Technical Report I – Existing Conditions

Ryan T. Blatz

Structural Option

Thesis Consultant: Dr. Hanagan

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Orange Regional Medical Center

Middletown, NY

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EXECUTIVE SUMMARY

When we peel away the brick façade, the artwork, the landscaping of this six story building, what are we left with? We're left with the intricate structural system of Orange Regional Medical Center, a 600,000 SF hospital in Middletown, NY. This report explores that structural system to determine how the many systems work in unison to defy gravity and lateral forces.

The latest codes were applied to analyze this steel frame, including ASCE7-10 and AISC 14th Edition. An analysis of the lateral forces from seismic and wind revealed that seismic controls in both shear and overturning moment. A seismic 2803.6 kip base shear proves greater than wind's 899.6 kips in the North/South and its 1008.7 kips in the East/West. Wind creates a moment of 44226.8 ft-kips East and West and 48938.6 ft-kips North and South. However, 176281.7 ft-kip tells us that seismic will be the condition to check when analyzing the eccentrically braced frame and concrete shear walls of this hospital. The geometry of this building has created different results than expected. The change in square footage at the third floor increases the gust factor while dropping seismic story shears.

Our spot checks of the composite deck with light weight concrete, beams, girders, and columns all checked out. In quite a few cases, however, the existing systems were over-designed in relation to the analysis methods from this report. We can only make educated guesses to explain these differences now, but these will become areas of interest in the future.

INTRODUCTION

This report explores the structural make-up of Orange Regional Medical Center. Through calculation and research, we will develop a greater understanding of the skeleton of this building, including the framing system, floor slab system, lateral resistance elements, and foundation. By carrying out an analysis of these systems and comparing it to the design of the project engineers, areas of discrepancy will become areas of interest, or perhaps a future thesis proposal. In order to understand these areas of discrepancy, we must understand how the structural system works as a whole, but let us first start with a building overview.

Building Introduction

The first hospital built in New York State in the last twenty-five years, Orange Regional Medical Center, can be found right off of Interstate 17 in the town of Middletown. This giant is 600,000 square feet spread over seven floors (six above grade and one below) and was designed anticipating future additions. As we can see in *Figure 1*, this structure follows a pod design, allowing for future additions to be constructed in the voids on the fifth and sixth floor roofs. We find this feature appearing in several areas throughout the building. For example, this hospital features a removable, full glass façade in multiple locations where future additions may be constructed. Later in this report, we will also see how the structure has been sized to account for these future loads.

***Figure 1**: Pod Construction*

When it comes to the building site, the original design had to be rotated 90 degrees to best fit the site. Although the design works better with the site grading, this change also moved the Emergency Room entrance to the back corner, on the opposite side from the street entrance (See *Figure 2*). This may be taken as an architectural drawback, but this can only be paired with a number of architectural

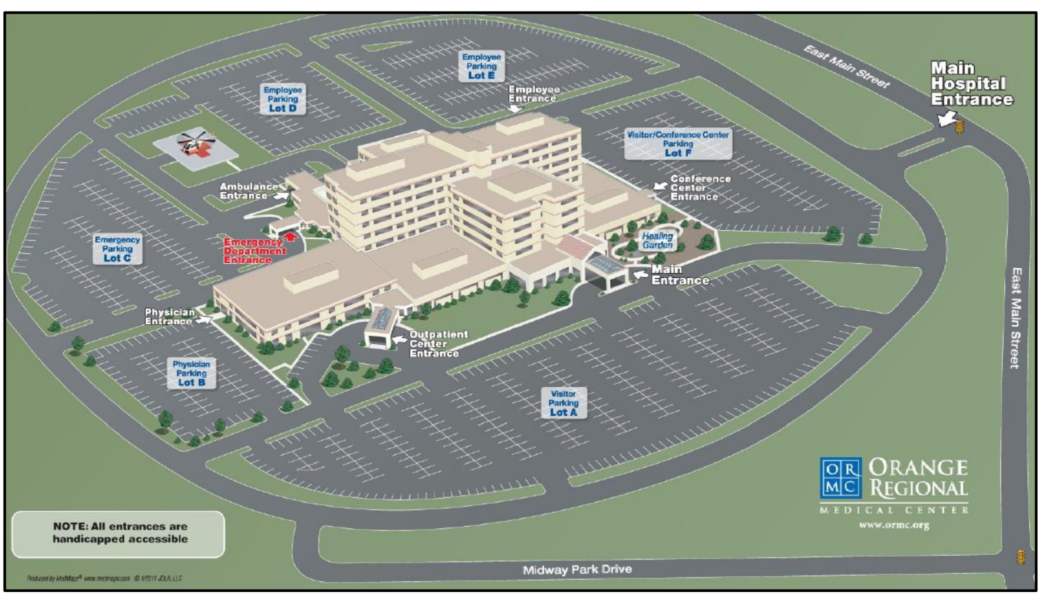


***Figure 2**: Hospital Site and Rotated Plan*

innovations in the healthcare field. Since the hospital's opening in August, patients have enjoyed rooms that rival that of hotels (See *Figure 3*). Carpeted hallways are also among some architectural features aimed at creating a quick recovery by creating comfortable, quiet spaces. Staying on the topic of architecture, this building has essentially been divided into two buildings: a healthcare building and a business administration building, each following a separate set of codes, as we will see later in this report. This separation is not so apparent in the façade, however. Tan brick with red soldier brick accents wrap completely around the building, leaving the EIFS façade of the lobby to stand apart as shown in *Figure 4*. The floor plan is also rather consistent from the second floor up. Each floor is in the shape of a Greek cross with the individual healthcare units branching off of the central elevator core, as seen in *Figure 5*. This not only allows for a uniform structural system, but it also allows first time visitors to be able to navigate the building with ease.

***Top - Figure 3**: Patient Rooms*
***Bottom - Figure 4**: Building Façade*

***Figure 5**: Typical Floor plan*

Framing System

The steel frame of this structure comes in a variety of sizes. On the first floor alone, there are a total of twelve different wide flange beams used, but in general, W16x26's and W16x31's serve as the primary joists throughout the building with an average spacing of about 7 feet and an average span of about 26 feet. W18x35's and W21x44's are the most common choice for girders with spans ranging between 14' 8" and 27' 1". Following the load path to the columns, we find just as much size dispersion. A majority of the columns are W12's with a small grouping of

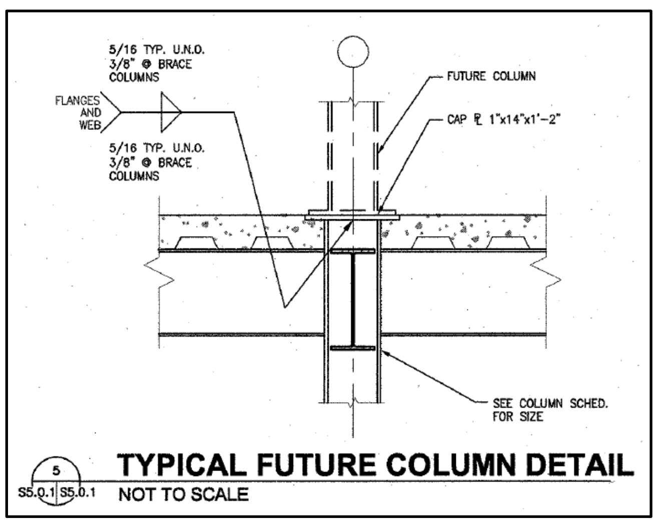


***Figure 6**: Future column specified on column schedule*

W10's and W8's. As mentioned earlier, structural columns for the future additions are also shown on the column schedule (Detail shown in *Figure 6*). Traveling up the building, the columns continue to carry less of the building load and therefore, reduce in size. Typically, each column has two splices occurring just above the second and fourth floors. However, there are special cases where splices occur on the third and fifth floors instead. The structural notes specify that all splice connections must be slip critical connections. Looking further into the frame connections, the structural notes also tell us to "detail steel beam connections as simple span beams, unless noted otherwise." There are only a handful of moment frames specified throughout the building which must be considered as continuous beams.

Lateral Load Resisting Elements

In order to resist the lateral forces from wind and seismic activity, the structure utilizes concrete shear walls on the ground level. From the first floor and above, the lateral forces are then resisted by eccentrically braced steel frame as shown in *Figure 7*.

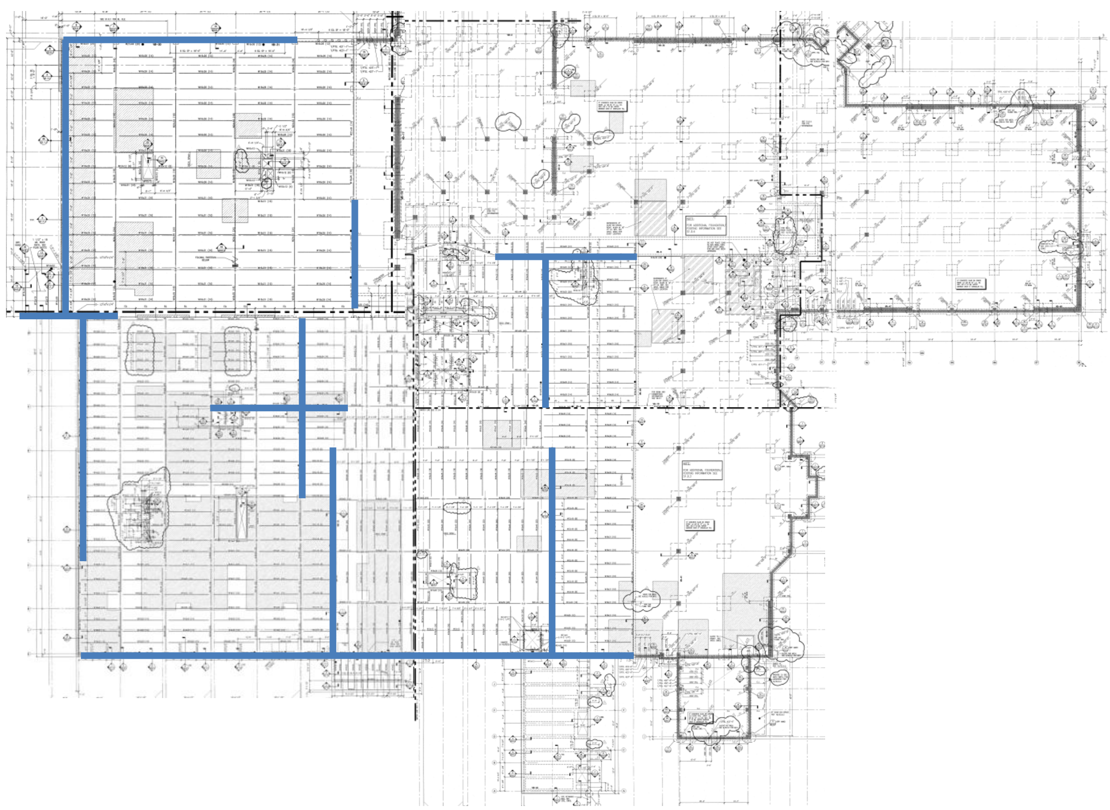


***Figure 7**: Braced Frames Location*

Floor System

Out of the Vulcraft catalog, the floor system of ORMC consists primarily of 2VLI20 composite deck with 3¼" of light weight concrete, making for a total floor thickness of 5¼". The decking runs three spans, perpendicular to the joists, where typical spans are in the range of 7'4". However, as mentioned earlier, the decking may see longer spans due to the lack of bay size uniformity.

Foundations

The foundations are determined by the recommendations of the geotechnical report by Melick-Tully and Associates. Square, concrete spread footings are set on with virgin soil or engineered, compacted soil with a bearing stress of 4000 psi.

General Structural Information

Throughout this report, the primary codes considered through the calculations were ASCE7-10 and AISC-14th Edition. ASCE was used for determining Live Loads and Lateral Loadings, where the Main Wind Force Resisting System (MWFRS) and Equivalent Lateral Force Method (ELF) were used for Wind and Earthquake analysis, respectively. It is important to note that the design team on this project had to follow the codes of New York State. This may contribute to discrepancy in values calculated for this report.

To better acquaint ourselves with the structural steel used throughout this report, refer to *Figure 8* for grades of steel used for the particular structural elements.

2. Materials shall conform to the following, unless noted otherwise.

a.	W's and WT's	ASTM A992
b.	Plates & other shapes	ASTM A36
c.	HSS:	ASTM A500, Grade B
d.	Pipe	ASTM A53, Grade B
e.	Bolts	ASTM A325, or F1852 where indicated, 3/4" diameter (min.), hex head in standard hole U.N.O.
f.	Anchor Rods	ASTM F1554, Grade 36 with washers and heavy hex nuts U.N.O.
g.	Threaded Rod	ASTM A36
h.	Headed Studs	AWS D1.1, Type B
i.	Electrodes	Matching strength, 70 ksi min.

***Figure 8**: Structural Materials*

Load Determination

Gravity Loads

Most loadings used in this report come directly from the codes, such as the live loads. For the purpose of this report, only three lives loads were used, all of which falling under the hospital category. The values shown in *Table 2* are not quite as accurate as the live loads, but by making realistic assumptions for the dead load elements, we are able to design within a reasonable percent error to the actual values. To estimate the dead load contributed by beam self-weight, a random sample, found in Appendix C, was taken to determine the typical size beam in a very diverse structure. Through these efforts, a total building weight was able to be calculated, as shown in Table 2, and applied in the seismic and wind analysis to come later.

Typical Floor Loading	
Component	Weight (psf)
Framing	6.00
Concrete & Decking	62.83
MEP & Misc.	20.00
	88.83
Roof Loading	
Component	Weight (psf)
Metal Roof Deck	2.00
Rigid Insulation	2.00
MEP & Misc.	20
Snow	8.4
	32.40

***Table 1**: Floor and Roof Gravity Loads*

Floor Loading			
Floor	SF	Loading (psf)	Floor Weight (k)
Ground	95676.14	60.42	5780.43
1	172143.54	88.83	15291.51
2	100166.97	88.83	8897.83
3	68865.15	88.83	6117.29
4	68865.15	88.83	6117.29
5	49774.58	88.83	4421.48
6	48782.31	88.83	4333.33
Roof	95676.14	32.40	3099.91
	604273.84		54059.07
Façade Loading			
Floor	Perimeter	Height	Weight on Floor
Ground	1307.90	8.00	397.60
1	1681.46	14.50	926.48
2	1276.00	13.00	630.34
3	1101.57	13.00	544.18
4	1101.57	13.00	544.18
5	1044.21	13.00	515.84
6	1039.21	13.25	523.24
Roof	1039.21	6.75	266.56
			4348.42
		Floor Load	54059.07
		Total Weight	58407.49

***Table 2**: Total Building Weight*

Gravity played an interesting role in the analysis of the building's snow load. Although we arrived at a reasonable flat load value of 42 psf, the drift value seems a little high. Our issue stems from the large roof drop from the sixth floor roof to the second floor roof where there is also a large l_u factor. Following the code, we arrive at 149.45 psf, but thinking about it realistically; any snow falling 52 ft will more than likely get blown about before it hits the lower roof. Therefore, to say that all snow will accumulate at the lower level seems unrealistic. Either way, drift loads should be accounted for in any snow load calculations, such as beam checks, since this increased loading will create a load imbalance, putting more stress on our structural system. For full snow load calculations, refer to Appendix A.

Wind Loads

Although wind applies a pressure to the building façade, the actual force is resisted internally once the force makes its way through the floor diaphragm and into the lateral elements. Therefore, since we will soon look to investigate lateral design further, it is important that we analyze wind's effects in this report. To do this, the shape of Orange Regional Medical Center first had to be simplified. *Figure 9* shows the simplified shape broken into and upper and lower section to better fit the building dimensions. This separation creates four different gust factors which all have a different effect on the building as we will see in the pressure diagram.

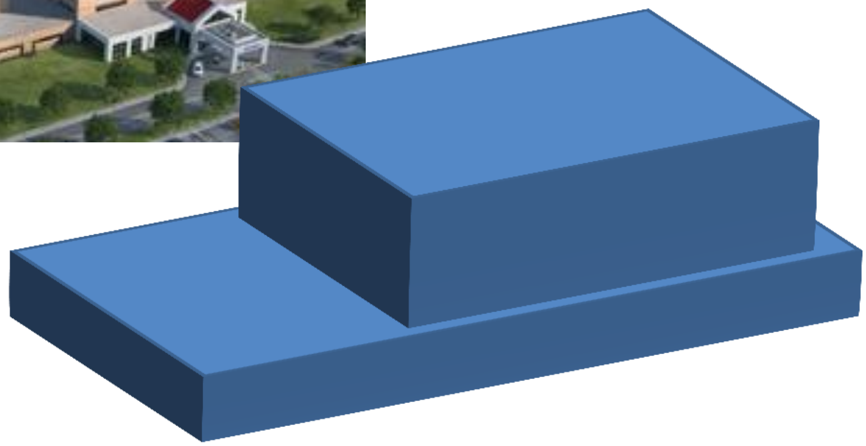


Figure 9: *Simplified Shape for Wind Analysis*

There was one discrepancy that emerged at the start of the wind analysis. The basic wind speed from ASCE7-10 for our design delivers a value of 120 mph, where the original drawings call for 90 mph. Since this is not calculation based, we can only assume that this difference comes from the difference in codes. New York State codes may allow a lower value for Middletown, NY. Despite this, the analysis still provided reasonable values as we can see in Tables 3 and 4 for the East/West and North/South directions. We arrived at the base shears and overturning moments shown in *Table 5*. The following figures (Figures 9 and 10) display how the pressures are distributed along the face of the building, and we can see how the change in the shape and gust factor creates different pressures along that face. For further wind calculations, see Appendix B.

Wind Pressures - North/South										
Floor	z	K_z	q_z	$p_{Windward}$ (psf)	WW (plf)	WW (k)	q_h	$p_{Leeward}$ (psf)	LW (plf)	LW (k)
Ground	0	0.85	26.63	18.1	145.1	70.8	39.32	-15.7	-125.8	-61.4
1	16	0.86	26.95	18.3	293.5	143.2	39.32	-15.7	-251.7	-122.8
2	32	0.99	31.08	21.2	306.8	149.7	39.32	-15.7	-228.1	-111.3
3	45	1.07	33.37	23.3	302.3	108.5	39.32	-16.4	-213.7	-76.7
4	58	1.12	35.16	24.5	318.5	114.3	39.32	-16.4	-213.7	-76.7
5	71	1.17	36.79	25.6	333.2	119.6	39.32	-16.4	-213.7	-76.7
6	84	1.22	38.29	26.7	353.5	126.9	39.32	-16.4	-217.8	-78.2
Roof	97.5	1.26	39.32	27.4	185.0	66.4	39.32	-16.4	-111.0	-39.8

Table 3: *North/South Wind Pressures*

Wind Pressures - East/West										
Floor	z	K_z	q_z	$p_{Windward}$ (psf)	WW (plf)	WW (k)	q_h	$p_{Leeward}$ (psf)	LW (plf)	LW (k)
Ground	0	0.85	26.63	17.9	143.4	81.9	39.32	-15.5	-124.4	-71.1
1	16	0.86	26.95	18.1	290.1	165.8	39.32	-15.5	-248.7	-142.1
2	32	0.99	31.08	20.9	303.2	173.3	39.32	-15.5	-225.4	-128.8
3	45	1.07	33.37	23.1	300.2	119.0	39.32	-16.3	-212.3	-84.2
4	58	1.12	35.16	24.3	316.3	125.4	39.32	-16.3	-212.3	-84.2
5	71	1.17	36.79	25.5	330.9	131.2	39.32	-16.3	-212.3	-84.2
6	84	1.22	38.29	26.5	351.1	139.2	39.32	-16.3	-216.3	-85.8
Roof	97.5	1.26	39.32	27.2	183.7	72.8	39.32	-16.3	-110.2	-43.7

***Table 4**: East/West Wind Pressure*

Shear	Moment
North/South	
70.8	0
143.2	2291.918
149.7	4791.709
108.5	4883.48
114.3	6631.14
119.6	8493.638
126.9	10660.81
66.4	6474.051
899.6	44226.75
East/West	
81.9	0
165.8	2652.54
173.3	5545.66
119.0	5356.439
125.4	7273.356
131.2	9316.236
139.2	11693.3
72.8	7101.053
1008.7	48938.58

***Table 5**: Wind Base Shear/ Overturning Moment*

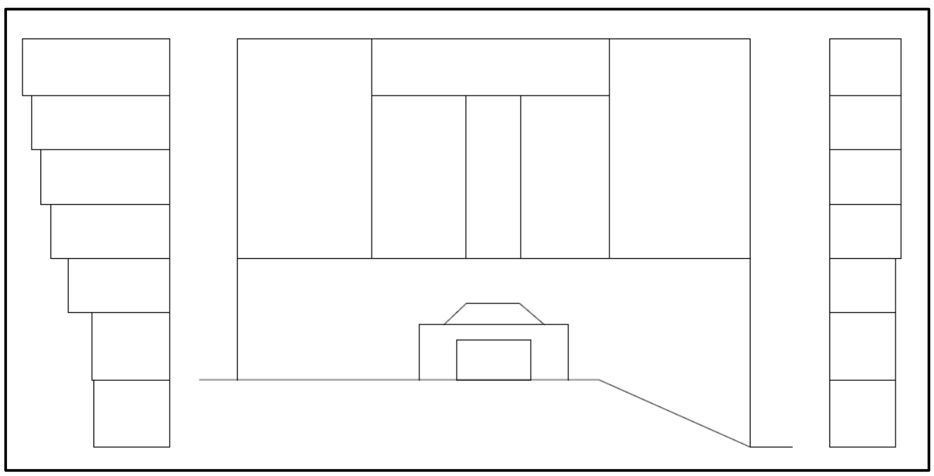


***Figure 9**: North/ South Wind Pressure*

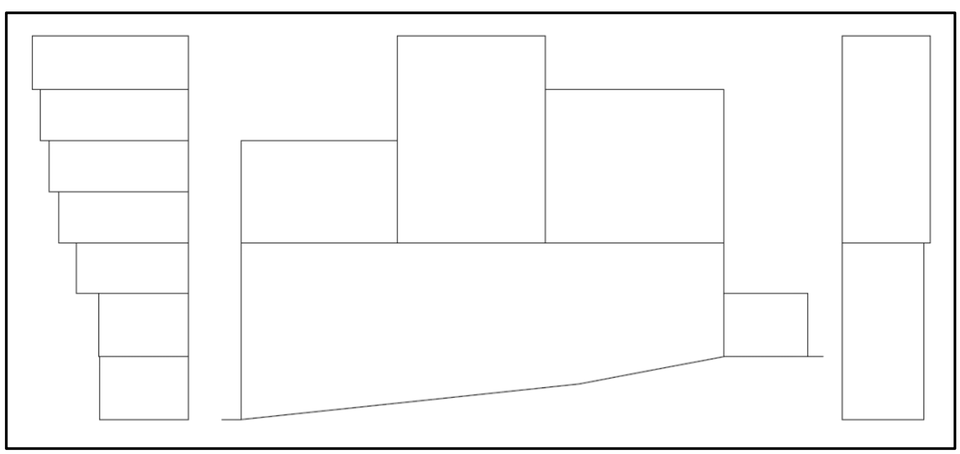


***Figure 10**: East/West Wind Pressure*

Seismic Loads

Equivalent Lateral Force Method was used to determine the seismic forces, from the individual story forces, to the base shear, to the overturning moment. The analysis in this report follows right along with the results from the structural drawings. The only discrepancy was arriving at category A for the seismic design category. However, this was paired with class C derived from table 11.6-2, so we chose the higher category, C, to be more conservative. So much of the seismic forces are dependent on building weight, so as we mentioned earlier, these values were determined using actual values and educated approximations. In fact, floor weights may be the answer to the discrepancies in *Figure 11*, which shows the seismic story forces. In most cases, we expect to see a nice curving story force as we climb the building, but from the analysis in this report, we find jumps between stories. Since story forces are proportional to story height and weight, these jumps must be credited to the fact that changes in floor geometry create floors of varying weights. In the end, we determined that ORMC has a base shear of 2,803.6 kips and an overturning moment of 176,281.7 ft-kips, which seems reasonable. *Table 6* shows how we arrived at these values, but for further calculations, check Appendix C.

Seismic Loads							
Floor	Weight (k)	Height (ft)	$w_x h_x^k$	C_{vx}	F_x (k)	V_x (k)	M (ft-k)
Roof	3099.9	97.5	827816.9	0.2	450.0	450.0	43870.1
6	4333.3	84.0	964812.1	0.2	524.4	974.4	44050.4
5	4421.5	71.0	801867.2	0.2	435.8	1410.2	30945.4
4	6117.3	58.0	866844.7	0.2	471.2	1881.4	27327.3
3	6117.3	45.0	636031.2	0.1	345.7	2227.1	15557.0
2	8897.8	32.0	610333.4	0.1	331.7	2558.8	10615.7
1	15291.5	16.0	450273.9	0.1	244.7	2803.6	3915.8
Ground			5157979.5		2803.6		176281.7

***Table 6**: Seismic Calculations*

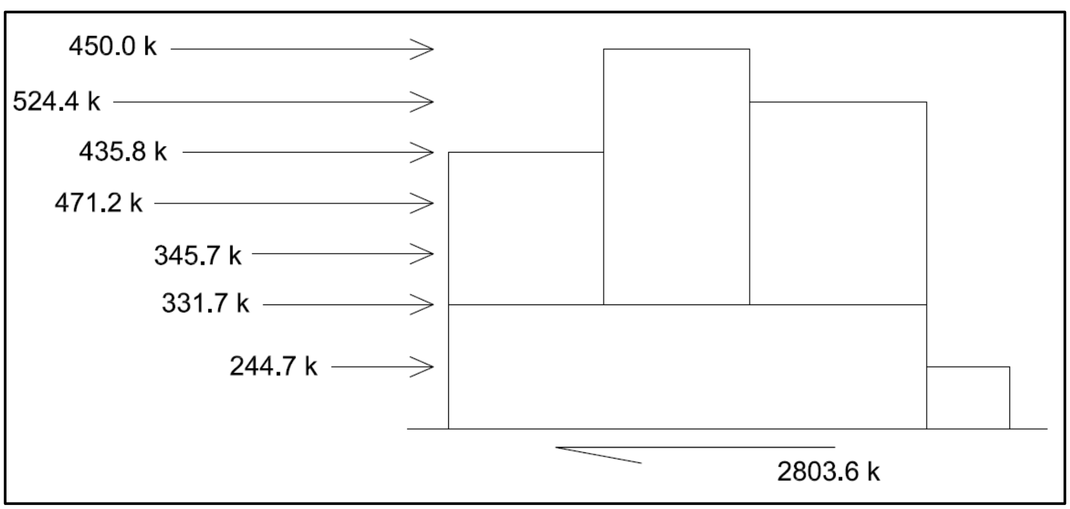


***Figure 11**: Seismic Story Forces*

System Evaluation

Typical Floor System

All checks in this report worked for the floor system. However, the floor deck is significantly over designed. This could be due to one of three things: this deck was chosen to achieve the 2 hour fire rating, regardless of loading, for constructability purposes where there may be longer spans, or this deck was chosen for serviceability reasons. At a hospital where patients are being rolled back and forth in stretchers all day, it probably is a good idea to design for vibration. Therefore, the deck may be oversized to account for vibrational dampening. To view the check calculations, refer to Appendix D.

Typical Beam and Girder

Values for the check came relatively close to actual values. The beam checks out okay and is reasonably close, where the girder also checks out but is a little over-designed. Again, I am claiming this is for serviceability reasons in an attempt to dampen vibrations.

Typical Columns

Both columns pass the spot check, with the interior column coming pretty close to the actual value. However, as with the other structural members, one is always a little over-designed. The exterior column may be accounting for the future additions, but I am unsure why we would see a greater difference in the exterior than the interior.

Conclusions

From the calculations performed in this report, we have achieved a greater understanding of Orange Regional Medical Center and its structural components. Although the actual building was designed to a different set of codes, by using ASCE7-10 and AISC we were able to find areas of discrepancy and determine if these differences were substantial or not.

We saw a difference in numbers for the composite floor deck, the girder, and exterior column. At this point, we can assume this is either for serviceability or this is compensating for future loads. As we continue our work with these buildings, we will begin to understand the true differences and perhaps explore them as a thesis proposal. At this point, vibrations may be one of those areas.

Appendix A: Snow Calculations

APPENDIX A | SNOW CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA - ASCE 7-10

$C_e = 1.0$ (TABLE 7-2) LOWER SECTION - PARTIALLY EXPOSED
$C_e = 1.0$ (TABLE 7-2) UPPER SECTION - PARTIALLY EXPOSED

$C_t = 1.0$ (TABLE 7-3)

$I_s = 1.20$ (TABLE 1.5-2)

$p_g = CS$ (FIGURE 7-1) $\longrightarrow$ $p_g = 50$ psf (FROM DRAWINGS)

$p_f = 0.7(1.0)(1.0)(1.20)(50) = 42$ psf

SNOW DRIFTS

$\gamma = 0.13(50) + 14 \leq 30$
$\gamma = 20.5$ pcf ≤ 30 ✓

- DRIFT ONTO FIFTH FLOOR ROOF

$l_u = 117'$
$h_c = 13.5'$

$h_d = 0.43 \sqrt[3]{117}\sqrt[4]{50+10} - 1.5 = 4.35'$
$w = 4h_d = 4(4.35) = 17.4'$

$P_d = (4.35)(20.5) = 89.18$ psf

- DRIFT ONTO SECOND FLOOR ROOF - NORTH/SOUTH

$l_u = 396'\ 7\,1/4''$

$h_d = 0.43 \sqrt[3]{396.6}\sqrt[4]{50+10} - 1.5 = 7.29'$
$w = 4(7.29) = 29.2'$

$P_d = (7.29)(20.5) = 149.45$ psf

- DRIFT ONTO SECOND FLOOR ROOF - EAST/WEST

$l_u = 213'\ 4''$

$h_d = 0.43 \sqrt[3]{213.3}\sqrt[4]{50+10} - 1.5 = 5.65'$
$w = 4(5.65) = 22.6'$

$P_d = (5.65)(20.5) = 115.84$ psf

Appendix B: Wind Calculations

APPENDIX B | WIND CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA – ASCE 7-10

BASIC WIND SPEED (FIGURE 26.5-1B): $V = 120$ mph

RISK FACTOR (TABLE 1.5.1): IV ESSENTIAL FACILITY

WIND DIRECTIONALITY FACTOR (TABLE 26.6-1): $K_d = 0.85$

EXPOSURE CATEGORY (SECTION 26.7.3): EXPOSURE C

TOPOGRAPHIC FACTOR (SECTION 26.8): DOES NOT APPLY, $K_{zt} = 1.0$

GUST FACTOR: SEE ATTACHED CALCULATIONS

- RIGIDITY CALCULATION

$$L_{eff} = \frac{16(488') + 32(359') + 45(359') + 58(359') + 71(249') + 84(145') + 97.5(145')}{16 + 32 + 45 + 58 + 71 + 84 + 97.5}$$

$L_{eff} = 248.5'(4) = 994' \gg 97.5'$ ⟶ CALCULATE n USING SECTION 26.9.3

$n_a = 75/h = 75/97.5 = 0.769$ Hz < 1.0 Hz ∴ STRUCTURE NOT CONSIDERED RIGID

$g_Q = 3.4$ $\quad g_v = 3.4$ $\quad g_R = \sqrt{2\ln(3600(.769))} + \dfrac{0.577}{\sqrt{2\ln(3600(.769))}}$

$g_R = 4.13$

1) GUST CALCULATION – EAST/WEST BOTTOM SECTION

$\bar{b} = 0.65$ $\quad \bar{\alpha} = 1/6.5 = 0.154$ $\quad \bar{V}_{\bar{z}} = 0.65\left(\frac{58.5}{33}\right)^{1/6.5}\left(\frac{88}{60}\right)(120) = 124.93$

$\bar{z} = 0.6h = 0.6(97.5) = 58.5 > 15$ ✓

$\ell = 500$ ft $\quad \bar{\epsilon} = 1/5.0$ $\quad L_{\bar{z}} = 500\left(\frac{58.5}{33}\right)^{1/5} = 560.66$

$R_n = \dfrac{7.47(3.45)}{(1 + 10.3(3.45))^{5/3}} = 0.064$ $\quad N_1 = \dfrac{0.769(560.66)}{124.93} = 3.45$

$R_h = \dfrac{1}{2.76} - \dfrac{1}{2(2.76)^2}\left(1 - e^{-2(2.76)}\right) = 0.297$ $\quad \eta_h = \dfrac{4.6(.769)(97.5)}{124.93} = 2.76$

$R_B = \dfrac{1}{16.18} - \dfrac{1}{2(16.18)^2}\left(1 - e^{-2(16.18)}\right) = 0.060$ $\quad \eta_B = \dfrac{4.6(.769)(571.5)}{124.93} = 16.18$

$R_L = \dfrac{1}{46.26} - \dfrac{1}{2(46.26)^2}\left(1 - e^{-2(46.26)}\right) = 0.021$ $\quad \eta_L = \dfrac{15.4(.769)(488)}{124.93} = 46.26$

Appendix B: Wind Calculations

APPENDIX B | WIND CALCULATIONS 2 | RYAN BLATZ

$\beta = 1.0\%$ AS RECOMMENDED IN ASCE7-10, pg. 521

$$R = \sqrt{(1/.01)(.064)(.297)(.06)(.53+.47(.021))} = 0.248$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{571.5+97.5}{560.66}\right)^{0.63}}} = 0.766$$

$$I_{\bar{z}} = 0.2\left(\frac{33}{58.5}\right)^{1/6} = 0.182 \qquad c = 0.20 \quad \text{TABLE 26.9-1}$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.766)^2+(4.13)^2(.248)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.841}$$

2) GUST CALCULATION - EAST/WEST TOP SECTION

- ALL CALCULATIONS NOT SHOWN ARE THE SAME AS PREVIOUS SECTION

$$R_B = \frac{1}{11.23} - \frac{1}{2(11.23)^2}\left(1-e^{-2(11.23)}\right) = 0.085 \qquad \eta_B = \frac{4.6(.769)(396.5)}{124.93} = 11.23$$

$$R_L = \frac{1}{34.03} - \frac{1}{2(34.03)^2}\left(1-e^{-2(34.03)}\right) = 0.029 \qquad \eta_L = \frac{15.4(.769)(359)}{124.93} = 34.03$$

$$R = \sqrt{(1/.01)(.064)(.297)(.085)(.53+.47(.029))} = 0.296$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{396.5+97.5}{560.66}\right)^{0.63}}} = 0.775$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.775)^2+(4.13)^2(.296)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.865}$$

3) GUST CALCULATION - NORTH/SOUTH BOTTOM SECTION

$$R_B = \frac{1}{13.82} - \frac{1}{2(13.82)^2}\left(1-e^{-2(13.82)}\right) = 0.070 \qquad \eta_B = \frac{4.6(.769)(488)}{124.93} = 13.82$$

$$R_L = \frac{1}{54.17} - \frac{1}{2(54.17)^2}\left(1-e^{-2(54.17)}\right) = 0.018 \qquad \eta_L = \frac{15.4(.769)(571.5)}{124.93} = 54.17$$

$$R = \sqrt{(1/.01)(.064)(.297)(.07)(.53+.47(.018))} = 0.268$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{488+97.5}{560.66}\right)^{0.63}}} = 0.779$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.779)^2+(4.13)^2(.268)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.851}$$

Appendix B: Wind Calculations

APPENDIX B | WIND CALCULATIONS 3 | RYAN BLATZ

4) GUST CALCULATION - NORTH/SOUTH TOP SECTION

$$B_B = \frac{1}{10.17} - \frac{1}{2(10.17)^2}\left(1-e^{-2(10.17)}\right) = 0.093 \qquad \eta_B = \frac{4.6(.769)(359)}{124.93} = 10.17$$

$$B_L = \frac{1}{37.59} - \frac{1}{2(37.59)^2}\left(1-e^{-2(37.59)}\right) = 0.026 \qquad \eta_L = \frac{15.4(396.5)(.769)}{124.93} = 37.59$$

$$B = \sqrt{(1/.01)(.064)(.297)(.093)(.53+.47(.026))} = 0.310$$

$$Q = \sqrt{\frac{1}{1+.63\left(\frac{359+97.5}{560.66}\right)^{0.63}}} = 0.802$$

$$G_f = 0.925\left(\frac{1+1.7(.182)\sqrt{(3.4)^2(.802)^2+(4.13)^2(.31)^2}}{1+1.7(3.4)(.182)}\right) = \boxed{0.871}$$

MAIN WIND FORCE RESISTING SYSTEM (MWFRS) - DIRECTIONAL PROCEDURE

ENCLOSURE CLASSIFICATION: ENCLOSED, $GC_{pi} = \pm 0.18$ • DO NOT NEED

WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.5$ EAST/WEST; NORTH/SOUTH: $C_p = -0.47$ BOTTOM, $C_p = -0.48$ TOP

SIDE WALL: $C_p = -0.7$

• THE REMAINDER IS CALCULATED USING AN EXCEL SPREADSHEET; SEE ATTACHED

Appendix C: Seismic Calculations

APPENDIX C | SEISMIC CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA – ASCE7-10

SITE CLASS: C (FROM GEOTECHNICAL REPORT)

RISK CATEGORY (TABLE 1.5.1): IV ESSENTIAL FACILITY

IMPORTANCE FACTOR (TABLE 1.5-2): $I_e = 1.50$

$S_s = 0.20$ (FIGURE 22-1) $S_1 = 0.06$ (FIGURE 22-2)

$F_a = 1.2$ (TABLE 11.4-1) $F_v = 1.7$ (TABLE 11.4-2)

$S_{MS} = (1.2)(0.20) = 0.24$ $S_{M1} = (1.7)(0.06) = 0.102$

$S_{DS} = 2/3 S_{MS} = 2/3(0.24) = 0.16$ $S_{D1} = 2/3 S_{M1} = 2/3(0.102) = 0.068$

SEISMIC DESIGN CATEGORY: A (TABLE 11.6-1), C (TABLE 11.6-2) — USE HIGHER CATEGORY CLASS C

RESPONSE MODIFICATION COEFFICIENT (TABLE 12.2-1): $R = 5$

- STEEL AND CONCRETE COMPOSITE ORDINARY SHEAR WALLS

EQUIVALENT LATERAL FORCE METHOD (ELF)

$T_a = C_t h_n^x = (0.03)(97.5)^{0.75} = 0.731\ s$ $C_t = 0.03$ (TABLE 12.8-2), $x = 0.75$ (TABLE 12.8-2)

$C_s = \frac{0.16}{(5/1.5)} = 0.048$

$V = C_s W = (0.048)(58407.49) = 2803.56$ kips

$F_x = C_{vx} V$, $C_{vx} = \frac{w_x h_x^k}{\sum_{i=1} w_i h_i^k}$ } COMPUTED IN TABLE $k = 1.22$ (SECTION 12.8.3)

Appendix C: Seismic Calculations

Beam Sample - From 16,267.2 SF Sample Area				
Beam Type	Unit Weight	# of linear feet	Weight (kips)	# of Beams
W12x19	19 plf	42.2	0.8018	2
W14x22	22 plf	16	0.352	1
W14x30	30 plf	42.2	1.266	2
W16x26	26 plf	1413.8	36.7588	56
W16x31	31 plf	683.9	21.2009	26
W16x36	36 plf	52.8	1.9008	2
W18x35	35 plf	293.5	10.2725	14
W21x44	44 plf	54.4	2.3936	2
W21x50	50 plf	31	1.55	1
W24x55	55 plf	154.1	8.4755	6
W24x62	62 plf	28	1.736	1
W24x76	76 plf	150.5	11.438	5
		SUM:	98.1459	118
BEAM WEIGHT CONTRIBUTION:		98,145.9 lbs / 16,267.2 SF = 6.0 psf		

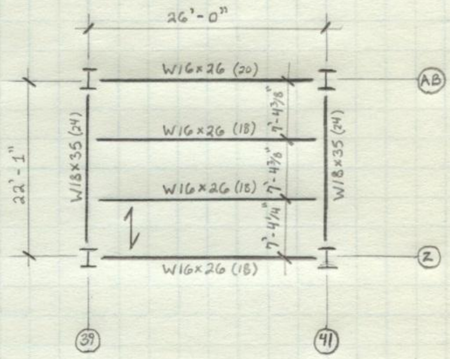
Appendix D: Spot Check – Decking

APPENDIX D | SPOT CHECK - DECKING | RYAN BLATZ

NOTE: THERE ARE NOT MANY BAYS WITH THE SAME DIMENSIONS AND SPACING BUT THESE SPOT CHECK CALCULATIONS USE THE MOST TYPICAL BAY.

FLOOR 6 - FLOOR CONSTRUCTION

- 2" COMPOSITE DECK (20 GAGE)
- 3 1/4" LWT, 3000 psi CONCRETE
- t_{TOTAL} = 5 1/4"



TYPICAL FLOOR LOADING

- LL = 80 psf (CORRIDOR ABOVE 1st FLOOR)
- DL = 20 psf (MEP AND MISC.)

  100 psf

FROM VULCRAFT CATALOG FOR 2VLI20

UNSHORED CLEAR SPAN (3 SPAN) = 10'-0" > 7'-4 3/8" ok ✓

SUPERIMPOSED LL AT 7'-6" CLEAR SPAN = 263 psf > 100 psf ok ✓

Appendix D: Spot Check – Beams

APPENDIX D | SPOT CHECK – BEAMS 1 | RYAN BLATZ

CHECKED AGAINST AISC STEEL MANUAL – 14^{th} EDITION

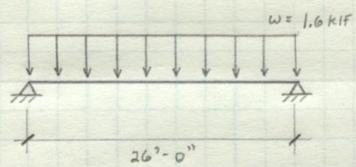
COMPOSITE BEAM W16×26 (18) : $F_y = 50$ ksi , $A = 7.68$ in^2 , $I_x = 301$ in^4

TYPICAL BEAM LOADING

- LL = 80 psf (CORRIDOR ABOVE 1^{st} FLOOR)
- DL = 20 psf (MEP AND MISC.)
  50.7 psf (COMPOSITE DECK W/ LW CONCRETE)
- SELF WT = 26 plf

$T_{WIDTH} = 7'\text{-}4\frac{3}{8}" = 7.36'$

$$\omega = 1.2\left[(20+50.7)(7.36)+26\right] + 1.6\left[(80)(7.36)\right] = 1.6 \text{ klf}$$



- GENERAL NOTES FROM DRAWING CALL FOR PIN CONNECTIONS

$$V_u = \frac{(1.6)(26)}{2} = 20.8 \text{ kips}$$

(TABLE 3-2)
$\phi V_n = 106$ kips > 20.8 kips ok ✓

$$M_u = \frac{(1.6)(26)^2}{8} = 135.2 \text{ ft}\cdot\text{kips}$$

CHECK COMPOSITE ACTION

$$b_{eff} = \min\begin{cases} 26/4 = 6.5' \leftarrow \text{CONTROLS} \\ 7.36 \end{cases}$$

$$a = \frac{96}{0.85(3)(6.5(12))} = 0.48 < 1.0 \leftarrow \text{CONTROLS}$$

(TABLE 3-19)
$\Sigma Q_n = 96.0$ @ PNA = 7

$$Y_2 = 5.25 - \tfrac{1}{2} = 4.75$$

$\phi M_n = 242.5$ ft·kips > 135.2 ft·kips ok ✓

(TABLE 3-21)
$Q_n = \dfrac{96.0}{17.2} = 5.58 \approx 6$ FOR HALF LENGTH

12 STUDS MIN. < 18 STUDS ok ✓

CHECK DEFLECTION

$$\Delta_{LL}: \frac{5\omega l^4}{384EI} < \frac{l}{360}$$

$I_{LB} = 545$ in^4 (TABLE 3-20)

$$\omega_L = \frac{80(7.36)}{1000} = 0.59 \text{ klf}$$

$$\frac{5(0.59)(26)^4(1728)}{384(29000)(545)} < \frac{(26)(12)}{360}$$

$0.384 < 0.867$ ok ✓

Appendix D: Spot Check – Beams

APPENDIX D | SPOT CHECK – BEAMS 2 | RYAN BLATZ

FIND I_{REQ} FOR WET CONCRETE DEFLECTION

$$\Delta_{MAX} = \frac{(26)(12)}{240} = 1.3 \text{ in} \qquad \omega = \frac{(50.7(7.36) + 26)}{1000} = 0.40 \text{ klf}$$

$$1.3 = \frac{5 \omega \ell^4}{384 E I_{REQ}} = \frac{(5)(0.4)(26)^4(1728)}{384(29000) I_{REQ}}$$

$$I_{REQ} = 109.1 \text{ in}^4 < 301 \text{ in}^4 \quad \text{ok} \checkmark$$

Appendix D: Spot Check – Girder

APPENDIX D | SPOT CHECK – GIRDER | RYAN BLATZ

CHECKED AGAINST AISC STEEL MANUAL – 14th EDITION

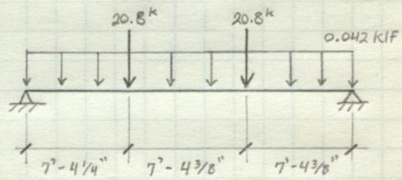
COMPOSITE GIRDER W18×35 (24): $F_y = 50$ ksi, $A = 10.3$ in^2, $I_x = 510$ in^4

TYPICAL GIRDER LOADING

- P = 20.8 kips (FROM JOISTS)
- $\omega = \frac{1.2(35)}{1000} = 0.042$ klf (SELF WEIGHT)

$V_u = 20.8 + 0.042(22.1)/2 = 21.3$ kips

$M_u = 20.8(7.36') + \frac{.042(22.1)^2}{8} = 155.7$ ft·kips



CHECK COMPOSITE ACTION

$b_{eff} = \min \begin{cases} 22.1/4 = 5.53 \longleftarrow \text{CONTROLS} \\ 26 \end{cases}$

$\Sigma Q_n = 129^k$ (TABLE 3-19) PNA = 7

$a = \frac{129}{(0.85)(3)(5.53(12))} = 0.76 < 1.0 \longleftarrow$ CONTROLS

$y_2 = 5.25 - 1/2 = 4.75$

$\phi M_n = 360.5$ ft·k > 155.7 ft·k ok ✓

$\phi V_n = 159^k > 21.3^k$ ok ✓

CHECK DEFLECTION

$P_L = \frac{(80)(7.36)(26)}{2(1000)} = 7.65$ kips

$\Delta_{LL}: \frac{5\omega l^4}{384EI} + \frac{Pl^3}{48EI} < \frac{l}{360}$

$I_{LB} = 892$ in^4

$0 + \frac{7.65(22.1)^3(1728)}{48(29000)(892)} < \frac{22.1(12)}{360}$

0.115 in < 0.737 in ok ✓

FIND I_{REQ} FOR WET CONCRETE DEFLECTION

$\Delta_{MAX} = \frac{22.1(12)}{240} = 1.1$ in

$P = (0.4)(26) = 10.4$

$I_{REQ} = \frac{5(.035)(22.1)^4(1728)}{384(29000)(1.1)} + \frac{(10.4)(22.1)^3(1728)}{48(29000)(1.1)}$

$I_{REQ} = 132.6$ in^4 < 510 in^4 ok ✓

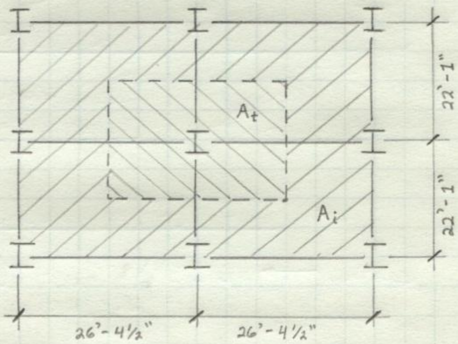
$Q_n = \frac{129^k}{17.2} = 7.5 \approx 8$ FOR HALF LENGTH

16 STUDS < 24 STUDS ok ✓

Appendix D: Spot Check – Column

APPENDIX D | SPOT CHECK – COLUMN 1 | RYAN BLATZ

COLUMN AB36 : W12 × 87 ANALYZED AT GROUND FLOOR



INTERIOR COLUMN

$A_t = 582.4\ ft^2$

$A_i = 2329.8\ ft^2 > 400\ ft^2$ ∴ REDUCIBLE

$DL = (88.83\ psf)(582.4)(6\ \text{FLOORS}) + (24\ psf)(582.4) = 324.4\ kips$

$S = (42\ psf)(582.4) = 24.5\ kips$

* NO DRIFT ON UPPER ROOF

$LL = (100\ psf)(582.4) + (44.9\ psf)(582.4)(5) = 189.0\ kips$

* CORRIDOR THROUGH BAY ON EACH FLOOR

$L = 80\left(0.25 + \dfrac{15}{\sqrt{2329.8}}\right)$

$L = 44.86 > .4L_o$ ✓

$DL + COL.\ WT = 324.4 + \dfrac{53(37.5)}{1000} + \dfrac{58(26)}{1000} = 328.0\ kips$

LOAD COMBINATIONS

$1.4D = 1.4(328.0) = 459.2\ kips$

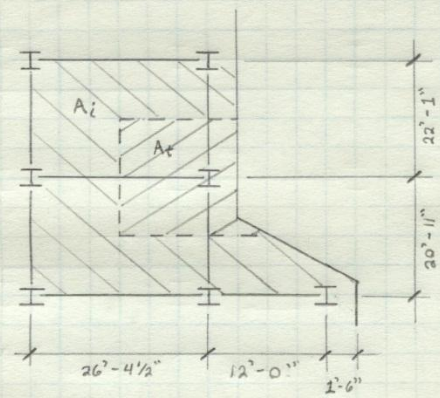
$1.2D + 1.6L + 0.5S = 1.2(328.0) + 1.6(189) + 0.5(24.5) = 708.3\ kips$ ← CONTROLS

AT $KL = 16\ ft$, $\phi P_n = 865\ kips > 708.3$ ok ✓

Appendix D: Spot Check – Column

APPENDIX D | SPOT CHECK – COLUMN 2 | RYAN BLATZ

COLUMN Z43: W12×53 ANALYZED AT FIRST FLOOR



EXTERIOR COLUMN

$A_t = (13.2 + 1.5)(10.5 + 11.0) = 316.1\ ft^2$

$A_i = (27.9)(43) + \frac{1}{2}(12)(11.2) = 1266.9\ ft^2$
$> 400\ ft^2$ ✓

$DL = (88.83)(316.1)(5) + (24)(316.1) + (38)(21.5)(73.5) = 208.0$ kips

$S = (42)(316.1) = 13.3$ kips

$LL = (26.9)(316.1) + (40.3)(316.1)(4) = 57.5$ kips

BRICK FACADE: 38 psf

- 2nd, 4th, 5th, 6th FLOORS – OPERATING ROOMS

$L = 60\left(0.25 + \frac{15}{\sqrt{1266.9}}\right)$

$L = 40.3\ psf > 0.4L_o$ ✓

$DL + COL.\ WT = 208 + \frac{40(37.5)}{1000} + \frac{45(26)}{1000}$

$DL + COL.\ WT = 210.8$ kips

- 3rd FLOOR – PATIENT ROOMS

$L = 40\left(0.25 + \frac{15}{\sqrt{1266.9}}\right)$

$L = 26.9\ psf > 0.4L_o$ ✓

LOAD COMBINATIONS

$1.4D = 1.4(210.8) = 295.1$ kips

$1.2D + 1.6L + 0.5S = 1.2(210.8) + 1.6(57.5) + 0.5(13.3) = 354.8$ kips ⟵ CONTROLS

AT $KL = 13ft$, $\phi P_n = 526$ kips > 354.8 kips ok ✓