26	0.19
25	0.31	0.24
26	0.34	0.25
27	0.29	0.20
28	0.30	0.20
total	14.69	15.13

H/400 =	9.39	in

allowable story drift (in)		
H/400	X difference	Y difference
0.47	0.35	0.37
0.41	0.12	0.17
0.40	0.00	0.06
0.40	-0.07	-0.02
0.40	-0.11	-0.06
0.39	-0.10	-0.08
0.54	0.10	0.09
0.32	-0.06	-0.13
0.28	-0.10	-0.41
0.28	-0.23	-0.52
0.28	-0.31	-0.55
0.28	-0.36	-0.55
0.28	-0.41	-0.56
0.28	-0.45	-0.55
0.28	-0.47	-0.54
0.28	-0.50	-0.53
0.28	-0.52	-0.52
0.28	-0.54	-0.52
0.28	-0.56	-0.51
0.28	-0.58	-0.51
0.35	-0.52	-0.44
0.35	-0.21	-0.16
0.32	0.06	0.12
0.32	0.06	0.13
0.32	0.01	0.09
0.35	0.01	0.10
0.26	-0.03	0.06
0.29	-0.02	0.09
9.39	-5.30	-5.74

The negative difference values show points at which the modeled deflection is greater than the permitted deflection of the building.

Appendix F

Shear Wall Checks

Loads on Piers

Pier	P	V2	V3	T	M2 (in-k)	M3 (in-k)
A	-526318	11207.89	-9181.41	-85324.4	999815.7	-7921538
B-1	-836416	22028.86	-2772.31	-12953.3	18537.32	-788859
B-2	-238981	14154.07	5506.38	6684.299	-29451.5	-19317.7
B-3	-205481	9931.86	-620.48	-2847.17	8490.576	-134742
C	-979264	26837.73	-1965.92	18974.24	-567154	-4058668
D	-1076983	22478.49	-17514.5	-32465.3	129570	-1286295
E	-183679	-5644.93	1386.48	-42191.4	-9409.6	-3392485
F-1	-2808.55	-6091.19	1219	-212.657	-10554.2	-288850
F-2	-460365	-17619.9	-153.83	-2848.78	13816.84	-68856
F-3	-171472	-12734.8	3995.7	3510.855	-28568.9	-284858
G	-519243	25847.95	12201.33	-805.026	-100607	2495619
H	-102198	29251.6	-66.68	-2372.92	1044.857	639521.2

Shear Wall "C" at ground level

Vertical: 52 #9 (≈ #9 @ 7) (2" cover)

horizontal: #5 @ 12" (1.5" cover)

$f'_c = 10{,}000$ psi

$V_u = 653^k$

$\phi V_c = .75(2)\sqrt{10000}\,(18)(183.5) = 495^k$

$V_s = 158^k$

$A_v = \frac{V_s S}{\phi f_y d} = \frac{158(12)}{(.9)(60)(150)} = 0.195 in^2$

$\rho = 0.000903 \ngtr .0025$ X

$A_v = .0025(18 \cdot 12) = 0.54 in^2$

#5 each face = $0.61 in^2$

$\boxed{\#5 @ 12"}$ good

Shear Wall "G" at ground level

Vert: #5 @ 12" each face

horiz: #5 @ 8" each face

$f'_c = 10{,}000$ psi

$V_u = 28.7^k$

$\phi V_c = .75(2)\sqrt{10000}\,(20)(266) = 800^k$

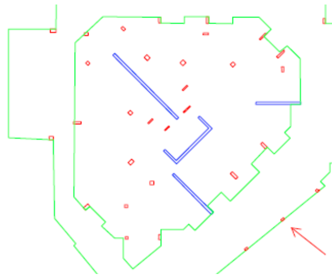
Use minimum S+T reinforcement.

OK

Appendix G

Computer Column Strength Checks

Example of critical column:

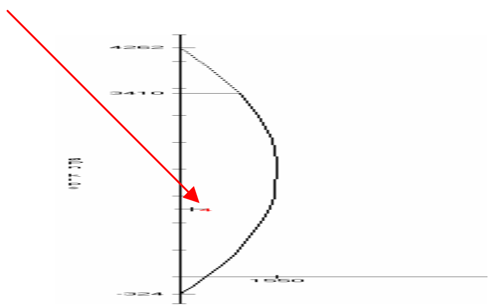


C337

	P	V2	V3	T	M2	M3
C337	1261416	43510.32	35813.36	12123.68	279450.5	327472.2

(name C337 auto-defined in ETABS)

Points on inside of interaction diagram confirm that column passes.

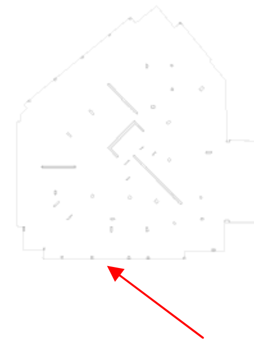


(PCACOL generated interaction diagram)

Appendix H

Hand Column Strength Checks

As an example I checked column 26. I used the values of loads that resulted from the ETABS analysis in conjunction with live load reduction in order to lessen the present axial load.



$A = 18' \times 15' = 270'^2$

$A_T = 1890'^2$

$D = \left(\frac{9.5}{12}\right)(150) + 20 = 140_{psf}$

$L = 42_{psf}$

$L_o = 17.75_{psf}$

$1.2(140) + 1.6(17.75) = 196.4_{psf}$

$196\,(270) = 53\ ^{k}/_{flr}$

$7 \text{ floors} = \underline{371\ k}$

I then used the reciprocal load method to qualify this column.

$P_{nxo} = .85f'_c ab + A_s'f_s'$ $\qquad a = 3.125$

$= .85(10)(3.1)(20) + (1.76)(60)$

$= 637^K$

$P_{nyo} = .85f'_c ab + A_s'f_s'$ $\qquad a = 3.525$

$= .85(10)(3.5)(18) + 1.76(60)$

$= 645^K$

$P_o = .85f'_c A_g = .85(10)(360) =$

$= 3060^k$

$$\phi P_n = \frac{1}{\frac{1}{637} + \frac{1}{645} + \frac{1}{3060}} = \underline{358^k}$$

This just failed.

$\boxed{371 \not< 358 \quad X}$

Upon further inspection the column to the northeast of column 26 will alter the tributary area of column 26 substantially enough to limit the loads that are seen by this column and allow it to pass strength checks.

Appendix I

Overturning

Overturning	(For each level)
0.00	
1870.68	
3451.06	
5292.88	
7351.23	
9516.12	
11590.74	
18633.37	
12486.54	
12041.53	
13183.18	
14342.31	
15517.98	
16709.35	
17915.70	
19136.34	
20370.69	
21618.19	
22878.34	
24150.68	
28428.55	
30222.68	
29147.73	
30676.38	
32218.39	
21022.60	
16204.81	
18464.95	
9099.28	
483542.29	ft-k

This yields 2700 kips that need to be resisted by the piles. Based on the values given in the drawings 6 piles would be needed in compression and 14 in tension. That is well short of the 250+ piles present and doesn't begin to account for the 5 foot thick concrete mat either.

Appendix J

Seismic Loading Check

Seismic Loads on the Structure

floor #	V (k)	M (ft-k)
1	0.0	0.00
2	0.2	0.19
3	0.5	0.48
4	0.8	0.84
5	1.3	1.26
6	1.7	1.74
7	2.3	2.25
8	3.2	56.70
9	2.9	52.92
10	3.3	59.76
11	3.7	66.78
12	4.1	74.16
14	4.5	81.72
15	5.0	89.64
16	5.4	97.56
17	5.9	105.84
18	6.4	114.48
19	6.8	123.12
20	7.3	132.12
21	7.8	141.12
22	8.5	152.82
23	5.3	26.25
24	5.3	26.60
25	5.7	28.30
26	6.0	30.05
27	6.4	32.05
28	5.6	28.20
29	4.7	23.35
30	3.7	18.65

TOTAL LOADING	level	A	B	C	D	E	F	G	H
	ground	0.00	0.00	0.00	0.00	0.00	0.00	0.00	0.00
	1	0.04	0.03	0.02	0.03	0.02	0.03	0.00	0.02
	2	0.14	0.11	0.09	0.09	0.06	0.11	0.01	0.06
	3	0.27	0.22	0.17	0.18	0.12	0.21	0.03	0.12
	4	0.44	0.36	0.27	0.28	0.19	0.34	0.04	0.18
	5	0.62	0.51	0.39	0.40	0.27	0.48	0.06	0.26
	6	0.83	0.68	0.52	0.54	0.36	0.64	0.08	0.35
	7	1.12	0.92	0.70	0.72	0.48	0.87	0.11	0.47
	8	13.03	10.11	8.12		5.55			
	9	23.57	9.14	14.46		9.07			
	10	26.60	10.31	16.32		10.24			
	11	29.71	11.52	18.23		11.43			
	12	32.98	12.79	20.24		12.69			
	14	36.33	14.09	22.29		13.98			
	15	39.84	15.45	24.44		15.33			
	16	43.35	16.81	26.60		16.68			
	17	47.02	18.23	28.85		18.10			
	18	50.84	19.71	31.20		19.57			
	19	54.67	21.20	33.55		21.04			
	20	58.65	22.74	35.99		22.57			
	21	62.70	24.31	38.47		24.13			
	22	66.24	25.69	40.65		25.49			
	23	18.89	10.45	6.75		4.23			
	24	20.48	11.78						
	25	21.78	12.53						
	26	23.14	13.32						
	27	23.92	13.77						
	28	14.09	18.78						
	roof	16.25	10.83						
	sum	727.55	326.40	368.34	2.24	231.61	2.68	0.33	1.46

Controlling Wind Values

sum	2327.60	940.40	626.27	115.46	518.79	218.89	14.62	90.94

The wind values are clearly higher as was expected. This confirms that wind does control the design of this building structure.