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Renaissance Schaumburg Hotel and Convention Center
Schaumburg, Illinois

Lateral Systems Analysis and Conformation Design
Due: November 21st, 2005

Contents

Executive Summary

This technical assignment includes a detailed analysis of the current lateral force resisting system of the 17 story Renaissance Schaumburg Hotel and Convention Center in Schaumburg, IL.

The first part of the technical paper discuses lateral load development and application. The shear wall system used is then analyzed with both a simplified approach and a computer model. The shear walls in the RSHCC resist a portion of the total shear per floor based off of their relative stiffness; the longer walls being in the critical direction are also the longest and thickest of the shear walls used on site.

Story drifts were also calculated for the entire building and resulted in ~ 3" displacement of the top shear wall to the ground. Though this value is small (corresponding to drift limit of L/700) it is within a reasonable drift given the analysis methods.

Shear wall "I", the east-most shear wall was spot checked for shear and flexural reinforcement as well as for overturning. This element was designed to resist the required shear based on the shear forces derived from ETABS, and passed the check while using the same reinforcement as specified in the drawings. After inspection of overturning moment it was found to be counter acted by the building and wall self-weight so that no tension needed to be countered by a foundation system.

This report is limited to analysis based on the most current design documents made available for the Renaissance Schaumburg Hotel and Convention Center by the lead structural engineer and architecture firm. Its function is to provide a detailed description and analysis of the lateral system currently in use. Simplified sketches have been included to further explain system layouts and details. Please see the appendix for other figures. This report will further detail the analysis of lateral force resisting shear walls constructed as part of the design of the Renaissance Schaumburg Hotel and Convention Center.

Introduction – Existing Design

The Renaissance Schaumburg Hotel and Convention Center (RSHCC) in Schaumburg, IL is composed of 17 stories of guest suites, conference rooms, restaurants, ballrooms, and many other impressive architectural spaces. The RSHCC succeeds in not only being able to create beautiful and functional spaces, but also is able to bring efficiency to many other building systems. This success is due, in large part, to the way in which the building was designed. Post-tensioned slabs account for most of the diaphragm elements in the structure, which take up very little ceiling space and work well with the lateral system. The major lateral force resisting elements are 11”-18” shear walls located (in orange) on the floor plan shown below in figure 1.

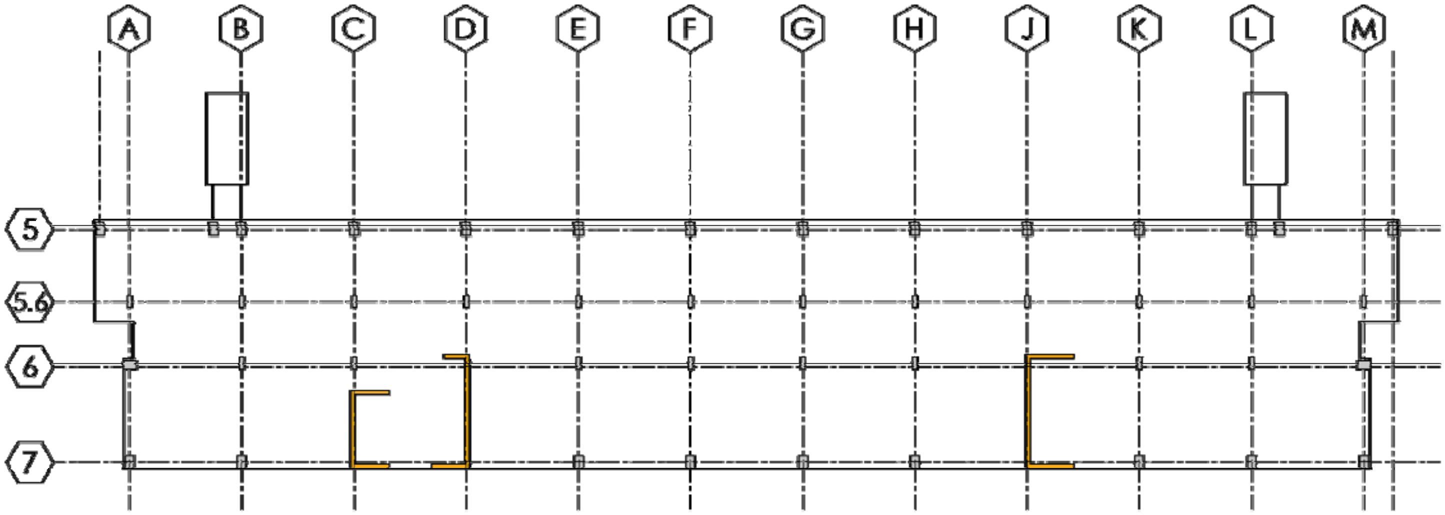


Figure 1 - Shear walls on typical floor

These shear walls are to be constructed of 8,000 psi concrete on lower floors (up to floor 6) and 6,000 psi concrete on the upper levels, this is also when they change from a maximum of 18” thick at the bottom, to no less than 11”. Reinforcement for the shear walls is typical ASTM A615 Grade 60 steel varying from #4’s at 12” as a minimum, to #8’s at 8”. As one can see from the figures on this page, there are 9 shear walls that create 3 C-shaped patterns. The naming convention used throughout the rest of this paper will refer to each wall with a letter as shown below in figure 2.

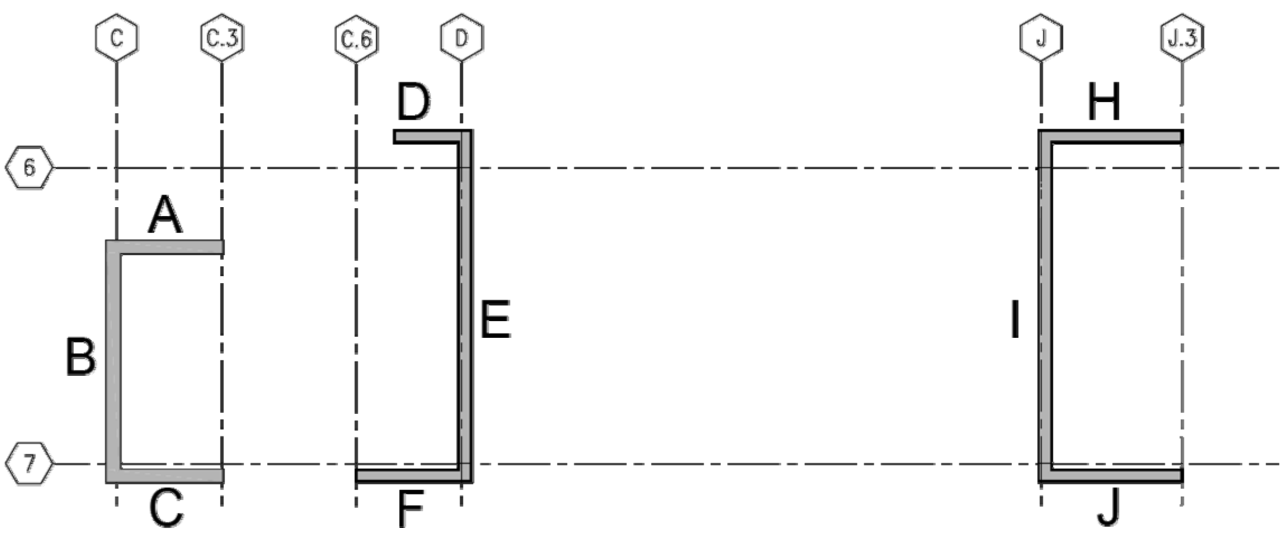


Figure 2 - Shear wall naming convention

This system is an excellent choice since the entire hotel structure is cast of concrete. Shear walls will transfer wind and other lateral loads through the building so that lateral loads including direct and torsional shears can be passed into competent soil through the foundations. The rest of this article will discuss the analysis process of the shear walls including: loading calculations, force distributions, model analysis, member spot checks, and most importantly the concluding impacts of the lateral force resisting shear wall system utilized in the Renaissance Schaumburg Hotel.

Loads and Load Cases

Gravity Loads

The dead load and live load figures are similar to those used in the actual design process and are referenced from ASCE 7-02. Introductions to the gravity loads that will be used through the rest of this study are as follows.

Live Loads (psf):

o Typical Floors (Hotels refer to residential)	40 psf
o Public rooms and Corridors	100 psf
	Design Total = 100 psf

Dead Loads (psf):

o Mechanical/Ceiling/Carpet/Miscellaneous	7 psf
o Partitions	20 psf
o Carpet/Miscellaneous	5 psf
	Total = 32 psf

Dead Loads (psf):

o Slab self-weight	10" @ 150 pcf
	Total = 125 psf

Snow Loads (psf):

o Mechanical/Ceiling/Carpet/Miscellaneous.	7 psf
o Partitions	20 psf
o Carpet/Miscellaneous	5 psf
	Total = 32 psf

Load Combinations (Controlling Case):

o U=1.2D+1.6L .... (Gravity)
o U=1.2D+1.6W+L..... (Overturning)
o U=1.2D+1.6L+0.8W .... (Wind and Gravity)
o U=1.2D+E+L .... (Seismic)

Wind Loads

The procedure used in the first technical assignment was used again to find the wind forces affecting the lateral system, however, this version is updated to account for small errors discovered in the worksheet. ASCE-02 was used to develop the wind loads as presented in figure 3. Wind blowing in the north-south direction controls the lateral system design since, as the next section states, seismic is not to be considered.

Load Analysis Summary

	Wind N-S	Wind E-W
Shear @ 18	52.66	44.68
Shear @ 17	104.36	88.40
Shear @ 16	128.17	108.56
Shear @ 15	124.62	105.36
Shear @ 14	124.30	105.05
Shear @ 12	123.36	104.12
Shear @ 11	123.03	103.80
Shear @ 10	121.61	102.39
Shear @ 9	120.91	101.71
Shear @ 8	119.64	100.45
Shear @ 7	121.62	101.91
Shear @ 6	121.26	101.45
Shear @ 5	116.47	97.21
Shear @ 4	114.08	94.97
Shear @ 3	112.44	93.36
Shear @ 2	197.03	162.88
Shear @ 1	216.26	176.71
Shear @ Ground	83.94	68.28
Base Shear	2,225.76	1,861.29
Overturning Moment	212,133.48	178,533.71
*Shear (kips) **Moment (ft-kips)		

Figure 3 - Wind Load Table

Seismic Loads

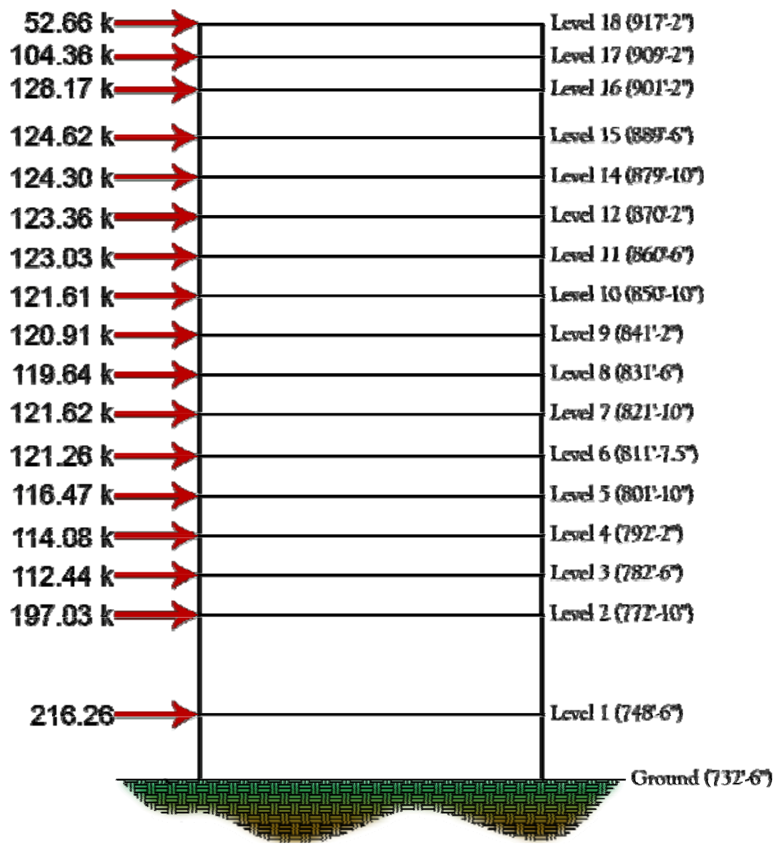
The same load development for seismic analysis as presented in technical assignment one was used again for this analysis (the procedure below follows those gravity loads presented above). However, since local building codes state that seismically induced forces need not be considered, they are also dropped from this analysis, though are available upon request or at the same location at the same web address as the wind spreadsheet cited in Appendix C.

Load Discussion

The process described above implies that wind controls the design of this project. The analysis to follow will concentrate on the distribution of wind pressures, and consequent analysis based on these load developments. Multiple load cases were not considered in this part of the assignment since lateral forces are all that were applied to both models of analysis. In later development of the model, columns and beams will be designed and load combinations that include live, and dead load will be considered. The live and dead loads listed above were used in seismic calculations, for lateral member spot checks, and will be referenced in the member check section of this paper when necessary.

Distribution *— How the system handles lateral force*

Description



The RSHCC is an entirely cast in place concrete structure, utilizing shear walls for lateral support; but first the load has to get there.

Loads were developed based from ASCE 7-02 for wind pressures, these pressures where distributed over the face of the exterior. The controlling direction would be a wind applied along the long side of the building blowing in the north-south direction.

After pressures are found based on the code, story forces are then calculated based on the length and magnitude of each pressure envelope. These story forces are displayed on the left hand side of the figure shown here. These story forces are then converted to story shears to analyze the diaphragm elements of the structure. These forces are used to arrive at the base shear which is used to ensure the shear wall at the base is strong enough to resist the entire buildings shear, bending and overturning moment.

The shear walls are considered as deep cantilevered beams extending vertically from the ground. Each element resists a portion of the total shear based on its stiffness relative to the other lateral resisting elements. Stiffness is a function of each element's width, length, modulus of Elasticity, and height, the shorter, longer, or wider the wall, the larger the stiffness will be. In the RSHCC there are 9 such shear walls, the three longest of which must resist shear development in the critical north-south direction. The process to determine each elements stiffness and relative proportion is highlighted in the Appendix (B) and in the next section of this paper.

Discussion

Major assumptions for both analysis types and distribution methods are outlined below:

- Shear walls are to resist all shear forces (columns will not contribute in simplified analysis)
- Openings in slabs are accounted for in ETABS
- When necessary a 5% incidental eccentricity is assumed
- Deflection analysis is to be completed in ETABS
- Direct shears are not to be reduced by negative torsional shear
- Foundation deformation is neglected
- Total shear forces will be compared from both the simplified shear wall calculations and the ETABS model

Analysis — *How the system works*

Wall B	
Floor	**Shear**
Roof	18.304
16	18.091
15	17.427
14	17.847
12	17.921
11	17.921
10	17.921
9	17.921
8	17.921
7	17.789
6	17.891
5	17.921
4	17.921
3	17.921
2	14.887
Ground	16.385
Base Shear 282 kips	

Wall E	
Floor	**Shear**
Roof	33.134
16	40.688
15	40.229
14	39.705
12	39.354
11	39.232
10	38.701
9	38.443
8	37.967
7	38.627
6	38.368
5	36.744
4	35.897
3	35.287
2	65.149
Ground	68.821
Base Shear 666 kips	

Wall I	
Floor	**Shear**
Roof	68.394
16	75.536
15	76.454
14	75.275
12	75.071
11	74.949
10	74.419
9	74.160
8	73.684
7	74.486
6	74.118
5	72.462
4	71.614
3	71.004
2	103.463
Ground	105.967
Base Shear 1241 kips	

Critical Walls

Summary of Analysis Procedures

The shear wall system described earlier was loaded with the development procedure introduced in the previous section. This system was first examined using excel to determine shear forces in each shear wall, then analyzed in ETABS using a simplified model of the hotel structure. These two methods where used as a double check to ensure that a reasonable answer would be determined. In this case, there was a great deal of difference in the two types of analysis since a simplified model was used in ETABS and only a typical floor plan was used to determine shear forces in the excel spreadsheet. The results of both methods are presented here and further explained in the Appendices (A and B) that accompany this paper. To the left you can examine the results from the simplified analysis using proportional stiffness calculations in excel, and below the results derived from the model created in ETABS.

ETABS solution:

Shear Wall	Max Shear (k)	Max Moment (k-ft)
B	287.21	8749 k-ft
E	573.89	22419 k-ft
I	998.03	34229 k-ft

Discussion

Although the shear in wall B is similar in both methods, the other walls are significantly different; this implies a distinct difference in analysis procedures. This difference is not entirely surprising since it was known that even though a simplified model was used in both procedures, differences in modeling were abundant.

When using ETABS a slightly more detailed floor plan, per floor, was considered. The plan included every major slab opening, where as the excel model was based off of a typical floor plan. The same lateral loads were used (the wind forces from technical assignment one) in both models to keep control of the outputs. Since the ETABS model was slightly more detailed in dimension, the rest of the member checks and reaction forces considered in this article will references those numbers from the ETABS analysis.

Shear Wall	ETABS	Stiffness	%
	Max Shear (k)	Max Shear (k)	Difference
B	287.2	282.0	1.831
E	573.9	666.0	14.858
I	998.0	1241.0	21.703

Drift Considerations

Although the system may initially look straight-forward, many complexities arose when modeling the structure, the first of which included varying strengths of the shear walls, and their thickness on a per floor basis. The three critical shear walls (labeled B, E, and I) all have 8,000 psi concrete at the base and are 18" thick, which then reduce to 6,000 psi concrete at 12" thick after you reach the sixth floor. The other shear walls had varying thickness from the ground level through floor three of 14", four through six of 12", and seven through the top of 11".

The drift calculated by ETABS resulted in a total building drift of 2.7" which is a rather reasonable performance of the lateral system since a typical L/400 deflection limit would place the value at a maximum of 5.2". This number is assumed to be closer to the actual value than a hand calculation would be since it takes into account the varying concrete strengths, wall thickness, and change in floor plan dimensions of the building. In the future an even more detailed model will be created in order to analyze the possible contributions to shear resistance of columns throughout the building floor plan.

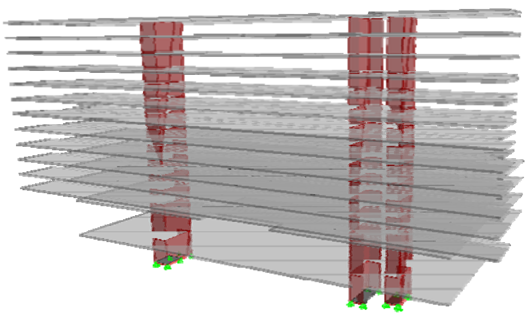
Member Check — *Analysis with context*

Shear wall "I" was checked to ensure that the numbers derived from a model were similar to the actual design. The shear wall's reinforcement calculation at the first level of the building resulted in the same reinforcement that was called for in the shear wall schedule. A summary of the design can be found below with a detailed processes going through the shear and flexural reinforcement design and double check of possible overturning moment problems in Appendix D. The process used followed a PCA design aid and the following reinforcement was found to satisfy applicable ACI codes.

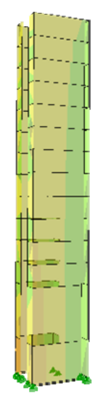
Shear Wall I	First Story
Verticle Reinforcement	#8's @ 12"
Horizontal Reinforcement	#6's @ 12"
Overturning	OK

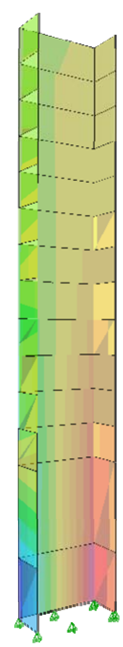
Since this wall preformed so well, and coincided with the reinforcement specified, additional checks we omitted in the interest of time. Additional structural analysis will be completed after a more complete computer model is drafted (to analyze gravity loads) and more checks will be presented in subsequent assignments.

Conclusion — *Summary of Analysis and Structural Impact*



After exploring lateral system in place at The Renaissance Schaumburg Hotel with two different methods of analysis and running simplified member checks it is easy to conclude that the shear wall system performs sufficiently. The member check of wall "I" resulted in a very reasonable reinforcement schedule and despite difference in the analysis procedures, both the computer model and the simplified shear wall approach yielded shear reactions that were within an acceptable vicinity.





Overturning was also considered for this wall and it was found to be stable since the moment created due to the wall's self-weight and tributary area was large enough to counter act that the lateral forces, this result implies that currently there is not need to develop a system to resist tension in the foundations. The drive steel piles will only be experiencing compression forces.

Further exploration of the entire building system will be necessary in order to correctly model and analyze the structure, including the addition of the gravity resisting system to the shear wall model created in ETABS. A final design conformation will be complete with a model which includes those items mentioned above, a process which will also help lead to a more complete drift analysis of the building.

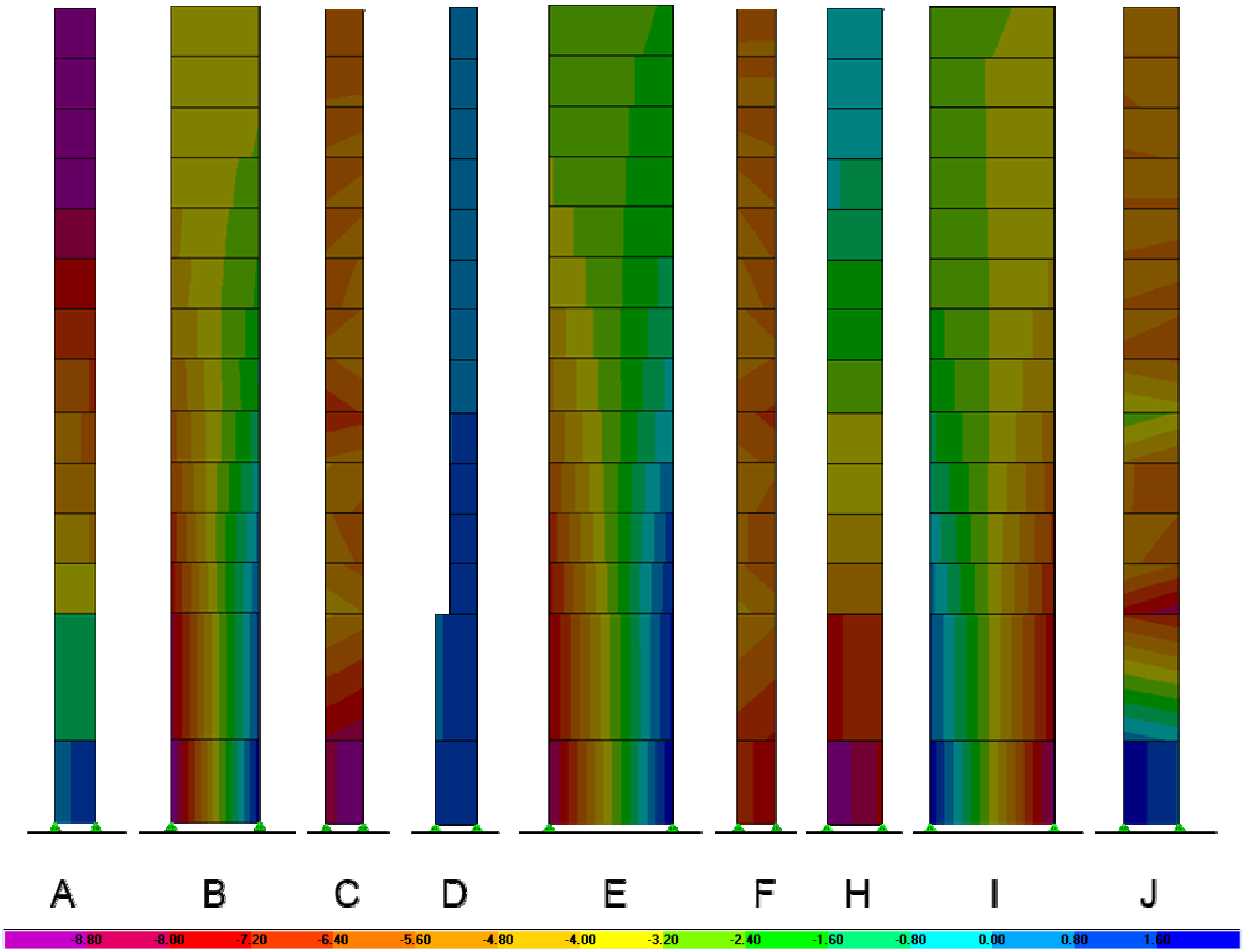
In conclusion, the Renaissance Schaumburg Hotel and Convention Center's lateral force resisting system appears to have passed the first round of detailed analysis based both on computer models and on hand verification, in the future the current 3D model will be expanded in order to refine the analysis presented in this paper.‡

‡ End of Report – Continue with Appendices

Appendix A – ETABS Data Output

This section includes data that ETABS Extended 3D Analysis of Building Systems Nonlinear V.8.2.6. The model used for testing can be found at

Stress Results



The image above is a composition of stress levels in each of the shear walls. As one can see from the image the entire stress range is apparent at the base of the structure, showing both compressive (toward the blue side of the color spectrum) and tensile stresses in the shear walls. As expected the stress levels decrease as you proceed to go higher in the building.

Shear wall B – ETAB Result

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
16	B	WY	Top	-0.05	15.34	0.01	227.175	-109.868	-707.623
16	B	WY	Bottom	-24.7	15.34	0.01	227.175	-109.252	1072.813
15	B	WY	Top	-24.77	36.41	0.16	220.362	-107.651	-1205.315
15	B	WY	Bottom	-49.43	36.41	0.16	220.362	-89.031	3019.775
14	B	WY	Top	-50.29	54.48	0.18	213.94	-90.954	-743.187
14	B	WY	Bottom	-74.94	54.48	0.18	213.94	-70.222	5578.698
12	B	WY	Top	-75.36	71.64	0.17	207.942	-72.12	459.18
12	B	WY	Bottom	-100.02	71.64	0.17	207.942	-52.31	8772.097
11	B	WY	Top	-99.06	88.37	0.15	198.183	-53.316	2384.828
11	B	WY	Bottom	-123.72	88.37	0.15	198.183	-35.983	12638.745
10	B	WY	Top	-120.64	105.71	0.08	182.318	-34.452	5042.478
10	B	WY	Bottom	-145.3	105.71	0.08	182.318	-24.613	17309.387
9	B	WY	Top	-139.98	123.74	0.12	150.225	-23.745	8629.777
9	B	WY	Bottom	-164.64	123.74	0.12	150.225	-9.277	22989.037
8	B	WY	Top	-156.4	136.58	-0.29	109.979	3.904	12861.137
8	B	WY	Bottom	-182.44	136.58	-0.29	109.979	-31.184	29595.197
7	B	WY	Top	-204.95	168.3	1.14	-14.957	-62.568	16520.416
7	B	WY	Bottom	-229.91	168.3	1.14	-14.957	70.975	36292.106
6	B	WY	Top	-299.5	170.14	0.14	74.837	7.198	25788.82
6	B	WY	Bottom	-336.49	170.14	0.14	74.837	23.049	45531.434
5	B	WY	Top	-361.57	177.46	0.51	32.115	0.891	32380.831
5	B	WY	Bottom	-398.56	177.46	0.51	32.115	59.759	52972.719
4	B	WY	Top	-399.32	185.98	-0.65	0.747	61.599	39614.379
4	B	WY	Bottom	-436.31	185.98	-0.65	0.747	-14.31	61196.004
3	B	WY	Top	-347.47	209.6	-0.38	70.424	66.109	26833.472
3	B	WY	Bottom	-440.52	209.6	-0.38	70.424	-45.574	88028.061
2	B	WY	Top	-412.71	287.21	0.37	23.412	-72.141	49844.971
2	B	WY	Bottom	-473.9	287.21	0.37	23.412	-0.904	104989.523
				Max Shear (k)	287.21	Max Moment (k-in)		104,990	8749 k-ft

Shear wall E – ETAB Result

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
16	E	WY	Top	-0.79	41.93	-0.01	78.627	2.18	-1550.718
16	E	WY	Bottom	-35.17	41.93	-0.01	78.627	1.17	3315.132
15	E	WY	Top	-38.79	76.99	-0.07	88.479	6.528	-887.977
15	E	WY	Bottom	-73.17	76.99	-0.07	88.479	-1.332	8045.411
14	E	WY	Top	-79.13	115.53	-0.04	100.084	5.819	1391.751
14	E	WY	Bottom	-113.52	115.53	-0.04	100.084	1.404	14798.398
12	E	WY	Top	-120.02	154.78	-0.05	111.794	9.484	5531.384
12	E	WY	Bottom	-154.41	154.78	-0.05	111.794	4.055	23492.118
11	E	WY	Top	-162.21	193.84	-0.05	123.091	14.198	11588.251
11	E	WY	Bottom	-196.6	193.84	-0.05	123.091	7.992	34081.384
10	E	WY	Top	-205.14	231.93	-0.04	133.922	19.616	19567.897
10	E	WY	Bottom	-239.53	231.93	-0.04	133.922	14.745	46481.518
9	E	WY	Top	-245.92	268.94	0.05	140.045	24.425	29712.336
9	E	WY	Bottom	-280.31	268.94	0.05	140.045	29.658	60920.507
8	E	WY	Top	-298.01	312.09	-0.01	149.601	38.156	39489.446
8	E	WY	Bottom	-334.32	312.09	-0.01	149.601	36.529	77727.032
7	E	WY	Top	-373.67	334.87	-0.87	126.53	68.547	56675.74
7	E	WY	Bottom	-408.48	334.87	-0.87	126.53	-33.312	96015.972
6	E	WY	Top	-505.78	389.06	-3.14	411.759	292.678	83169.619
6	E	WY	Bottom	-557.36	389.06	-3.14	411.759	-71.236	128316.519
5	E	WY	Top	-633.17	434.74	-1.63	429.802	71.979	105402.307
5	E	WY	Bottom	-684.75	434.74	-1.63	429.802	-117.145	155849.078
4	E	WY	Top	-625.28	484.91	9.82	966.712	-210.74	121378.316
4	E	WY	Bottom	-676.86	484.91	9.82	966.712	928.337	177647.176
3	E	WY	Top	-527.68	553.21	0.81	255.736	-242.293	89921.018
3	E	WY	Bottom	-657.46	553.21	0.81	255.736	-5.458	251435.9
2	E	WY	Top	-564.24	573.89	-1.05	82.78	128.698	158839.054
2	E	WY	Bottom	-649.59	573.89	-1.05	82.78	-72.698	269025.251
				Max Shear (k)	573.89	Max Moment (k-in)		269,025	22419 k-ft

Shear wall I – ETAB Result

Story	Pier	Load	Loc	P	V2	V3	T	M2	M3
16	I	WY	Top	-1.85	67.33	0.16	32.272	-8.374	-3313.966
16	I	WY	Bottom	-36.24	67.33	0.16	32.272	9.65	4498.434
15	I	WY	Top	-39.55	134.75	0.07	21.769	-1.703	-4461.931
15	I	WY	Bottom	-73.93	134.75	0.07	21.769	6.483	11174.406
14	I	WY	Top	-78.96	201.26	0.13	3.248	-3.645	-2865.368
14	I	WY	Bottom	-113.34	201.26	0.13	3.248	11.624	20488.596
12	I	WY	Top	-119.7	267.58	0.19	-16.997	-3.739	1218.729
12	I	WY	Bottom	-154.09	267.58	0.19	-16.997	18.153	32268.711
11	I	WY	Top	-163.08	333.09	0.26	-38.032	-3.526	7609.313
11	I	WY	Bottom	-197.47	333.09	0.26	-38.032	26.991	46261.04
10	I	WY	Top	-209.31	398.23	0.32	-57.694	-1.717	16304.518
10	I	WY	Bottom	-243.7	398.23	0.32	-57.694	35.933	62514.92
9	I	WY	Top	-256.18	462.49	0.43	-75.673	1.734	27791.298
9	I	WY	Bottom	-290.56	462.49	0.43	-75.673	51.282	81459.009
8	I	WY	Top	-318.05	528.24	0.17	-78.38	23.804	38769.551
8	I	WY	Bottom	-354.36	528.24	0.17	-78.38	44.086	103489.869
7	I	WY	Top	-366.86	593.07	0.22	-62.143	24.51	59825.858
7	I	WY	Bottom	-401.68	593.07	0.22	-62.143	50.702	129500.262
6	I	WY	Top	-473.6	655.18	0.66	-253.941	-18.509	98109.575
6	I	WY	Bottom	-525.18	655.18	0.66	-253.941	58.281	174137.234
5	I	WY	Top	-567.26	715.15	0.75	-351.619	-3.52	123472.513
5	I	WY	Bottom	-618.84	715.15	0.75	-351.619	82.95	206458.613
4	I	WY	Top	-608.26	767.36	-2.99	-543.49	176.042	152458.66
4	I	WY	Bottom	-659.84	767.36	-2.99	-543.49	-171	241502.661
3	I	WY	Top	-493.67	875.08	-0.91	-2.362	165.138	106198.075
3	I	WY	Bottom	-623.44	875.08	-0.91	-2.362	-101.71	361686.326
2	I	WY	Top	-583.35	998.03	0.47	-48.488	-78.111	219129.031
2	I	WY	Bottom	-668.7	998.03	0.47	-48.488	12.427	410750.997
				Max Shear (k)	998.03	Max Moment (k-in)		410,751	34229 k-ft

Drift Considerations

Story	Item	Load	Point	X	Y	Z	DriftX	DriftY	Story Height	Story Drift
16	Max Drift X	WY	90	3748	703	1884.36	0.000227		9.67	0.0022
16	Max Drift Y	WY	90	3748	703	1884.36		0.00164	9.67	0.0159
15	Max Drift X	WY	90	3748	703	1768.32	0.000227		9.67	0.0022
15	Max Drift Y	WY	90	3748	703	1768.32		0.001651	9.67	0.0160
14	Max Drift X	WY	90	3748	703	1652.28	0.000227		9.67	0.0022
14	Max Drift Y	WY	90	3748	703	1652.28		0.001657	9.67	0.0160
12	Max Drift X	WY	90	3748	703	1536.24	0.000224		9.67	0.0022
12	Max Drift Y	WY	90	3748	703	1536.24		0.001655	9.67	0.0160
11	Max Drift X	WY	90	3748	703	1420.2	0.000219		9.67	0.0021
11	Max Drift Y	WY	90	3748	703	1420.2		0.001642	9.67	0.0159
10	Max Drift X	WY	90	3748	703	1304.16	0.00021		9.67	0.0020
10	Max Drift Y	WY	90	3748	703	1304.16		0.001614	9.67	0.0156
9	Max Drift X	WY	90	3748	703	1188.12	0.000195		9.67	0.0019
9	Max Drift Y	WY	90	3748	703	1188.12		0.001569	9.67	0.0152
8	Max Drift X	WY	90	3748	703	1072.08	0.000171		10.21	0.0017
8	Max Drift Y	WY	90	3748	703	1072.08		0.001502	10.21	0.0153
7	Max Drift X	WY	141	3345	-745	949.56	0.000154		9.79	0.0015
7	Max Drift Y	WY	154	1380	0	949.56		0.004544	9.79	0.0445
6	Max Drift X	WY	141	3345	-745	832.08	0.000149		9.67	0.0014
6	Max Drift Y	WY	90	3748	703	832.08		0.001255	9.67	0.0121
5	Max Drift X	WY	141	3345	-745	716.04	0.000143		9.67	0.0014
5	Max Drift Y	WY	90	3748	703	716.04		0.001151	9.67	0.0111
4	Max Drift X	WY	79	-16	703	600	0.000279		9.67	0.0027
4	Max Drift Y	WY	79	-16	703	600		0.001349	9.67	0.0130
3	Max Drift X	WY	288	2940.5	-1124	483.96	0.000158		24.33	0.0038
3	Max Drift Y	WY	139	3520.375	-696	483.96		0.000706	24.33	0.0172
2	Max Drift X	WY	314	2803.5	-6	192	0.00001		16	0.0002
2	Max Drift Y	WY	315	2803.5	278.5	192		0.000269	16	0.0043

$$\frac{L}{Drift^{-1}} = Story \quad Drift$$

Total Drift (in)	
Y	X
2.736943	0.330699

Appendix B – Shear-wall Spreadsheets

Wall Rigidity Calculation															
	f'c (ksi)	Shear (k)	Shear (k)	Thickness (in)	Wall Height (ft)	Wall Length (ft)									
		E-W	N-S			A	B	C	D	E	F	H	I	J
Roof	6	104.3604	88.40473	8	8	8	19	8	8	26	8	10	26	10
16	6	128.1686	108.558	8	8	8	19	8	8	26	8	10	26	10
15	6	124.6234	105.3649	8	11.67	8	19	8	8	26	8	10	26	10
14	6	124.3021	105.0482	8	9.67	8	19	8	8	26	8	10	26	10
12	6	123.3601	104.1197	8	9.67	8	19	8	8	26	8	10	26	10
11	6	123.0315	103.7959	8	9.67	8	19	8	8	26	8	10	26	10
10	6	121.6075	102.3924	8	9.67	8	19	8	8	26	8	10	26	10
9	6	120.9137	101.7086	8	9.67	8	19	8	8	26	8	10	26	10
8	6	119.6358	100.4491	8	9.67	8	19	8	8	26	8	10	26	10
7	6	121.6186	101.914	8	10.21	8	19	8	8	26	8	10	26	10
6	8	121.2575	101.4452	12	9.79	8	19	8	8	26	8	10	26	10
5	8	116.4687	97.21464	12	9.67	8	19	8	8	26	8	10	26	10
4	8	114.0785	94.9718	12	9.67	8	19	8	8	26	8	10	26	10
3	8	112.4427	93.35957	12	9.67	8	19	8	8	26	8	10	26	10
2	8	197.0281	162.8762	12	24.33	8	19	8	8	26	8	10	26	10
Ground	8	216.2567	176.709	12	16	8	19	8	8	26	8	10	26	10

	Wall locations (from A7) (ft)									Floor CG	
	A	B	C	D	E	F	H	I	J	N-S	E-W
Roof	17	54	0	26	81	0	26	216	0	155.5	28.17
16	17	54	0	26	81	0	26	216	0	155.5	28.17
15	17	54	0	26	81	0	26	216	0	155.5	28.17
14	17	54	0	26	81	0	26	216	0	155.5	28.17
12	17	54	0	26	81	0	26	216	0	155.5	28.17
11	17	54	0	26	81	0	26	216	0	155.5	28.17
10	17	54	0	26	81	0	26	216	0	155.5	28.17
9	17	54	0	26	81	0	26	216	0	155.5	28.17
8	17	54	0	26	81	0	26	216	0	155.5	28.17
7	17	54	0	26	81	0	26	216	0	155.5	28.17
6	17	54	0	26	81	0	26	216	0	155.5	28.17
5	17	54	0	26	81	0	26	216	0	155.5	28.17
4	17	54	0	26	81	0	26	216	0	155.5	28.17
3	17	54	0	26	81	0	26	216	0	155.5	28.17
2	17	54	0	26	81	0	26	216	0	155.5	28.17
Ground	17	54	0	26	81	0	26	216	0	155.5	28.17

Center of Ridigity	
N-S	E-W
124.8377	11.590

Torsional Moment @ Base		
(ft-k)	N-S**	E-W
	5418.307	3584.881

$$R = Et\left[4\left(\frac{h}{L}\right)^3 + \frac{3h}{L}\right]^{-1}$$

	Rigidity									ΣP	
	A	B	C	D	E	F	H	I	J	N-S	E-W
Roof	1.143	5.060	1.143	1.143	7.575	1.143	1.907	7.575	1.907	20.209	8.386
16	1.143	5.060	1.143	1.143	7.575	1.143	1.907	7.575	1.907	20.209	8.386
15	0.477	2.848	0.477	0.477	4.600	0.477	0.871	4.600	0.871	12.047	3.649
14	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
12	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
11	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
10	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
9	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
8	0.749	3.845	0.749	0.749	5.954	0.749	1.310	5.954	1.310	15.754	5.616
7	0.659	3.535	0.659	0.659	5.537	0.659	1.168	5.537	1.168	14.609	4.972
6	1.090	5.655	1.090	1.090	8.781	1.090	1.913	8.781	1.913	23.218	8.187
5	1.123	5.767	1.123	1.123	8.932	1.123	1.965	8.932	1.965	23.631	8.424
4	1.123	5.767	1.123	1.123	8.932	1.123	1.965	8.932	1.965	23.631	8.424
3	1.123	5.767	1.123	1.123	8.932	1.123	1.965	8.932	1.965	23.631	8.424
2	0.099	0.960	0.099	0.099	1.919	0.099	0.201	1.919	0.201	4.798	0.797
Ground	0.316	2.399	0.316	0.316	4.227	0.316	0.612	4.227	0.612	10.853	2.488

	Wall Proportion									
	A	B	C	D	E	F	H	I	J	Check
Roof	0.136	0.250	0.136	0.136	0.375	0.136	0.227	0.375	0.227	OK
16	0.136	0.250	0.136	0.136	0.375	0.136	0.227	0.375	0.227	OK
15	0.131	0.236	0.131	0.131	0.382	0.131	0.239	0.382	0.239	OK
14	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
12	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
11	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
10	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
9	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
8	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
7	0.133	0.242	0.133	0.133	0.379	0.133	0.235	0.379	0.235	OK
6	0.133	0.244	0.133	0.133	0.378	0.133	0.234	0.378	0.234	OK
5	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
4	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
3	0.133	0.244	0.133	0.133	0.378	0.133	0.233	0.378	0.233	OK
2	0.124	0.200	0.124	0.124	0.400	0.124	0.253	0.400	0.253	OK
Ground	0.127	0.221	0.127	0.127	0.389	0.127	0.246	0.389	0.246	OK

Direct Shear

Wall A			Wall B			Wall C		
Floor	Proportion	Shear	Floor	Proportion	Shear	Floor	Proportion	Shear
Roof	0.136	14.223	Roof	0.250	22.136	Roof	0.136	14.223
16	0.136	17.467	16	0.250	27.182	16	0.136	17.467
15	0.131	16.283	15	0.236	24.906	15	0.131	16.283
14	0.133	16.577	14	0.244	25.638	14	0.133	16.577
12	0.133	16.451	12	0.244	25.411	12	0.133	16.451
11	0.133	16.408	11	0.244	25.332	11	0.133	16.408
10	0.133	16.218	10	0.244	24.990	10	0.133	16.218
9	0.133	16.125	9	0.244	24.823	9	0.133	16.125
8	0.133	15.955	8	0.244	24.515	8	0.133	15.955
7	0.133	16.120	7	0.242	24.660	7	0.133	16.120
6	0.133	16.148	6	0.244	24.710	6	0.133	16.148
5	0.133	15.532	5	0.244	23.726	5	0.133	15.532
4	0.133	15.214	4	0.244	23.179	4	0.133	15.214
3	0.133	14.995	3	0.244	22.785	3	0.133	14.995
2	0.124	24.380	2	0.200	32.578	2	0.124	24.380
Ground	0.127	27.448	Ground	0.221	39.067	Ground	0.127	27.448

Wall D			Wall E			Wall F		
Floor	Proportion	Shear	Floor	Proportion	Shear	Floor	Proportion	Shear
Roof	0.136	14.223	Roof	0.375	33.134	Roof	0.136	14.223
16	0.136	17.467	16	0.375	40.688	16	0.136	17.467
15	0.131	16.283	15	0.382	40.229	15	0.131	16.283
14	0.133	16.577	14	0.378	39.705	14	0.133	16.577
12	0.133	16.451	12	0.378	39.354	12	0.133	16.451
11	0.133	16.408	11	0.378	39.232	11	0.133	16.408
10	0.133	16.218	10	0.378	38.701	10	0.133	16.218
9	0.133	16.125	9	0.378	38.443	9	0.133	16.125
8	0.133	15.955	8	0.378	37.967	8	0.133	15.955
7	0.133	16.120	7	0.379	38.627	7	0.133	16.120
6	0.133	16.148	6	0.378	38.368	6	0.133	16.148
5	0.133	15.532	5	0.378	36.744	5	0.133	15.532
4	0.133	15.214	4	0.378	35.897	4	0.133	15.214
3	0.133	14.995	3	0.378	35.287	3	0.133	14.995
2	0.124	24.380	2	0.400	65.149	2	0.124	24.380
Ground	0.127	27.448	Ground	0.389	68.821	Ground	0.127	27.448

Wall H			Wall I			Wall J		
Floor	Proportion	Shear	Floor	Proportion	Shear	Floor	Proportion	Shear
Roof	0.227	23.735	Roof	0.375	33.134	Roof	0.227	23.735
16	0.227	29.149	16	0.375	40.688	16	0.227	29.149
15	0.239	29.746	15	0.382	40.229	15	0.239	29.746
14	0.233	28.997	14	0.378	39.705	14	0.233	28.997
12	0.233	28.777	12	0.378	39.354	12	0.233	28.777
11	0.233	28.701	11	0.378	39.232	11	0.233	28.701
10	0.233	28.368	10	0.378	38.701	10	0.233	28.368
9	0.233	28.207	9	0.378	38.443	9	0.233	28.207
8	0.233	27.908	8	0.378	37.967	8	0.233	27.908
7	0.235	28.569	7	0.379	38.627	7	0.235	28.569
6	0.234	28.334	6	0.378	38.368	6	0.234	28.334
5	0.233	27.170	5	0.378	36.744	5	0.233	27.170
4	0.233	26.612	4	0.378	35.897	4	0.233	26.612
3	0.233	26.230	3	0.378	35.287	3	0.233	26.230
2	0.253	49.754	2	0.400	65.149	2	0.253	49.754
Ground	0.246	53.232	Ground	0.389	68.821	Ground	0.246	53.232

Torsional Shear

Torsional Shear @ 16 ft					
Ground	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.316	42.410	567.99	0.0002	1.29
B	2.399	70.838	12039.88	0.0030	16.38
C	0.316	11.590	42.42	0.0001	0.35
D	0.316	13.952	61.47	0.0001	0.42
E	4.227	43.838	8122.78	0.0033	17.86
F	0.316	11.590	42.42	0.0001	0.35
H	0.612	13.952	119.22	0.0002	0.82
I	4.227	91.162	35126.94	0.0069	37.15
J	0.612	11.590	82.26	0.0001	0.68

Torsional Shear @ 40 ft					
Floor 2	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.099	42.410	177.37	0.0002	0.92
B	0.960	70.838	4815.86	0.0027	14.89
C	0.099	11.590	13.25	0.0000	0.25
D	0.099	13.952	19.20	0.0001	0.30
E	1.919	43.838	3688.31	0.0034	18.42
F	0.099	11.590	13.25	0.0000	0.25
H	0.201	13.952	39.17	0.0001	0.61
I	1.919	91.162	15950.08	0.0071	38.31
J	0.201	11.590	27.03	0.0001	0.51

Torsional Shear @ 50 ft					
Floor 3	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.123	42.410	2020.56	0.0004	2.09
B	5.767	70.838	28939.88	0.0033	17.92
C	1.123	11.590	150.89	0.0001	0.57
D	1.123	13.952	218.68	0.0001	0.69
E	8.932	43.838	17164.34	0.0032	17.18
F	1.123	11.590	150.89	0.0001	0.57
H	1.965	13.952	382.51	0.0002	1.20
I	8.932	91.162	74227.13	0.0066	35.72
J	1.965	11.590	263.95	0.0002	1.00

Torsional Shear @ 60 ft					
Floor 4	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.123	42.410	2020.56	0.0004	2.09
B	5.767	70.838	28939.88	0.0033	17.92
C	1.123	11.590	150.89	0.0001	0.57
D	1.123	13.952	218.68	0.0001	0.69
E	8.932	43.838	17164.34	0.0032	17.18
F	1.123	11.590	150.89	0.0001	0.57
H	1.965	13.952	382.51	0.0002	1.20
I	8.932	91.162	74227.13	0.0066	35.72
J	1.965	11.590	263.95	0.0002	1.00

Torsional Shear @ 69 ft					
Floor 5	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.123	42.410	2020.56	0.0004	2.09
B	5.767	70.838	28939.88	0.0033	17.92
C	1.123	11.590	150.89	0.0001	0.57
D	1.123	13.952	218.68	0.0001	0.69
E	8.932	43.838	17164.34	0.0032	17.18
F	1.123	11.590	150.89	0.0001	0.57
H	1.965	13.952	382.51	0.0002	1.20
I	8.932	91.162	74227.13	0.0066	35.72
J	1.965	11.590	263.95	0.0002	1.00

Torsional Shear @ 79 ft					
Floor 6	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.090	42.410	1961.04	0.0004	2.07
B	5.655	70.838	28378.41	0.0033	17.89
C	1.090	11.590	146.45	0.0001	0.56
D	1.090	13.952	212.23	0.0001	0.68
E	8.781	43.838	16875.36	0.0032	17.19
F	1.090	11.590	146.45	0.0001	0.56
H	1.913	13.952	372.40	0.0002	1.19
I	8.781	91.162	72977.43	0.0066	35.75
J	1.913	11.590	256.97	0.0002	0.99

Torsional Shear @ 89 ft					
Floor 7	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.659	42.410	1185.34	0.0004	1.99
B	3.535	70.838	17738.28	0.0033	17.79
C	0.659	11.590	88.52	0.0001	0.54
D	0.659	13.952	128.28	0.0001	0.65
E	5.537	43.838	10640.75	0.0032	17.24
F	0.659	11.590	88.52	0.0001	0.54
H	1.168	13.952	227.35	0.0002	1.16
I	5.537	91.162	46015.87	0.0066	35.86
J	1.168	11.590	156.88	0.0002	0.96

Torsional Shear @ 99 ft					
Floor 8	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.09
B	3.845	70.838	19293.25	0.0033	17.92
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.69
E	5.954	43.838	11442.90	0.0032	17.18
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.72
J	1.310	11.590	175.97	0.0002	1.00

Torsional Shear @ 109 ft					
Floor 9	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.09
B	3.845	70.838	19293.25	0.0033	17.92
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.69
E	5.954	43.838	11442.90	0.0032	17.18
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.72
J	1.310	11.590	175.97	0.0002	1.00

Torsional Shear @ 118 ft					
Floor 10	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.09
B	3.845	70.838	19293.25	0.0033	17.92
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.69
E	5.954	43.838	11442.90	0.0032	17.18
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.72
J	1.310	11.590	175.97	0.0002	1.00

Torsional Shear @ 128 ft					
Floor 11	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.09
B	3.845	70.838	19293.25	0.0033	17.92
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.69
E	5.954	43.838	11442.90	0.0032	17.18
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.72
J	1.310	11.590	175.97	0.0002	1.00

Torsional Shear @ 138 ft					
Floor 12	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.09
B	3.845	70.838	19293.25	0.0033	17.92
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.69
E	5.954	43.838	11442.90	0.0032	17.18
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.72
J	1.310	11.590	175.97	0.0002	1.00

Torsional Shear @ 147 ft					
Floor 14	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.749	42.410	1347.04	0.0004	2.08
B	3.845	70.838	19293.25	0.0033	17.85
C	0.749	11.590	100.60	0.0001	0.57
D	0.749	13.952	145.78	0.0001	0.68
E	5.954	43.838	11442.90	0.0032	17.10
F	0.749	11.590	100.60	0.0001	0.57
H	1.310	13.952	255.01	0.0002	1.20
I	5.954	91.162	49484.75	0.0066	35.57
J	3.845	11.590	516.44	0.0005	2.92

Torsional Shear @ 159 ft					
Floor 15	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	0.477	42.410	857.46	0.0003	1.75
B	2.848	70.838	14289.72	0.0032	17.43
C	0.477	11.590	64.03	0.0001	0.48
D	0.477	13.952	92.80	0.0001	0.57
E	4.600	43.838	8839.35	0.0032	17.42
F	0.477	11.590	64.03	0.0001	0.48
H	0.871	13.952	169.53	0.0002	1.05
I	4.600	91.162	38225.73	0.0067	36.22
J	0.871	11.590	116.98	0.0002	0.87

Torsional Shear @ 167 ft					
Floor 16	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.143	42.410	2055.58	0.0005	2.45
B	5.060	70.838	25392.64	0.0033	18.09
C	1.143	11.590	153.51	0.0001	0.67
D	7.575	13.952	1474.44	0.0010	5.33
E	7.575	43.838	14556.32	0.0031	16.76
F	1.143	11.590	153.51	0.0001	0.67
H	1.907	13.952	371.25	0.0002	1.34
I	7.575	91.162	62948.73	0.0064	34.85
J	1.907	11.590	256.17	0.0002	1.12

Torsional Shear @ 175 ft					
Roof	R	x	Rx^2	$Rx/\Sigma Rx^2$	Torsional Shear
A	1.143	42.410	2055.58	0.0005	2.47
B	5.060	70.838	25392.64	0.0034	18.30
C	1.143	11.590	153.51	0.0001	0.68
D	1.143	13.952	222.47	0.0002	0.81
E	7.575	43.838	14556.32	0.0031	16.96
F	1.143	11.590	153.51	0.0001	0.68
H	1.907	13.952	371.25	0.0003	1.36
I	7.575	91.162	62948.73	0.0065	35.26
J	1.907	11.590	256.17	0.0002	1.13

Shear Totals

Wall B	
Floor	Shear
Roof	18.304
16	18.091
15	17.427
14	17.847
12	17.921
11	17.921
10	17.921
9	17.921
8	17.921
7	17.789
6	17.891
5	17.921
4	17.921
3	17.921
2	14.887
Ground	16.385
Base Shear 282 kips	

Wall E	
Floor	Shear
Roof	33.134
16	40.688
15	40.229
14	39.705
12	39.354
11	39.232
10	38.701
9	38.443
8	37.967
7	38.627
6	38.368
5	36.744
4	35.897
3	35.287
2	65.149
Ground	68.821
Base Shear 666 kips	

Wall I	
Floor	Shear
Roof	68.394
16	75.536
15	76.454
14	75.275
12	75.071
11	74.949
10	74.419
9	74.160
8	73.684
7	74.486
6	74.118
5	72.462
4	71.614
3	71.004
2	103.463
Ground	105.967
Base Shear 1241 kips	

Critical Walls

The total shear for the controlling shear wall elements can be found on the left. These values can be compared to those found through analysis in ETABS, re-printed below for convince.

ETABS Results: (re-summarized)

Shear Wall	Max Shear (k)	Max Moment (k-ft)
B	287.21	8749 k-ft
E	573.89	22419 k-ft
I	998.03	34229 k-ft

Appendix C – Wind Analysis

As previously developed for technical assignment one. (Revised 11-05)

Wind Load Analysis

Building Properties	
B (ft)	117
L (ft)	311
h (ft)	184.67
K_{zt}	1
K_d	0.85
V (mph)	90
Importance	III
I_w	1.15
Exposure	C
α	9.5
z_g	900
z_{min}	15
c	0.2
$\in$	0.2
l	500
~~b~~	0.154
~~α~~	0.65
a	0.105
b	1

Period Parameters	
Struct. Type	Concrete
C_t	0.016
x	0.9
(check eq) T	1.753354
Natural *f*	0.570335
Rigidity	Flex

Rigid	
$g_Q=g_v$	3.4
ž	110.8
$I_ž$	0.1634
$L_ž$	637.05
Q	0.8472
G	0.8563

Windward	
Cp	0.8

Flexible	
g_R	4.05
R_n	0.037
N_1	8.14
η_h	10.86
η_B	0.059
η_L	61.21
R_h	0.088
R_B	0.962
R_L	0.016
$V_ž$	44.62
β	0.05
R	0.18
G_f	0.8688

Leeward	
Ratio	C_p
N-S I 0.376	-0.500
E-W I 2.658	-0.267

K_z and q_z		
Z(ft)	K_z	q_z
0-15	0.85	17.23
20	0.90	18.24
25	0.94	19.05
30	0.98	19.86
40	1.04	21.08
50	1.09	22.09
60	1.13	22.90
70	1.17	23.72
80	1.21	24.53
90	1.24	25.13
100	1.26	25.54
120	1.31	26.55
140	1.36	27.57
160	1.39	28.17
180	1.43	28.99
200	1.46	29.59
184.67	1.44	29.13

Pressure Coefficients		
Internal		
Enc. Type	Enclosed	
Internal (GC_{pi})	0.18	+/-

Velocity Pressure Envelope

	Windward		Leeward		MWFRS	MWFRS
Z(ft)	N-S	E-W	N-S	E-W	N-S	E-W
0-15	16.53	16.29	-17.21	-11.15	33.74	27.44
20	17.23	16.99	-17.21	-11.15	34.44	28.14
25	17.80	17.54	-17.21	-11.15	35.01	28.69
30	18.36	18.10	-17.21	-11.15	35.57	29.25
40	19.21	18.93	-17.21	-11.15	36.42	30.08
50	19.91	19.63	-17.21	-11.15	37.12	30.78
60	20.48	20.18	-17.21	-11.15	37.68	31.33
70	21.04	20.74	-17.21	-11.15	38.25	31.89
80	21.60	21.29	-17.21	-11.15	38.81	32.44
90	22.02	21.71	-17.21	-11.15	39.23	32.86
100	22.31	21.99	-17.21	-11.15	39.52	33.14
120	23.01	22.68	-17.21	-11.15	40.22	33.83
140	23.72	23.37	-17.21	-11.15	40.92	34.53
160	24.14	23.79	-17.21	-11.15	41.35	34.94
180	24.70	24.35	-17.21	-11.15	41.91	35.50
200	25.12	24.76	-17.21	-11.15	42.33	35.91
184.67	24.80	24.44	-17.21	-11.15	42.01	35.59

Load Analysis Summary

	Wind N-S	Wind E-W
Shear @ 18	52.66	44.68
Shear @ 17	104.36	88.40
Shear @ 16	128.17	108.56
Shear @ 15	124.62	105.36
Shear @ 14	124.30	105.05
Shear @ 12	123.36	104.12
Shear @ 11	123.03	103.80
Shear @ 10	121.61	102.39
Shear @ 9	120.91	101.71
Shear @ 8	119.64	100.45
Shear @ 7	121.62	101.91
Shear @ 6	121.26	101.45
Shear @ 5	116.47	97.21
Shear @ 4	114.08	94.97
Shear @ 3	112.44	93.36
Shear @ 2	197.03	162.88
Shear @ 1	216.26	176.71
Shear @ Ground	83.94	68.28
Base Shear	2,225.76	1,861.29
Overturning Moment	212,133.48	178,533.71
*Shear (kips) **Moment (ft-kips)		

For the Spreadsheet used for the calculation of wind loading, please see:

http://www.arche.psu.edu/thesis/eportfolio/current/portfolios/ejy112/tech-assign.htm

Appendix D – Lateral Member Checks

Shear Reinforcing

Check the 1st Story of Shear wall I:

$V_u = 1.6(998k) = 1596.8k$

$\phi V_c = 2\sqrt{f'_c}hd$ for $d = 0.8l_w$ $\phi V_c = 2\sqrt{8{,}000}(18")(0.8(330)) = 850.063k$

$\phi V_c = (0.85)850.063 = 722.55k < V_u = 1596.8k \therefore$ Additional reinforcement is needed.

$V_s = \dfrac{A_v f_y d}{s}$, with $s = 12" \therefore A_v = \dfrac{V_s s}{f_y d} = \dfrac{(1596.8 - 772.55)(12)}{60(330(0.8))} = 0.6623in^2$

Use #8 bars at 12" (horizontally)

$A_v = 0.79in^2$

Vertical Reinforcing:

$\rho_{t-\min} = 0.0025$ $\rho_v = 0.0025 + 0.5\left(2.5 - \dfrac{h_w}{l_w}\right)(\rho_h - 0.0025), where, \dfrac{h_w}{l_w} = \dfrac{175}{27.5} = 6.364$

$\rho_h = A_{vh}/s_2 h = 0.79/12(18) = 0.00366$ $\rho_v = 0.0025 + 0.5(2.5 - 6.364)(0.00366 - 0.0025) = 2.59E - 4$

Use #6 bars at 12" (vertically)

$A_v = 0.44in^2$

Flexural Reinforcing

Check of Shear wall I:

$U = 0.9D + 1.6W$ Tributary Area=742.5ft^2

Wall Dead Load = $0.150(18(330))/144 = 6.1875k/ft$ or $0.150(12(330))/144 = 4.125k/ft$

$P_u = 0.9[(0.145)742.5 + 0.157(16)742.5 + 6.1875(89.33) + 4.125(95.34)] = 2{,}918.83k$

$M_u = 34{,}229.25\text{ft - k}$

Moment Strength at 1st Story:

$A_{st} = 0.79(27.5) = 21.725in^2$

$w = \left(\dfrac{A_{st} f_y}{l_w h f'_c}\right) = \left(\dfrac{21.725(60)}{330(18)4}\right) = 0.05486$ $a = \left(\dfrac{P_u}{l_w h f'_c}\right) = \left(\dfrac{2918.83}{330(18)4}\right) = 0.12285$

$\dfrac{c}{l_w} = \left(\dfrac{w + a}{2w + 0.85\beta_1}\right) = \left(\dfrac{0.05486 + 0.12285}{2(0.05486) + 0.85(0.85)}\right) = 0.21353$

$\phi M_n = \phi\left[0.5A_{st} f_y l_w \left(1 + \dfrac{P_u}{A_{st} f_y}\right)\left(1 - \dfrac{c}{l_w}\right)\right] = 0.9\left[(0.5)21.725(60)330\left(1 + \dfrac{2{,}918.83}{21.725(60)}\right)(1 - 0.21353)\right] = 493129in - k$

$\phi M_n = 41{,}094\,ft - k > M_u = 34{,}229.25\,ft - k \therefore OK$

Overturning Check

Overturning Moment – Moment due to Self Weight

$M_{OT} = 34{,}229.25\text{ft - k}$ $P_u = 2{,}918.83k$

$M_{WALL} = \dfrac{P_u l_w}{2} = \dfrac{2{,}918.83(330/12)}{2} = 40{,}133.9\,ft - k > M_{OT} = 34{,}300\,ft - k \therefore$ Overturning is countered, thus OK.