Katie Ritter
Structural Option

Advisor: Dr. Ali Memari
September 29, 2008

Technical Report 1

Existing Conditions / Structural Concepts

Residence Inn
By Marriott

Norfolk, Virginia

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EXECUTIVE SUMMARY

The Residence Inn by Marriott is a nine story hotel located in downtown Norfolk, Virginia. When it is delivered to its owner in January 2009, it will offer itself to the public as a modern upscale, yet comfortable, place to call home while away from home. Each of its 160 suites on floors 2-9 will feature all of the necessities for extended-stay guests, including separate living and sleeping areas, a fully equipped kitchenette, and even closet storage. The ground floor will serve a variety of functions. Guest features include an indoor pool and adjacent exercise room, coin laundry, as well as study areas and a private meeting room.

This report is intended to unveil and gain an understanding of this building's structural systems and the loads they must support, analyzing any differences that may exist between the calculations here and those of the original designer.

The Residence Inn is almost entirely structurally supported by reinforced concrete elements, including a two-way flat plate floor and roof system with concrete columns transferring gravity loads. 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 loads are resisted by reinforced concrete shear walls that are continuous throughout the height of the building. Due to the coastal soil conditions, the foundation consists of precast concrete piles driven to 70', cast-in-place concrete piles and grade beams.

Design codes used in this analysis differ only slightly from those employed by the designer, and in most cases had little effect on the overall results.

Using the most current codes and design standards, a typical floor dead load was found to be 15 psf, and had a live load of 40 psf (+15 psf for partitions). The roof's dead and live loads were determined to be 30 psf and 20 psf respectively. Snow drift on the roof along the parapet will need to be considered more extensively in future analyses.

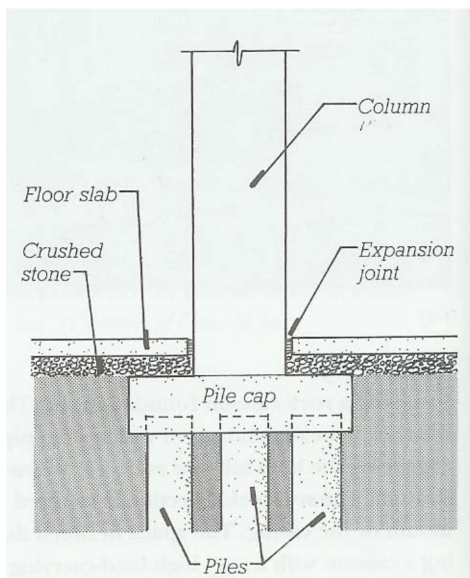
Wind pressures were calculated according to ASCE 7-05 and range between 15 and 23 psf on the windward side. The controlling lateral forces occurred in the North-South direction and resulted in a critical base shear of 569 k and an overturning moment of 30,660 ft-k. Wind loads were found to be more critical than seismic loads, with seismic loads resulting in 379 k of base shear and 25,778 ft-k of overturning moment. In general, the loads calculated here are comparable to the design loads, and where they differ is discussed in further detail within the report.

Spot checks were performed using the calculated dead and live loads and comparing the results with the design. The gravity load column check proved to be inaccurate due to its over-simplifying assumptions that did not include lateral loads, which caused the column to appear to be over-designed. An analysis of the two-way slab was also performed to verify adequate steel reinforcing in the design for the calculated loads, and the result was affirmative.

STRUCTURAL SYSTEMS

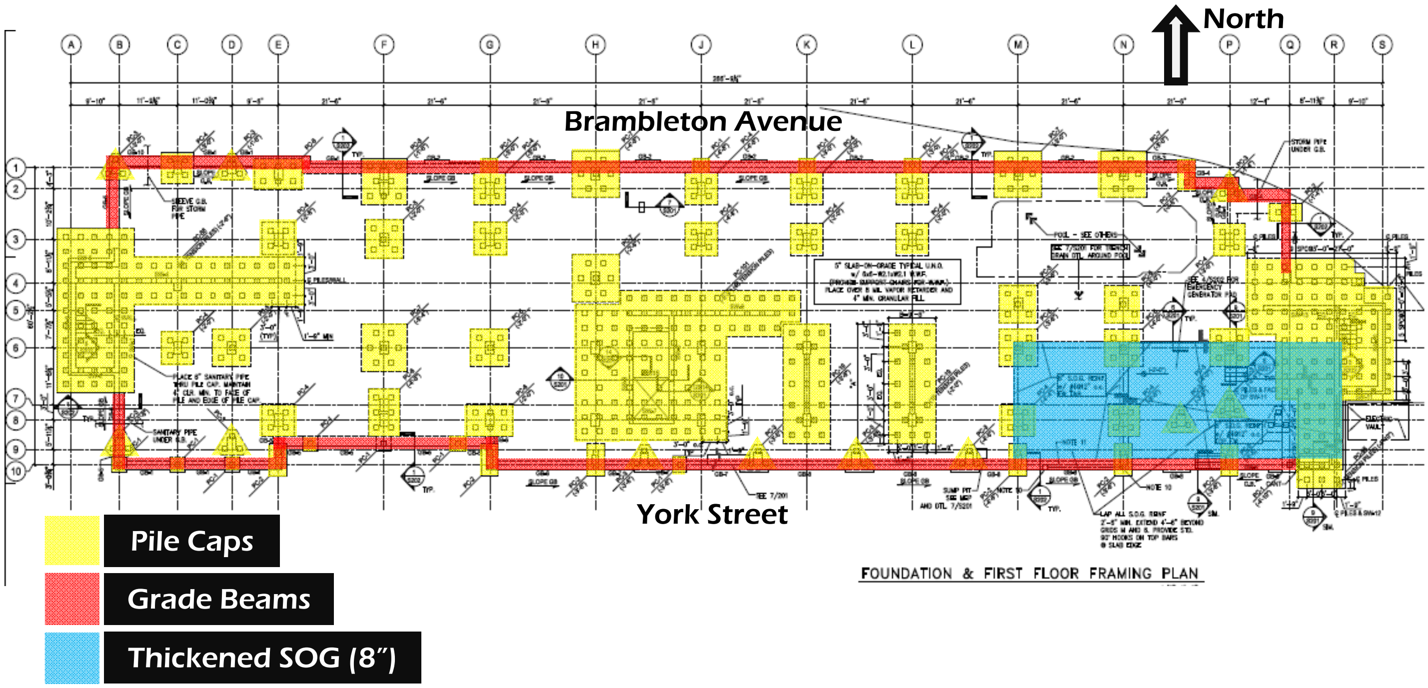
SOILS & FOUNDATIONS

Located in a coastal area, the Residence Inn site requires special attention to its foundation systems. Friction piles are necessary because of the high water table and lack of a firm bearing stratum. Due to the highly compressible soils found at the site by the geotechnical engineer, McCallum Testing Laboratories, the hotel utilizes high capacity (100 ton) 12" square precast, pre-stressed concrete piles, driven to depths between 60' and 70' (Figure 1). All piles are capable of resisting 5,000 psi in compression and up to 35 tons of uplift. Tendons are to be subjected to 700 psi of prestress. Clusters of piles are joined together by reinforced concrete pile caps (f'c=4,000psi), the largest of which are located in areas supporting shear walls above (Figures 2&3). Depths of pile caps range from 1'-4" at a perimeter column over 3 piles to 5'-8" over 46 tension piles at the shear walls near the elevator core at the center of the building.



(FIGURE 1) Foundation System: Concrete Piles & Pile Caps

A continuous reinforced concrete grade beam (f'c=4,000psi) ranging in size from 24"x24" to 24"x40" is utilized around the perimeter of the building to transfer loads from the walls into the piles (Figure 2). A 5" concrete slab on grade (f'c=3,500psi) with 6x6-W2.1xW2.1 welded wire fabric is typical of the first floor, except where additional support is required for mechanical and service areas. Here, an 8" concrete slab on grade (f'c=3,500psi) with #4@12" o.c. each way, top and bottom, is required (Figure 2).

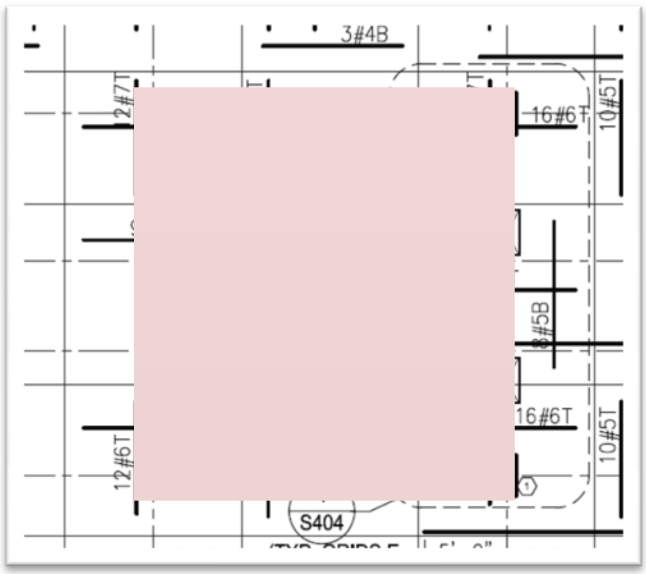


(FIGURE 2) Foundation & First Floor Framing Plan

(FIGURE 3) Foundations & 1st Floor Columns Under Construction

FLOOR SYSTEM

Like many hotels, the Residence Inn utilizes an economical 8" two-way flat-plate concrete floor system on all floors including the roof, with a typical bay spacing of 21'-6", and a maximum span of 22'-0". At the lower levels (third floor and below) 5,000 psi concrete is used for all slabs and beams; whereas, 4,000 psi concrete is reserved for use on the upper levels (fourth floor to the roof). Typical reinforcement consists of a bottom mat of #4@12" o.c. everywhere, and top reinforcement varies based on location (Figure 4).

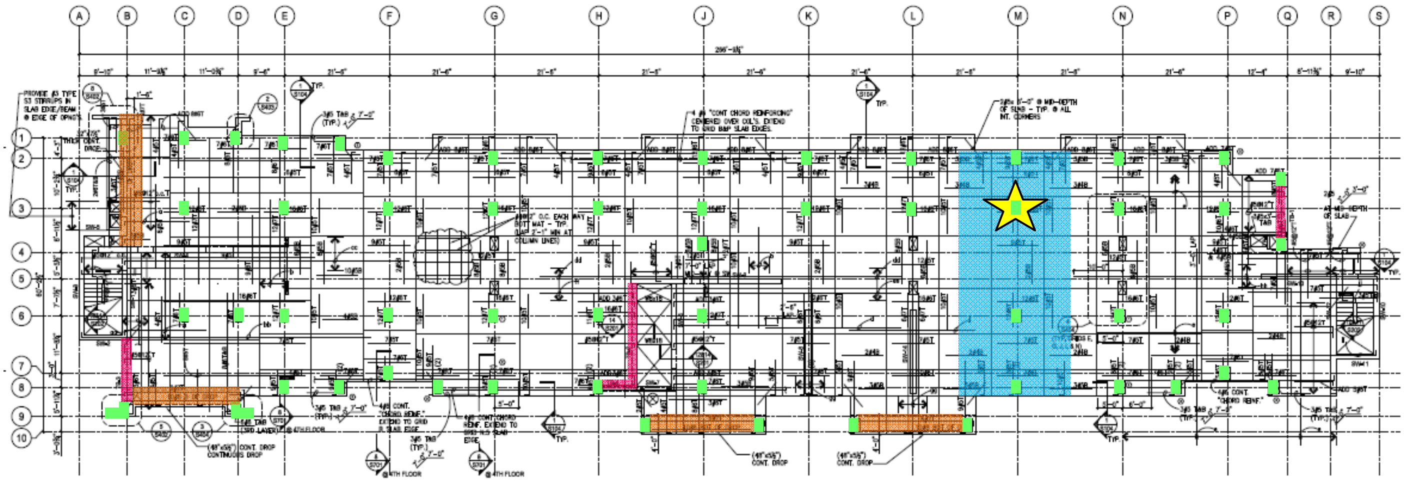


(FIGURE 4) Typical Bay 2-Way Flat Plate Floor & Roof Slab System

COLUMNS & BEAMS

Reinforced concrete columns, ranging in size from 12"x24" reinforced with (8)#8 bars on the upper floors to 20"x30" with (12) #5 bars at the first floor, support the two-way slab system. From the foundation level up to level five, compressive concrete strength is 5,000 psi, whereas levels five and above have a compressive strength of 4,000 psi. While the Residence Inn is primarily a flat-plate system, a few specific areas on each floor utilize reinforced concrete beams to support the slab near openings. These areas are highlighted in the typical floor plan shown below (Figure 5).

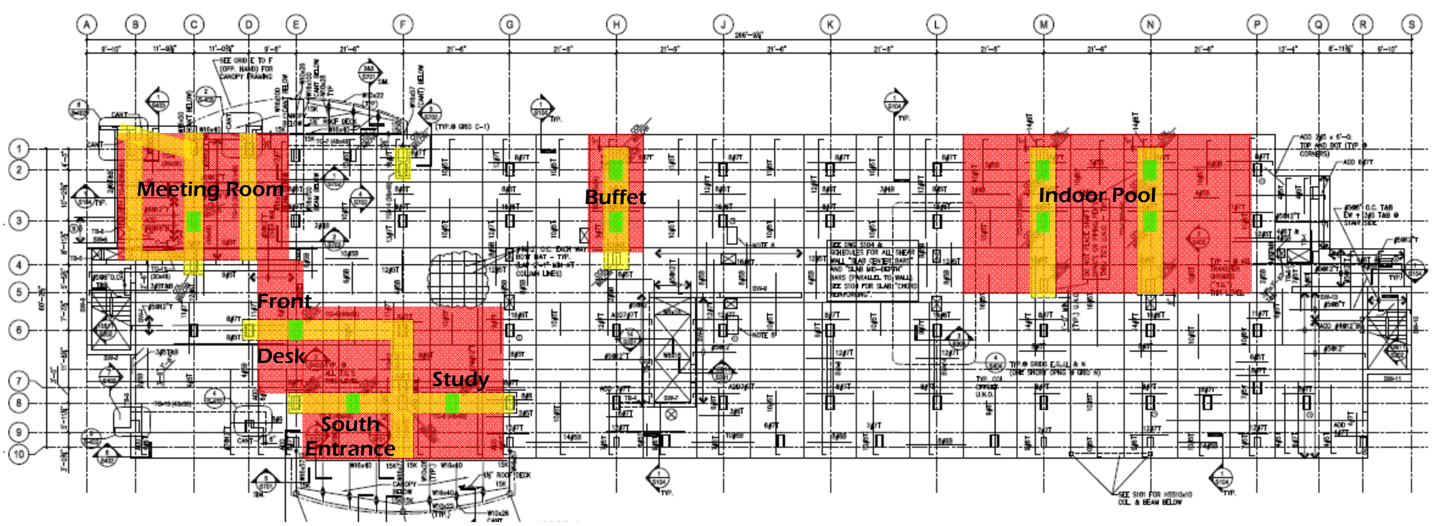
Along the exterior where the two-way slab ends at columns without a cantilever, drops are necessary to resist additional stresses due to the lack of structural continuity. These areas are also highlighted in the figure below (Figure 5).



(FIGURE 5) Typical Floor Plan

TRANSFER GIRDERS

At the second floor, reinforced concrete transfer girders are employed to discontinue columns on the first floor, where they are undesired near the open lobby, meeting room, breakfast buffet, and indoor pool areas (Figure 6). The sizes of these vary, the largest of which is 72" wide and 54" deep. The large depth of these girders is permissible since the first floor has a height of 19'-0". These girders can be seen under construction in (Figures 7 & 8) below.



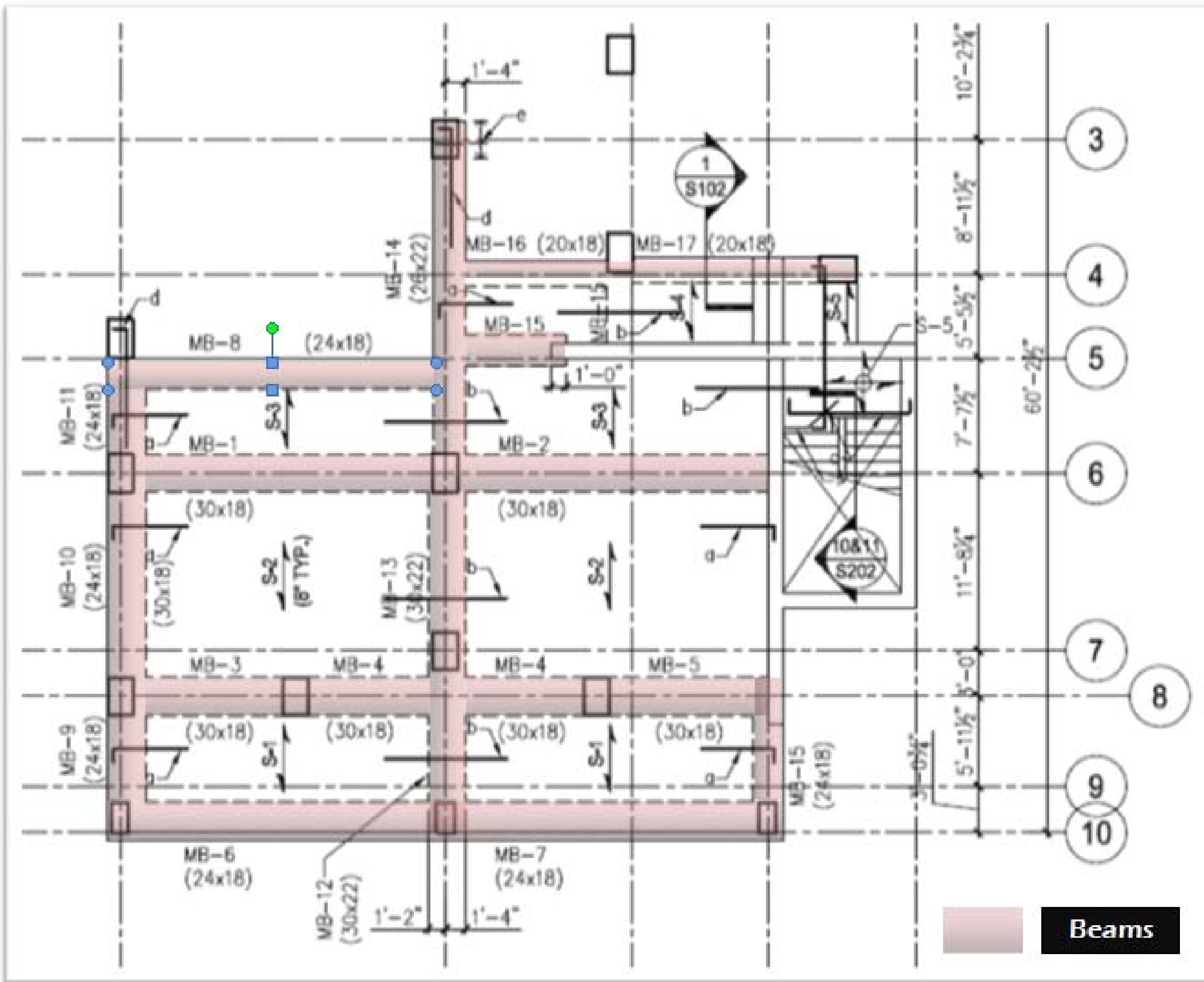
(FIGURE 6) 2nd Floor Framing Plan

(FIGURE 7) Transfer Girder Reinforcing

(FIGURE 8) West End 2nd Floor Reinforcing Prior to Concrete Placement

MECHANICAL MEZZANINE LEVEL

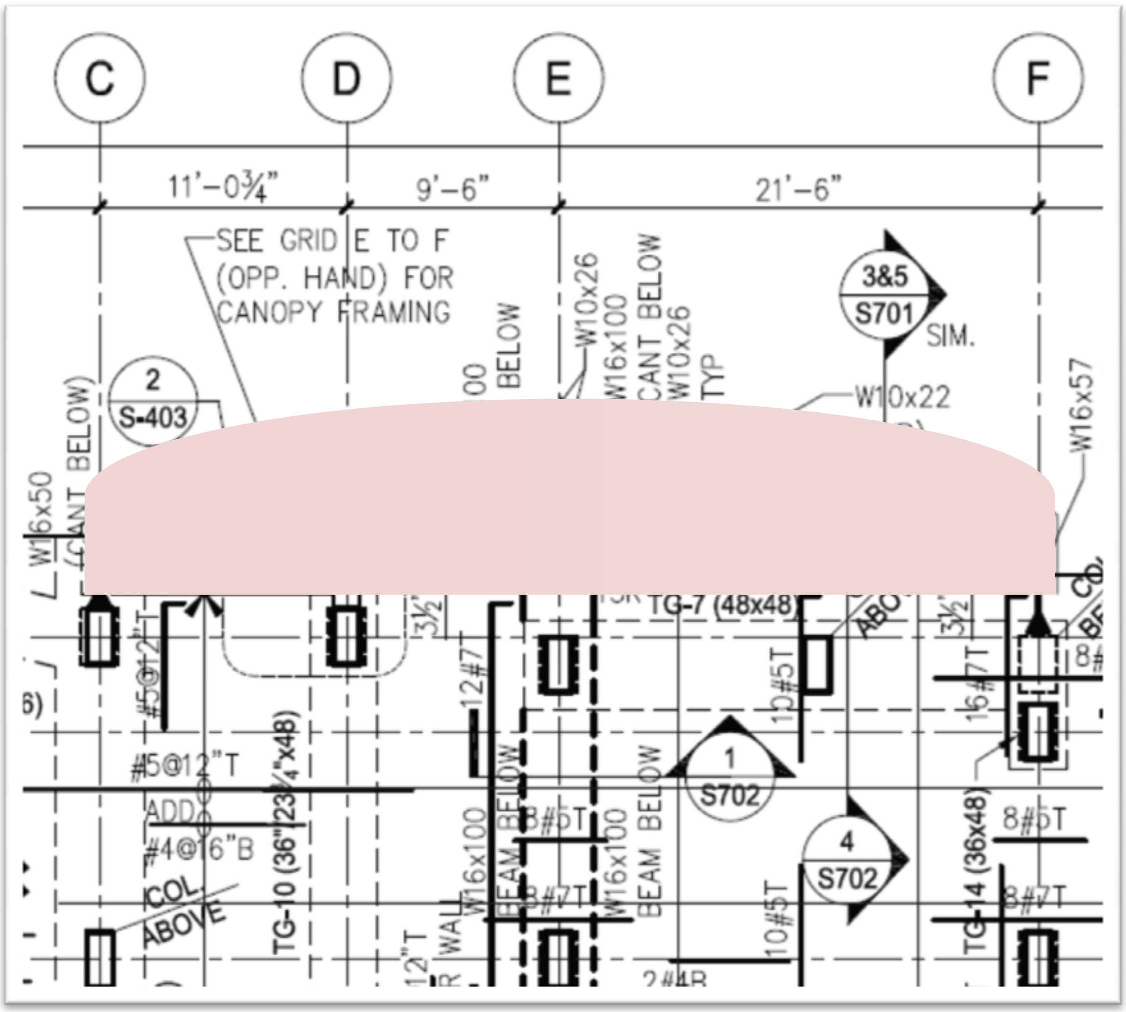
Located between the first and second floors, the mechanical mezzanine level provides additional floor area for mechanical equipment. Due to the heavier loads anticipated by such equipment, an 8" one-way flat plate floor system with beams is used here. The maximum span is 21'-6" between frames and 14'-8" in the direction of the one-way slab span. Reinforced concrete beams typically 18" deep support the slab and transfer loads into the columns (Figure 9).



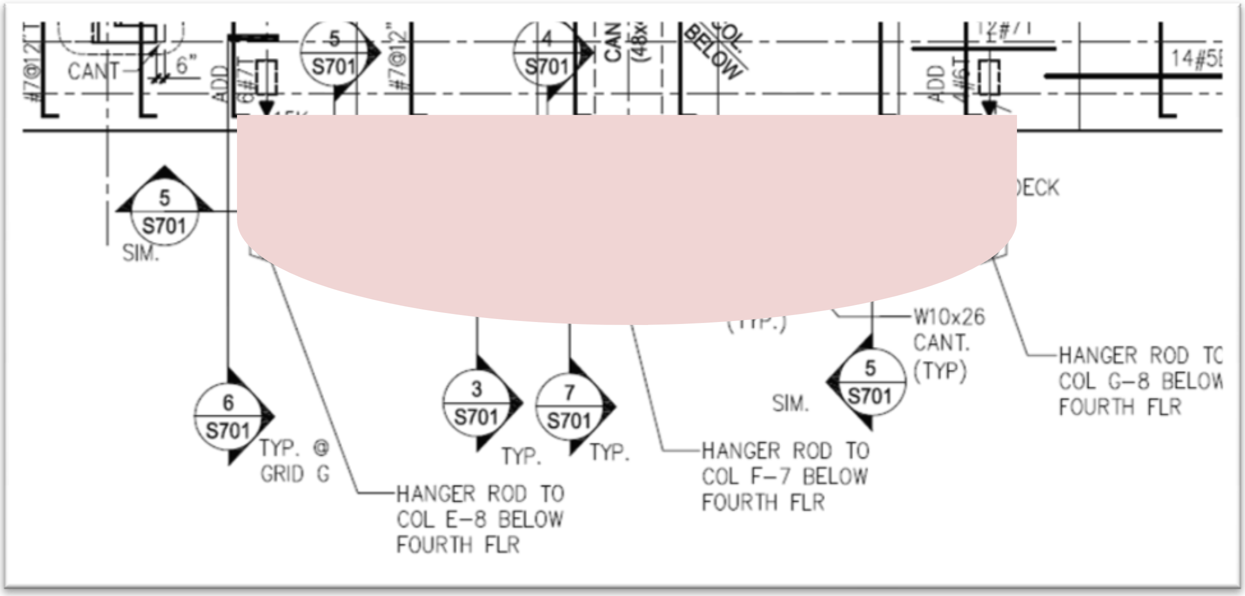
(FIGURE 9) Mechanical Mezzanine Floor Framing Plan

CANOPY FRAMING

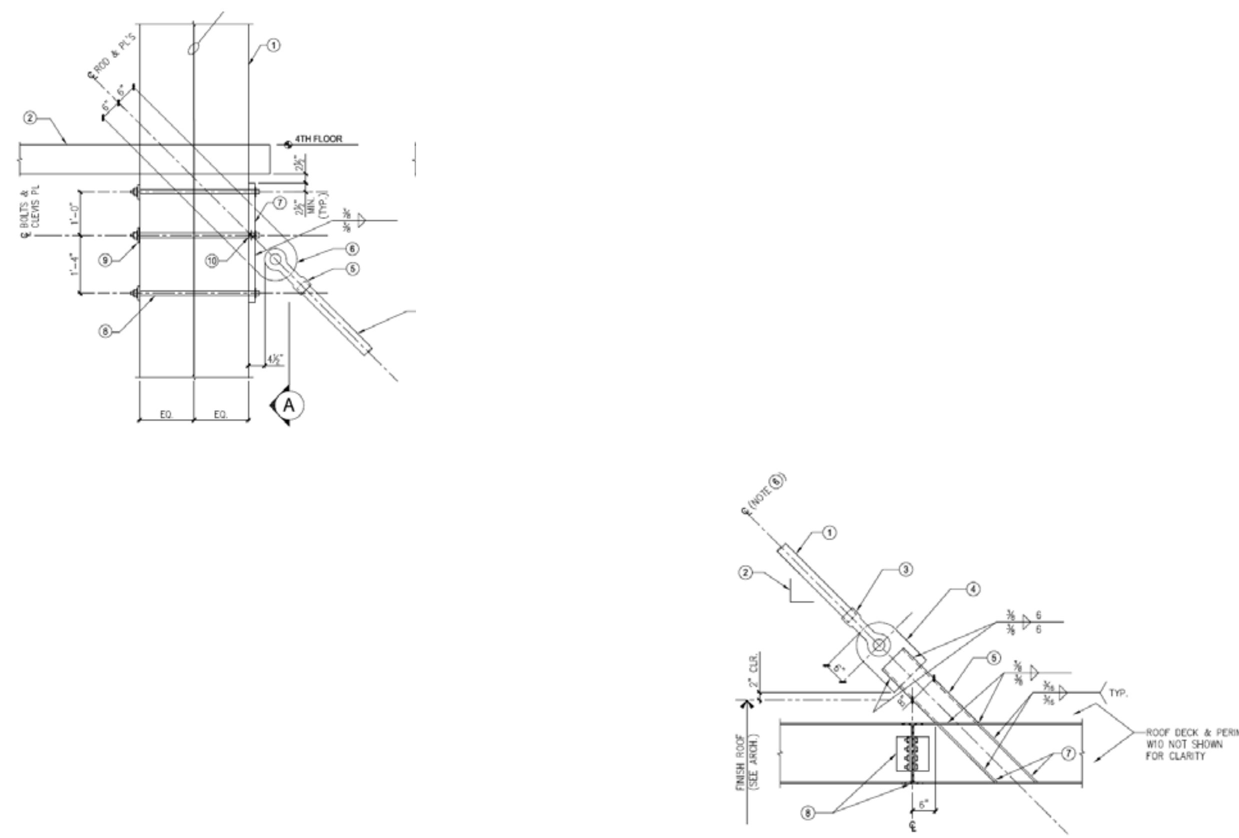
Canopies are located near each lobby entrance; one to the North along the Brambleton Avenue elevation, and the other to the South, along the York Street elevation. Moment connections are utilized to cantilever the canopies up to 10' beyond the building structure, tying into the first floor columns. Each canopy is supported with steel wide flange framing. Typical sizes include W10x26, W16x40, W16x57, and larger varied sizes at the center supports of each canopy. The York Street canopy has steel hanger rods that are attached just below the fourth floor to provide additional support for the longer cantilever length (Figures 10, 11, &12). These canopies feature a light-weight roofing system of metal deck and a single-ply EPDM.



(FIGURE 10) 2nd Floor Framing Plan – Brambleton Avenue Canopy

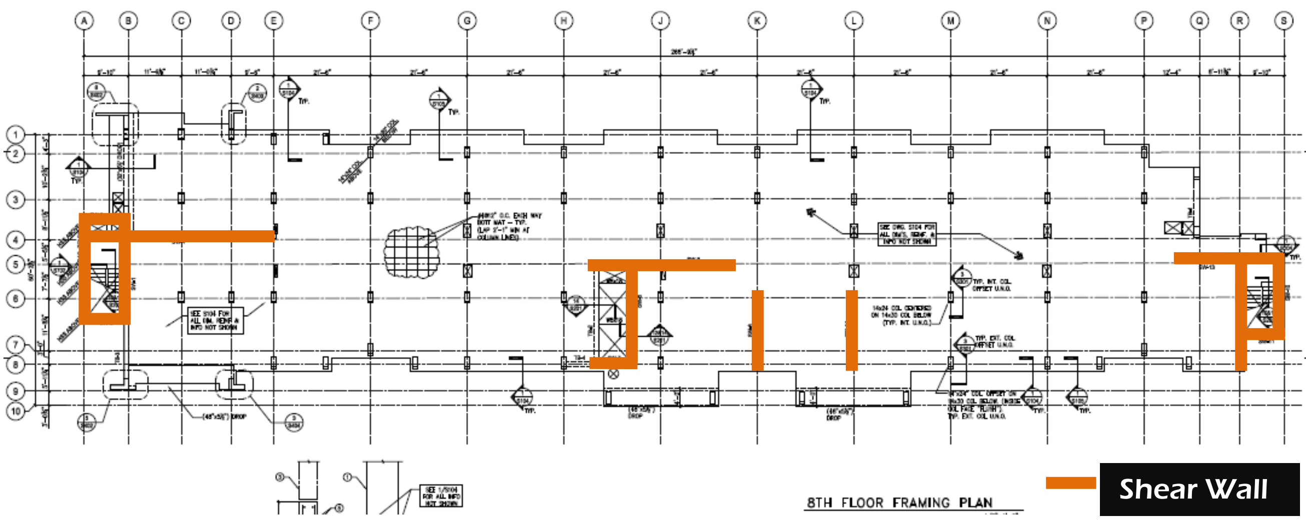


(FIGURE 11) 2nd Floor Framing Plan – York Street Canopy



(FIGURE 12) York Street Canopy Hanger Detail

The Residence Inn by Marriott employs cast-in-place reinforced concrete shear walls to resist lateral forces (Figure 14). There are a total of fourteen shear walls (shown in orange in the Figure 13 below), between 1'-0" and 1'-2" thick, and oriented in such a way to resist forces in both directions. These shear walls are continuous from the foundation to the top of the building, and behave as fixed cantilevers. They surround both the East and West stairwells, as well as the elevator shaft central to the building. Shear walls can also be found in between these areas to provide additional support. There are more shear walls oriented from North – South, which resist an overturning moment in the more susceptible direction. Lateral loads are transmitted to the shear walls through the floor diaphragms.



(FIGURE 13) Typical Floor Plan Highlighting Lateral Force Resisting System

(FIGURE 14) Shear Walls & 1st Floor Columns Under Construction

CURTAIN WALLS

The West stairwell requires special attention to support its three-story expanse of curtain wall. Steel HSS6x6 beams and columns transfer loads down to a cast-in-place concrete load-bearing wall at the seventh floor.

Curtain walls located in the guest rooms on the eighth and ninth floors span a smaller distance vertically, and therefore, additional framing is not required. The slabs above and below provide the anchoring points for this system.

APPLICABLE DESIGN CODES & STANDARDS

- IBC 2003**
- Virginia Uniform Statewide Building Code – 2003 Edition
- ASCE 7-02: Minimum Design Loads for Buildings and Other Structures
- ACI 318-02: Building Code Requirements for Structural Concrete**
- CRSI: Manual of Standard Practice
- AISC: Manual of Steel Construction – Allowable Stress Design, 9th Edition, 1989**
- Steel Deck Institute's Design Manual for Floor Decks & Roof Decks, 2001

**Denotes that a newer version was used in all calculations contained within this report

Specifically, the following references were used to calculate loads and perform spot checks:

- IBC 2006
- ACI 318-08
- AISC: Manual of Steel Construction – Load and Resistance Factor Design, 13th Edition, 2005

GRAVITY LOADS

The following is a summary of the superimposed dead and live loads, both as originally designed and according to the newest building code – IBC 2006 (Figure 15). The current code allows dead loads to be estimated based on actual material weights. Differences between the design and the assumed dead loads are seen as a result of the flexibility of assumptions. As can be seen in later calculations of the effective seismic weight, the assumed dead loads are considered conservative. It is important to note that the designer has included equipment weights for the mechanical mezzanine in the live load, thus the significant difference of values.

Snow loads are included separately below for comparison purposes with the design loads. It appears as though the canopies were designed for snow drifting, which was more conservative than was calculated here.

The assumed dead loads and the assumptions that make up these values listed below are described in more detail in the Appendix where the effective seismic weight is calculated.

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 15) Gravity Load Summary

WIND LOADS

Wind loads were calculated in accordance with ASCE7-05, Chapter 6. At this time, consideration was only given to the Main Wind Force Resisting System (MWFRS), following the Analytical Procedure (Method 2). A basic wind speed of 110 mph (3-sec gust) is required for the Norfolk, Virginia area. The occupancy category was determined as Type II using IBC 2006. An initial assumption that the structure was rigid based on its systems was later verified in the seismic calculations. Design wind pressures were determined, as shown below (Figures 16 & 18), for both the North-South and the East-West directions. The resulting story forces and overturning moments were then calculated, as shown in (Figures 17 & 19) for the N-S and E-W directions respectfully. Note: internal pressures were assumed to be zero. For a detailed list of assumptions and coefficients used, see Appendix.

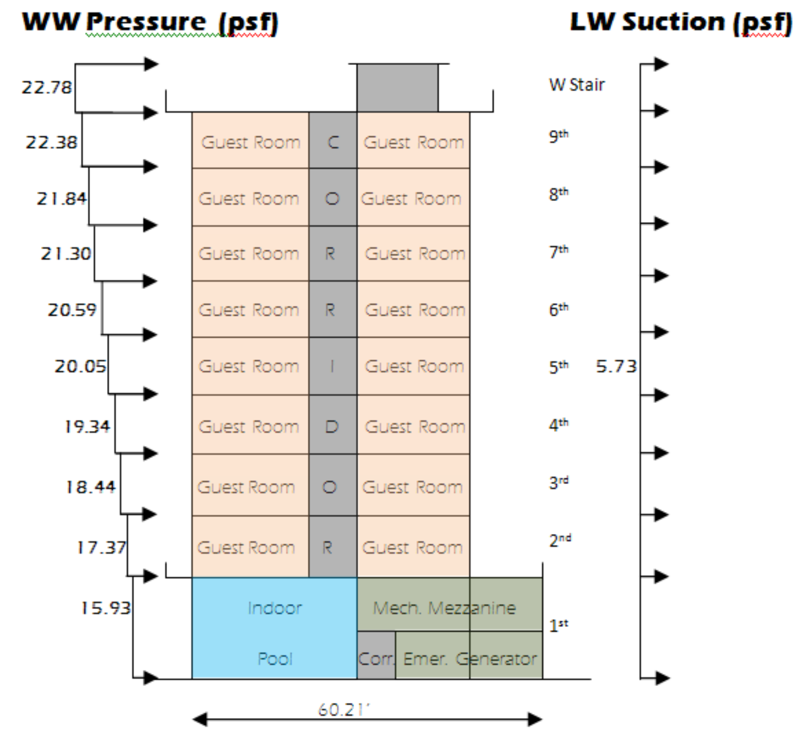
It is apparent from the results that the North-South direction for wind is controlling. This is not surprising since these elevations are significantly larger than in the East-West direction.

Controlling Wind Base Shear: 565 k

Corresponding Overturning Moment: 30,660 ft-k

While these results cannot be directly compared with the designer's, since the designer reported Components & Cladding pressures, a comparison of the external pressure coefficient (GC_p) is feasible. In the windward direction, a pressure coefficient of 0.68 was determined. The designer had a slightly lower value of 0.61. The difference could be attributed to different versions of ASCE7 and/or the designer may have performed a more detailed analysis to determine the gust factor G. The more critical leeward pressure's coefficient was found to be 0.42, which is very close to the designer's value of 0.43.

North-South Wind Pressures

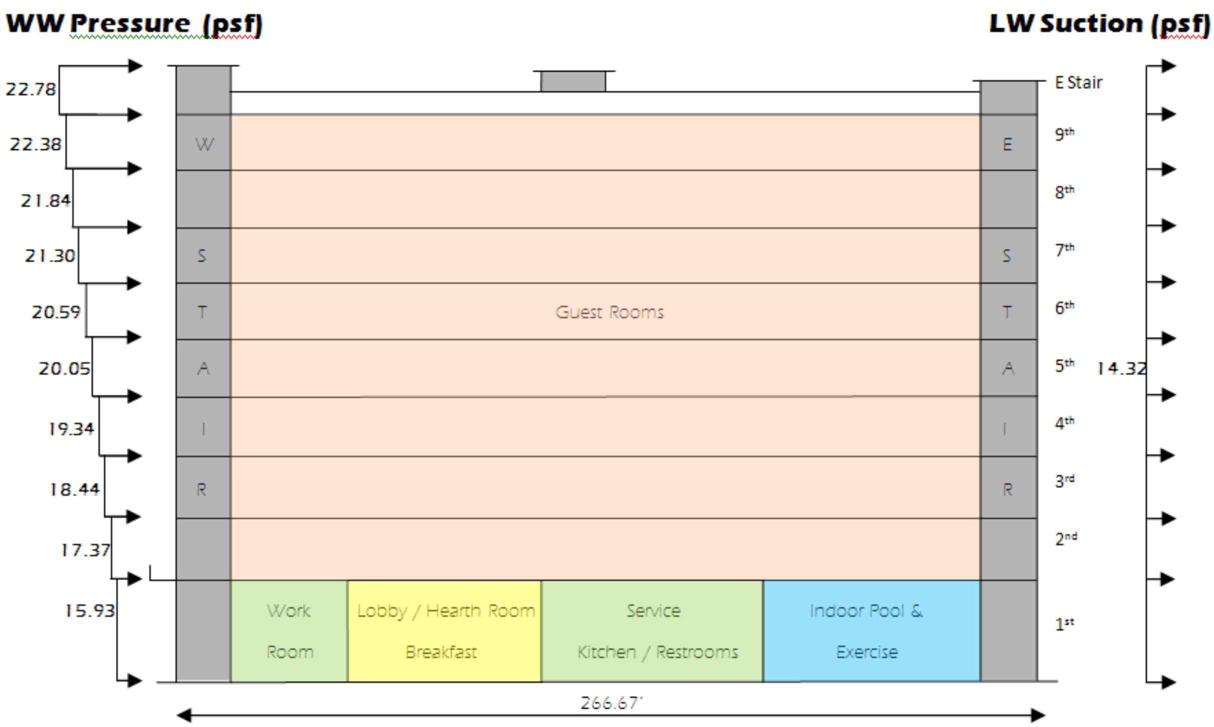


(FIGURE 16) North-South Wind Pressure Diagram

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

(FIGURE 17) North-South Wind Load Summary

East-West Wind Pressures



(FIGURE 18) East-West Wind Pressure Diagram

EAST-WEST WIND LOAD										
Floor	Location	Height Above Ground Level	Tributary Height	Tributary Width	Velocity Pressure	External Pressure	Total Pressure WW+(-LW)	Story Force	Story Shear	Overturning Moment
		h (ft)	(ft)	(ft)	q (psf)	qGC_p (psf)	p_t (psf)	F_x (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	5.38	569.19
Roof		93.67	4.67	48.00	32.91	22.38	36.70	8.23	13.61	770.62
9th		84.33	9.34	48.00	32.12	21.84	36.16	16.21	29.82	1,367.25
8th		75.00	9.34	48.00	31.33	21.30	35.63	15.97	45.79	1,197.92
7th		65.67	9.34	48.00	30.28	20.59	34.91	15.65	61.45	1,027.88
6th		56.33	9.34	48.00	29.49	20.05	34.38	15.41	76.86	868.12
5th		47.00	9.34	48.00	28.44	19.34	33.66	15.09	91.95	709.29
4th		37.67	9.34	48.00	27.12	18.44	32.76	14.69	106.64	553.33
3rd		28.33	9.34	48.00	25.54	17.37	31.69	14.21	120.84	402.49
2nd		19.00	14.17	60.21	23.43	15.93	30.25	25.81	146.66	490.44
	Leeward	ALL			33.70	-14.32	Base Shear =	146.66	M =	7,956.52

(FIGURE 19) East-West Wind Load Summary

SEISMIC LOADS

Seismic loads were determined using ASCE7-05, Chapter 12, IBC 2006, and the USGS's website for finding the design spectral acceleration for the exact latitude and longitude of the building site. Based on the spectral acceleration values, it was found that the more critical SDC B was in effect. It was then permissible to use the Equivalent Lateral Force Procedure. The site was considered to be Site Class D, based on the recommendation given in the geotechnical report. For a complete list of assumptions governing the load calculation, as well as a detailed calculation of the effective seismic weight, see the Appendix.

The table below (Figure 20) gives the results of this analysis, which are as follows:

Calculated Base Shear:	379 k
Corresponding Overturning Moment:	25,778 ft-k

The designer reported a base shear of 444 k, significantly greater than that which was calculated below. However, after close inspection of the assumptions made and coefficients used, it was determined that this difference would be a reasonable expectation. The designer used a seismic response coefficient that was almost 25% greater than that used here. Differences in these values traced back to the design spectral acceleration values, S_{DS} and S_{D1}. The designer reported that S_{DS}=0.143 and S_{D1}=0.097. The values from USGS differed from these obtained using figures within ASCE7. To check the impact of this difference, a calculation of the base shear using the designer's value of C_s=0.018 was performed and the results were a base shear of 477.5 k, only 7.5% greater than V_{design}. The overestimation indicates that the effective seismic weight was conservative. This would be expected since a typical 15 psf superimposed dead load was used here, as opposed to the designer's value of 10 psf.

LATERAL LOAD CONCLUSIONS

Based on the above results, it is clear that wind in the North-South direction would control the design. The base shear of the N-S wind is over 150% greater and the overturning moment almost 20% greater than the associated values with seismic loading. This would be expected given the considerably large façade facing in these directions.

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	118.65	105.67	48,436	0.0110	4.16	4.16	440.00
Penthouse Roof	108.51	102.17	42,413	0.0096	3.65	7.81	372.52
East Stair Roof	24.04	99.67	9,101	0.0021	0.78	8.59	77.98
Main Roof	2,164.94	93.67	756,503	0.1714	65.03	73.63	6091.79
9th	2,569.81	84.33	784,184	0.1777	67.41	141.04	5685.05
8th	2,569.81	75	674,109	0.1527	57.95	198.99	4346.36
7th	2,615.53	65.67	578,047	0.1310	49.69	248.69	3263.35
6th	2,615.53	56.33	474,257	0.1075	40.77	289.46	2296.61
5th	2,615.53	47	375,462	0.0851	32.28	321.73	1517.04
4th	2,615.53	37.67	282,224	0.0639	24.26	346.00	913.95
3rd	2,615.53	28.33	195,415	0.0443	16.80	362.80	475.92
2nd	3,770.61	19	168,269	0.0381	14.47	377.26	274.85
Mech. Mezzanine	1,223.59	10.33	24,879	0.0056	2.14	379.40	22.09
1st	901.28	0	0	-	-	379.40	-
TOTALS	26,528.89			1.0000	379.40		25,777.53

(FIGURE 20) Seismic Load Distribution & Overall Results

SPOT CHECKS

*Note: See Figure 5 above for locations of all spot checks.

GRAVITY COLUMN

A spot check was performed on gravity columns at column line M-3. The assumed loads described above were tabulated and used to decide if the column was adequate as designed (14"x30" w/ 12#9's). See Appendix (Figure A-15) for this load tabulation and (Figure A-16) for hand calculations. Results indicated that gravity loads on this column are not controlling and there must be additional lateral loads and moments that need to be accounted for before making a realistic comparison, as the column appeared to be severely oversized. The capacity appeared to be almost 400% greater than necessary. Oversimplification of the actual conditions has caused this null and void result. In comparing the tabulated load with the design load listed on the column schedule as 530 k of unfactored load, there is little discrepancy to speak of.

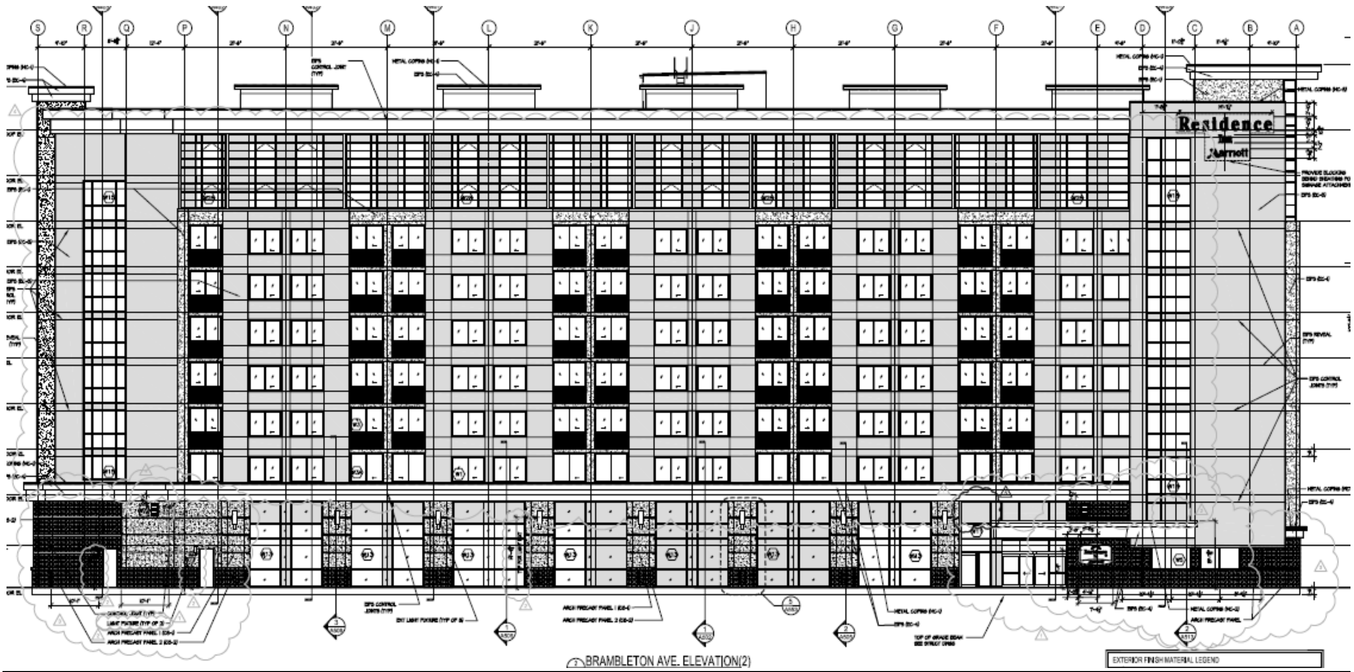
The intention was to proceed with analyzing the load of this column line on the transfer girder below, however, without resolving the true loads found above, this evaluation would not be very accurate.

TWO-WAY FLAT PLATE SLAB

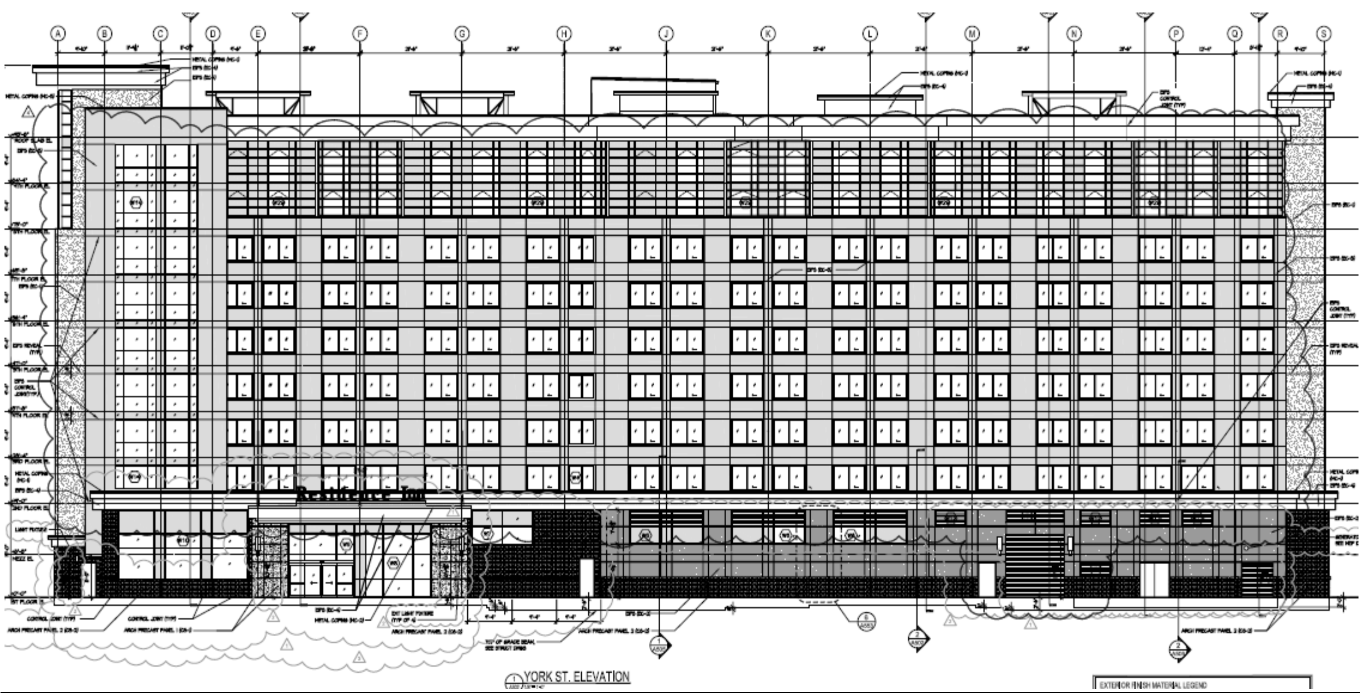
A separate analysis of the two-way flat plate slab system, which is typical of all upper floors, was performed on the sixth floor to check for adequate reinforcing along the column strip at column line M. It was difficult to choose a frame area that consisted of three successive spans similar in length, a requirement of the Direct Design Method. However, to avoid an excessively detailed analysis, the spans were considered equal, and this method was employed, using the longest dimension, which would result in a conservative design. Dimensions of columns, as designed, were used in this analysis to simulate the practical situation where a design must work to achieve architectural goals.

The results verify that the reinforcement design in general is adequate for the intended loads. The bottom reinforcing on the exterior span appears to be slightly inadequate, however, this is due to the fact that a longer span was assumed than actually exists, for sake of using the Direct Design Method. Other factors that could influence the results include the relative rigidities of nearby structural elements like shear walls, and also the width of column and middle strips. Frequently, when using computer programs to design two-way slabs, the column and middle strip widths will vary based on the input parameters, and do not necessarily match the simplified equation used to determine these values in the Direct Design Method. Without knowing the designer's values for these widths, the only way to estimate the provided reinforcement is by using the widths obtained in this calculation, and the given information that a typical bottom reinforcing mat is spaced at 12". There is some room for interpretation here that could cause the differences in steel required versus steel provided. Alternatively, an additional investigation of the required middle strip reinforcing could be combined with the results of the column strip analysis and instead make a comparison of total steel along the entire width of the frame. To achieve a more accurate result, computer modeling in programs such as ADOSS can be used, which better represent the actual situation.

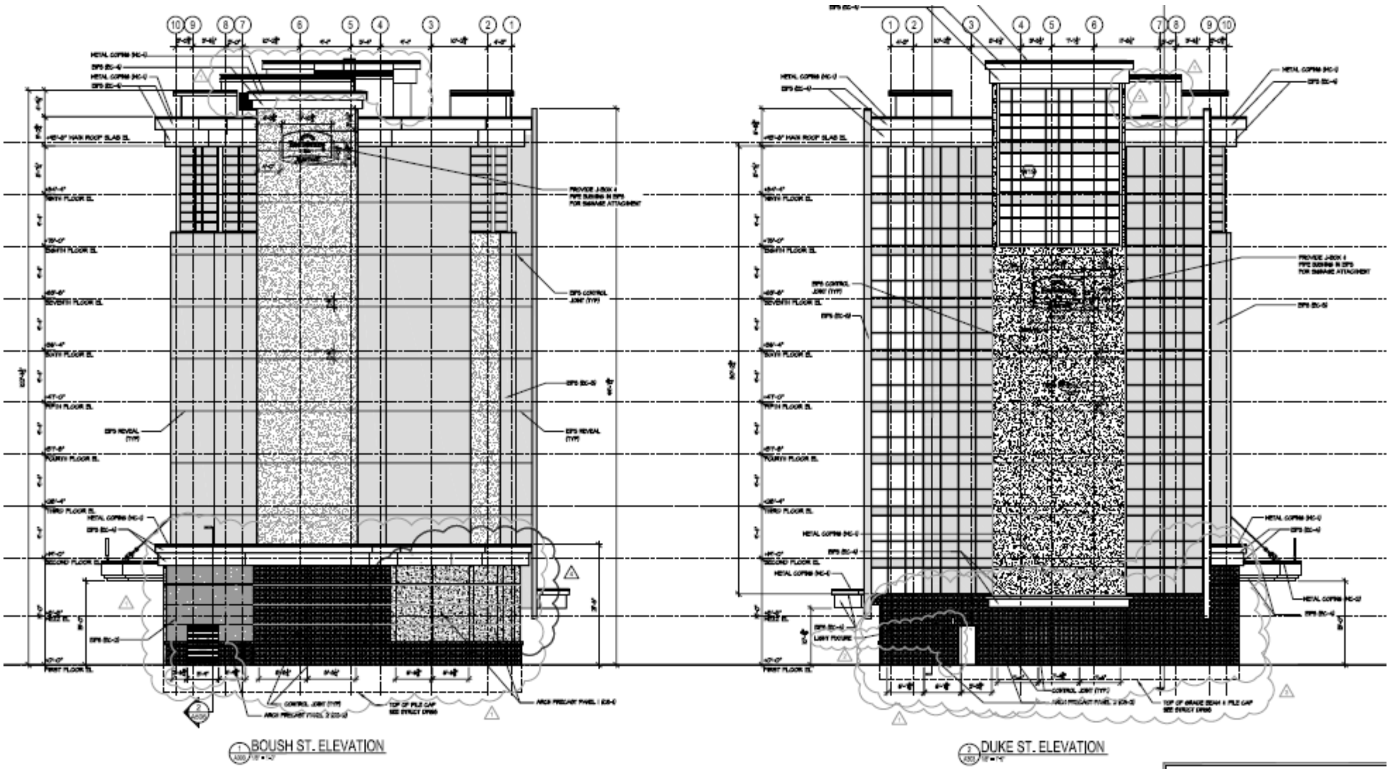
APPENDIX



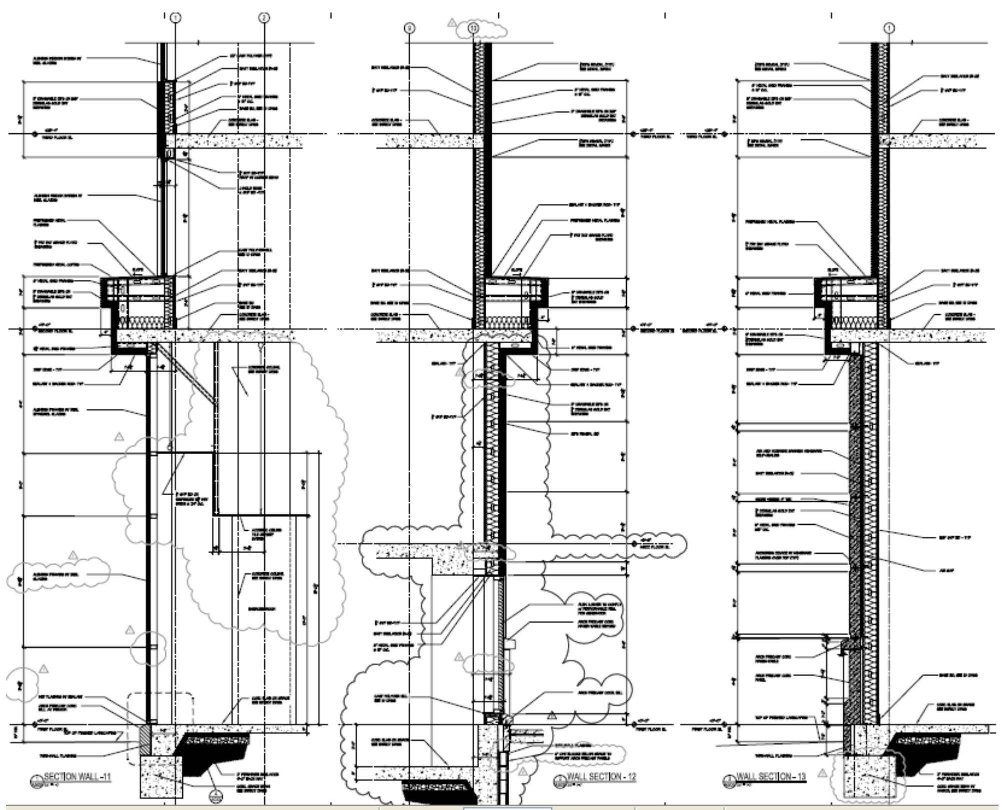
(FIGURE A-1) Brambleton Avenue (North) Elevation



(FIGURE A-2) York Street (South) Elevation



(FIGURE A-3) Boush Street (East) & Duke Street (West) Elevations



(FIGURE A-4) Typical Exterior Wall Sections

LIVE LOADS	RESIDENCE INN	09.13.08 KMR	1/1

REFERENCES : ASCE 7-05

(TBL. 4-1)

HOTELS → RESIDENTIAL

- PVT. ROOMS & CORRIDORS SERVING THEM : LL = 40 PSF
- PUBLIC ROOMS " " : LL = 100 PSF
- 1^{ST} FLR CORRIDORS LL = 100 PSF
- ORD. FLAT ROOF LL = 20 PSF
- CANOPIES LL = 20 PSF

(FIGURE A-5) Live Load Determination

SNOW LOADS | RESIDENCE INN | 09.24.08 KMR | 1/2

REFERENCES: ASCE 7-05

GROUND SNOW LOAD (7.2)

$p_g = 10$ PSF

FLAT ROOF SNOW LOAD (7.3)

$p_f = 0.7 C_e C_t I p_g > I p_g = (1)(10 \text{ PSF}) = 10 \text{ PSF}$

$C_e = 1.0$ (TBL. 7-2) EXPOSURE FACTOR

$C_t = 1.0$ (TBL. 7-3) THERMAL FACTOR

$I = 1.0$ (TBL. 7-4) IMPORTANCE FACTOR

$p_f = 0.7(1)(1)(1)(10 \text{ PSF}) = 7 \text{ PSF} < 10 \text{ PSF MIN} \therefore$

$\boxed{p_f = 10 \text{ PSF}}$

DRIFTS ON LOWER ROOFS (7.7)

Drift Height $h_d = 1.9'$ (LEEWARD DRIFTS)

(FIG 7-9) $\ell_u = 60'$ MORE CRITICAL*

$h_d = 3/4(0.5') = 0.375'$ (WINDWARD DRIFTS)

$\ell_u = 10'$

Drift Width $w = 4h_d = 4(1.9) = 7.6'$

Snow Density $\gamma = 0.13 p_g + 14 < 30$ PCF

$\gamma = 0.13(10) + 14 = 15.3$ PCF

(FIGURE A-6) Snow Load Determination (1/2)

DRIFT SURCHARGE LOAD

$$p_d = h_d \gamma = 1.9'(15.3\ PCF) = 29.07\ PSF$$

~ $\boxed{\text{USE 30 PSF}}$ ADD'L WHERE 1ST FLR ROOF EXTENDS FROM BLDG. ABOVE

ROOF PARAPET (7.8)

$h_d = 4'$ $(l_u = 267')$

$\times 3/4$

$h_d = 3'$

$$p_d = h_d \gamma = 3'(15.3\ PCF) = 45.9\ PSF \sim \boxed{\text{USE 46 PSF}}\ \text{ADD'L}$$

IS THIS REASONABLE?

RAIN LOADS (8.3)

$$R = 5.2(d_s + d_h)$$

NOTE: ELEV'S OF DRAINAGE SYSTEMS UNAVAILABLE ∴ CANNOT CALC. AT THIS TIME

(FIGURE A-7) Snow Load Determination (2/2)

WIND LOADS	RESIDENCE INN	09.14.08 KMR	1/2

REFERENCES: ASCE7-05
IBC 2006

MWFRS ONLY – METHOD 2, ANALYTICAL PROCEDURE

- BASIC WIND SPEED (NORFOLK, VA) (3-SEC. GUST) (FIG. 6-1) — $V = 110$ MPH
- DIRECTIONALITY FACTOR (TABLE 6-4) — $K_d = 0.85$
- IMPORTANCE FACTOR (TABLE 6-1) — $I = 1.0$
- EXPOSURE CATEGORY (6.5.6.3) — C
- OCCUPANCY CATEGORY: (TABLE 1-1) IBC 2006 — II
- TOPOGRAPHIC FACTOR (6.5.7.2) — $K_{zt} = 1.0$
- VELOCITY PRESSURE EXP. COEFF. (TABLE 6-3) INTERPOLATED AS NECESSARY — K_z (VARIES BASED ON HEIGHT – SEE SPREADSHEET)
- VELOCITY PRESSURE — q_z (SEE SPREADSHEET)

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I \ (\text{lb/ft}^2)$$ EQN 6-15

- GUST EFFECT FACTOR (6.5.8.1) — $G = 0.85$

*ASSUMPTION: STRUCTURE IS RIGID
→ (NEED TO VERIFY THAT NAT. FREQUENCY OF BLDG $n_1 > 1.0$ Hz.)

✓ OK

*SEE SEISMIC CALCS

(FIGURE A-8) Wind Load Determination (1/2)

• COMBINED NET PRESSURE @ PARAPET

$GC_{pn} = +1.5$ (WW PARAPET)

$GC_{pn} = -1.0$ (LW PARAPET)

$p_p = q_p GC_{pn}$ (lb/ft²)

PARAPET HT (TYP)

$p_p = (33.70\ \text{lb/ft}^2)(1.5) = 50.55$ PSF × 4.5' = 227.5 PLF (WW)
(@ SCREEN WALLS) × 9.33' = 471.6 PLF

$p_p = (33.70\ \text{lb/ft}^2)(-1.0) = -33.70$ PSF × 4.5' = −151.7 PLF (LW)
(@ SCREEN WALLS) × 9.33' = −314.4 PLF

• WALL PRESSURE COEFF'S
(FIG. 6-6)

E-W

WW WALL → $C_p = 0.8$

LW WALL → $L/B = 60.21'/266.77' = 0.226 \rightarrow C_p = -0.5$

N-S

WW WALL → $C_p = 0.8$

LW WALL → $L/B = 266.77'/60.21' = 4.43 \rightarrow C_p = -0.2$

• DESIGN WIND PRESSURES
(6.5.12.2.1)

$p = qGC_p - \cancel{q_i(GC_{pi})}$ (lb/ft²) NEGLECTING INT. PRESSURES AT THIS STAGE

$\therefore\ p = qGC_p$ (SEE SPREADSHEET)

(FIGURE A-9) Wind Load Determination (2/2)

SEISMIC LOADS	RESIDENCE INN	09.20.08 KMR	1/5

REFERENCES: ASCE7-05
IBC 2006
geocoder.us (36.854776N LAT, -76.291680 W LONG)

SITE CLASS D (PER GEOTECH REPORT – MCCALLAM)

S_S (0.2s SPECTRAL RESPONSE ACCEL.) FIG. = 0.118
S_1 (1.0s SPECTRAL RESPONSE ACCEL.) FIG. = 0.048
} SITE CLASS B, PER USGS

(SEE OUTPUT FROM USGS SITE)

$S_{DS} = 0.126$
$S_{D1} = 0.077$
} SITE CLASS D, PER GEOTECH REPORT (I WILL USE THESE VALUES)

OCCUPANCY CATEGORY II

FUNDAMENTAL PERIOD OF STRUCTURE (T_a) (12.8.2.1)

$T_a = C_t h_n^x = (0.016)(108)^{0.9} = 1.08\,s$

$h_n = 108'$

CONC. MOMENT-RESISTING FRAMES $\therefore$ $C_t = 0.016$

$x = 0.9$

$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.077}{0.126} = 0.611\,s$ (11.4.5)

$0.8T_s = 0.8(0.611s) = 0.489s \overset{?}{>} T_a = 1.08\,s \rightarrow$ NO

$\therefore$ TBL. 11.6-1 NOT PERMITTED FOR SDS

(FIGURE A-10) Seismic Load Determination (1/5)

MCE SPECTRAL RESP. ACCEL. (11.4.3)

$S_{MS} = F_a S_s = 1.6(0.118) = \boxed{0.189\,s}$

$F_a = 1.6$ (TBL. 11.4-1), SITE CLASS D, $S_s < 0.25s$

$S_{M1} = F_v S_1 = 2.4(0.048) = \boxed{0.115\,s}$

$F_v = 2.4$ (TBL. 11.4-2), SITE CLASS D, $S_1 < 0.1s$

VERIFY W/ USGS
DESIGN SPECTRAL ACCEL.

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.189s) = \boxed{0.126s}$ ✓ SAME AS USGS

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.115s) = \boxed{0.077s}$ ✓ SAME AS USGS

SDC

- BASED ON $S_{DS} \rightarrow$ SDC = A (TBL. 11.6-1) $S_{DS} = 0.126 < 0.167$; OCC. CAT. II

- BASED ON $S_{D1} \rightarrow \boxed{\text{SDC} = B}$ MOST SEVERE ∴ USE THIS (TBL. 11.6-2) $0.067 < S_{D1} = 0.077 < 0.133$ OCC. CAT. II

$\rightarrow$ SINCE SDC = B $\rightarrow$ EQUIVALENT LATERAL FORCE PROCEDURE IS PERMITTED (12.8)

(FIGURE A-11) Seismic Load Determination (2/5)

RESPONSE MODIFICATION COEFF. (R)
(TBL. 12.2-1)

BLDG. FRAME SYSTEM - ORDINARY REINF. CONC. SHEAR WALLS $\longrightarrow$ $R = 5$

ASSUMPTIONS PERMITTING BLDG HT UP TO 240':
- NO EXTREME TORSIONAL IRREGULARITY
- ANY ONE SHEAR WALL RESISTS ≤ 60% TOTAL SEISMIC FORCES IN EA DIRECTION, NEGLECTING ACCIDENTAL TORSIONAL EFFECTS

IMPORTANCE FACTOR (TBL. 11.5-1) $I = 1.0$

- BASED ON OCCUP. II

LONG-PERIOD TRANSITION PERIOD (FIG. 22-15) $T_L = 8s$

SEISMIC RESPONSE COEFF

$T = 1.08s < T_L = 8s \therefore$

$$C_S = \frac{S_{D1}}{T\left(\frac{R}{I}\right)} \le \frac{S_{DS}}{\left(\frac{R}{I}\right)}$$ (EQNS. 12.8-2, 12.8-3)

0.1286

$$= \frac{0.077}{1.08\left(\frac{5}{1}\right)} = 0.0143 \overset{?}{\le} \frac{0.126}{\left(\frac{5}{1}\right)} = 0.0252$$

0.018 ✓ YES

$\therefore \boxed{C_S = 0.0143} > C_{S,MIN} = 0.01 \therefore$ OK

0.018

(FIGURE A-12) Seismic Load Determination (3/5)

(FIGURE A-12a) Effective Seismic Weight Determination:

PENTHOUSE ROOF			
Approximate Area:			Floor-to-Floor Height
552 SF			0
	Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances			
Roofing	5		2.76
MEP Hung Below	10		5.52
Slab Self Weight			
10" Reinf. Conc. Slab	125		69.00
	Qty. (LF)	Weight (LB/FT)	
Concrete Beams			
12"x30"	19	375	7.13
24"x20"	35	500	17.50
12"x24"	22	300	6.60
		TOTAL FLOOR WEIGHT (k)	108.51

WEST STAIR ROOF			
Approximate Area:			Floor-to-Floor Height
416 SF			0
	Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances			
Roofing	5		2.08
MEP Hung Below	10		4.16
Slab Self Weight			
8" Reinf. Conc. Slab	100		41.60
	Qty. (LF)	Weight (LB/FT)	
Concrete Beams			
30"x22"	39	687.5	26.81
72"x22"	12	1650	19.80
36"x22"	20	825	16.50
14"x24"	22	350	7.70
		TOTAL FLOOR WEIGHT (k)	118.65

EAST STAIR ROOF			
Approximate Area:			Floor-to-Floor Height
209	SF		0
	Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances			
Roofing	5		1.05
MEP Hung Below	10		2.09
Slab Self Weight			
8" Reinf. Conc. Slab	100		20.90
		TOTAL FLOOR WEIGHT (k)	24.04

MAIN ROOF				
Approximate Area:				Height to Upper Roof: Varies
14,376	SF		West Stairwell	12'
			Penthouse	8.5'
			East Stairwell	6'
				ITEMIZED WEIGHT (k)
Mechanical Equipment				27.65
	Allowance (PSF)			
Superimposed Allowances				
Roofing	10			143.76
MEP Hung Below	10			143.76
MISC - Ducts, Pipes, Wear Course	10			143.76
Slab Self Weight				
8" Reinf. Conc. Slab	100			1437.60
	Qty. (LF)	Weight (LB/FT)		
Concrete Columns				
1 14"x24" @ W Stair	12	350		4.20
1 14"x24" @ PH	8.5	350		2.98
3 12"x24" @ PH	8.5	300		7.65
1 24"x24" @ PH	8.5	600		5.10
Concrete Beams				
12"x16"	14	200		2.80
12"x20"	3	250		0.75
36"x26"	11	975		10.73
12"x24"	24	300		7.20
24"x30"	23	750		17.25
14"x16"	12	233.3		2.80
Shear Walls (1'-0" Thick, Typ.)				
@ W Stair	28	1800		50.40
@ PH	60	1275		76.50
@ E Stair	52	900		46.80

Parapet Walls	(4'-6" HT, Typ.)			
	Braced Metal Studs	654	46	30.08
Steel Framing	(Screen Walls)			
	HSS 7x7x5/16	34	27.54	0.94
	6" Dia. ES Pipe	60	28.6	1.72
	5" Dia. Std. Pipe	36	14.6	0.53
			TOTAL FLOOR WEIGHT (k)	2164.94

FLOORS 8-9				
Approximate Area:				Floor-to-Floor Height
	14,376 SF			9.33'
		Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances				
	Floor Finishes	5		71.88
	MEP Hung Below	10		143.76
	Partitions	20		287.52
Slab Self Weight				
	8" Reinf. Conc. Slab	100		1437.60
		Qty. (LF)	Weight (LB/FT)	
Concrete Columns				
56	14"x24"	9.33	350	182.87
2	14"x30"	9.33	437.5	8.16
Concrete Beams				
	12"x16"	44	200	8.80
	14"x16"	12	233.3	2.80
Shear Walls				
	1'-0" Thick	204	1399.5	285.50
	1'-2" Thick	22	1632.8	35.92
Exterior Walls				
	Curtain Wall	750	140	105.00
			TOTAL FLOOR WEIGHT (k)	2569.81

FLOORS 3-7				
Approximate Area:				Floor-to-Floor Height
	14,376 SF			9.33'
		Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances				
	Floor Finishes	5		71.88
	MEP Hung Below	10		143.76
	Partitions	20		287.52
Slab Self Weight				
	8" Reinf. Conc. Slab	100		1437.60
		Qty. (LF)	Weight (LB/FT)	
Concrete Columns				
58	14"x30"	9.33	437.5	236.75
Concrete Beams				
	12"x16"	44	200	8.80
	14"x16"	12	233.3	2.80
Shear Walls				
	1'-0" Thick	204	1399.5	285.50
	1'-2" Thick	22	1632.8	35.92
Exterior Walls				
	Drainable EIFS	750	140	105.00
			TOTAL FLOOR WEIGHT (k)	2615.53

FLOOR 2					
Approximate Area:					Floor-to-Floor Height
	14,376	SF			9.33'
Canopies	650	SF			
					ITEMIZED WEIGHT (k)
Canopy Steel Framing (Wide Flanges)					16.96
		Allowance (PSF)			
Canopy Roof		10			6.5
Superimposed Allowances					
	Floor Finishes	5			71.88
	MEP Hung Below	10			143.76
	Suspended Ceiling Below	5			71.88
	Partitions	20			287.52
Slab Self Weight					
	8" Reinf. Conc. Slab	100			1437.60
		Qty. (LF)	Weight (LB/FT)		
Concrete Columns					
58	14"x30"	19	437.5		482.13
Concrete Beams					
	12"x16"	42	200		8.40
	8"x8"	2	66.7		0.13
	14"x16"	16	233.3		3.73
Concrete Transfer Girders					
	48"x36"	26	1800		46.80
	48"x48"	70	2400		168.00
	48"x44"	28	2200		61.60
	48"x54"	30	2700		81.00
	36"x48"	57	1800		102.60
	30"x48"	9	1500		13.50
	36"x36"	12	1350		16.20
	72"x54"	55	4050		222.75
	60"x54"	30	3375		101.25

Shear Walls				
	1'-0" Thick	204	1399.5	285.50
	1'-2" Thick	22	1632.8	35.92
Exterior Walls				
	Drainable EIFS	750	140	105.00
			TOTAL FLOOR WEIGHT (k)	3770.61

MECHANICAL MEZZANINE					
Approximate Area:					Floor-to-Floor Height
		1,576 SF			10.33'
		Allowance (PSF)			ITEMIZED WEIGHT (k)
Superimposed Allowances					
	Floor Finishes	5			7.88
	MEP Hung Below	10			15.76
	Mech. Equipment	10			15.76
Slab Self Weight					
	8" Reinf. Conc. Slab	100			157.60
		Qty. (LF)	Weight (LB/FT)		
Concrete Columns					
58	20"x30"	10.33	625		374.46
7	20"x24"	10.33	500		36.16
Concrete Beams					
	24"x18"	76	450		34.20
	30"x18"	116	562.5		65.25
	30"x22"	22	687.5		15.13
	26"x22"	22	595.8		13.11
	20"x18"	25	375		9.38
Shear Walls					
	1'-0" Thick	204	1399.5		285.50
	1'-2" Thick	22	1632.8		35.92

Exterior Walls				
	Arch. Precast/Drainable EIFS	750	210	157.50
			TOTAL FLOOR WEIGHT (k)	1223.59

FLOOR 1				
Approximate Area:				Floor-to-Floor Height
	14,376	SF		19'
		Allowance (PSF)		ITEMIZED WEIGHT (k)
Superimposed Allowances				
	Floor Finishes	5		71.88
	Partitions	20		287.52
		Qty. (LF)	Weight (LB/FT)	
Concrete Columns				
58	20"x30"	9.67	625	350.54
7	20"x24"	9.67	500	33.85
Exterior Walls				
	Arch. Precast/Drainable EIFS	750	210	157.50
			TOTAL FLOOR WEIGHT (k)	901.28

SEISMIC LOADS | RESIDENCE INN | 09.21.08 KMR | 4/5

CALCULATION OF EFFECTIVE SEISMIC WEIGHT (W) (12.7.2)

(SEE SPREADSHEET)

KEY ASSUMPTIONS:

- REINF. CONC. WT = 150 PCF
- PARTITION ALLOWANCE BASED ON TYPICAL FLOOR.
- FOUNDATION SYSTEMS EXCLUDED FROM CALC: (IE. GRADE BMS, PILES, PILE CAPS)

$W =$	108.51^k	PH ROOF
	118.65	W STAIR ROOF
	24.04	E STAIR ROOF
	2164.94	MAIN ROOF
	2(2569.81)	FLRS 8-9
	5(2615.53)	FLRS 3-7
	3770.61	FLR 2
	1223.59	MECH MEZZ.
	+ 901.28	FLR 1

$$\boxed{W = 26,528.89^k}$$

BASE SHEAR (V) (12.8-1)

$$V = C_s W = 0.0143\,(26,528.89^k) = \boxed{379.4^k}$$

COMPARISON W/ DESIGN BASE SHEAR $V_{DES} = 444^k$

DIFFERENCE: $S_{DS} = 0.143$
$S_{D1} = 0.097$

$$C_s = \frac{0.097}{1.08\left(\frac{5}{1}\right)} = 0.0180 \overset{?}{\leq} \frac{0.143}{\left(\frac{5}{1}\right)} = 0.0286$$

YES ✓

$\therefore C_{s,DESIGN} = 0.0180$

V_{DESIGN} USING MY CALC. W $= 0.018\,(26,528.89^k) = 477.5^k \sim 7.5\%$ GREATER THAN V_{DESIGN}

(FIGURE A-13) Seismic Load Determination (4/5)

SEISMIC LOADS | RESIDENCE INN | 09.21.08 KMR | 5/5

VERTICAL DISTRIBUTION OF SEISMIC FORCES

(12.8.3)

$F_x = C_{vx} V$ (SEE SPREADSHEET)

$$C_{vx} = \frac{w_x h_x^k}{\sum_{i=1}^{n} w_i h_i^k}$$

$k = 0.75 + 0.5T = 0.75 + 0.5(1.08) = 1.29$

DESIGN STORY SHEAR

(EQN. 12.8-13)

$V_x = \sum_{i=x}^{n} F_i$ (SEE SPREADSHEET)

DEFLECTION AMPLIFICATION FACTOR (FOR FUTURE USE)

$C_d = 4.5$ (TBL 12.2-1)

(FIGURE A-14) Seismic Load Determination (5/5)

SPOT CHECK - GRAVITY COLUMN M-3 (using design loads)								
Floor	Tributary Area	Dead Load	Live Load	K_{LL}	Live Load Reduction Factor	Reduced Live Load	Factored Load 1.2D + 1.6L	Total Factored Load
	(ft^2)	(psf)	(psf)	(Int. Col.)		(psf)	(psf)	(k)
Main Roof	344.00	134	30	4	-	30.00	208.80	71.83
9th	344.00	110	50	4	0.65	32.72	184.35	63.42
8th	344.00	110	50	4	0.65	32.72	184.35	63.42
7th	344.00	110	50	4	0.65	32.72	184.35	63.42
6th	344.00	110	50	4	0.65	32.72	184.35	63.42
5th	344.00	110	50	4	0.65	32.72	184.35	63.42
4th	344.00	110	50	4	0.65	32.72	184.35	63.42
3rd	344.00	110	50	4	0.65	32.72	184.35	63.42
2nd	344.00	110	50	4	0.65	32.72	184.35	63.42
TOTALS								579.16
							Self Weight of Cols Above	30.00
								609.16

SPOT CHECK - GRAVITY COLUMN M-3 (using assumed loads)								
Floor	Tributary Area	Dead Load	Live Load	K_{LL}	Live Load Reduction Factor	Reduced Live Load	Factored Load 1.2D + 1.6L	Total Factored Load
	(ft^2)	(psf)	(psf)	(Int. Col.)		(psf)	(psf)	(k)
Main Roof	344.00	139	20	4	-	20.00	198.80	68.39
9th	344.00	115	55	4	0.65	35.99	195.58	67.28
8th	344.00	115	55	4	0.65	35.99	195.58	67.28
7th	344.00	115	55	4	0.65	35.99	195.58	67.28
6th	344.00	115	55	4	0.65	35.99	195.58	67.28
5th	344.00	115	55	4	0.65	35.99	195.58	67.28
4th	344.00	115	55	4	0.65	35.99	195.58	67.28
3rd	344.00	115	55	4	0.65	35.99	195.58	67.28
2nd	344.00	115	55	4	0.65	35.99	195.58	67.28
TOTALS								606.64
							Self Weight of Cols Above	30.00
								636.64

(FIGURE A-15) Spot Check – Gravity Column M-3 Accumulated Loads

SPOT CHECK GRAVITY COLUMN | RESIDENCE INN | 09.22.08 KMR | 1/1

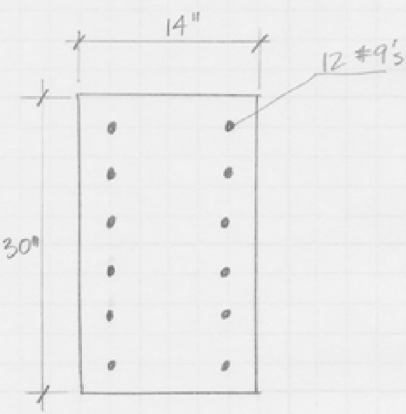
CHECK COL. M-3 FOR GRAVITY – VERIFY DESIGN IS ADEQUATE @ 2ND FLR – 3RD FLR

TRIB. AREA = 21'-6" × 16' = 344 SF

$f'_c = 5000$ PSI

$f_y = 60{,}000$ PSI

$P_u = 636.64^k$ (ASSUMING PURE AXIAL)



$A_s = 12(1.0\,in^2) = 12\,in^2$

$\varepsilon_y = \frac{f_y}{E_s} = \frac{60}{29{,}000} = 0.00207$

$\rho = \frac{A_s}{bd} = \frac{12}{30 \times 14} = 0.029$

$$P_o = 0.85 f'_c A_c + A_s f_y$$
$$= 0.85(5)(30 \times 14 - 12) + 12(60)$$
$$= 2454^k > P_u = 636.64^k$$

∴ COLUMN IS MORE THAN ADEQUATE

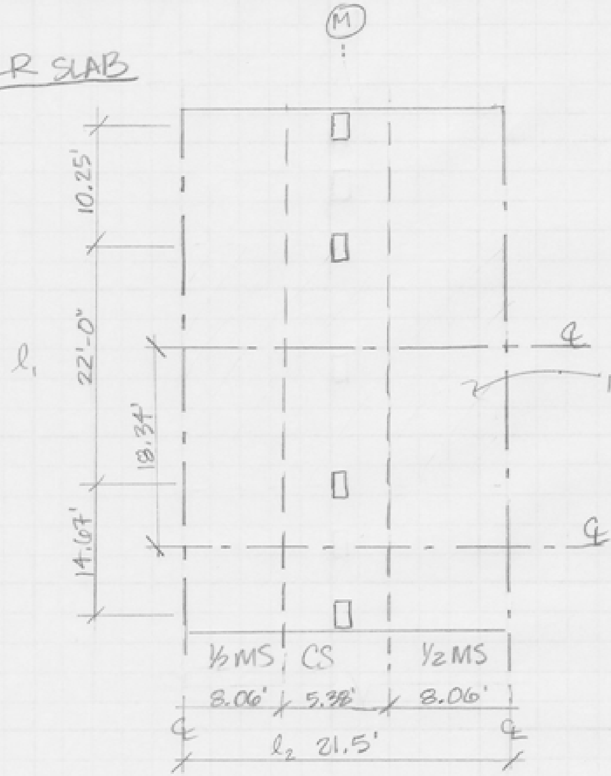
* MOMENTS NEED TO BE ACCOUNTED FOR IN ORDER TO HAVE A MORE RELEVANT COMPARISON

(FIGURE A-16) Gravity Column Spot Check

SPOT CHECK – 2-WAY SLAB | RESIDENCE INN | 09.26.08 KMR | 1/5

6^{TH} FLR SLAB

ACI 318-08, Ch. 13



FRAME M

$w_u = 1.2 w_{DL} + 1.6 w_{LL}$
$= 195.58$ PSF
$= 0.196$ K/SF
(SEE GRAVITY COL. SPOT CHECK SPREADSHEET)

COLS: 14" × 30"

$f_y = 60000$ PSI

$f'_c = 4000$ PSI

DIRECT DESIGN METHOD:

1) ≥ 3 SPANS ✓ OK

2) $l_2/l_1 = 21.5/22 = 0.98 < 2 \therefore$ OK

3) $l_2 - l_1 = -0.5' < \frac{1}{3} l_2 = \frac{1}{3}(21.5) = 7.17 \therefore$ OK ✓YES

4) COL. LINES ARE NOT OFFSET ✓

5) $w_{LL} = 35.99 \overset{?}{\leq} 2 w_{DL} = 2(115) = 230 \therefore$ OK ✓YES

10.25' SPAN DOES NOT MEET THIS CRITERIA; HOWEVER THERE ARE NO OTHER CHOICES OF FRAMES OF 3 SUCCESSIVE SPANS THAT SATISFY THIS. COMPUTER ANALYSIS WILL BE NECESSARY.

TOTAL FACTORED STATIC MOMENT: (INT. SPAN)

$$M_0 = \frac{w_u l_2 l_n^2}{8} = \frac{1}{8}(0.196 \text{ KSF})(21.5')\left(22 - \frac{30}{12}\right)^2 = \boxed{200^{'k}}$$

(FIGURE A-17) Two-Way Slab Spot Check (1/5)

MIN. SLAB THICKNESS FOR SLABS W/O BMS:

TBL. 9.5C → $\frac{l_n}{33} = \frac{(22-\frac{30}{12})(12)}{33} = \boxed{7.09'' = t_{min}} \sim 8''$

$< t_{actual} = 8''$ ∴ OK

COL. STRIP WIDTH:

$0.25\, l_2 = 0.25(21.5') = 5.38'$

DISTRIBUTION OF TOTAL STATIC MOMENT

INT. SPAN: M^- (0.65)

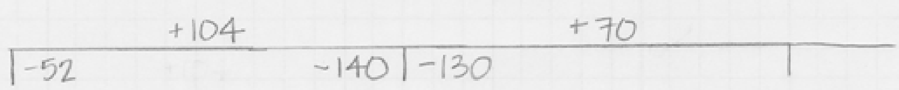
M^+ (0.35)

END SPAN: M^-_{INT} (0.70)

M^+ (0.52)

M^-_{EXT} (0.26)

$\times\ 200'^k = M_0$



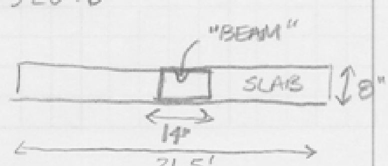
FACTORED MOMENTS IN COL. STRIPS [13.6.4.1]

NEG. MOM. @ INT. SUPPORT

$0.75(-130) = \boxed{-97.5'^k}$ CS

$0.25(-130) = \boxed{-32.5'^k}$ MS

$\alpha_f = \frac{I_b E_{cb}}{I_s E_{cs}} = \frac{\frac{14(8)^3}{12}}{\frac{(21.5)(12)(8)^3}{12}} = \frac{7168}{132096} = 0.054$ (SAME = 1)



$\beta_t = \frac{C}{2I_s}$

(FIGURE A-18) Two-Way Slab Spot Check (2/5)

SPOT CHECK – 2-WAY SLAB	RESIDENCE INN		09.26.08 KMR	3/5

$$C = \Sigma\left(1 - 0.63\frac{x}{y}\right)\frac{x^3 y}{3} = \left(1 - 0.63\left(\frac{14}{8}\right)\right)\frac{(14)^3(8)}{3} = \text{NEG. VALUE}$$

→ ASSUME NO TORSIONAL RESIST. SINCE NO BMS ∴ $\beta_t = 0$

$$\alpha_{f1}\left(\frac{\ell_2}{\ell_1}\right) = 0.054(0.98) = 0.053$$

REGARDLESS, INTERP. FROM TABLE IN 13.6.4.2 IS BETW. VALUE OF 100, SO

CS – % OF M^-_{EXT} = 100%

NEG. MOM. @ EXT. SUPPORT → 100% $\boxed{CS = -52^{'k}}$ → 0% $\boxed{MS = 0^{'k}}$

POS. MOMENT (INT. SPAN) → 60.8% $\boxed{CS = 42.6^{'k}}$ 39.2% $\boxed{MS = 27.4^{'k}}$

	0.5	0.98	1.0
0	60	60	60
0.053		$\boxed{60.8}$	
1.0	90	75.6	75

SUMMARY – FRAME M : TOTAL WIDTH = 21.5'
COL. STRIP = 5.38'
MID. STRIP = 16.12'

(ft·k)

TOTAL MOMENT	-52	+104	-140	-130	+70
CS	-52	+63.2	-105	-97.5	+42.6
MS	0	+40.8	-35	-32.5	+27.4

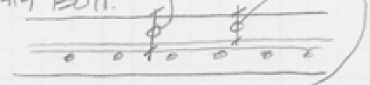
(FIGURE A-19) Two-Way Slab Spot Check (3/5)

SPOT CHECK - 2-WAY SLAB | RESIDENCE INN | 09.26.08 KMR | 4/5

REINF. DESIGN

$d_{long} = 7 - 0.5 = 6.5"$

#4 BOTT.



$d_{short} = 8 - 3/4 - 1/2(0.5) = 7$

$A_{s,MIN} = 0.0018\,bt$ $[f_y = 60\text{ KSI}]$

Max Spacing = $2(t) = 2(8") = 16"$

DESIGN OF SLAB REINF. IN COL. STRIP

Item #	Description	EXT. SPAN M^-_{EXT}	M^+	M^-_{INT}	INT. SPAN M^-	M^+
(1)	Moment M_n (ft·k)	−52	63.2	−105	−97.5	+42.6
(2)	CS Slab Width b $= \frac{1}{2}(5.38')(12)$ (in)	32.28	32.28	32.28	32.28	32.28
(3)	Eff. Depth d (in)	~~6.5"~~ 6.91	~~6.5"~~ 6.91	~~6.5"~~ 6.91	~~6.5"~~ 6.91	~~6.5"~~ 6.91
(4)	$M_u = \frac{M_n}{\Phi} = \frac{M_n}{0.9}$	−57.8	+70.22	−116.7	−108.3	+47.3
(5)	$\frac{M_u(12)}{b}$	−21.49	+26.1	−43.38	−40.26	+17.58
(6)	$R = \frac{M_n}{bd^2}$	405	492	817	759	332
(7)	ρ (Tbl. A.5a)	0.0072	0.0089	0.0158	0.0145	0.0057
$A_{s,req'd}$ = (8)	$A_{st} = \rho bd$ (in²)	$\boxed{1.61}$ ✓	$\boxed{1.99}$ ✓	$\boxed{3.52}$ ✓	$\boxed{3.23}$ ✓	$\boxed{1.27}$ ✓
(9)	$A_{st,min} = 0.002bt$	0.52	0.52	0.52	0.52	0.52
(10)	N = larger (8)(9) / 0.20 in² (area of #4)	8.05=9 ✓	9.95=10 ✓	17.6=18 ✓	16.2=17 ✓	6.35=7 ✓
(11)	$N_{min} = \frac{\text{width strip}}{2t}$; $\frac{32.28}{2(8)} = 2$	2	2	2	2	2
		8 #5	6 #4	12 #6	12 #6	6 #4 / 2 #5
	$A_{s,\text{PROVIDED}}$ (in²)	2.48 ✓	1.20 X	5.28 ✓	5.28 ✓	1.82 ✓

(FIGURE A-20) Two-Way Slab Spot Check (4/5)

SPOT CHECK - 2-WAY SLAB	RESIDENCE INN		09.26.08 KMR	5/5

CHECK ρ_{max} FOR $f'_c = 4000$ psi, $f_y = 60$ ksi

TBL. A-4 (TEXTBOOK - "DESIGN OF CONC. STRUCTURES." NILSON, DARWIN, DOLAN, 13TH ED.)

$$\rho_{max} = 0.0206$$

$$\frac{M_u}{\phi b d^2} = R = \rho f_y \left(1 - 0.59 \frac{\rho f_y}{f'_c}\right)$$

$$\Rightarrow d^2 = \frac{M_u}{(0.9)(0.0206)(60000)(12)\left(1-0.59\left(\frac{0.0206(60)}{4}\right)\right)} = \frac{M_u}{10{,}915}$$

Using $M_u = -43.38$ (MOST CRITICAL)

$$\rightarrow d_{min} = \sqrt{\frac{43.38(12000)}{10915}} = 6.91'' > d_{used} = 6.5''$$

$\therefore$ USE $d = 6.91''$

(FIGURE A-21) Two-Way Slab Spot Check (5/5)