Hampton Inn & Suites – National Harbor, MD

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John Pillar
Dr. Memari Thesis Advisor

Technical Report #3
11/19/06

Executive Summary

This report was intended to further analyze the lateral elements of the Hampton Inn & Suites in National Harbor, Maryland from the findings in Technical Report one. From this continued analysis, the lateral distribution to shear walls was refined and further analysis of seismic and wind impacts were carried out.

After posting Technical Report one, the seismic loads were revised to account for the first floor being flush with the ground, and therefore no shear will develop until just above the floor level. This reduced the effective seismic weight of the building and the design base shear.

An in-depth study of lateral load distribution was again carried out, along with torsion, but with revised controlling seismic design forces. These results were compared to a RAM computer model and were found to be validated.

Validation was proved through a series of calculation results with closely matched the computer output. Tabulations for centers of mass and rigidity were within at least 2.4% of the hand calculated values, and shear values made intuitive sense even though they did not exactly match hand tabbed values in all cases.

The computer model was then used to conduct what would have been a tedious drift calculation by hand. These values were found to be within the acceptable limits of L/400 for total drift, and 0.02 multiplied by the story height for story drift. These limits were found in ASCE 7-05 and exist due to serviceability requirements. In this case, the total drift was calculated to be 1.996” for wind and 2.88” for seismic, or drift ratios of L/781 and L/542, respectively.

Spot checks were performed on critical lateral elements and on the centers of mass and rigidity, which, as stated before, were validated through computer results. Design base shear for wind and seismic were also computed, as well as overturning moments for each.

Load paths and discrepancies were explained near the end of the report, and a conclusion which summarizes the findings is also included at the end of the report. Calculations immediately follow this report in the appendix.

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Overview

Columns

All columns are 12"x24" with chamfered edges, where exposed. There are 32 columns which span from the foundation to the roof, over 115 feet, with number 4 ties spaced at 12 inches all the way up. Vertical reinforcing ranges from ten number 11 bars to six number 8 bars. In all cases, the vertical reinforcing is distributed along the 24" face of the column in two sheets, one on each side. In all cases, class B lap splices are required for vertical splicing. Concrete strength is normal weight 6000 psi from the foundation to the third floor, where it drops to 5000 psi until it reaches the roof. Typical floor to floor heights are close to 10'.

There is a double-height pool structure on the first floor that rests on grade. Because it intersects with two column lines, the two columns start at the second floor and proceed to the roof. They cannot continue down to the foundation, so their weight is picked up by a transfer beam that is 36" deep, 44" wide, and heavily reinforced with six number 8 bars on top, ten number 11 bars on the bottom with an additional row of six number 9 bars also on the bottom. The reinforcing is tied together with number 5 closed stirrups spaced at ten inches on center. This transfer beam also frames into to two similar girders, tied into columns, at either end.

The last two columns start at the roof and help hold up a mechanical screen wall. The roof of the screen wall consists of W14x22 curved steel members with 1-1/2" galvanized metal roof deck resting on top.

Floor Slabs

The floor slabs are usually 10-1/2" thick when not near columns. At each column there is a 2-1/2" drop panel to combine for a 13" slab thickness. A typical drop panel size is 5'-6"x6'-9" and accounts for 38 square feet. Steel reinforcing is laid out longitudinally and transversely on both the bottom and top. The slab reinforcing ranges from number 4 bars to number 6 bars spaced approximately 12 inches apart. Where not specified, number 5 bars spaced at 6" is the minimum required.

For slabs on level 3 and below, concrete strength is normal weight 6000 psi. Slabs resting on the fourth floor and up have a strength of 5000 psi. Minimum reinforcing protection for floor slabs is 3/4".

The slabs on this project are considered to act as two way slabs, meaning that they carry load in both lateral directions. The three largest bays have dimensions of 29'x26'-10". There are no beams spanning between columns in this case. In the largest bay, the drop panels cover roughly 6 feet of the span, or 20.7%.

Lateral System

The lateral components of this building are comprised of twelve shear walls of varying length. Five of the twelve are aligned with Plan North, while the other seven are aligned plan East-West. Each shear wall is one foot thick and is vertically reinforced with number 5 bars at 18" on center. They are each tied into the foundation by rebar that matches vertical reinforcing called out in the plans. All rebar is to have class B splices and extend one foot into the foundation with 90^o hooks. In most cases, two columns act as bookends for each shear wall. In these cases, the shear wall reinforcement of number 5 bars spaced at 18 inches is continued into the columns and hooked 90^o.

The longest shear walls are 21'-4" along grid lines B and C running North to South. In the East-West direction, the longest shear wall is located along grid line 6, and is 21'-0" long. Nine of the twelve shear walls wrap around the two stair cases and lone elevator shaft that are spaced evenly throughout the building's long dimension.

The total length of the shear walls in the North-South direction is 99'-4", and 79'-0" in the East-West direction. Because the building is rectangular, forces acting on the wide side of the building have a much greater affect on the building's response than forces acting on the narrow side. Thus, more total shear wall length was provided to resist North-South loads. Refer to figure 1 on the next page for a graphical shear wall layout.

With a total height of 130 feet, the shear walls travel the full height of the building and are in the same position, relative to each other, on every floor (although some individual floor layouts may vary).

To assist in the analysis of this structure, a RAM model was created following the building's floor plans. While the model has some limitations, and spot checks were made with some simplifying assumptions, the results were confirmed through hand calculations. However, the accuracy of the RAM output depends directly on the model generated, and there were some areas and conditions that were not feasible to model for this report.

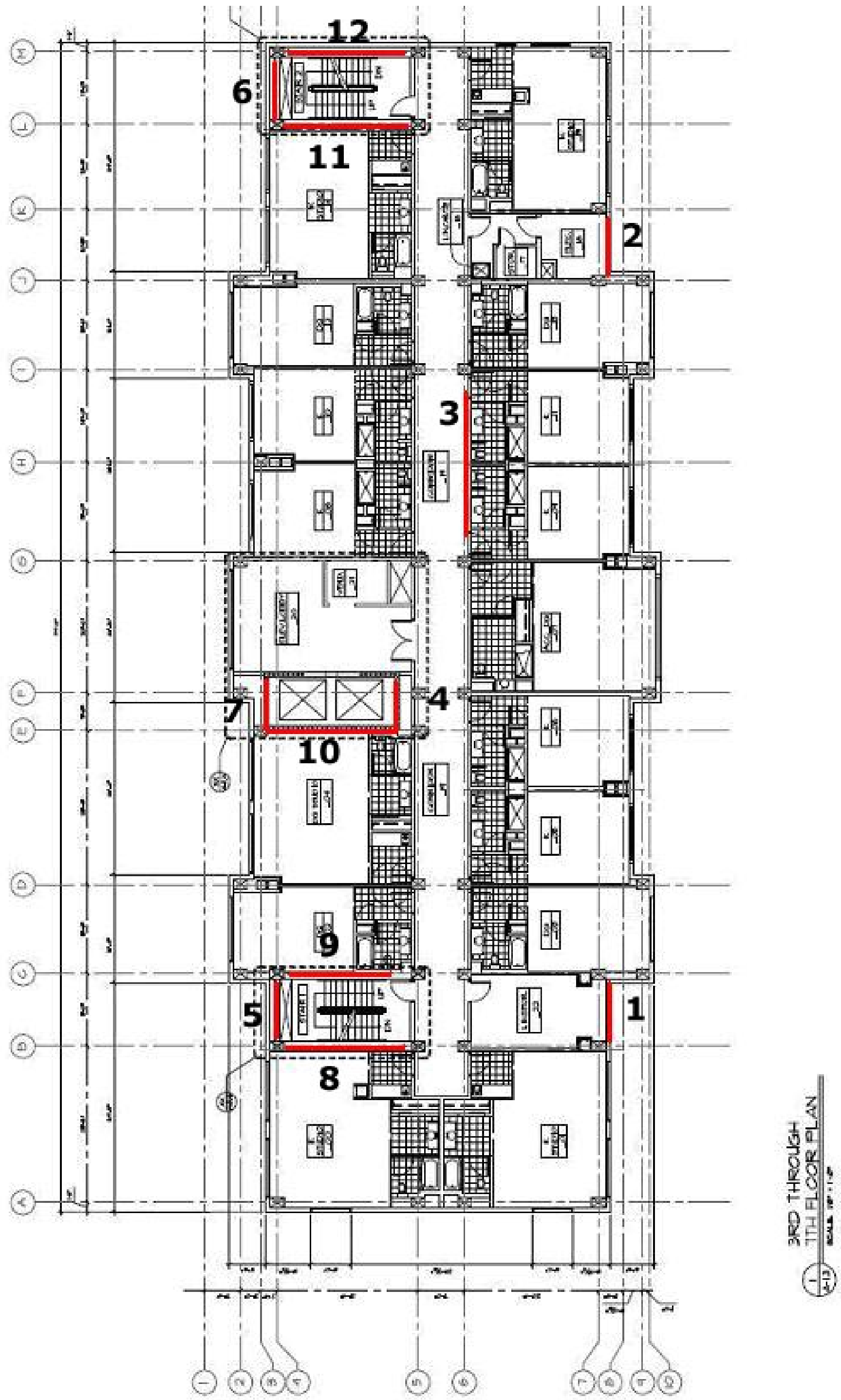


Fig. 1

Shown is the shear wall layout for a typical floor of the Hampton Inn & Suites in National Harbor, MD.

Code List

Building Code
Maryland Building Performance Standards (MBPS) – based on IBC 2003 and IRC

Structural Concrete Code
The American Concrete Institute (ACI) – sections 301, 318 and 315
Aggregate shall comply with ACI 304, and slump with 211.1
Reinforcing shall comply with ASTM A615, Grade 60

Masonry Code
ACI – section 530.1
Reinforcing shall comply with ASTM A615, Grade 60

Structural Steel Code
Load and Resistance Factor Design Specification (LRFD) conforming with the American Institute of Steel Construction (AISC) specification for structural steel for buildings, and AWS D1.1, latest edition
Connection bolts shall conform to ASTM A325

W shapes, columns	ASTM A992 or ASTM 572-50
S, M, and HP shapes	ASTM A36
column baseplates, web doubler plates	ASTM A992 or ASTM 572-50
channels, tees, bars, angles and plates	ASTM A36
HSS rectangular or square	ASTM A500 – GR. B (Fy=46ksi)
steel pipe	ASTM A500 – GR. B (Fy=42ksi)
anchor rods	ASTM A307, A449 where noted

Load Summary

	Corridor	Storage	Guest	Roof	Canopy
Slab	148	148	148	148	--
M/E/C/L	8	8	8	8	8
Roof	--	--	--	2	2
Insulation	--	--	--	8	8
Total Dead	156	156	156	166	16
Live	100	125	40	30	30
Partition	--	--	20	--	--
Total	256	281	216	196	48

Controlling Load Cases

Both wind and seismic forces were analyzed for this report. The design wind pressures remained the same as those found in Technical Report one, while the seismic loads were re-analyzed after the weight of the first floor was eliminated from the calculation.

Design Wind Pressure

Design Pressure		⊥55'			⊥178'		
Level	Height	p w-w	p l-w	p roof	p w-w	p l-w	p roof
1	0	8.371749	-4.5166	-18.965	8.147242	-9.5140	-19.789
2	12	8.371749	-4.5166	-15.642	8.147242	-9.5140	
3	22.25	9.399858	-4.5166		9.147781	-9.5140	
4	32.5	10.5014	-4.5166		10.21979	-9.5140	
5	42.75	11.34592	-4.5166		11.04166	-9.5140	
6	53	12.09351	-4.5166		11.76919	-9.5140	
7	63.25	12.681	-4.5166		12.34093	-9.5140	
8	74.25	13.33605	-4.5166		12.97841	-9.5140	
9	84.5	13.8501	-4.5166		13.47868	-9.5140	
10	94.75	14.3201	-4.5166		13.93607	-9.5140	
11	105	14.724	-4.5166		14.32914	-9.5140	
Low Roof	115.25	15.09118	-4.5166		14.68648	-9.5140	
High Roof	130	15.64195	-4.5166		15.22248	-9.5140	

Story Shear and Overturning Moment - Wind

Story Shear		
Level	⊥55'	⊥178'
1	8.506319	37.72452
2	7.265814	32.22303
3	7.845411	34.04851
4	8.466408	36.00438
5	8.942505	37.50389
6	9.363954	38.83127
7	10.40455	42.79204
8	10.06444	41.0375
9	10.35424	41.95024
10	10.6192	42.78475
11	10.84689	43.5019
Low Roof	15.90682	63.53848
Total	118.5866	491.9405

Overturning Moment		
Level	⊥55'	⊥178'
1	51.03792	226.3471
2	124.4271	551.8193
3	214.7681	932.0779
4	318.5486	1354.665
5	428.1224	1795.499
6	544.2799	2257.068
7	715.3131	2941.953
8	798.8648	3257.351
9	927.9985	3759.79
10	1060.592	4273.127
11	1194.514	4790.647
Low Roof	1950.574	7791.406
Total	8329.041	33931.75

Wind load calculations were performed according to ASCE 7-05 using method 2 – analytical procedure. K_{zt} was assumed to be equal to 1.0 and the building was considered enclosed when analyzing the main wind force resisting system (mwfrs) according to case 1. Through seismic calculations, the building was determined to be rigid. Linear interpolation was used where permitted.

Seismic Criteria

As the vertical distribution of forces shows, seismic analysis was the controlling factor in both directions. That is, the seismic base shear, which is the same in both directions, was larger than either direction of wind base shear. This result is not surprising, as the seismic response is based on the building weight. Concrete buildings tend to carry more mass per story, and consequently are often controlled by seismic design criteria.

The overturning moment also turned out to be larger for seismic than wind. This can be attributed to larger forces being present at higher elevations for the seismic design. The vertical distribution of forces equation attempts to take a whiplash effect into account. As the base of the building moves one way, the top wants to catch up to it. As it does this, the base of the building switches directions and moves back, thus pulling the top of the building back to its original position with much greater force.

Once the seismic and wind forces are determined, the analysis of the lateral elements of the building can begin. Because the seismic load controls, the shear walls will be analyzed according to their relative stiffness within the group using seismic load.

Total Weight by Floor		
Floor	Total Weight	Elevation
1		0
2	1472841.5	12
3	1803184	22.25
4	1803184	32.5
5	1803184	42.75
6	1803184	53
7	1803184	63.25
8	1803184	74.25
9	1803184	84.5
10	1803184	94.75
11	1327969	105
Low Roof	1055250	115.25
High Roof	44464	130

Vertical Distribution of Forces		
Floor	C_{vx}	F_x (k)
High Roof	0.00555783	3.30274673
Low Roof	0.11505062	68.3689417
11	0.13025935	77.406749
10	0.15740865	93.5402432
9	0.13822721	82.1416545
8	0.11935803	70.9286284
7	0.09949812	59.1268555
6	0.08140748	48.3764721
5	0.06378575	37.9047448
4	0.04673032	27.7695407
3	0.030397	18.0634478
2	0.01231964	7.32095544
1	0	0
	1	594.25098

Overturning Moment	
Level	
1	48294.9748

The values for base shear and overturning moment have been modified from Technical Report one. After removing the weight of the first floor, the seismic weight of the building was reduced. This was done because the first floor lies on the same plane as the surrounding earth, and as the ground moves, the first floor will effectively move with it. Hence, shear does not begin to develop until the load reaches just above the first floor.

Distribution of Lateral Loads

The distribution of loads depends directly on each member's relative stiffness. Because each shear wall is the same thickness, relative stiffnesses can be closely approximated by each member's length. In this case, all shear walls are orthogonal to each other and do not need to be broken down into components. Each shear wall's relative stiffness value is listed in the column on the far right.

Force Distribution Calculations

Element	Height	Depth	h/d	(h/d)^3	3(h/d)	ΔF	R
SW1	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW2	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW3	10	21	0.476190476	0.1079797	1.428571429	1.536551128	0.650808152
SW4	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW5	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW6	10	11	0.909090909	0.751314801	2.727272727	3.478587528	0.287473002
SW7	10	9	1.111111111	1.371742112	3.333333333	4.705075446	0.212536443
SW8	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW9	10	15.33	0.652315721	0.277570646	1.956947162	2.234517808	0.447523844
SW10	10	20	0.5	0.125	1.5	1.625	0.615384615
SW11	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486
SW12	10	21.33	0.468823254	0.103045121	1.406469761	1.509514882	0.662464486

Load Path

The more efficient the load path, the more lateral force a building's frame is able to transfer to the foundation. In the case of wind, the largest forces are transferred from the very top of the building all the way down. The wind hits the façade, is transferred to intermediate elements, and then to the columns and shear walls. Once the load reaches these lateral elements, they are transferred down to the foundation. Because the shear wall and column layout does not change as the floors go up, there is no diminishing of strength towards the top of the building. Because seismic forces control in this study, larger forces need to be resisted, but the maximum applied force occurs at story level 10, not the top. The load path is the same as before, and rigid diaphragm action from the floor slab helps keep drift to a minimum. The drop panels at each column and shear wall location further assist in minimizing each individual lateral displacement, and thus reduce sway even more.

A potential weakness of the system is the fact that only one shear wall lies completely within the floor slab. Shear wall 3 has rigid diaphragm action and bracing in all directions, but the others do not. Because they either lie on the exterior of the building or along an elevator or stairwell shaft, all the other shear walls have at least one side without bracing. Consideration must be taken into account to adequately tie the floor slab into each shear wall, where possible.

Spot Checks

Included are some quick spot checks to validate the RAM output.

<u>Center of Mass</u>

Center of Mass Calculations

					Distance from Reference			
Element	Area	Height	Unit Weight	W	x	y	Wx	Wy
Floor	9790	0.875	0.15	1285	64.66	18.33	83088.1	23554.05
SW1	11	10	0.15	16.5	5.5	0	90.75	0
SW2	11	10	0.15	16.5	145.16	0	2395.14	0
SW3	21	10	0.15	31.5	111.16	20.33	3501.54	640.395
SW4	9	10	0.15	13.5	76	31.16	1026	420.66
SW5	11	10	0.15	16.5	29.17	48.66	481.305	802.89
SW6	11	10	0.15	16.5	168.83	48.66	2785.695	802.89
SW7	9	10	0.15	13.5	76	51.16	1026	690.66
SW8	21.33	10	0.15	32	0	45.66	0	1461.12
SW9	15.33	10	0.15	23	48.665	11	1119.295	253
SW10	20	10	0.15	30	48.83	47.84	1464.9	1435.2
SW11	21.33	10	0.15	32	45.66	139.66	1461.12	4469.12
SW12	21.33	10	0.15	32	45.66	150.66	1461.12	4821.12

Xmass	Ymass
64.100715	25.24934552

Center of Rigidity

Center of Rigidity Calculations

	Distance from Reference					
Element	x	y	Rx	Ry	RxY	RyX
SW1		0	0.287473002		0	
SW2		0	0.287473002		0	
SW3		20.33	0.650808152		13.23092973	
SW4		31.16	0.212536443		6.622635569	
SW5		48.66	0.287473002		13.98843629	
SW6		48.66	0.287473002		13.98843629	
SW7		51.16	0.212536443		10.87336443	
SW8	0			0.662464486		0
SW9	11			0.447523844		4.922762289
SW10	47.83			0.615384615		29.43384615
SW11	139.66			0.662464486		92.51979008
SW12	150.66			0.662464486		99.80689942
			2.225773047	3.050301917	58.7038023	226.6832979

Xrigidity	Yrigidity
74.315036	26.37456788

When compared to the RAM output, it is clear that the model accurately demonstrates the lateral characteristics of the building. From hand calculations, the center of mass was found to be (64.1, 25.24), and the center of rigidity to be (74.3, 26.37). RAM gave coordinates of (64.53, 24.64) and (75.24, 26.9), respectively. These numbers came within 2.4% of the hand calculated values, so the RAM model is justified to calculate more complicated lateral analyses.

Strength Check

Shear wall 3 was checked to see if it had adequate strength to withstand seismic forces acting in its direction. Through hand calculations of relative strength and lateral distribution, it was found that shear wall 3 took 175.9k of seismic force, and RAM echoed this finding with a result of 163.5k after its distribution. At its base, shear wall 3 is 21’-0” long, takes 176k of force, and has a strength of 6000 psi. After calculating its strength and reinforcing, the results matched those called out on the drawings of #5 bars @ 18” O.C. each way, and it was found to have more than adequate strength.

Story Drift and Total Drift Check

From the RAM model, total wind drift was calculated to be 2", or a ratio of L/781. This drift seems appropriate since the building is made of concrete and has rigid diaphragm action at every floor, including drop panels at each shear wall. Seismic drift came out to be 2.88", or a ratio of L/542. The small drift in each case also might be explained by the fact that there are numerous shear walls, and one or more of them might be sacrificed to save money and still be within the allowable limits for drift.

Story Drift (⊥ to long direction)

Level	Wind	Seismic
1	0.0478	0.1187
2	0.059	0.148
3	0.0725	0.1826
4	0.0834	0.2105
5	0.0919	0.232
6	0.0985	0.2478
7	0.1109	0.2777
8	0.1066	0.265
9	0.1085	0.2672
10	0.1094	0.267
11	0.1097	0.2658
12	0.9982	0.3965
	1.9964	2.8788

	Building Height to Low Roof
	130'-0"
	Equivalent Drift, Seismic
L/	541.8924552
	Equivalent Drift, Wind
L/	781.4065318

The maximum story drift comes from wind, and is almost one inch at the top floor. From ASCE 7-05, the maximum allowable story drift is 2.46" for a 10'-3" floor height, which is greater than 1", so it is acceptable per ASCE 7-05.

Torsion Issues

Because the centers of mass and rigidity do not coincide, any wind or seismic loading will create inherent torsion on the building. The distance between the two centers is 9.84' East-West and 4.23' North-South. By taking a consistent sign convention, the results from the hand analysis closely match those found from the RAM output.

Hand Calculations

Lateral Load Distributions, Forces Parallel to Short Dimension

Controlling Shear (k):	594.25							
Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.287473002	28.83			238.938619	0	1.007907489	-1.00791
SW2	0.287473002	28.83			238.938619	0	1.007907489	-1.00791
SW3	0.650808152	8.51			47.13159143	0	0.67353708	-0.67354
SW4	0.212536443	2.32			1.143956152	0	0.059965348	0.059965
SW5	0.287473002	19.798			112.6781491	0	0.692145421	0.692145
SW6	0.287473002	19.798			112.6781491	0	0.692145421	0.692145
SW7	0.212536443	22.29			105.5974778	0	0.57613259	0.576133
SW8			0.662464486	75.24	3750.249612	129.0591985	6.061642977	135.1208
SW9			0.447523844	64.24	1846.831377	87.18515472	3.496235495	90.68139
SW10			0.615384615	27.41	462.3434462	119.8872497	2.051325769	121.9386
SW11			0.662464486	64.44	2750.892787	129.0591985	5.191550684	123.8676
SW12			0.662464486	75.44	3770.213642	129.0591985	6.077755797	122.9814

Lateral Load Distributions, Forces Parallel to Long Direction

Controlling Shear (k):	594.25							
Element	Ksn	Cn	Ksn	Cn	KsnCn^2	Direct Shear	Torsional Shear	Hn
SW1	0.287473002	28.83			238.938619	76.75123561	3.261955148	80.01319
SW2	0.287473002	28.83			238.938619	76.75123561	3.261955148	80.01319
SW3	0.650808152	8.51			47.13159143	173.7565943	2.179810913	175.9364
SW4	0.212536443	2.32			1.143956152	56.74423164	0.194069672	56.55016
SW5	0.287473002	19.798			112.6781491	76.75123561	2.24003427	74.5112
SW6	0.287473002	19.798			112.6781491	76.75123561	2.24003427	74.5112
SW7	0.212536443	22.29			105.5974778	56.74423164	1.864574563	54.87966
SW8			0.662464486	75.24	3750.249612	0	19.61768091	-19.6177
SW9			0.447523844	64.24	1846.831377	0	11.31508942	-11.3151
SW10			0.615384615	27.41	462.3434462	0	6.638836125	-6.63884
SW11			0.662464486	64.44	2750.892787	0	16.80174585	16.80175
SW12			0.662464486	75.44	3770.213642	0	19.66982785	19.66983

Hn, the column on the far right, is the design net shear. It accounts for direct shear plus or minus torsional shear according to sign convention. Looking at shear wall 12, the design shear forces are 122.9k for seismic acting along its direction, and 19.66k for seismic acting perpendicular to its direction (all 19.66k are attributed to torsion). The computer model calculates each case to be

110.77k and 19.54k, respectively. Clearly, these numbers are close to the actual values obtained from hand calculations, and are thus justified. These torsion values also include a 5% “accidental” eccentricity in their calculations.

Discrepancies

Direct Shear

There were some irregularities with the computer model that could not be explained by hand calculations. One such difference was the fact that two identical shear walls with the same length did not carry the same direct shear force. It seemed that, the farther away from the point at which the load was applied, the more diffused the load became. That is, the loads from the computer model became less intense as the distance increased from the center of rigidity or stiffness. This can be attributed to redistribution of lateral loads in two-way concrete slab systems. The hand calculation method assumes that a load is perfectly distributed among lateral elements according to stiffness, when in reality, gravity columns and non-lateral frames do posses some stiffness and therefore absorb some of the load.

Torsional Shear

Hand tabulated torsional shear values were very close to the computer output in some locations, and approximate in other locations. Seismic torsional shear depends on both the center of mass and the center of rigidity. On levels one and two, the center of mass changes due to variations in the floor plan. This was not taken into consideration for hand calculations, as only a typical floor (levels 3 to 11) was analyzed for torsion.

Conclusion

This report was intended to further analyze the lateral elements of the Hampton Inn & Suites in National Harbor, Maryland from the findings in Technical Report one. From this continued analysis, the lateral distribution to shear walls was refined and further analysis of seismic and wind impacts were carried out.

After posting Technical Report one, the seismic loads were revised to account for the first floor being flush with the ground, and therefore no shear develops until just above the floor level. This reduced the effective seismic weight and the design base shear.

An in-depth study of lateral load distribution was again carried out, along with torsion, but with revised controlling seismic design forces. These results were compared to a RAM computer model and were found to be validated. The computer model was then used to conduct what would have been a tedious drift calculation by hand. These values were found to be within the acceptable limits of L/400 for total drift, and 0.02 multiplied by the story height for story drift. These limits were found in ASCE 7-05 and exist due to serviceability requirements. In this case, the total drift was calculated to be 1.996" for wind and 2.88" for seismic, or drift ratios of L/781 and L/542, respectively.

These actual drifts are considerably below the allowable limits, and therefore, provide the owner with a few options to save costs. Either a few shear walls could be completely eliminated from the lateral system, or they could all be shortened, repositioned, or reduced in strength and still possibly meet the drift criteria.

It is clear that, in the case of high-rise construction, drift controls over strength. This was found from simple strength checks of critical shear walls – each one is over designed for strength because drift design is much more stringent.

Torsional shear was not a factor for the shear walls near the center of stiffness, as expected, but amounted to a considerable force in shear walls aligned in the North-South direction. Shear walls 8 and 12, which are farthest away from the center of stiffness, each had 20k all attributed to torsional shear. However, in the East-West direction, each only had 6k of torsional shear. This is due to the fact that the torsional component is larger in the East-West direction than the North-South direction.

In summary, the findings from hand calculations and from the RAM model both parallel each other, and therefore are justified. The building is within drift limits, has sufficient strength, and possesses an adequate load path to get both wind and seismic loads to the foundation.

Appendix

Table of Contents

Wind Calculations

Wind Variables	
Variable	Value
h	130
V	100
Kd	0.85
I	1
Kzt	1
GCpi	0.18
qh	21.8144
Iz	0.259931
Q ⊥178'	0.812881
Q ⊥55'	0.853623
G ⊥178'	0.821083
G ⊥55'	0.843709

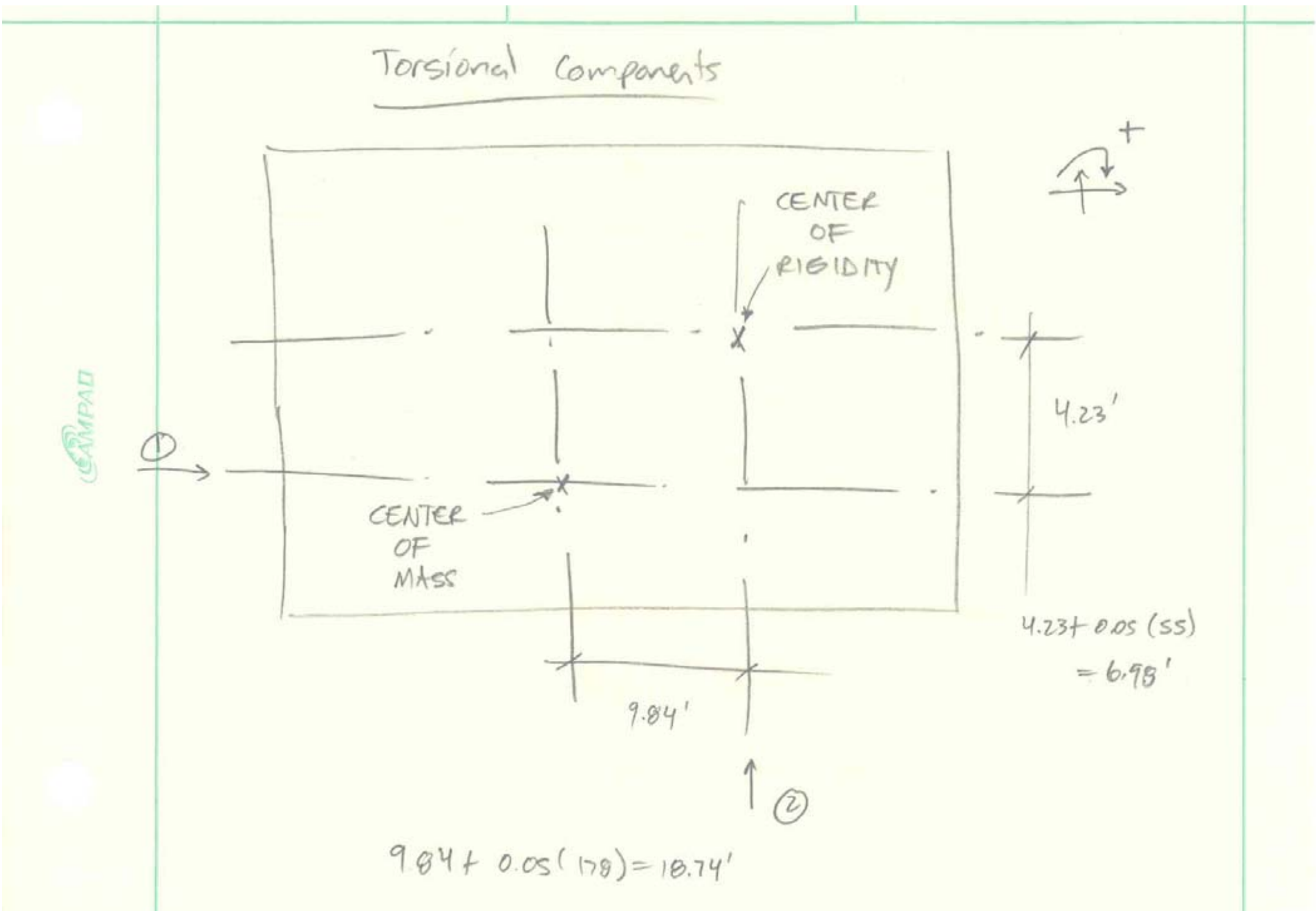
Velocity Pressures by Floor				
Level	Height	Kz	qz	
1	0	0.57	12.4032	
2	12	0.57	12.4032	
3	22.25	0.64	13.9264	
4	32.5	0.715	15.5584	
5	42.75	0.7725	16.8096	
6	53	0.8234	17.91718	
7	63.25	0.8634	18.78758	
8	74.25	0.908	19.75808	
9	84.5	0.943	20.51968	
10	94.75	0.975	21.216	
11	105	1.0025	21.8144	
Low Roof	115.25	1.0275	22.3584	
High Roof	130	1.065	23.1744	qh
Parapet	132	1.07	23.2832	

Seismic Calculations

Seismic Inputs	
Variable	Value
S_s	0.152
S_1	0.5
F_a	1.6
F_v	2.4
I	1
SM_s	0.2432
SM_1	1.2
SD_s	0.16213333
SD_1	0.8
R	5
C_s	0.03242667
C_t	0.02
h_n	130
x	0.75
T_a	0.7699943
T_o	0.98684211
T_s	4.93421053
V (k)	594.25098

Weight Seen by Floor		
Floor	Weight	Story Shear
High Roof	44464	1.44181931
Low Roof	1099714	35.6600593
11	2427683	78.7216674
10	4230867	137.192914
9	6034051	195.66416
8	7837235	254.135407
7	9640419	312.606653
6	11443603	371.0779
5	13246787	429.549146
4	15049971	488.020393
3	16853155	546.491639
2	18325996.5	594.25098
1	18325996.5	594.25098

Torsional Components



Story Drift

Story Drift (seismic)

$S_{D1} = 0.033$ Occupancy I or II

Seismic Design Category A

Table 12.12-1 Allowable Story Drift

$\Delta a = 0.02\, h_{sx}$, where h_{sx} is the story height below level x

@ level 12, $h_{sx} = 10'$

$(10')(12)(0.02) = 2.4''$

$\Delta_{TOP} = 1.0137'' < 2.4''$ ∴ ok

Strength Check

Shear wall calculation (revised)

$\boxed{SW\ 3}$ $l = 21'$

$V = 178k$

ACI 21.7.2.2

$2(12)(21)(12)\sqrt{6000}/1000 = 468.5 < V_u \therefore 2$ curtains

$\rho_l, \rho_t = \frac{As_l}{A_{cv}} \geq 0.0025$

$A_{cv} = (144 in^2/ft)(0.0025) = 0.36 in^2/ft$ req'd

Assume #5

$As_l = 0.62 in^2 / \$$, $\$ =$ spacing

$\frac{0.36}{12} = \frac{0.62}{\$} \Rightarrow \$ = 20.67''$ MAX

Try #5 @ 18" o.c. Both Directions

$\frac{h_w}{l_w} = \frac{130}{21} = 6.19 > 2 \therefore \alpha = 2.0$

$V_u = (12)(12)(21)(2\sqrt{6000} + 0.0043(60,000))/1000 = 1249k$

$\phi V_n = 0.6(1249) = 749.4k > V_u \therefore$ ok