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Technical Report 3

Lateral Force Resisting Systems

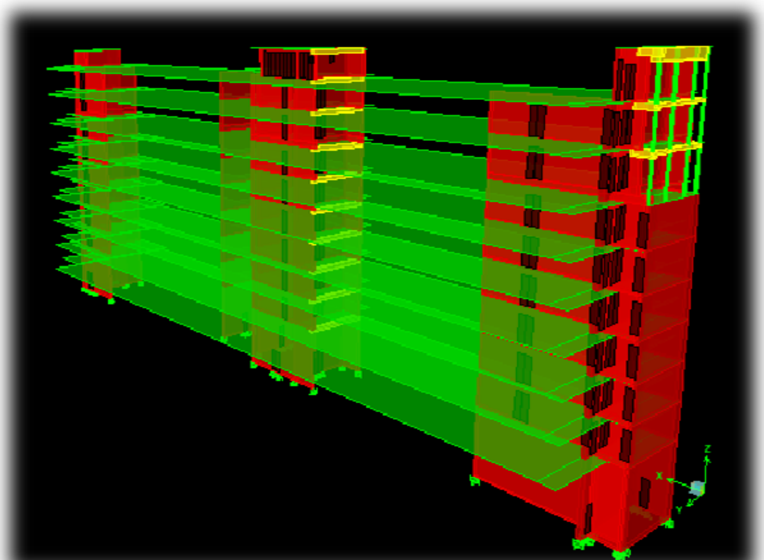
TABLE OF CONTENTS

EXECUTIVE SUMMARY

This report analyzes the lateral load resisting systems for the Residence Inn by Marriott located in downtown Norfolk, VA. The nine story hotel will rise to over one hundred feet above a lively coastal city, and feature 160 guest suites designed for extended-stay guests.

In an effort to understand the lateral load resisting systems, a combination of simple hand calculations, a three-dimensional computer model, and visual observations were exploited. All fourteen of the Residence Inn's cast-in-place concrete shear walls, which provide all of the lateral resistance, were modeled as membrane elements in a structural analysis program called ETABS, where they were analyzed for stiffness and drift. Hand calculations were performed, confirming the accuracy of the model. The model proves to be accurate enough to use as a basis for future models of the Residence Inn with continued refining.

Among other observations, the key findings include that wind controls in the North-South direction and seismic controls the design in the East-West direction. Torsion forces were not significant with this particular building, although a more precise model and check may be worthwhile to address the differences between the calculated center of rigidity and that which ETABS determined. The weight of the building is drastically larger than any uplift forces; therefore, this is not a concern either.



(FIGURE 1) 3-D View of ETABS Model

INTRODUCTION

The new Residence Inn by Marriott will be situated in a lively downtown Norfolk, Virginia area, surrounded on all sides by busy streets. The hotel will serve as an upscale temporary residence with extensive amenities for its extended stay patrons. The building itself boasts a unique combination of simple structural components and fascinating architectural features. A tasteful combination of architectural precast, drainable EIFS, and curtain wall will be used to make this building an impressive and distinguished landmark in the community.

There will be 160 guest suites on eight upper floors, with public functions, such as lobbies, gathering areas, and an indoor swimming pool, located on the first floor. The extensive program on the first floor requires large open spaces desired for architectural allure. The upper floors generally have the same layout; only minor differences exist to accommodate various room types. A main corridor connecting the emergency stairwells at either end of the building separates 10 guest suites each on the North and South sides of the building. A pair of elevators is located at a central core along this corridor. Many of the upper floor suites will have magnificent views of the surrounding city and inner-coastal bays.

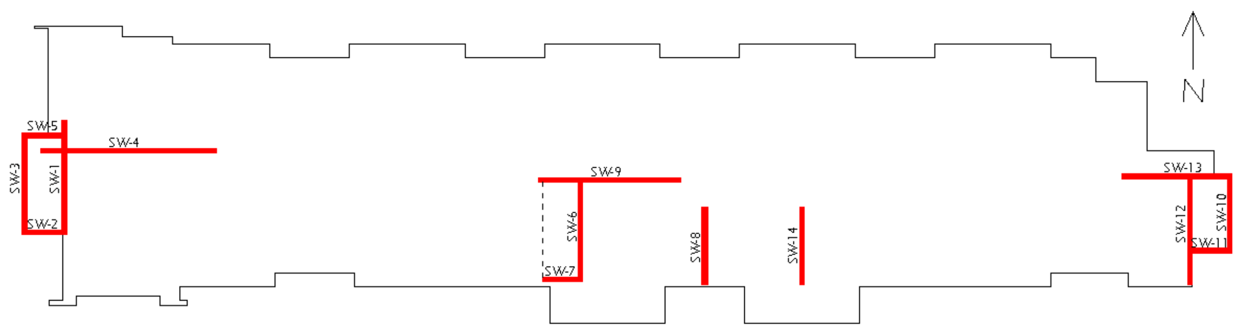
Typical floor-to-floor heights are 9'-4", with the first floor having a height of 19'-0". The total height of the building as designed is approximately 95 feet, excluding parapets and stair towers that extend beyond the main roof. Floor plans and building sections illustrating the architecture and general configuration of the building may be referenced in Appendix A.

FOUNDATIONS & GRAVITY LOAD SYSTEMS

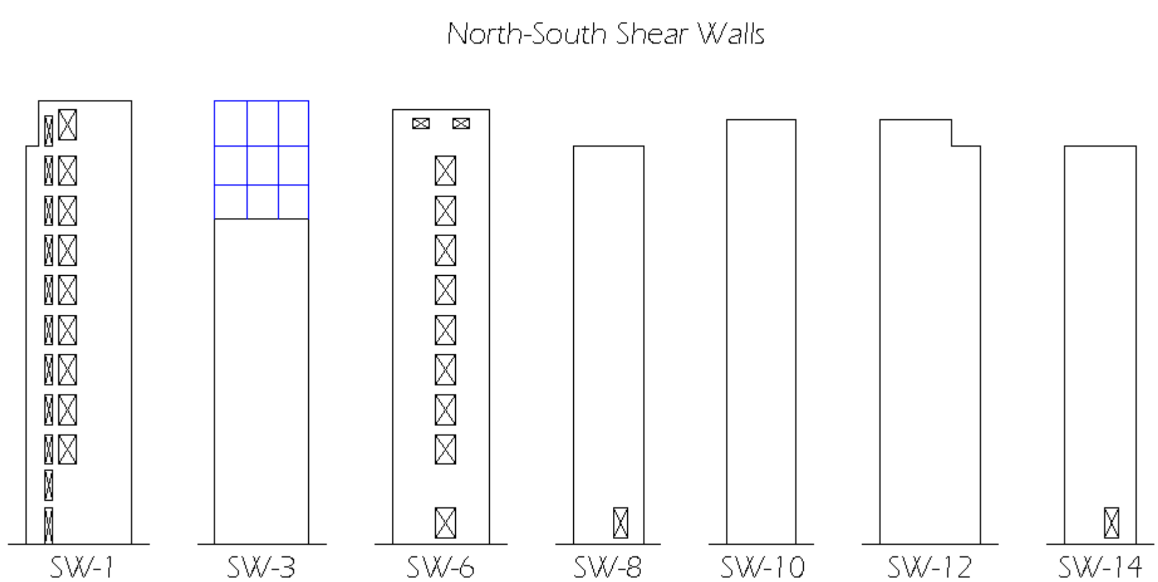
Foundations consist of precast concrete piles (100 ton capacity; 35 ton uplift capacity) driven to 70', cast-in-place concrete pile caps and grade beams. Above grade, the Residence Inn is almost entirely structurally supported by reinforced concrete elements. The floor system as well as the roof consists of an 8" two-way flat plate slab. Reinforced concrete columns, ranging in size from 12"x24" on the upper floors to 20"x30" at the first floor, support the two-way slab system. Typical interior columns are 14"x30". At the second floor, reinforced concrete transfer girders are used to discontinue several columns from above, providing larger open spaces on the ground floor below.

LATERAL FORCE RESISTING SYSTEMS

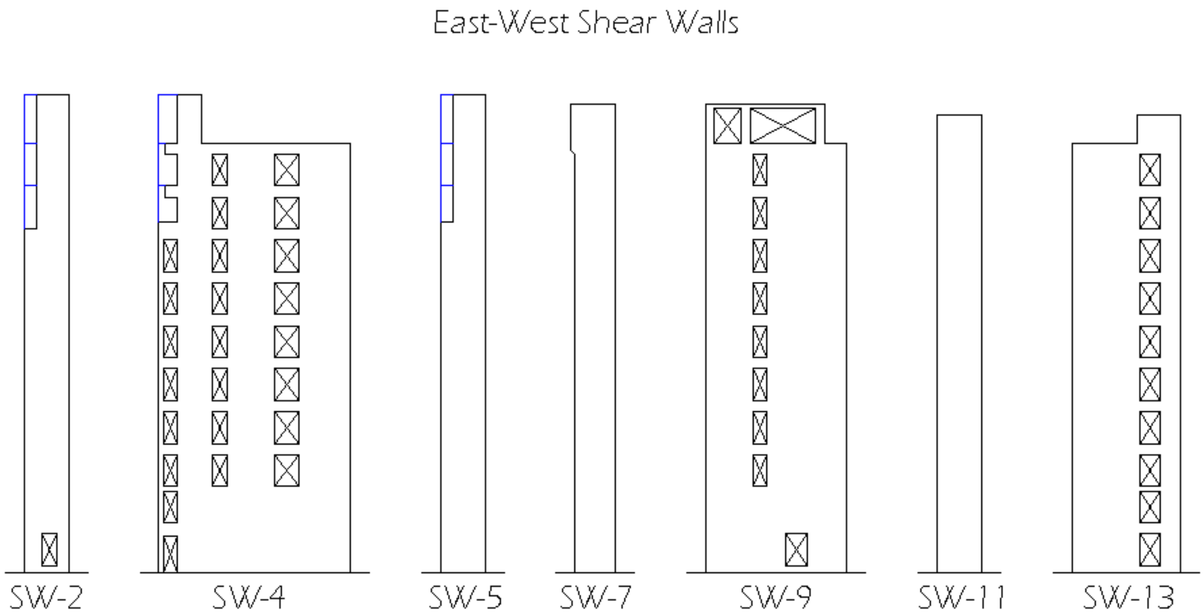
Cast-in-place reinforced concrete shear walls are employed to resist lateral forces. There are a total of fourteen shear walls, the majority of which are 1'-0" thick, with a few slightly larger at 1'-2". These shear walls are continuous from the foundation to the top of the building, and behave as fixed cantilevers. Lateral loads are transmitted to the shear walls through the floor diaphragm. Several shear walls located at the west stair tower contain three stories of HSS steel tubing to support an expanse of curtain wall (shown in blue in the elevations below). These frames are rigidly connected to the surrounding concrete shear walls; however, they provide little lateral force resistance as compared with the shear walls. See the figures below for an outline of a typical floor showing shear wall locations and separate figures follow illustrating shear wall elevations.



(FIGURE 2) Typical Floor Diaphragm & Shear Wall Layout



(FIGURE 3) Concrete Shear Wall Elevations – N-S Direction



(FIGURE 4) Concrete Shear Wall Elevations – E-W Direction

The purpose of this report is to perform an in-depth analysis of these systems, identifying areas of interest or concern and gaining an understanding for how they work. In order to do so, the following subjects will be explored as they relate to the Residence Inn:

- Lateral loads – wind and seismic, adjusted from Tech 1
- Load cases & a discussion of criticality
- Applicable design standards and drift criteria
- Lateral load distribution based on relative stiffness of shear walls
- Torsion/center of mass/center of rigidity
- Story drift & building deflections
- Overturning moments & impact on foundations

LOADS & LOAD CASES

Since the focus of this report is on lateral force resisting systems, wind and seismic loads will be discussed in detail. For sake of thoroughness, gravity loads have been included in Appendix B for reference, but they were omitted in this analysis for simplicity. Once a complete three-dimensional model is constructed, all loads will be considered simultaneously.

WIND LOADS

Wind loads were previously calculated in Technical Report 1 using the analytical method prescribed in ASCE 7-05, Chapter 6. A summary table of the results is shown below. The only difference here is that an additional column has been added to include factored story forces (1.6 multiplier for the most critical case) in order to obtain a factored base shear for comparison with seismic lateral loads and determine which loads control the design.

EAST-WEST WIND LOAD											
Floor	Location	Height Above Ground Level	Tributary Height	Tributary Width	Velocity Pressure	External Pressure	Total Pressure WW+(-LW)	Story Force	Factored Story Force	Story Shear	Overturning Moment
		h (ft)	(ft)	(ft)	q (psf)	qGC_p (psf)	p_t (psf)	F_x (k)	1.6*Fx (k)	V_x (k)	M_x (ft-k)
W Stairwell	Windward	105.77	6.05	24.00	33.44	22.74	37.06	5.38	8.61	5.38	569.19
Roof		93.67	4.67	65.00	32.91	22.38	36.70	11.14	17.83	16.52	1,043.55
9th		84.33	9.34	65.00	32.12	21.84	36.16	21.96	35.13	38.48	1,851.48
8th		75.00	9.34	65.00	31.33	21.30	35.63	21.63	34.61	60.11	1,622.18
7th		65.67	9.34	65.00	30.28	20.59	34.91	21.20	33.91	81.30	1,391.92
6th		56.33	9.34	65.00	29.49	20.05	34.38	20.87	33.39	102.17	1,175.58
5th		47.00	9.34	65.00	28.44	19.34	33.66	20.44	32.70	122.61	960.49
4th		37.67	9.34	65.00	27.12	18.44	32.76	19.89	31.83	142.50	749.30
3rd		28.33	9.34	65.00	25.54	17.37	31.69	19.24	30.78	161.74	545.04
2nd		19.00	14.17	65.00	23.43	15.93	30.25	27.87	44.59	189.60	529.46
	Leeward	ALL			33.70	-14.32	Base Shear =	189.60	303.37	M =	10,438.18

(FIGURE 5) East-West Wind Pressures, Forces, & Overturning Moment Summary

NORTH-SOUTH WIND LOAD											
Floor	Location	Height Above Ground Level	Tributary Height	Tributary Width	Velocity Pressure	External Pressure	Total Pressure WW+(-LW)	Story Force	Factored Story Force	Story Shear	Overturning Moment
		h (ft)	(ft)	(ft)	q (psf)	qGC_p (psf)	p_t (psf)	F_x (k)	1.6*Fx (k)	V_x (k)	M_x (ft-k)
W Stairwell	Windward	105.77	6.05	12.00	33.44	22.74	28.47	2.07	3.31	2.07	218.61
Roof		93.67	4.67	266.77	32.91	22.38	28.11	35.02	56.03	37.09	3,280.17
9th		84.33	9.34	266.77	32.12	21.84	27.57	68.70	109.92	105.78	5,793.33
8th		75.00	9.34	266.77	31.33	21.30	27.03	67.36	107.78	173.14	5,051.98
7th		65.67	9.34	266.77	30.28	20.59	26.32	65.58	104.93	238.72	4,306.69
6th		56.33	9.34	266.77	29.49	20.05	25.78	64.24	102.79	302.97	3,618.77
5th		47.00	9.34	266.77	28.44	19.34	25.07	62.46	99.94	365.43	2,935.77
4th		37.67	9.34	266.77	27.12	18.44	24.17	60.23	96.36	425.66	2,268.74
3rd		28.33	9.34	266.77	25.54	17.37	23.10	57.55	92.08	483.21	1,630.38
2nd		19.00	14.17	266.77	23.43	15.93	21.66	81.89	131.02	565.09	1,555.85
	Leeward	ALL			33.70	-5.73	Base Shear =	565.09	904.15	M =	30,660.29

(FIGURE 6) North-South Wind Pressures, Forces, & Overturning Moment Summary

SEISMIC LOADS

Seismic loads were also previously calculated in Technical Report 1 using ASCE 7-05, Chapter 12; however, several adjustments have since been made to obtain more accurate results. Initially, the estimated superimposed dead loads incorporated in calculating the seismic weight were overly conservative. A more realistic superimposed dead load of 10 psf (instead of 15 psf) was assigned to each floor, which significantly reduced the resulting seismic base shear. There still exists a discrepancy between the design spectral acceleration values; therefore, until this is resolved, the designer's seismic response coefficient will be used for analysis because it is more conservative than that which was calculated in Technical Report 1. See the figure below for a summary of the adjusted seismic loads.

SEISMIC LOAD DISTRIBUTION							
Floor	Weight	Height		Vertical Distribution Factor	Story Force	Story Shear	Overturning Moment
	w_x (k)	h_x (ft)	$w_x{*}h_x^k$	C_{vx}	F_x (k)	V_x (k)	M_x (ft-k)
West Stair Roof	122.81	105.67	50,135	0.0119	5.32	5.32	561.90
Penthouse Roof	114.03	102.17	44,568	0.0106	4.73	10.04	482.97
East Stair Roof	26.13	99.67	9,890	0.0024	1.05	11.09	104.55
Main Roof	2,093.06	93.67	731,386	0.1743	77.57	88.67	7266.32
9th	2,426.05	84.33	740,315	0.1764	78.52	167.19	6621.65
8th	2,426.05	75	636,398	0.1516	67.50	234.69	5062.41
7th	2,471.77	65.67	546,275	0.1302	57.94	292.63	3804.92
6th	2,471.77	56.33	448,190	0.1068	47.54	340.16	2677.74
5th	2,471.77	47	354,825	0.0845	37.63	377.80	1768.80
4th	2,471.77	37.67	266,712	0.0635	28.29	406.09	1065.63
3rd	2,471.77	28.33	184,674	0.0440	19.59	425.67	554.91
2nd	3,554.97	19	158,646	0.0378	16.83	442.50	319.70
Mech. Mezzanine	1,223.59	10.33	24,879	0.0059	2.64	445.14	27.26
1st	384	0	0	-	-	445.14	-
TOTALS	**24,729.91**			**1.0000**	**445.14**		**30,318.76**

(FIGURE 7) Seismic Forces & Overturning Moment Summary

LOAD COMBINATIONS

The following factored load combinations, prescribed by ASCE 7-05, Chapter 2, are applicable to this lateral load analysis:

(Note: D_i, F, F_a, H, R, T, & W_i are assumed to be zero)

1. 1.4D
2. 1.2D + 1.6L + 0.5(L_r or S)
3. 1.2D + 1.6(L_r or S) + (L or 0.8W)
4. 1.2D + 1.6W + L + 0.5(L_r or S)
5. 1.2D + 1.0E + L + 0.2S
6. 0.9D + 1.6W
7. 0.9D + 1.0E

It is apparent that regardless of the impact of gravity loads, the critical factored lateral load will be either 1.6W or 1.0E. Therefore, it is adequate to assess the controlling load in each direction based on applying these factors. As can be seen in the summary lateral loading figures above, wind is controlling in the North-South direction, with a factored base shear of approximately 905 kips, which is more than twice that of seismic. This is not surprising considering the sizeable façade facing this direction. In the East-West direction, however, seismic controls, with a critical base shear of approximately 450 kips, as compared with the less critical wind load of just over 300 kips in this direction.

APPLICABLE DESIGN STANDARDS/REFERENCES

- IBC 2006
- Virginia Uniform Statewide Building Code – 2003 Edition
- ASCE 7-05
- ACI 318-08

Building Drift Limitations:

- H/400 (Accepted value for service loads (D+L+W); Structural Engineering Handbook, 1968)
- $0.020h_{sx}$ Story Drift – Seismic (for a typical story $\Delta_{s,\,max}$ =2.28" (9'-6" story height))

LATERAL LOAD DISTRIBUTION

ETABS MODEL - OVERVIEW OF ASSUMPTIONS

Lateral load resisting systems including shear walls, steel framing at the West stairwell, and critical concrete beams at shear walls was modeled in ETABS as a basis for hand calculation comparison. With a few simplifying assumptions, a fairly representative and accurate model was successfully created. Shear walls were modeled as membrane elements meshed at a maximum size of 24"x24", and fixed at their bases, thus behaving as cantilevers. Mass was applied to the structure with a uniform area mass that included the self weight of the typical 8" two-way flat-plate slab, a 10 psf superimposed dead load, and column weights were distributed also as a uniform load on the diaphragm. In an effort to represent actual conditions, openings in all shear walls were made by using the door and window opening features in ETABS. Having limited experience at gauging modeling assumptions, I exercised caution when computer modeling in an effort to avoid over-simplification and inaccurate results. This proved to be a valuable lesson, developing the ability to recognize what sizes/orientations of openings have the most significant impact on the stiffness of each shear wall. A rigid diaphragm was assumed, and minor niches were omitted for simplification and ease of construction.

RELATIVE STIFFNESS OF SHEAR WALLS

In addition to modeling the lateral systems with ETABS, hand calculations were performed to confirm the accuracy of the model. The relative stiffness of each wall in the E-W direction was calculated by applying an arbitrary 100 kip load at the uppermost level of each wall. Since each shear wall has a unique profile, with varying widths, openings, and concrete strengths at different floors within the same wall, it was necessary to make some simplifying assumptions. Simple averages were computed for the moment of inertia, modulus of elasticity, and cross-sectional areas, based on a percentage of the wall's height with a certain attribute. After getting a feel for the importance of considering openings, some relative comparisons between similar walls was made to expedite the calculations. It was confirmed that shear deformations were negligible, and, therefore, for the remaining calculations, they were omitted. A similar discovery was made in Dr. Lepage's Computer Modeling class when comparing a steel structure to a concrete one. In general, shear deformations are less significant in the concrete structure because of a larger cross sectional area. By taking a ratio of relative stiffness to the summation of stiffness in each direction, a percentage is obtained that represents the portion of lateral load a particular wall is capable of resisting. For extensive calculations of relative stiffness, refer to Appendix C.

Similarly, in the ETABS model, an arbitrary 1000 kip load was applied at the center of mass, and the resulting story shears at the ninth floor were recorded. During this analysis, the program was set to neglect torsion, so as to obtain a more relevant comparison with the hand calculations, as they did not incorporate the effects of torsion. Percentages of lateral load that each shear wall resists were determined by the ratio of story shear in a particular member to the total story shear. After comparing these values with the aforesaid hand calculations, it was concluded that they generally agree to within an acceptable range of error. Therefore, relative stiffness of shear walls in the N-S direction could be accurately obtained by a similar procedure using the model output. The figure below summarizes the relative stiffness of each shear wall and the corresponding percentage of lateral load taken by each.

Relative Stiffness & Lateral Force Distribution in Shear Walls					Hand Calculation Results
9th Floor	Shear Wall	Story Shear @ 9th Flr. (k)	Relative Stiffness	% of Total Lateral Force Resisted	% of Total Lateral Force Resisted
X-Direction Loading (E-W)	SW-2	14.23	0.01	1.36	0.64
	SW-4	526.96	0.51	50.52	62.58
	SW-5	4.97	0.00	0.48	0.64
	SW-7	7.67	0.01	0.74	0.52
	SW-9	340.76	0.33	32.67	23.54
	SW-11	14.23	0.01	1.36	0.78
	SW-13	134.16	0.13	12.86	11.29
	TOTAL	**1042.98**	~1000		
Y-Direction Loading (N-S)	SW-1	142.83	0.14	13.86	
	SW-3	2.34	0.00	0.23	
	SW-6	439.16	0.43	42.62	
	SW-8	67.58	0.07	6.56	
	SW-10	81.66	0.08	7.92	
	SW-12	229.36	0.22	22.26	
	SW-14	67.58	0.07	6.56	
	TOTAL	**1030.51**	~1000		

(FIGURE 8) Summary of Relative Stiffness & Lateral Force Distribution in Shear Walls – 9th Flr.

As can be seen above, the lateral force distribution obtained by ETABS is comparable to that obtained through hand calculations. There are a number of reasons justifying the slight differences in these values. All assumptions used to calculate relative stiffness by hand (ex. averaging properties) obviously will cause slight differences. In regards to the ~10% differences seen in Shear Walls 4 & 9, significant irregularities exist in each of these walls, as seen in the elevations. It may have been a wiser choice to evaluate relative stiffness at a central floor level where there is more consistency in shape and opening patterns because the steel curtain wall frame, large openings, and stepped profiles drastically alter the percentages of shear seen in these sensitive areas. In order to see if the floor level may affect the relative stiffness, an additional calculation was performed, this time at the fourth floor, as shown in the figure below.

Relative Stiffness & Lateral Force Distribution in Shear Walls				
4th Floor	Shear Wall	Story Shear @ 4th Flr. (k)	Relative Stiffness	% of Total Lateral Force Resisted
X-Direction Loading (E-W)	SW-2	5.51	0.01	0.53
	SW-4	654.66	0.63	62.77
	SW-5	4.22	0.00	0.40
	SW-7	6.05	0.01	0.58
	SW-9	329.30	0.32	31.57
	SW-11	5.51	0.01	0.53
	SW-13	141.94	0.14	13.61
	TOTAL	**1147.19**	~1000	
Y-Direction Loading (N-S)	SW-1	128.19	0.12	12.44
	SW-3	176.47	0.17	17.12
	SW-6	440.66	0.43	42.76
	SW-8	51.33	0.05	4.98
	SW-10	51.03	0.05	4.95
	SW-12	181.13	0.18	17.58
	SW-14	51.33	0.05	4.98
	TOTAL	**1080.14**	~1000	

(FIGURE 9) Summary of Relative Stiffness & Lateral Force Distribution in Shear Walls – 4^{th} Flr.

Relatively speaking, the distribution is approximately the same. The obvious difference can be seen in Shear Wall 3, as anticipated. Taking a section cut through a lower floor where the shear wall portion exists reveals drastically different results, confirming the stated assumption that the steel frame at the West stair serves the primary function of supporting a large expanse of curtain wall, and offers little resistance to lateral forces itself other than to transfer localized lateral loads into the adjacent shear walls.

To comment on the reliability of ETABS output in calculating relative stiffness in the opposite direction, which was not exhaustively checked by hand calculations, the results seem logical. By visual inspection, the relative stiffness of shear walls in the N-S direction appears roughly distributed according to relative widths and size of openings. In general, the wider walls are seen taking more shear and similar walls share similar load responsibility. Here again, however, the results for Shear Wall 3 on Floor 9 appear skewed because investigation took place where the HSS steel tube frame is present. Shear Walls 1 & 6 appear, at a first glance, to be capable of resisting similar loads, but again, the presence of a step at the top of Shear Wall 1 is contributing to a significantly lower relative stiffness. In addition, the location of openings in Shear Wall 1 closer to one side makes it more susceptible to overturning, and, therefore, less stiff. This example is similar to comparing a table with two symmetrical solid legs to one with one large and one small leg trying to work together to prevent overturning. Obviously, the symmetrical case would be stiffer.

Actual distribution of direct story forces is shown in the figures that follow. Note that on Floors 8 and above, the percentages corresponding to the relative stiffness at Floor 9 was used. Below this level, it was assumed that Floor 4 is representative in terms of the relative stiffness of shear walls and distribution of lateral loads proceeded accordingly.

X-Direction Loading (East-West) [1.0 * SEISMIC]																	
Floor	Height (ft)	Total Factored Load (k)	SW-2		SW-4		SW-5		SW-7		SW-9		SW-11		SW-13	
			Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment
Upper Roof	105.67	**5.32**	**0.07**	7.67	**2.69**	284.03	**0.03**	2.68	**0.04**	4.13	**1.74**	183.67	**0.07**	7.67	**0.68**	72.31
Roof	93.67	**77.57**	**1.06**	99.13	**39.19**	3671.10	**0.37**	34.62	**0.57**	53.43	**25.34**	2373.92	**1.06**	99.13	**9.98**	934.63
9th	84.33	**78.52**	**1.07**	90.34	**39.67**	3345.52	**0.37**	31.55	**0.58**	48.69	**25.65**	2163.39	**1.07**	90.34	**10.10**	851.74
8th	75	**67.50**	**0.92**	69.07	**34.10**	2557.80	**0.32**	24.12	**0.50**	37.23	**22.05**	1654.01	**0.92**	69.07	**8.68**	651.20
7th	65.67	**57.94**	**0.00**	0.00	**33.06**	2171.33	**0.21**	14.00	**0.31**	20.07	**16.63**	1092.20	**0.28**	18.28	**7.17**	470.78
6th	56.33	**47.54**	**0.00**	0.00	**27.13**	1528.20	**0.17**	9.85	**0.25**	14.12	**13.65**	768.70	**0.23**	12.86	**5.88**	331.34
5th	47	**37.63**	**0.00**	0.00	**21.47**	1009.28	**0.14**	6.51	**0.20**	9.33	**10.80**	507.68	**0.18**	8.49	**4.66**	218.83
4th	37.67	**28.29**	**0.00**	0.00	**16.14**	608.15	**0.10**	3.92	**0.15**	5.62	**8.12**	305.90	**0.14**	5.12	**3.50**	131.86
3rd	28.33	**19.59**	**0.00**	0.00	**11.18**	316.71	**0.07**	2.04	**0.10**	2.93	**5.62**	159.31	**0.09**	2.67	**2.42**	68.67
2nd	19	**16.83**	**0.00**	0.00	**9.60**	182.48	**0.06**	1.18	**0.09**	1.69	**4.83**	91.79	**0.08**	1.54	**2.08**	39.56

(FIGURE 10) Lateral Seismic Load Distribution in the East-West Direction

Y-Direction Loading (North-South) [1.6 * **WIND**]																
Floor	Height (ft)	Total Factored Load (k)	**SW-1**		**SW-3**		**SW-6**		**SW-8**		**SW-10**		**SW-12**		**SW-14**	
			Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment	Story Force (k)	Overturning Moment
Upper Roof	105.67	3.31	0.46	48.48	0.01	0.79	1.41	149.06	0.22	22.94	0.26	27.72	0.74	77.85	0.22	22.94
Roof	93.67	56.03	7.77	727.43	0.13	11.92	23.88	2236.62	3.67	344.18	4.44	415.89	12.47	1168.12	3.67	344.18
9th	84.33	109.92	15.24	1284.77	0.25	21.05	46.84	3950.29	7.21	607.89	8.71	734.54	24.46	2063.12	7.21	607.89
8th	75	107.78	14.94	1120.38	0.24	18.36	45.93	3444.85	7.07	530.11	8.54	640.56	23.99	1799.14	7.07	530.11
7th	65.67	104.93	12.45	817.79	17.14	1125.79	42.81	2811.19	4.99	327.46	4.96	325.55	17.60	1155.52	4.99	327.46
6th	56.33	102.79	12.20	687.17	16.79	945.98	41.93	2362.19	4.88	275.16	4.86	273.55	17.24	970.96	4.88	275.16
5th	47	99.94	11.86	557.46	16.33	767.41	40.77	1916.29	4.75	223.22	4.72	221.91	16.76	787.68	4.75	223.22
4th	37.67	96.36	11.44	430.79	15.74	593.04	39.31	1480.87	4.58	172.50	4.55	171.49	16.16	608.70	4.58	172.50
3rd	28.33	92.08	10.93	309.59	15.04	426.19	37.57	1064.23	4.38	123.97	4.35	123.24	15.44	437.44	4.38	123.97
2nd	19	131.02	15.55	295.44	21.41	406.71	53.45	1015.58	6.23	118.30	6.19	117.61	21.97	417.45	6.23	118.30

(FIGURE 11) Lateral Wind Load Distribution in the North-South Direction

TORSION

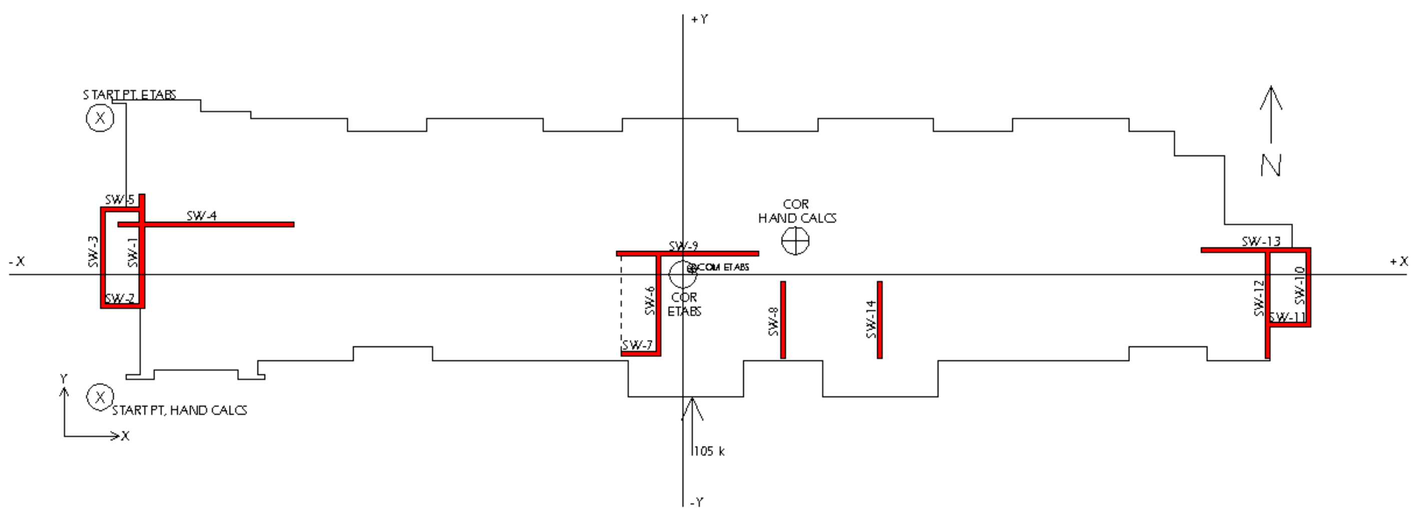
For most designers, ideal conditions occur when a building can take advantage of some form of symmetry, both geometrically and in terms of relative stiffness. The reason for this is that torsion issues are not present when the center of mass of the structure coincides with the center of rigidity (COR). Any offset that exists between these two locations presents a torsion moment (a force acting at an eccentricity) on the lateral force resisting systems. If large enough, this may introduce significant out-of-plane shears that the designer must account for.

Based on the relative stiffness coefficients determined in the previous section, the center of rigidity was located. See Appendix C for detailed calculations. The coordinate obtained was then compared with the COR calculated by ETABS. The y-component was found to be almost exactly the same as ETABS. However, the calculated x-component is significantly different than the ETABS result. The ninth floor calculation yielded expected results, moving the center of rigidity away from the more flexible west end of the building. This flexibility must play a role in the result because when the center of rigidity was then calculated at the fourth floor, a closer value was obtained. Differences in these values are likely due to the oversimplification of the shape of diaphragms used in the computer model, causing the center of mass to be slightly different than its true value. Also, the computer model will need to be refined to ensure that the assumptions and behavior as modeled reflect the designer's intent. This is especially true in specialized areas, like that of the West stairwell, and at the elevator shafts, where reinforced concrete beams are located.

For sake of calculation, it was assumed that the ETABS center of rigidity location was correct. Resultant net forces were obtained by combining the effects of torsion with the direct forces. The figure below illustrates the insignificance of torsion in the case of seismic loading, with a maximum influence of a mere three percent of the total net force on a single shear wall. Floor 7 was chosen for a spot check analysis to prove this in the figure below. It is assumed that if torsion forces are minimal even with one of the largest lateral loads acting at an eccentricity (wind in the N-S direction, seismic in the E-W direction), they are even less critical for other less severe loading conditions.

Center of Mass/Rigidity	X	Y	X	Y
Story	COM	COM	COR	COR
9TH FLR	1564	397	1530	414
9TH FLR CALCULATED	-	-	1837	413
4TH FLR	1564	397	1536	410
4TH FLR CALCULATED	-	-	1483	418

(FIGURE 12) Center of Mass / Center of Rigidity Calculated vs. ETABS



(FIGURE 13) Center of Mass / Center of Rigidity Calculated vs. ETABS Diagram

Torsional Moments & Forces in Shear Walls							
Shear Wall	k	d_i (ft)	$k_i*d_i^2$	Torsional Force (k)	Direct Force @ Floor 7 (k)	Net Force (k)	% Of Net Force Due to Torsion
SW-2	0.00	-7	0.24	0.00	0	0.00	100.00
SW-4	0.57	11	69.05	0.16	33.06	33.22	0.48
SW-5	0.00	14.42	0.77	0.00	0.21	0.21	0.64
SW-7	0.01	-17.33	1.59	0.00	0.31	0.31	0.76
SW-9	0.29	4.58	6.02	0.03	16.63	16.66	0.20
SW-11	0.00	-11	0.58	0.00	0.28	0.28	0.48
SW-13	0.12	5.33	3.51	0.02	7.17	7.19	0.23
SW-1	0.12	-119	1680.91	-0.36	12.45	12.09	2.98
SW-3	0.16	-127.83	2670.04	-0.53	17.14	16.61	3.21
SW-6	0.41	-5.33	11.59	-0.06	42.81	42.75	0.13
SW-8	0.05	22	22.99	0.03	4.99	5.02	0.53
SW-10	0.05	137.67	894.58	0.17	4.96	5.13	3.23
SW-12	0.17	128.83	2783.35	0.55	17.6	18.15	3.04
SW-14	0.05	43.5	89.88	0.05	4.99	5.04	1.04
		SUM	8235.11				

(FIGURE 14) Torsion Force Summary

STORY DRIFT & BUILDING DEFLECTIONS

Confidence so far in using the ETABS model built for this project led to using output generated by the program for story drift. The maximum story drift occurs as a result of the North-South wind. This is logical since the building is less stiff in this direction. Service loads (without load factors) were applied and the following story drift ratios were extracted from the output:

Story Drift & Building Deflection					
Story	Max Story Drift Ratio	Max Story Drift (in.)	Acceptable Story Drift - Seismic (in.)	Building Deflection (in.)	Acceptable Building Deflection (h/400) (in.)
Upper Roof	0.000464	0.0668	2.88	0.478	3.17
Roof	0.000462	0.0527	2.28	0.411	2.81
9th	0.000480	0.0547	2.28	0.358	2.53
8th	0.000456	0.0520	2.28	0.303	2.25
7th	0.000447	0.0510	2.28	0.251	1.97
6th	0.000428	0.0488	2.28	0.201	1.69
5th	0.000397	0.0453	2.28	0.152	1.41
4th	0.000351	0.0400	2.28	0.106	1.13
3rd	0.000287	0.0327	2.28	0.066	0.85
2nd	0.000148	0.0337	4.56	0.034	0.57

(FIGURE 15) Story Drift & Building Deflection Summary

By multiplying the story drift ratio at each story by the story height, the relative story displacements, otherwise known as story drifts, were found. As can be seen in the figure above, all of these values are well within the accepted limits for story drift based on seismic criteria. A summation of individual story drifts reveals overall building deflections at each level, the maximum of which is just under a half inch and occurs at the roof level. The overall building deflection values are also well within the acceptable deflection criteria under wind loading for meeting serviceability needs. Because I initially questioned the low story drift values, a short hand calculation of Shear Wall 14 was performed to verify the deflection at the roof. This particular wall was chosen because it does not intersect other shear walls, thus there would be less chance for it to be taking significant amounts of out-of-plane shear. The resulting 0.30" max deflection was comparable to the 0.358" – 0.411" values obtained in the figure above. See Appendix C for the actual calculation. The Residence Inn does not appear to have any significant serviceability issues related to deflections under normal loading.

OVERTURNING MOMENT & FOUNDATIONS

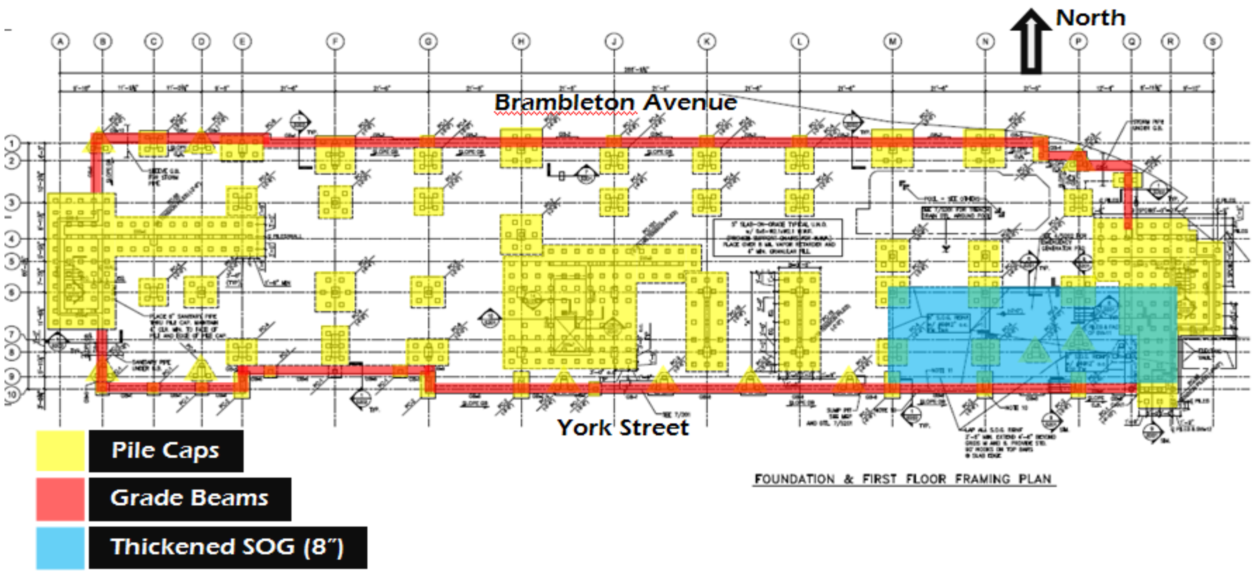
A simplified analysis of the impact of overturning moments on the originally designed foundations was performed. Factored critical moments in both the E-W and N-S directions were divided by the moment arm to determine the coupling force acting on the foundation. Since the building has a self-weight that is more than fifteen times the more critical uplift from the N-S wind loading condition, uplift does not present itself as an issue with this building. Furthermore, the precast concrete piles that form the foundations for the Residence Inn inherently are capable of resisting up to 70 kips of uplift each. See Appendix C for calculations involving uplifting forces on foundation systems.

CONCLUSIONS

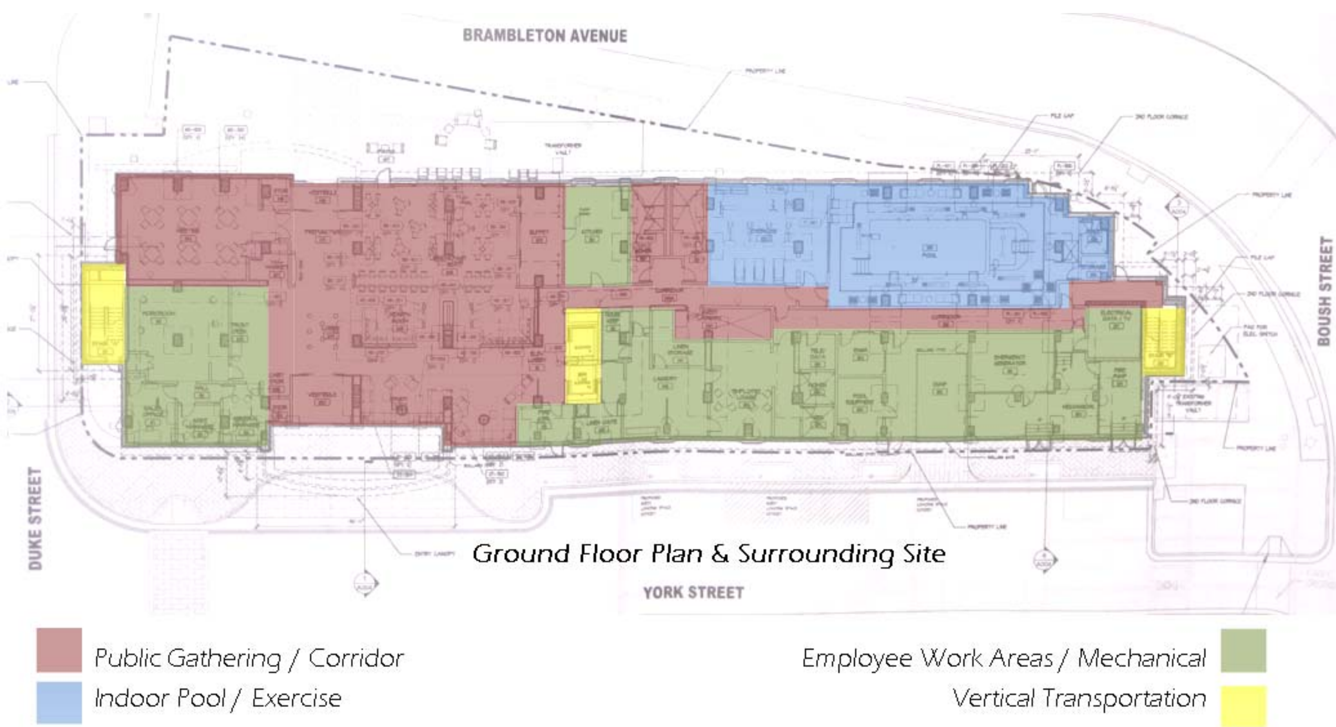
This thorough study of the lateral loads and how they are distributed amongst the fourteen shear walls throughout the building has been very informative. At first, it was not apparent to me the logic in the orientation of the walls; now, however, I can begin to understand how important it is to strategically locate these systems to extricate their full advantage. Applying load factors and combinations helped to clarify the controlling wind loads in the N-S direction and seismic loads in the E-W direction. After distributing these loads according to relative stiffness of the shear walls, it was apparent what properties are most critical to the stiffness and how this stiffness affects torsion. It is important to locate lateral force resisting elements in an arrangement that gives stiffness to the structure in a symmetrical fashion, if at all possible. Although torsion forces were not significant for this building, it was quite obvious how an increased eccentricity might cause undesired torsional shear.

Computer modeling as part of this exercise has been very helpful; however, understanding the assumptions that go into such models is critical for success. I was able to perform multiple hand calculation checks to be sure that I had not inadvertently misrepresented the building, which was reassuring, especially for a computer modeling novice. An additional calculation to determine the adequacy of one of the shear walls is included in Appendix C, confirming agreement among designer and the models and calculations performed here.

APPENDIX A: Additional Figures



(FIGURE 16) Foundation Plan



(FIGURE 17) Ground Floor Plan

Guest Suites

S Studio

1 1-Bedroom

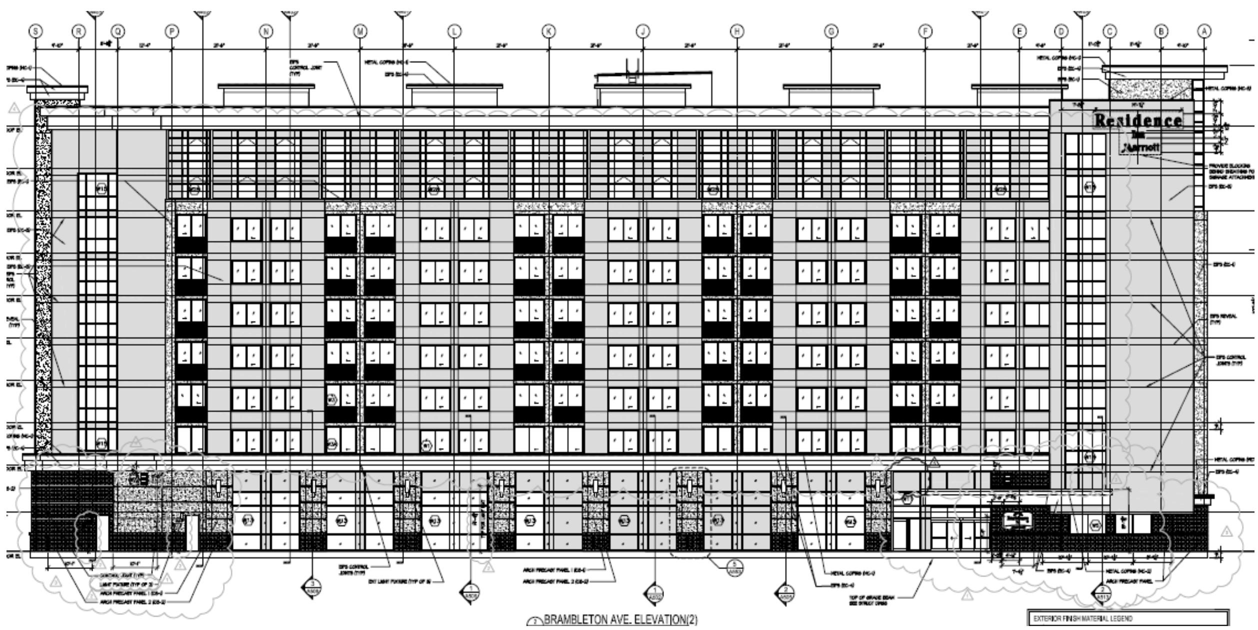
2 2-Bedroom

Employee Work Areas / Mechanical

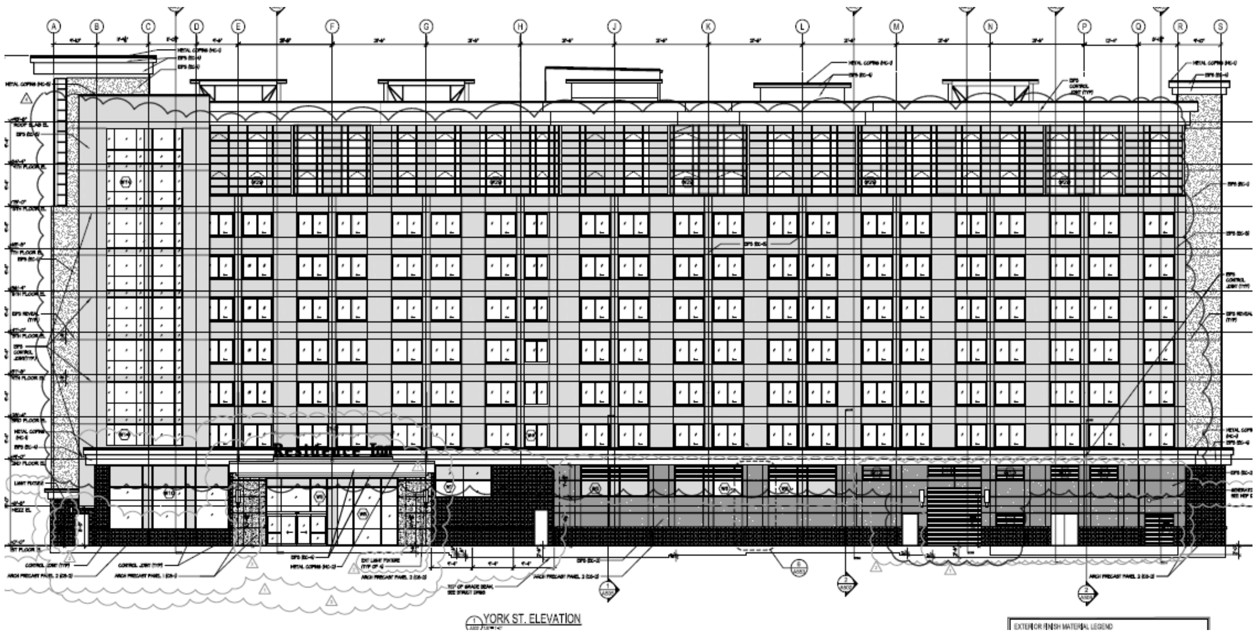
Vertical Transportation

Corridor

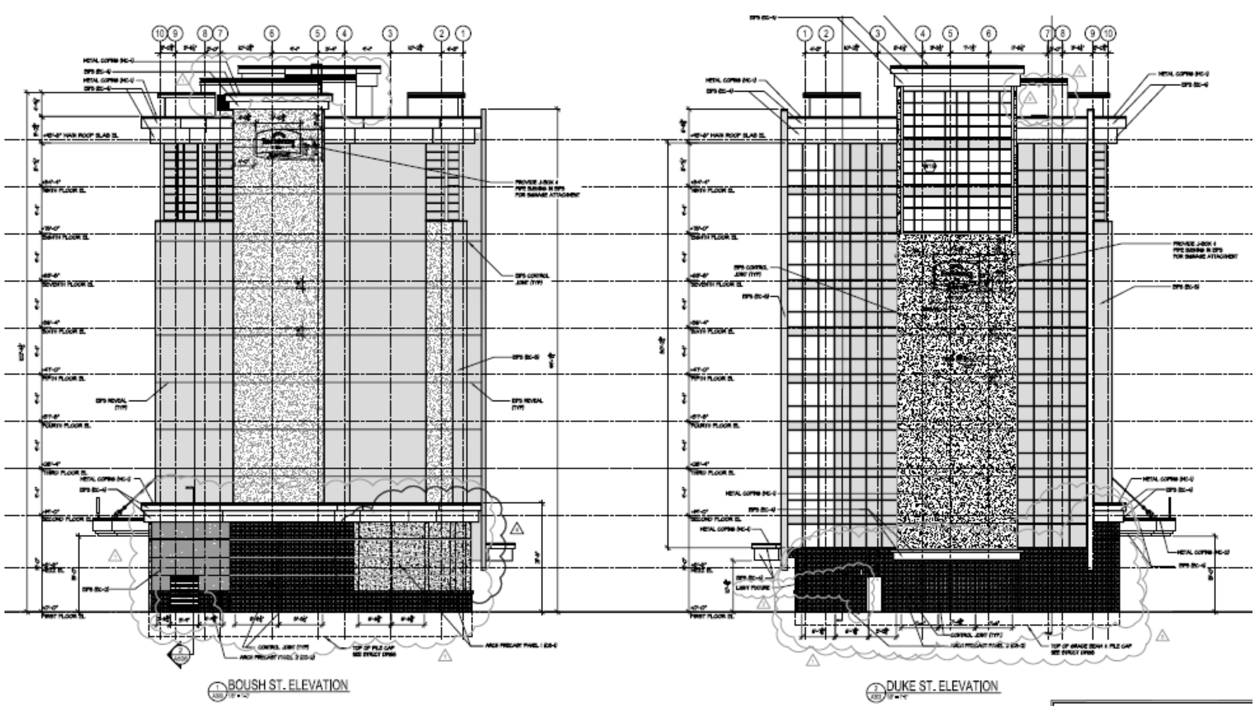
(FIGURE 18) Typical Upper Floor Plan



(FIGURE 19) Brambleton Ave. (North) Elevation



(FIGURE 20) York St. (South) Elevation



(FIGURE 21) Boush St. (East) & Duke St. (West) Elevations

APPENDIX B: Gravity Loads

GRAVITY LOADS (psf)				
Location	Design Dead Load	Assumed Dead Load	Design Live Load	IBC 2006 Live Load
Typical Floors Incl. Corridors Serving them	10	15	40 + 10 (partitions)	40 + 15 (partitions)
Mechanical Mezzanine	10	25	150	40
Roof	25	30	30	20 + 46 (Snow Drift Surcharge only where necessary near parapet)
Canopies	N/A	10	75	20 + 10 (Snow) + 30 (Snow Drift Surcharge) = 60
Lobbies, All Floors / Public Rooms	10	15	100	100

(FIGURE 22) Gravity Loads

APPENDIX C: CALCULATIONS

ETABS Mass Definitions	Tech 3 Residence Inn	Katie Ritter	1/1

FOR TYP. FLRS & ROOF:

Slab Weight $= \frac{8''}{12}(150\text{ PCF}) = 100\text{ PSF}$

SDL 10 PSF

CONC. COLS. TYP. 14" × 30"
9'-4" HT

58 COLS / FLR
14,376 SF / FLR

$$\frac{58\left(\frac{14''\times 30''}{144\text{ in}^2/\text{ft}^2}\right)(9.33')(150\text{ PCF})}{14{,}376\text{ SF}} = 16.5\text{ PSF}$$

$126.5\text{ PSF} = 0.127\text{ KSF}$

$$\frac{\frac{0.127\text{ KSF}}{32.2}}{12^3} = \boxed{2.282\times 10^{-6}\ \frac{\text{MASS}}{\text{AREA}}}$$

FOR STAIRWELL / PH ROOFS:

$$\frac{\frac{0.100\text{ PSF}}{32.2}}{12^3} = \boxed{1.797\times 10^{-6}\ \frac{\text{MASS}}{\text{AREA}}}$$

(FIGURE 23) ETABS Mass Definitions [1/1]

PERMISSIBLE SEISMIC DRIFT | RESIDENCE INN | KMR | 1/1

OCCUPANCY CATEGORY → II

↳ TBL. 12.12-1

↳ All other Structures → $\Delta_{allow} = 0.020 h_{sx}$

→ For Typical Flr → $h = 9.5' = 114"(0.020) = 2.28"/\text{STORY}$

(FIGURE 24) ASCE 7-05 Permissible Seismic Drift [1/1]

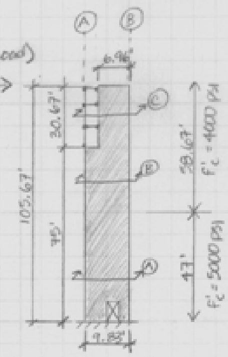
SHEAR WALLS – RELATIVE STIFFNESS | RESIDENCE INN | KMR | 1/6

SW-2 (X-DIRECT.)
& SW-5

Shearwall thickness : 1'-0"

Stl. Frame : HSS 6×6×½

Assumption :
opening negligible



$$K = \frac{AE}{L} = \frac{P}{\delta}$$

$$\delta_{CANT} = \frac{Ph^3}{3EI} + \frac{1.2Ph}{E_r A}$$

$$(\Delta_f) + (\Delta_s)$$

$(E = 57000\sqrt{f'_c})$
$(E_r = 0.4E)$

→ VARYING PROPERTIES:

SECT. A $\left[\frac{47'}{105.67'} = 44\% \text{ OF HT}\right]$ 118" 12"

$I = \frac{(12)(118)^3}{12} = 1{,}643{,}032 \text{ in}^4$

$A = (12)(118) = 1{,}416 \text{ in}^2$

$E = 57000\sqrt{5000} = 4.03\times10^6 \text{ psi}$

$E_r = 0.4(4.03\times10^6) = 1.61\times10^6 \text{ psi}$

SECT. B $\left[\frac{28'}{105.67'} = 26\% \text{ OF HT}\right]$ 118" 12"

$I = 1{,}643{,}032 \text{ in}^4$

$A = 1{,}416 \text{ in}^2$

$E = 57000\sqrt{4000} = 3.60\times10^6 \text{ psi}$

$E_r = 0.4(3.60\times10^6) = 1.44\times10^6 \text{ psi}$

SECT C [30% OF HT] 75.54" 34.5" 83.5" 12"

HSS6×6×½
A = 9.74 in²
I = 48.3 in⁴
PER AISC STL MANUAL

$$\bar{x} = \frac{9.74(3) + (83.5)(12)(76.25)}{9.74 + (83.5\times12)} = 75.54"$$

$$I = 48.3 + 9.74(72.54)^2 + \frac{(12)(83.5)^3}{12} + (83.5)(12)(0.71)^2$$

$$I = 633{,}989 \text{ in}^4$$

(FIGURE 25) Relative Stiffness of Shear Walls [1/6]

SHEAR WALLS – RELATIVE STIFFNESS | RESIDENCE INN | KMR | 2/6

SW-2 CONT'D
& SW-5

.... SECT. C CONT'D

~1% OF AREA ~99% OF AREA

$$A = 9.74 + (83.5 \times 12) = 1011.74 \text{ in}^2$$

$$E_{avg,c} = 0.01\,(29{,}000{,}000 \text{ psi}) + 0.99\,(3.60 \times 10^6 \text{ psi})$$

$$= 3.85 \times 10^6 \text{ psi}$$

$$E_{r,avg,c} = 0.4\,(3.85 \times 10^6) = 1.54 \times 10^6 \text{ psi}$$

$$I_{avg} = 0.44\,(1{,}643{,}032 \text{ in}^4) + 0.26\,(1{,}643{,}032) + 0.3\,(633{,}989)$$

$$= 1{,}340{,}319 \text{ in}^4$$

$$E_{avg} = 0.44\,(4.03 \times 10^6) + 0.26\,(3.60 \times 10^6) + 0.3\,(3.85 \times 10^6)$$

$$= 3.864 \times 10^6 \text{ psi}$$

$$E_{r,avg} = 0.44\,(1.61 \times 10^6) + 0.26\,(1.44 \times 10^6) + 0.3\,(1.54 \times 10^6)$$

$$= 1.545 \times 10^6 \text{ psi}$$

$$A_{avg} = 0.70\,(1416) + 0.3\,(1011.74) = 1295 \text{ in}^2$$

$$\delta_{CANT,\,SW\text{-}2,\,SW\text{-}5} = \frac{100^k\,(1000\,\tfrac{lb}{kip})\,(1268'')^3}{3\,(3.864 \times 10^6 \text{ psi})\,(1{,}340{,}319 \text{ in}^4)} + \frac{1.2\,(100{,}000^{\#})\,(1268'')}{(1.545 \times 10^6 \text{ psi})\,(1{,}295 \text{ in}^2)}$$

$$13.12'' + 0.076''$$

$$= \boxed{13.2''}$$

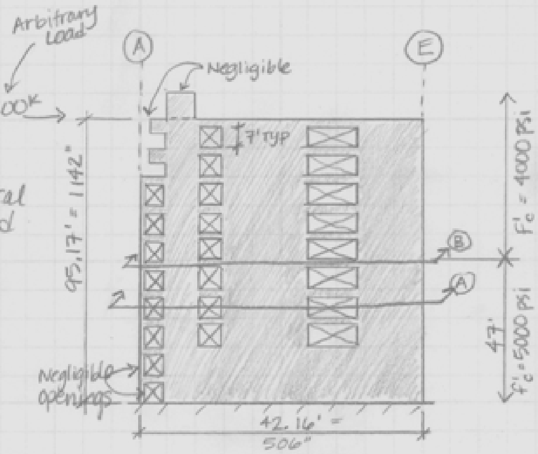
$$K = \frac{P}{\delta} = \frac{100^k}{13.2''} = \boxed{7.578}$$

(FIGURE 26) Relative Stiffness of Shear Walls [2/6]

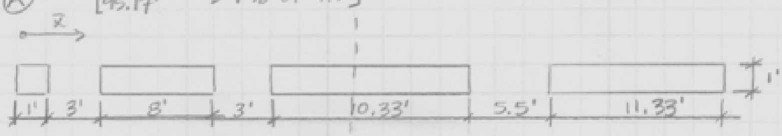
SW-4 (X-DIRECT.)

Shearwall thickness: 1'-0"

Assumption:
I calculated for one typical section w/ 3 openings & solid wall case



SECT (A) $\left[\frac{56'}{95.17'} \sim 59\% \text{ OF HT}\right]$



$$\bar{x} = \frac{(12\times12\times6)+(96\times12\times96)+(124\times12\times242)+(136\times12\times438)}{(12\times12)+(96\times12)+(124\times12)+(136\times12)}$$

$$= \frac{1186368}{4416} = 268.7"$$

$$I_{openings} = \frac{(12)(12)^3}{12} + (12\times12)(268.7-6)^2 = 9,939,354 \text{ in}^4$$

$$+ \frac{(12)(96)^3}{12} + (12\times96)(268.7-96)^2 = 35,243,470$$

$$+ \frac{(12)(124)^3}{12} + (12\times124)(268.7-242)^2 = 2,967,404$$

$$+ \frac{(12)(136)^3}{12} + (12\times136)(438-268.7)^2 = 49,292,640$$

$$97,442,869 \text{ in}^4$$

SECT (B) [~41% OF HT]

$$I_{solid} = \frac{(12)(506")^3}{12} = 129,554,216 \text{ in}^4$$

$$I_{avg} = 0.59(97,442,869) + 0.41(129,554,216) = \underline{110608521 \text{ in}^4}$$

(FIGURE 27) Relative Stiffness of Shear Walls [3/6]

SW-4 CONT'D

$$E_{avg} = 0.49\left(57000\sqrt{5000}\right) + 0.51\left(57000\sqrt{4000}\right) = 3.813\times10^6\ psi$$

$$E_{r,avg} = 0.4\left(3.813\times10^6\right) = 1.525\times10^6\ psi$$

$$A_{avg} = 0.59\left(4416\ in^2\right) + 0.41\left(6072\right) = 5{,}095\ in^2$$

$$\delta_{CANT,\,SW-4} = \frac{100^k(1000)(1142)^3}{3(3.813\times10^6)(110{,}608{,}521)} + \frac{1.2(100000)(1142)}{(1.525\times10^6)(5095)}$$

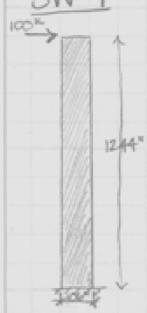
$$= 0.1177'' + 0.0176$$

$$= \boxed{0.135''}$$

$$K = \frac{P}{\delta} = \frac{100^k}{0.135''} = \boxed{738.894}$$

** → Shear deformations are minimal ∴ δ of remaining x-direct. shear walls can be determined using a ratio of moment of inertia.

SW-7



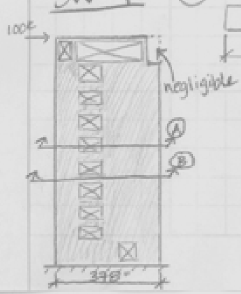
$$I_{SW-7} = \frac{(12)(106'')^3}{12} = 1{,}191{,}016\ in^4 \Big/ 110608521 = [0.0108]$$

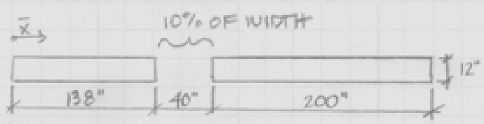
$$h = 1244''$$

$$\frac{1244^3}{1142^3} = \frac{1925134784}{1489355288} = 1.293 \qquad \frac{1}{1.293} = [0.7734]$$

$$\frac{3E\overset{\text{same}}{I}}{h^3} \rightarrow \underset{I_{factor}}{[0.0108]}\ \underset{h_{diff.\,factor}}{[0.7734]}\ (738.894) = \boxed{6.172} = K$$

SW-9 Ⓐ





$$\bar{x} = \frac{(138\times12)(69) + (200\times12)(278)}{(138+200)(12)} = 192.7''$$

$$I_{openings} = \frac{(12)(138)^3}{12} + (138\times12)(192.7-69)^2 + \frac{(12)(200)^3}{12} + (200\times12)(278-192.7)^2$$

$$I_{openings} = 53{,}443{,}129\ in^4 \quad [\sim 85\%\ \text{OF HT}]$$

(FIGURE 28) Relative Stiffness of Shear Walls [4/6]

SW-9 CONT'D

$I_{SOLID} = \frac{(12)(378)^3}{12} = 54{,}010{,}152 \text{ in}^4$ [~15% OF HT]
(SECT Ⓑ)

$I_{avg} = 0.85(53{,}443{,}129) + 0.15(54{,}010{,}152) = 53{,}639{,}880 \text{ in}^4$

(~0.07% diff.)

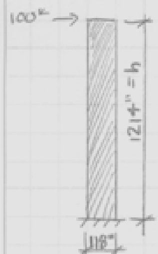
Based on how close this is to I_{solid} (w/ no openings accounted for) openings negligible.

$I_{factor\ w/SW-7} = \frac{53{,}639{,}880}{1{,}191{,}016} = [45.04]$

(h is same as SW-7)

$\rightarrow k = 45.04(6.172) = \boxed{277.969}$

SW-11



$I_{SW-11} = \frac{(12)(118)^3}{12} = 1{,}643{,}032 \Big/ 1{,}191{,}016 = [1.38]$

↑ SW-7

$h_{factor} = \frac{1}{\frac{1214^3}{1244^3}} = [1.076]$

$k_{SW-11} = [1.38][1.076](6.172) = \boxed{9.165}$

SW-13

→ ~18% opening as percentage of SW width compared w/ 10% SW-9

→ I will decrease I_{SW-13} by 2% to approximate the impact of larger openings on I_{avg}

$I_{SW-13} = \frac{(12)(290")^3}{12} = 24{,}389{,}000$

$-[2\%] = 487{,}780$

$I_{avg} \cong 23{,}901{,}220 \text{ in}^4$

$I_{factor\ w/SW-11} = \frac{23{,}901{,}220}{1{,}643{,}032} = [14.547]$

(all other variables same as SW-11)

$k_{SW-13} = [14.547](9.165) = \boxed{133.323}$

(FIGURE 29) Relative Stiffness of Shear Walls [5/6]

SHEAR WALLS — RELATIVE STIFFNESS | RESIDENCE INN | KMR | 6/6

SUMMARY OF SW STIFFNESSES IN X-DIRECT. (W/ 100^k ARB. LOAD)

	SW	REL. K	% OF TOTAL K	ETABS % BASED ON SHEAR FORCE	
W STAIR	SW-2	7.578	0.642	1.88	✓
	SW-4	738.894	62.582	50.432	*
	SW-5	7.578	0.642	0.288	✓
ELEV CORE	SW-7	6.172	0.523	0.739	✓
	SW-9	277.969	23.543	32.7	*
E STAIR	SW-11	9.165	0.776	1.151	✓
	SW-13	133.323	11.292	12.9	✓
	SUM =	1180.679	100% ✓		

ETABS TOTAL @ 9^{TH} FLR, X-DIR $= 1042.2^k$

~4.2% error < 5% OK

→ COMPARISON ABOVE OF ETABS W/ HAND CALCS CONFIRMS METHOD FOR EXTRACTING REL. STIFFNESSES USING ETABS MODEL ↳ see spreadsheet

(FIGURE 30) Relative Stiffness of Shear Walls [6/6]

CALC. OF CNTR. OF RIGIDITY | RESIDENCE INN | KMR | 1/2

[SEE FIGURE IN BODY OF REPORT FOR LOCATIONS x_i & y_i]

$$\bar{x} = \frac{\Sigma k_i x_i}{\Sigma k_i} = \frac{\underset{SW\text{-}3}{0(0)} + \underset{SW\text{-}1}{0.14(9.25')} + \underset{SW\text{-}6}{0.43(123')} + \underset{SW\text{-}8}{0.07(150.5')} + \underset{SW\text{-}14}{0.07(171.75')} + \underset{SW\text{-}12}{0.22(257.33')} + \underset{SW\text{-}10}{0.08(266')}}{(0.00 + 0.14 + 0.43 + 2(0.07) + 0.22 + 0.08)}$$

$$\bar{x} = \frac{154.6351}{1.01} = 153.10' = \underline{\underline{1837 \text{ in.}}}$$

$$\bar{y} = \frac{\underset{SW\text{-}4}{0.51(38')} + \underset{SW\text{-}2}{0.01(20')} + \underset{SW\text{-}5 \text{ negligible}}{0(\ \ )} + \underset{SW\text{-}7}{0.01(9.67')} + \underset{SW\text{-}9}{0.33(31.5')} + \underset{SW\text{-}11}{0.01(16')} + \underset{SW\text{-}13}{0.13(32.33')}}{(0.51 + 0.01 + 0.01 + 0.33 + 0.01 + 0.13)}$$

$$\bar{y} = \frac{34.4346}{1} = 34.43' = \underline{\underline{413''}}$$

* COR ABOVE IS FOR $\boxed{\text{9TH FLR}}$ B/C REL. K VALUES @ 9TH FLR USED HERE)

w/ STL. FRAME @ W. STAIR, COR TENDS TO EAST OF COM.

(FIGURE 31) Center of Rigidity [1/2]

CALC. OF CNTR. OF RIGIDITY | RESIDENCE INN | KMR | 2/2

@ FLR. 4 (TYPICAL FLR)

$$\bar{x} = \frac{0.12(9.25') + 0.17(0') + 0.43(123') + 0.05(150.5') + 0.05(266') + 0.18(257.33') + 0.05(171.75')}{(0.12 + 0.17 + 0.43 + 3(0.05) + 0.18)}$$

$$\bar{x} = 123.554' = \underline{\underline{1483''}}$$

$$\bar{y} = \frac{0.01(20') + 0.63(38') + 0.01(9.67') + 0.32(31.5') + 0.01(16') + 0.14(32.33')}{0.01 + 0.63 + 0.01 + 0.32 + 0.01 + 0.14}$$

$$\bar{y} = 34.824' = \underline{\underline{418''}}$$

@ FLR 7 → story load ≅ 105^k (more critical than 96^k @ story 4 & both have comparable rigidities)

more critical in x dimension

$$M = 105^k (2') = 210'^k$$

Forces caused by Torsional Moment

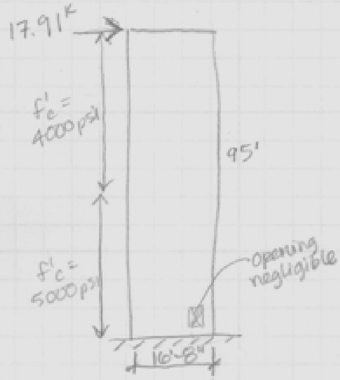
$$F_{iT} = \frac{k_i d_i}{\Sigma k_i d_i^2} \times M$$

↳ SEE SPREADSHEET FOR CALCS OF EA SW.

(FIGURE 32) Center of Rigidity [2/2]

DEFLECTION | RESIDENCE INN | KMR | 1/1

SPOT CHECK OF DEFL. IN SW-14



$$\frac{2723'^{k}}{1.6} = 1{,}701.74'^{k}$$

$$\frac{1701.74}{95'} = 17.91^{k}$$

FROM SW-4 CALC → $E_{avg} = 3.813 \times 10^6$ PSI

$E_{r,avg} = 1.525 \times 10^6$ PSI

$$A = 16.67'(12)(12) = 2400.5 \text{ in}^2$$

$$I = \frac{(12)(200")^3}{12} = 8{,}000{,}000 \text{ in}^4$$

$$\delta_{cant} = \frac{17.91^k(1000)(95\times12)^3}{3(3.813\times10^6 \text{ PSI})(8000000)} + \frac{1.2(17910^{\#})(95\times12)}{1.525\times10^6 \text{PSI}(2400.5)}$$

$$0.29" + 0.0067"$$

$\delta = \boxed{0.30"}$ ≈ compare w/ 0.411" TOTAL BLDG. DEFL. PER ETABS

∴ Δ. values found by ETABS in ballpark :)

(FIGURE 33) Deflection Check [1/1]

SHEAR WALL MEMBER CHECK | RESIDENCE INN | KMR | 1/2

SW-14 @ STORY 4

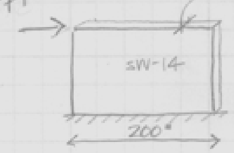
$f'_c = 5000$ psi

$E = 4.03 \times 10^6$ PSI

$I = 8{,}000{,}000$ in^4

$A = 2400.5$ in^2

$V_u = 32.79^k$ (FACTORED STORY SHEAR @ 5TH FLR SLAB)



$a = \frac{200"}{2} = 100"$ OR $a = \frac{114"}{2} = 57"$ ← governs

$d = 0.8 \ell_w = 0.8(200") = 160"$

MAX PERMISSABLE SHEAR STRENGTH:

$V_u \le \phi V_n = 0.75\left(10\sqrt{5000}\,(12")(160")\right)/1000 = 1{,}018^k$

$V_u = 32.79^k < \phi V_n = 1{,}018^k \therefore$ OK

SHEAR STRENGTH BY V_c (CONC)

→ USING SIMPLE EQN: $V_c = 2\sqrt{5000}\,(12)(160")/1000 = 271.5^k$

REQ'D HORIZ. SHEAR REINF.

IS $V_u = 32.79^k \overset{?}{>} 0.5(0.75(271.5^k)) = 101.8^k$

NO ∴ PROVIDE SHEAR REINF. PER CH. 14 [ACI 318-08]

PER CH. 14: $\rho_{vert,min} = 0.0012 \rightarrow A_{s,min\,vert} = 0.0012(2400.5\,in^2)$

$A_{s,min,vert} = 2.88\ in^2$

$\rho_{horiz.,min} = 0.0025 \rightarrow A_{s,min\,horiz} = 0.0025(2400.5\,in^2)$

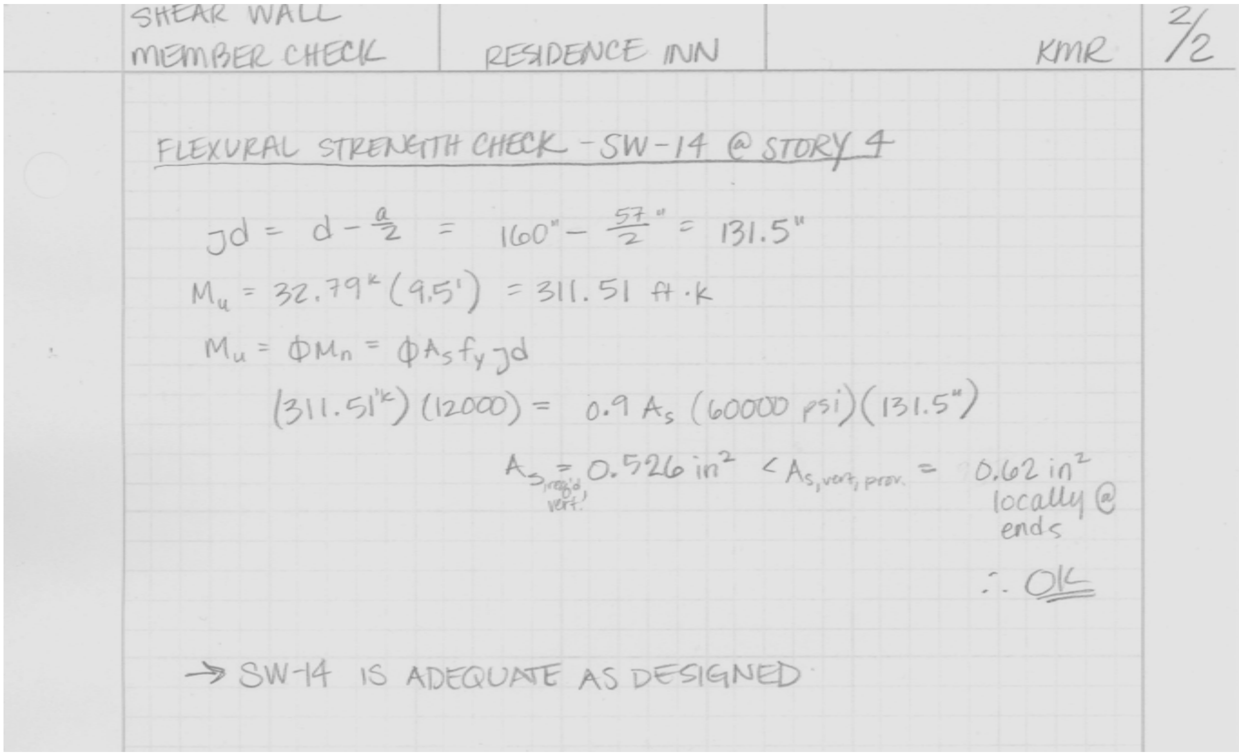
$A_{s,min,horiz.} = 6.00\ in^2$

$A_{s,prov.}$ = #5 @ 12" EF, EW → $0.31 \times \underset{\text{2 faces}}{2} = 0.62\ in^2/12"$

$A_{s,vert,prov.} = 16(0.62\ in^2) = 9.92\ in^2 \gg 2.88\ in^2 \therefore$ OK

$A_{s,HORIZ,prov.} = 10(0.62\ in^2) = 6.2\ in^2 > 6.0\ in^2 \therefore$ OK

(FIGURE 34) Shear Wall Member Check [1/2]



SHEAR WALL MEMBER CHECK	RESIDENCE INN		KMR	2/2

FLEXURAL STRENGTH CHECK - SW-14 @ STORY 4

$jd = d - \frac{a}{2} = 160'' - \frac{57''}{2} = 131.5''$

$M_u = 32.79^k (9.5') = 311.51 \text{ ft}\cdot\text{k}$

$M_u = \phi M_n = \phi A_s f_y jd$

$(311.51'^k)(12000) = 0.9 A_s (60000 \text{ psi})(131.5'')$

$A_{s,req'd,vert.} = 0.526 \text{ in}^2 < A_{s,vert,prov.} = 0.62 \text{ in}^2$ locally @ ends

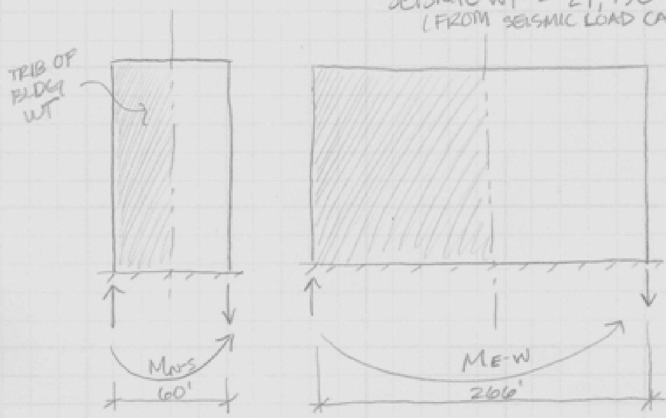
$\therefore$ OK

→ SW-14 IS ADEQUATE AS DESIGNED

(FIGURE 35) Shear Wall Member Check [2/2]

UPLIFT FORCES RESIDENCE INN KMR 1/1

SEISMIC WT = $24{,}730^k \sim \frac{1}{2} = \underline{\underline{12{,}365^k}}$
(FROM SEISMIC LOAD CALCS, TECH 1)



N-S $\quad 1.6(30{,}660.29'^k) = \dfrac{49{,}056.5'^k}{60'} = 818^k$

$\dfrac{12{,}365^k}{818^k} \approx 15^x$ GREATER $\therefore$ UPLIFT NOT ISSUE

E-W $\quad \dfrac{1.0(30{,}318.76'^k)}{266'} = 114^k$

$\dfrac{12{,}365^k}{114^k} \approx 108^x$ GREATER $\therefore$ NO CONCERN

(FIGURE 36) Uplift Force Check/Overturning Moment [1/1]