Technical Report III – Existing Conditions, Alternate Floor Systems, & Lateral System Analysis

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Structural Option

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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 49172 ft-kips East and West and 44393 ft-kips North and South. However, 295197 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.

In the second part of this report, the lateral frames of the existing structure were analyzed. The distribution of lateral story forces was determined through relative stiffness and torsional shear calculations. The results proved that the existing structure is adequate to carry these loads, but the calculations produce lower member sizes than that of the existing frames. This may be due to the deliberate addition of redundancy, seeing that seismic loading in the East and West direction was the predominant lateral load at every story. The existing frame sizes allowed for story drifts to fall within the acceptable limits for wind and seismic, as well.

Continuing analysis on the effects of gravity loads, three alternative floor systems were explored in addition to the existing system. These four systems are the existing one-way composite, one-way precast hollow core planks, one-way non-composite, and two-way flat slab with drop plates. Through some quick preliminary calculations it was determined that both the precast planking system and non-composite system were not viable for this application. The 4' module size of the planks pushed for limited bay sizes and less plenum space, and the weight of the planks puts much more stress on the foundations. The non-composite system proved slightly more expensive than the existing system for about 3.5" more plenum space and no other notable benefits, so it didn't seem to be a better option. The alternative that does grant further research is the flat slab system. For a longer construction ould achieve a lighter, shallower system at $1.5 million less. This is a decision that would ultimately be made by the owner, but more detailed calculations would have to be made first before calling this the better option.

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

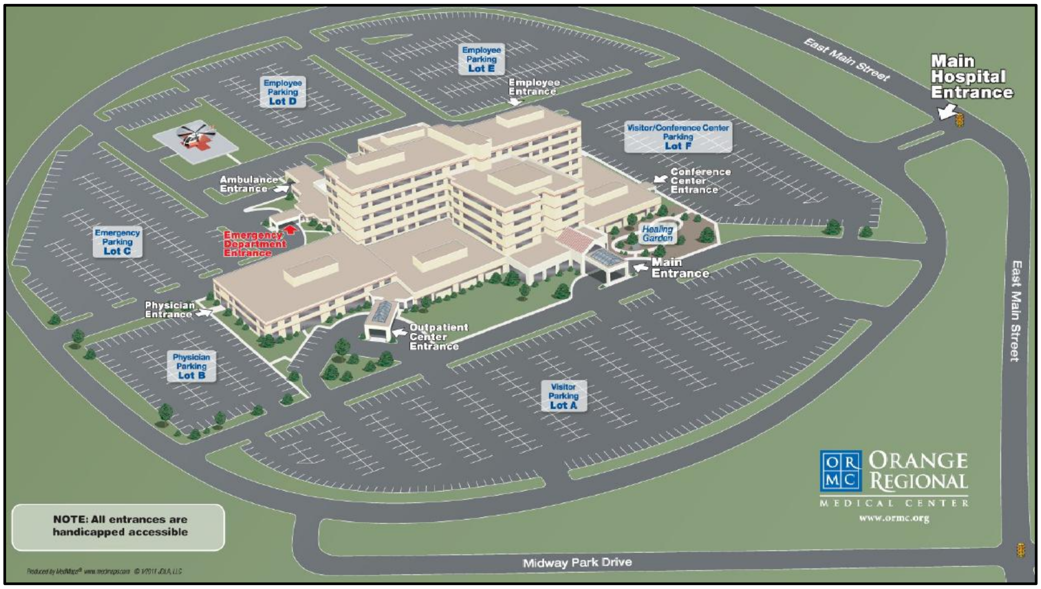


***Figure 2**: Hospital Site and Rotated Plan*

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

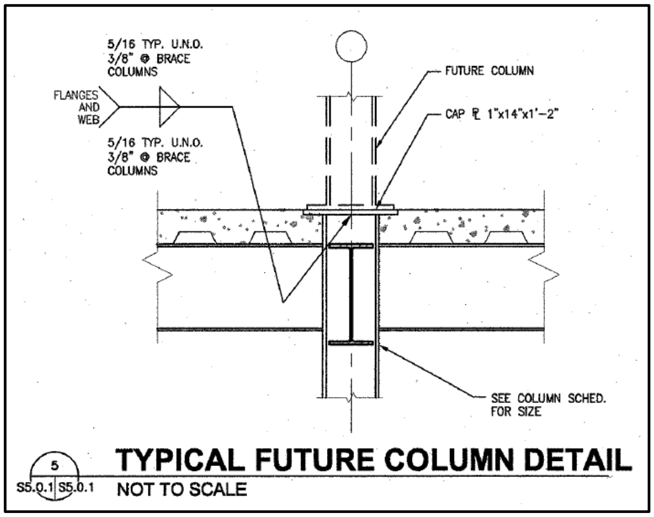


Figure 6: *Future column specified on column schedule*

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*.

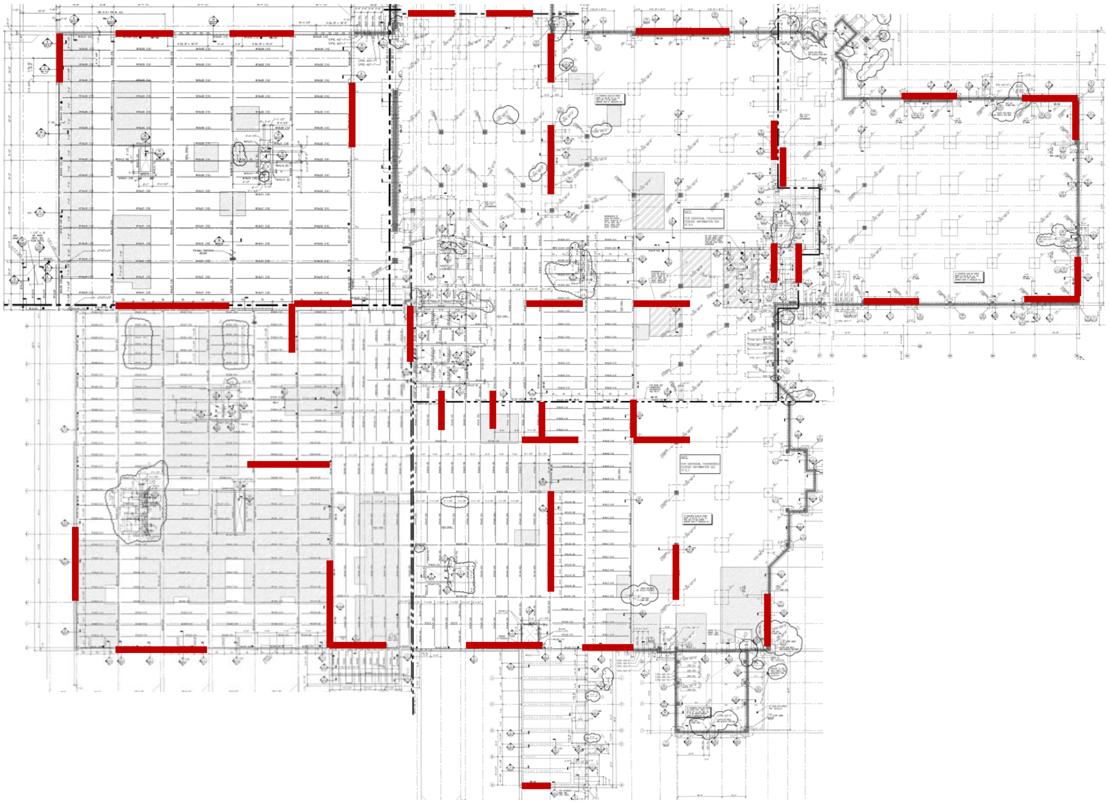


Figure 7*: Braced Frames Location*

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.

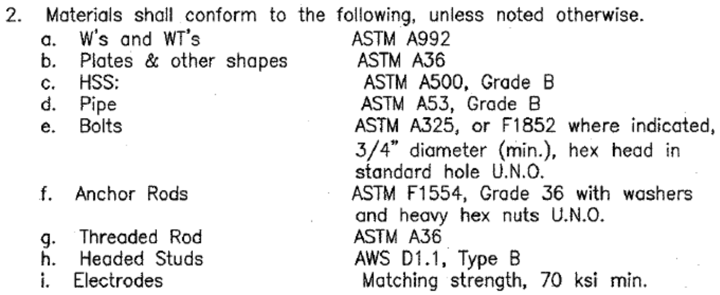
Foundations

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
1	172143.54	88.83	15292
2	100166.97	88.83	8898
3	68865.15	88.83	6117
4	68865.15	88.83	6117
5	49774.58	88.83	4421
6	48782.31	88.83	4333
Roof	95676.14	32.40	3100
	604273.84		54059
Façade Loading			
Floor	Perimeter	Height	Weight on Floor
Ground	1307.90	8.00	398
1	1681.46	14.50	926
2	1276.00	13.00	630
3	1101.57	13.00	544
4	1101.57	13.00	544
5	1044.21	13.00	516
6	1039.21	13.25	523
Roof	1039.21	6.75	267
			4348
		Floor Load	54059
		Total Weight	58407

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

Typical Live Loading	
Component	Weight (psf)
Operating Rms, Labs	60
Patient Rooms	40
Corridors Above 1st	80
Corridor 1st Floor	100
Lobby	100
Dining Area	100
Offices	50
Roof	20

***Table 3 (Left)**: Floor Live Loads*

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.

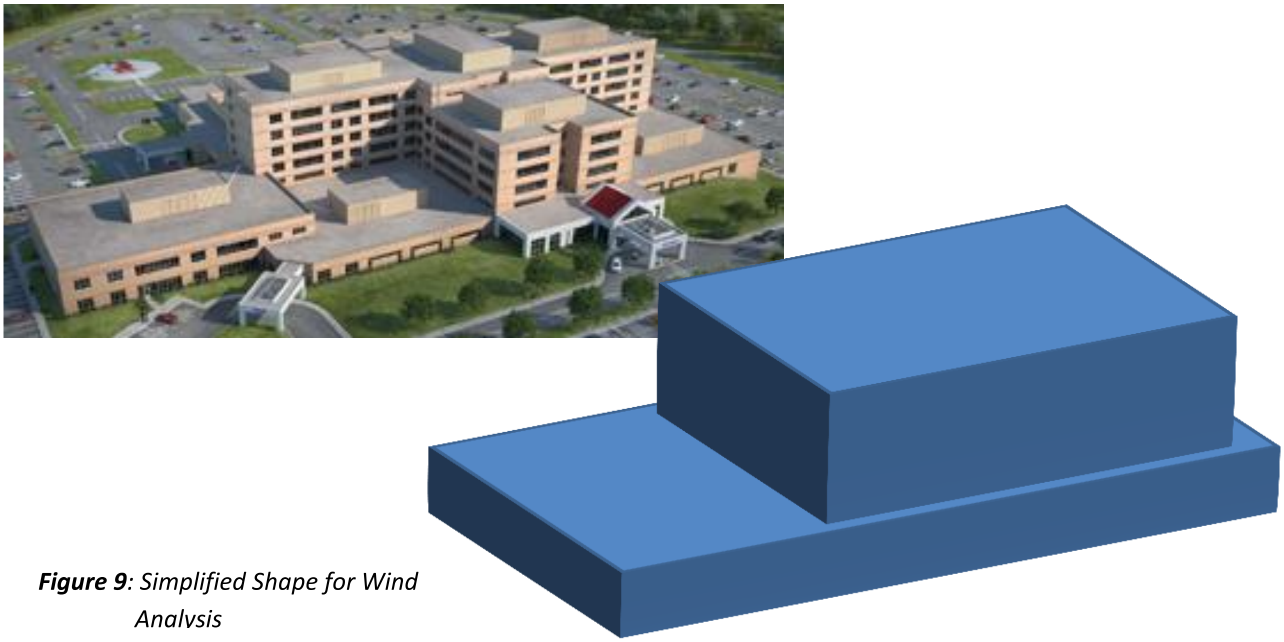


***Figure 9**: Simplified Shape for Wind Analysis*

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 4 and 5 for the East/West and North/South directions. We arrived at the base shears and overturning moments shown in *Table 6*. 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	18.6	148.5	72.5	39.32	-16.1	-128.5	-62.7
1	16	0.86	26.95	18.8	300.4	146.6	39.32	-16.1	-257.0	-125.4
2	32	0.99	31.08	21.7	314.1	153.3	39.32	-16.1	-232.9	-113.7
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 4**: 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)
1	16	0.86	26.95	18.6	298.4	170.5	39.32	-16.5	-264.6	-151.2
2	32	0.99	31.08	21.5	311.9	178.2	39.32	-16.5	-239.8	-137.0
3	45	1.07	33.37	23.1	300.2	119.0	39.32	-17.0	-221.1	-87.7
4	58	1.12	35.16	24.3	316.3	125.4	39.32	-17.0	-221.1	-87.7
5	71	1.17	36.79	25.5	330.9	131.2	39.32	-17.0	-221.1	-87.7
6	84	1.22	38.29	26.5	351.1	139.2	39.32	-17.0	-225.4	-89.4
Roof	97.5	1.26	39.32	27.2	183.7	72.8	39.32	-17.0	-114.8	-45.5

***Table 5**: East/West Wind Pressure*

Shear	Moment
North/South	
70.8	0
143.2	2345.783
149.7	4904.322
108.5	4883.48
114.3	6631.14
119.6	8493.638
126.9	10660.81
66.4	6474.051
899.6	44393.23
East/West	
81.9	0
165.8	2728.237
173.3	5703.919
119.0	5356.439
125.4	7273.356
131.2	9316.236
139.2	11693.3
72.8	7101.053
1008.7	49172.54

***Table 6**: Wind Base Shear/ Overturning Moment*

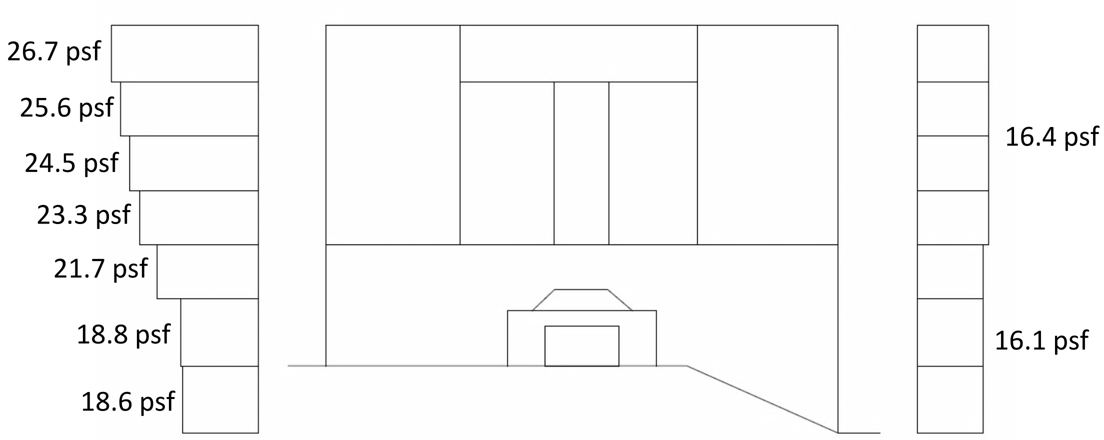


***Figure 9**: North/ South Wind Pressure*

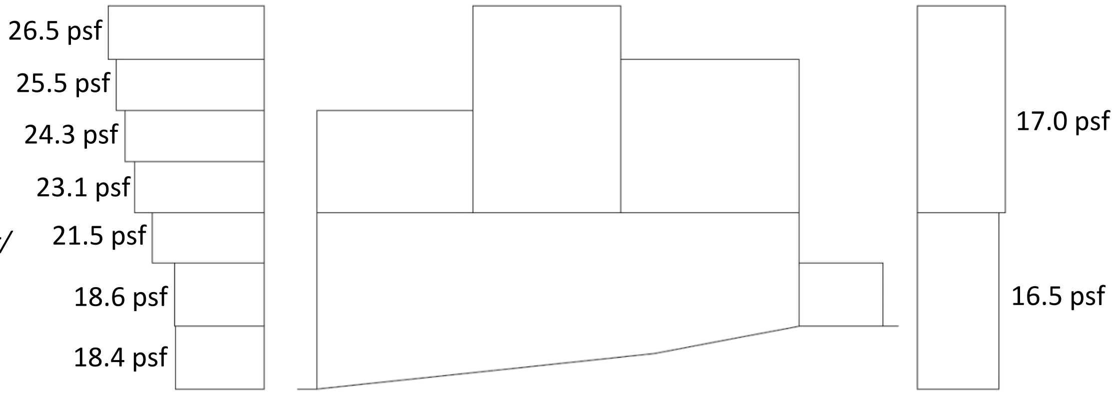


***Figure 10**: East/West Wind Pressure*

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 7* 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	3366	97.5	899001	0.159	744	744	72574
6	4857	84	1081310	0.192	895	1640	75205
5	4937	71	895419	0.159	741	2381	52638
4	6661	58	943957	0.167	782	3163	45331
3	6661	45	692611	0.123	573	3736	25806
2	9528	32	653571	0.116	541	4277	17316
1	16218	16	477555	0.085	395	4673	6326
Ground			5643425		4673		295197

***Table 7**: Seismic Calculations*

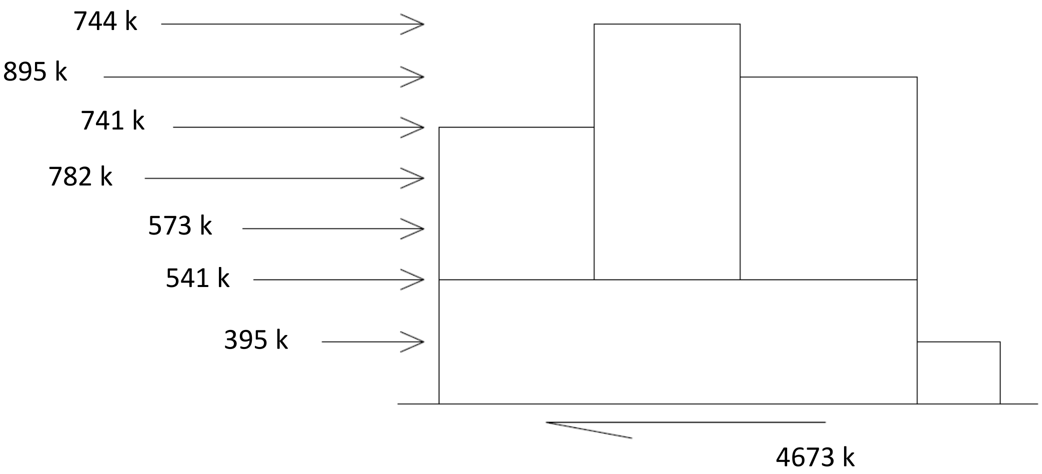


***Figure 11**: Seismic Story Forces*

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.

Lateral Systems

Lateral Load Distribution

The stiffest members will take the most lateral load. Loading first starts as a pressure transferred from the building façade into the floor diaphragms. Both wind and seismic loading is then transferred through the diaphragm into the lateral braced frames. Again, the stiffest members will take a majority of this load since force follows stiffness. A majority of the frames in this structure take the lateral loading to the sub-grade shear walls where it is then transferred into the foundation. Other frames do not have ground floor shear walls and instead, transfer the shear force right into the footings. After modeling all lateral frames, the relative stiffness was computed at two locations since a majority of the exterior frames only extend up through the second floor, where there building footprint steps back. A 100 kip lateral load was applied to the rigid diaphragm at the top of the frames at the second floor and roof levels. Finding the shear in the frames at these levels allowed for stiffness to be calculated, using the constant story deflection across the diaphragm. *Figure 12* illustrates the location of the braced frames, with full height braced frames shown in red and two-story frames shown in blue. The actual and relative stiffness of all frames at the two vertical heights can be found in Appendix H with a sample hand calculation shown in Appendix L. Later in this report, the distribution of torsional shear will also be illustrated in the lateral spot checks section of this report.

***Figure 12**: Braced Frames Location*

Lateral Load Analysis

Computer Model

Given the size and complexity of Orange Regional Medical Center, a computer model was used to efficiently analyze the structural response to lateral loading. Only the eccentrically braced frames, shear walls, and foundation walls were modeled in ETABS since these systems resist the loads from wind and seismic. *Figures 13 and 14* show the model with rigid diaphragms, which distribute the load to the lateral elements. Figure 15 shows just the lateral frames mentioned earlier. In each of these figures, one will notice the varying base levels and heights of the braced frames. The ground floor is partially below grade, so some of the northern frames have pinned supports starting at the first floor. Additionally, this model accounts for the future additions above the fifth floor since this is how the lateral loading would have accurately been handled by the design team.

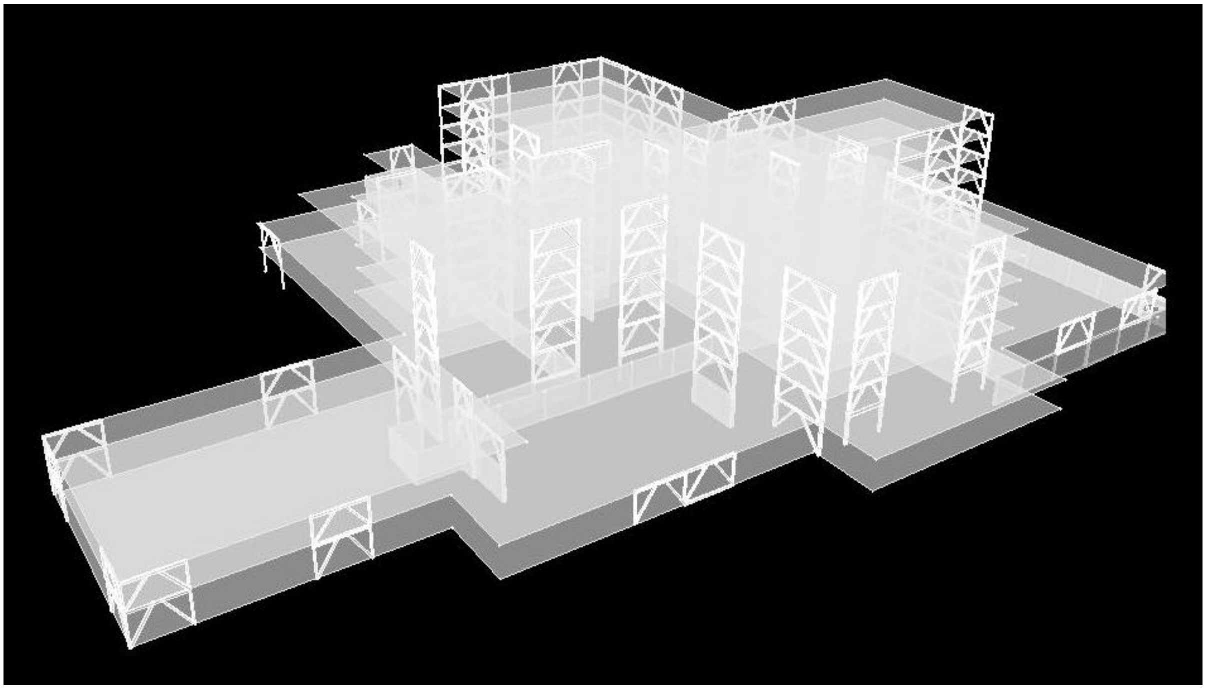


***Figure 13**: ETABS Model Looking Southeast*

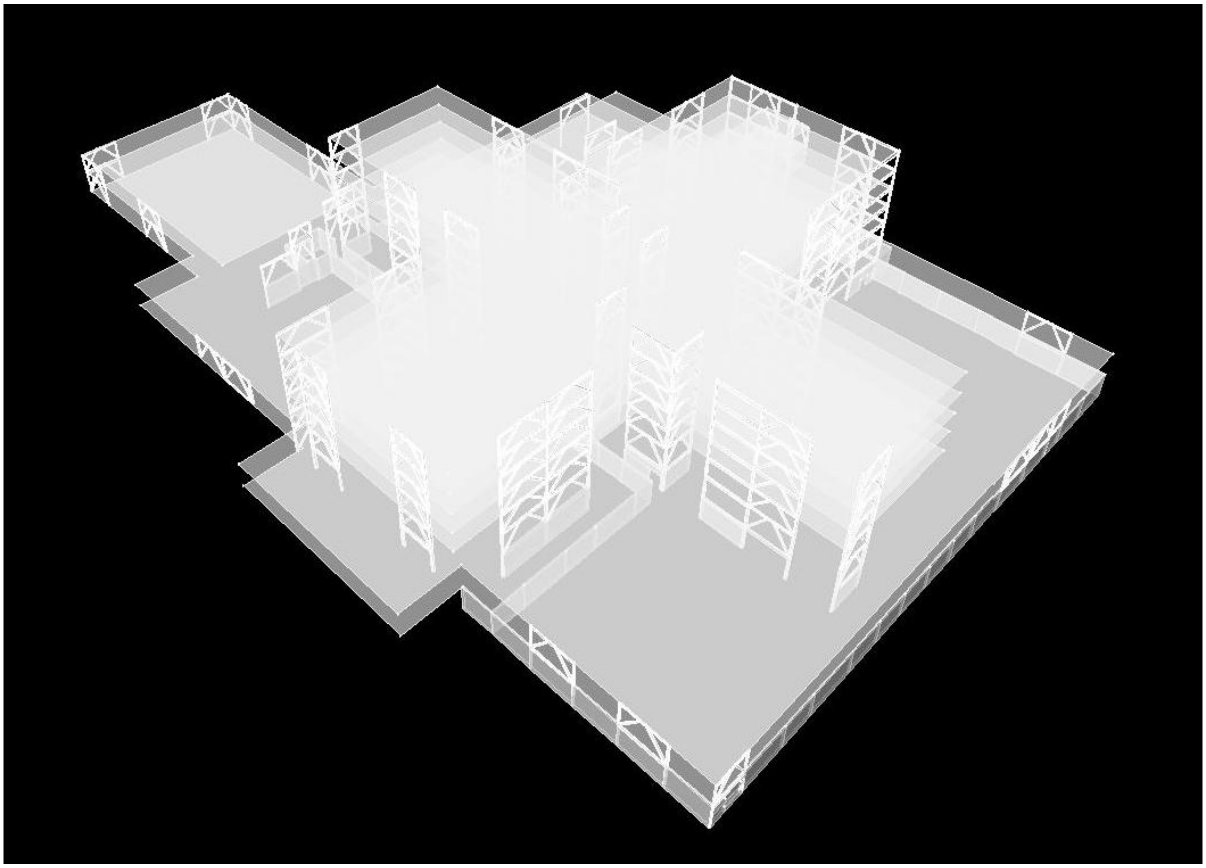


***Figure 14**: ETABS Model Looking Northeast*

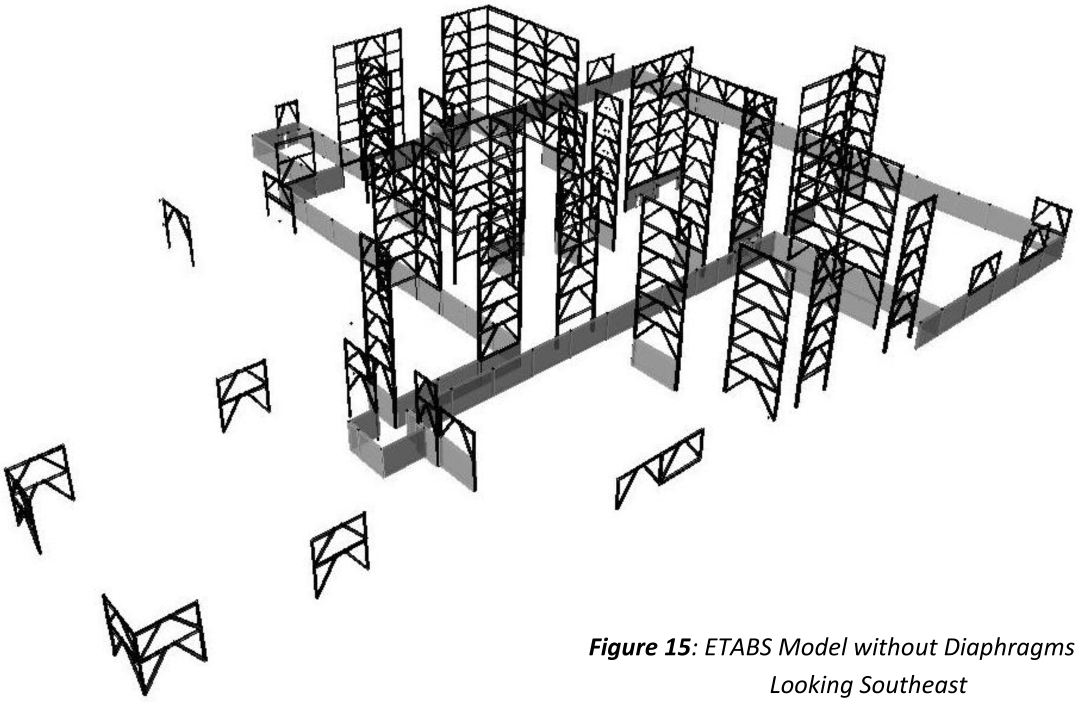


***Figure 15**: ETABS Model without Diaphragms Looking Southeast*

Lateral Loads

Load combinations from ASCE7-10 give wind and seismic a 1.0 load factor. Essentially, this means that the same lateral load can be applied to check strength and serviceability. Wind however, is given a factor for MWFRS in the different cases from ASCE7-10, as shown in Figure 16. Each of these load cases were evaluated to determine the worst case loading. Only cases in which the torsional moment would be additive were considered. This concept is illustrated in Appendix I, along with sample calculations in Appendix L. Plugging these loadings for wind and seismic into ETABS produced results showing earthquake as the predominant load on every story. In particular, earthquake in the North and South direction yielded the largest story shears and deflections.

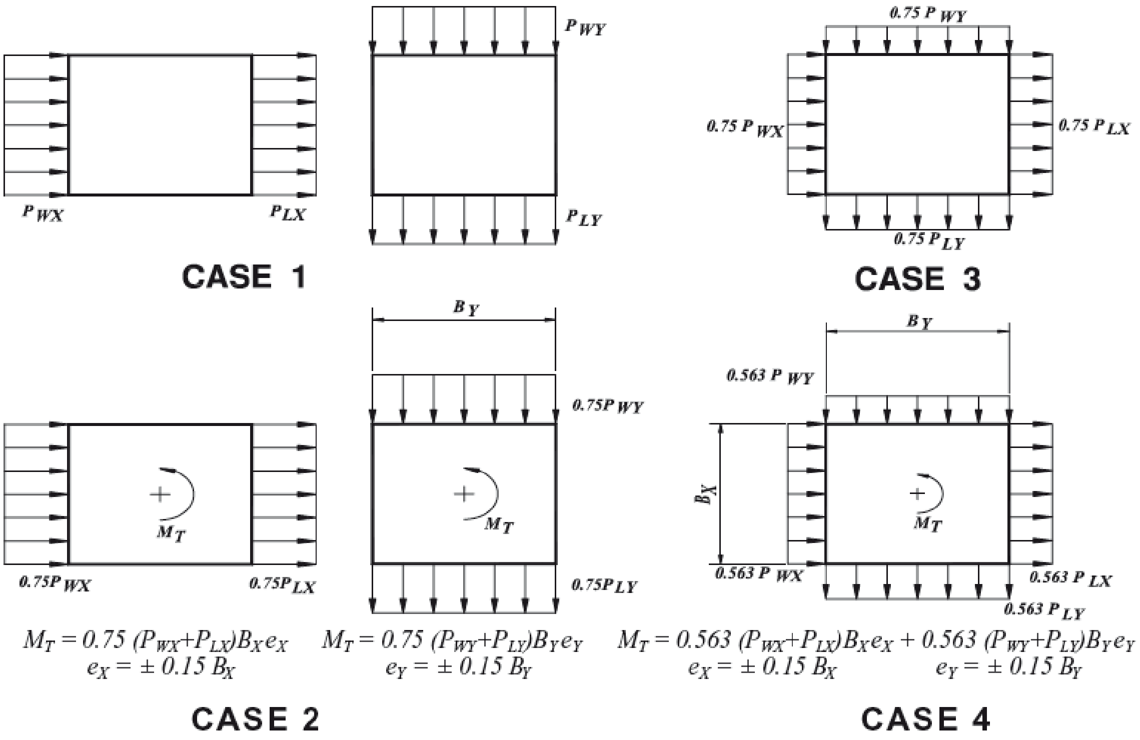


***Figure 16**: ASCE7-10 Wind Load Cases*

Story Drifts

ETABS was used on numerous occasions throughout this report to gain a better understanding of the existing structure. One place this model was employed was to output story drifts that could be checked against the accepted values. For wind, drift values should be less that L/400, and for seismic drift, the values for and occupancy category IV should fall underneath $0.01h_{sx}$. *Tables 8 and 9* show the model story drifts compared to the accepted values. All drifts for both wind and seismic fall within the acceptable limits for this structure. For sample calculations from the tables below, refer to Appendix L.

WIND STORY DRIFTS					
STORY	ETABS Drift E/W	ETABS Drift N/S	DRIFT (in)	ALLOWABLE	PASS?
7	0.000278		0.045	0.405	Yes
7		0.000365	0.059	0.405	Yes
6	0.000378		0.059	0.39	Yes
6		0.000495	0.077	0.39	Yes
5	0.000466		0.073	0.39	Yes
5		0.000607	0.095	0.39	Yes
4	0.000519		0.081	0.39	Yes
4		0.000687	0.107	0.39	Yes
3	0.000615		0.096	0.39	Yes
3		0.000655	0.102	0.39	Yes
2	0.000577		0.111	0.48	Yes
2		0.000565	0.108	0.48	Yes

***Table 8**: Actual Wind Story Drifts vs. Accepted*

SEISMIC STORY DRIFTS					
STORY	ETABS DriftX	ETABS DriftY	DRIFT (in)	ALLOWABLE	PASS?
7	0.001366		0.443	1.62	Yes
7		0.001984	0.643	1.62	Yes
6	0.001739		0.543	1.56	Yes
6		0.002514	0.784	1.56	Yes
5	0.002005		0.626	1.56	Yes
5		0.002882	0.899	1.56	Yes
4	0.00215		0.671	1.56	Yes
4		0.003149	0.982	1.56	Yes
3	0.002341		0.730	1.56	Yes
3		0.002801	0.874	1.56	Yes
2	0.001904		0.731	1.92	Yes
2		0.002132	0.819	1.92	Yes

***Table 9**: Actual Seismic Story Drifts vs. Accepted*

Overturning Moments

As mentioned earlier, once the lateral members take the wind or seismic load, the forces are then carried down to the foundation. It is crucial that these foundations can handle the moment forces developed by the lateral forces. Again, earthquake controls for overturning moment with 295,197 ft-kips acting on the soil. From the geotechnical report, the soil is capable of withstanding 4,000 psi, so they footings must be properly designed to spread out the load. *Table 10* gives the overturning moment for wind and seismic in kip-ft.

Overturning Moments			
Floor	Earthquake	Wind E/W	Wind N/S
Roof	72574	7101	6474
6	75205	11693	10661
5	52638	9316	8494
4	45331	7273	6631
3	25806	5356	4883
2	17316	5704	4904
1	6326	2728	2346
Ground	295197	49173	44393

***Table 10**: Overturning Moment from*
Wind and Earthquake

Lateral Spot Checks

Although analysis using a computer model often times seems advantageous, there is also much room for human error. Therefore, to confirm the ETABS model and to confirm the adequacy of the lateral elements to withstand load, several spot checks were run. Appendix K is the spot check of a typical brace and column at the second floor level. The second floor was chosen for checks because this is the level where the shorter lateral frames drop out and the taller frames must now carry that extra lateral load. Therefore, a check was necessary at this point of interest to confirm the elements' strength. The calculations shown in Appendix K confirm that both the brace and column are adequate to carry the required loading. However, the calculated values are much smaller than the required strength for both the column and brace. This difference can most likely be credited to the application of redundancy in the lateral system. Since seismic loading was the predominant load in this structure, it is good practice to include redundancy to increase the over strength of a structure. This way, appropriate strength will be available in the extreme case earthquake. Torsional shear was also calculated by hand in the East and West direction to confirm the results of the ETABS model. The table for all torsional shears in East/West running lateral frames can be found in Appendix J. Overall, the combination of direct and torsional shear from hand calculations were comparable to the shears from ETABS. The values were within a couple kips with the hand calculations being consistently larger than the model. Since these values are so close, it is tough to say where the error comes from. The most likely option is either rounding errors or a missed frame somewhere in calculation. A sample calculation for torsion can be found in Appendix L.

Alternate Systems

Multiple floor systems are analyzed in the remainder of this report. Exploring three preliminary alternative floor systems, and comparing them with the existing system, allows for the pros and cons to transpire. What effects does this system have on the other disciplines of the building? Is this cost effective? Are the results comparable to or better than the existing system? These are some of the questions that will be answered as the following systems are examined:

- Existing one-way composite concrete slab
- One-way precast hollow core planks on steel frame
- One-way non-composite concrete slab on steel frame
- Two-way flat slab with drop panels

All floor systems are designed in relation to the typical bay shown in *Figure 12*. This allows for close comparisons to be made in order to determine which alternate systems may be viable. Of course, the existing system likely fits the needs of this building quite well, which is why the project team chose the system in the first place. However, there is a multitude of floor combinations that a structural engineer may choose from, so chances are, this report may stumble upon other viable systems which will warrant further investigation.

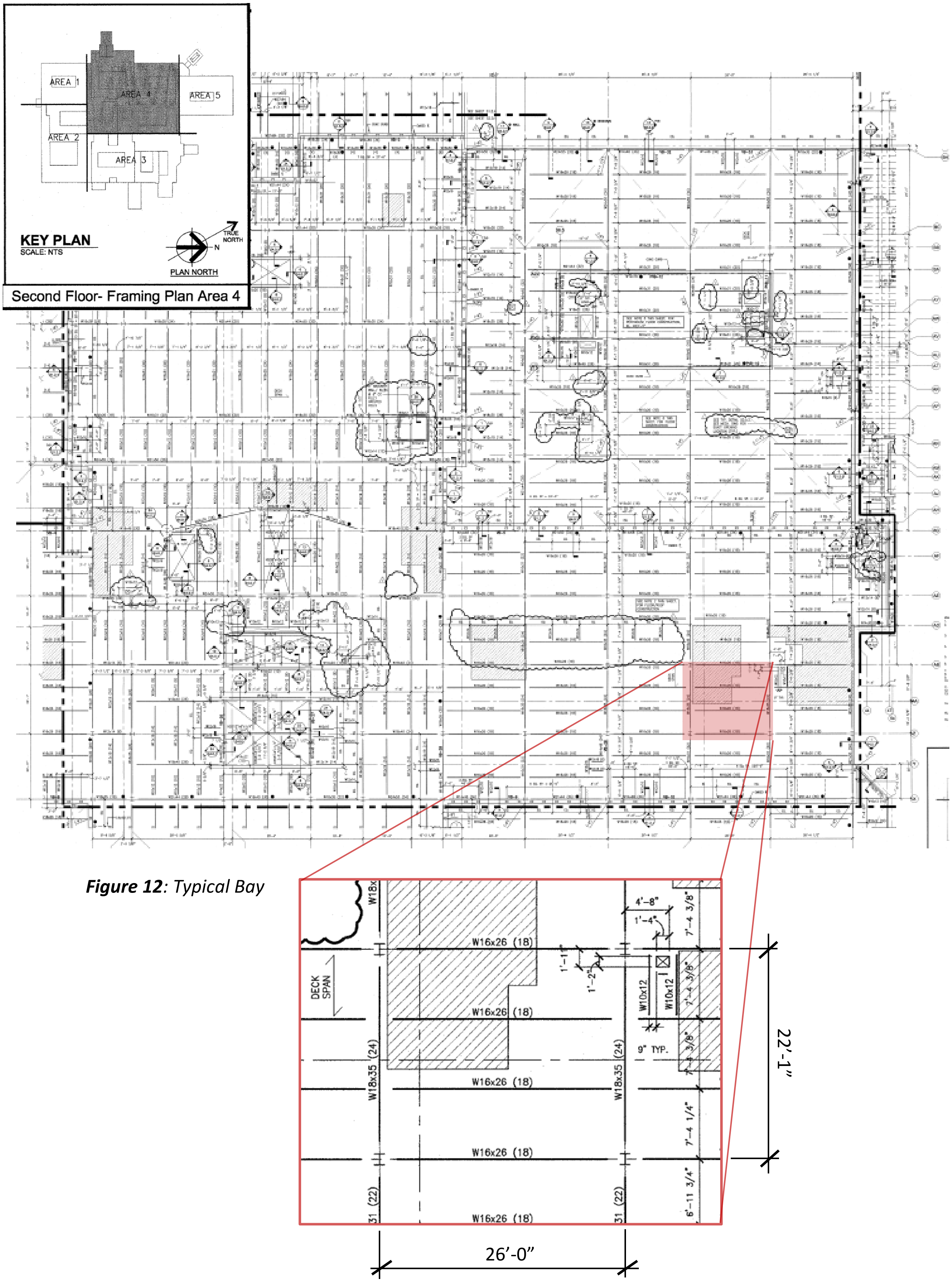

Second Floor- Framing Plan Area 4

***Figure 12**: Typical Bay*

Existing One-Way Composite Concrete Slab

Making use of composite action, the existing steel frame uses 2VLI20 composite deck with 3¼" of lightweight concrete, running in the 22'-1" direction of the 26'-0" x 22'-1" typical bay. The decking rests on W16x26 beams, typically spaced at 7'-4 3/8" with 18 shear studs a piece. These then frame into W18x35 girders, which span the 22'-1" direction and have 24 shear studs a piece. In total, the cost of the existing floor system can be estimated by RS Cost Works to be about $11,380,000. This system, like the other system in this comparison study, is subjected to the loads mentioned earlier in this report, and further calculations for these loadings can be found in Appendix D. These loadings, as well as self-weight, put a 305 psi pressure on the soil from the footings. Also, refer back to *Figure 12* for the bay layout of this system.

Advantages

By putting the concrete in compression and placing the steel beam in tension, composite systems are very efficient systems. This enables the designer to use a smaller beam or girder and therefore reduce the structural depth. The composite floor system is also fairly light, being the second lightest system studied in this report. This allows for smaller footings and therefore, less concrete. A third advantage is the ease of constructability since the metal composite decking serves as the formwork for the concrete. Lastly, the estimated system cost is comparable to the other systems in this report, meaning that it is not too expensive to take advantage of the composite action.

Disadvantages

A lighter system such as this could have potential vibration issues, which would need to be investigated. Additionally, although it is structurally efficient to use composite action, installation and inspection of the shear stubs could prove time consuming and costly. Fireproofing may also be a concern in this system with all the exposed steel. In order to achieve the two hour fire rating, the beams, girders, and underside of the decking will need to be fireproofed, which again, is time consuming and adds cost. Despite these disadvantages though, the existing floor system still fits the needs of this building fairly well, which is why it was chosen by the designers as a viable system.

One-Way Precast Hollow Core Planks on Steel Frame

From the Nitterhouse specifications, untopped 10" x 4'-0" hollow core planks with 6-1/2" diameter strand pattern were chosen to withstand the typical floor loading. Starting with the typical bay size of 26'-0" x 22'-1", the 4 ft width planks were assessed for the best fit. It was determined that planks spanning the long direction (26'-0"), had the smallest effect on the architectural floor plan. However, this 4 ft module size meant that the short direction had to be changed to either 24'-0" or 20'-0". *Table 8* gives the load capacity for the 26 ft span. These precast planks are supported by a steel frame which was determined by the AISC manual to be W18x86 girders.

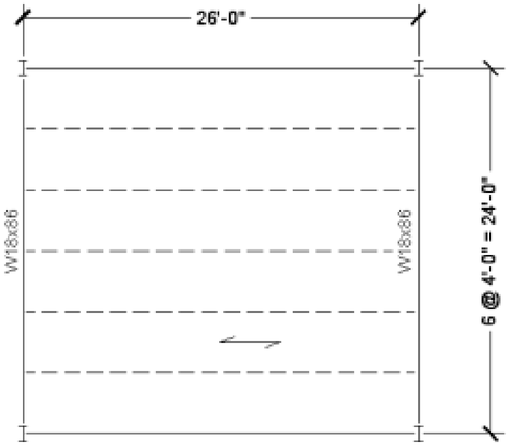
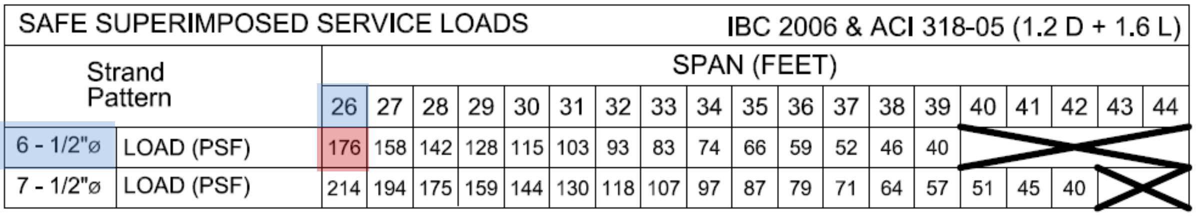


***Figure 13**: Typical Bay*

All of this can be seen in *Figure 13* of the typical bay. For further calculations, refer to Appendix E.



SAFE SUPERIMPOSED SERVICE LOADS IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	176	158	142	128	115	103	93	83	74	66	59	52	46	40					
7 - 1/2"ø	LOAD (PSF)	214	194	175	159	144	130	118	107	97	87	79	71	64	57	51	45	40		

***Table 8**: Plank Loading Specs*

Advantages

Precast planks offer quite a few advantages, some of which, the existing composite system can't offer. For one, hollow core planks allow for easy construction since everything is cast off-site and can simply be put in place once they arrive on site. Additionally, since a majority of the flooring throughout the hospital is carpet, a leveling top coat isn't necessary for the planks. The joints can simply be feathered with a latex cement or grout. This all allows for the construction schedule to move along quicker and deliver the building earlier. A second advantage is the 2 hour fire rating that the planks provide, meaning that only the steel support girders would need to be fireproofed, rather than the entire system. This is also the second cheapest system being evaluated in this report at about $10 million. This is about $1 million less than the existing system.

Disadvantages

As mentioned earlier, in order to accommodate the 4 ft module width of the planks, the bay sizes had to be adjusted in the plan E-W direction. Luckily, in most locations on the typical floor plan, the columns fall in the center of a wall and may be moved east or west with little impact on the architectural layout. However, there are areas where adjustments will not be so easy and would need to be coordinated with the architect. In order to withstand the typical floor loading with this system, the structural depth had to be increased from that of the existing composite system. At 28.4", this system is just short of 5.5" deeper, which translates to larger floor-to-floor heights or smaller plenum space for the other disciplines to work with. A larger story height would consequently add cost to this project from the expanded façade around the perimeter of the building. Additional costs may be accrued up front, considering transportation costs and the additional cranes that would be required to hoist the precast planks to the constructed floors. This system also adds a lot of weight to the foundation at 415 psi on the soil. This is over 100 psi more than the existing structure. The connection to the lateral system would also have to be reworked. Overall, the disadvantages outweigh the advantages of this system. Difficulty with the module bay sizes along with added weight and structural depth makes this system tough to justify.

One-Way Non-Composite Concrete Slab on Steel Frame

Using form deck rather than the composite decking, it was found that 2C20 deck, from the Vulcraft Catalog, could adequately withstand the floor loading. For comparison purposes, lightweight concrete was used, which requires a topping thickness of 3½". For the slab to hold these loads, the concrete had to be paired with 6x6 – w2.9 x w2.9 welded wire fabric. All other criteria such as unshored clear span and deflections checked out, as can be found in Appendix F. The decking then transfers the floor load to

W14x48 beams, as determined by the AISC Steel Manual, which span the 26’-0” direction. Loading is then transferred to W14x68 girders, spanning the 22’-1” direction, which frame nicely with the W14 beams. This framing is illustrated in the bay of *Figure 14.* Calculations for required moment and moment of inertia, used in determining these framing sizes, can also be found in Appendix F. The appendix also shows calculations for the system weight which was slightly larger than the composite system at a 343 psi soil pressure.

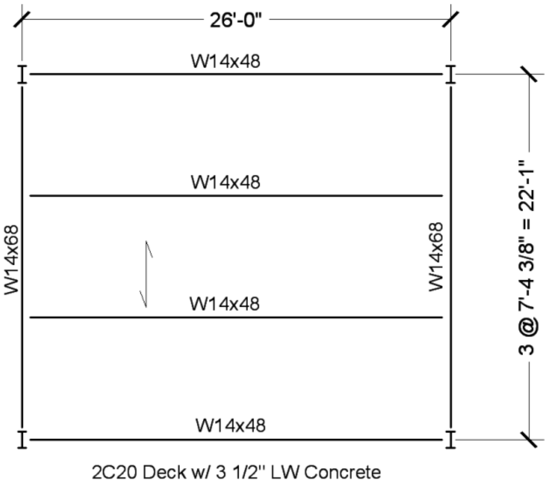


Figure 14*: Typical Bay*

Advantages

As mentioned with the composite system, installing shear studs can be costly and time consuming, but since a non-composite system does not use shear studs, construction may move along quicker than a composite system. Again, as with the composite system, construction may also move along quicker due to the form deck serving as the concrete formwork. In terms of structural depth, this system is about 3.5” less than the existing, which would leave more room for other disciplines to install their equipment.

Disadvantages

Because part of the steel beam will be in compression with the non-composite system, members with larger flexural strength will have to be chosen. This translates to a heavier system and heavier foundation loads. This system puts roughly 40 psi more on the soil than the composite system, and the site soils will have to be analyzed for that. In some cases, the cost of shear studs may be more than the cost of larger beams. This is not the case for this structure, however. According to RS Means Cost Works, this system totaled $11.5 million (the most expensive system evaluated), which is about $200,000 more than the existing structure. These results were also confirmed in Appendix F on a material load basis. With shear studs counting for roughly ten pounds a piece, the proportion shows that non-composite is much heavier, and therefore more expensive in material costs. Lastly, as mentioned with the existing floor, all exposed steel would still need to be fireproofed. This shows that the only thing to really gain from non-composite is 3.5” of plenum space and slightly shorter construction schedule. These benefits do not seem to make up for the added costs and weight, and can therefore be ruled out as a viable option.

Two-Way Flat Slab with Drop Panels

Switching over to concrete framing, a flat slab with drop panels was evaluated for the typical bay, given the appropriate spans, as shown in *Figure 15*, and its popular use in hospitals. The CRSI Handbook was used for preliminary design to arrive at a 9" slab with drop panels 8.67' wide and 6.25" in depth. The handbook mentioned that for rectangular bays with l_2/l_1 close to 1.0, to use the longer span for design and reinforcement. Therefore, the design bay size is 26'-0" x 26'-0", but the actual bay size is still 26'-0" x 22'-1". Reinforcement and dimensions can be found in the bay layout in *Figure 16*, but for additional calculations and the CRSI design table, refer to Appendix G.

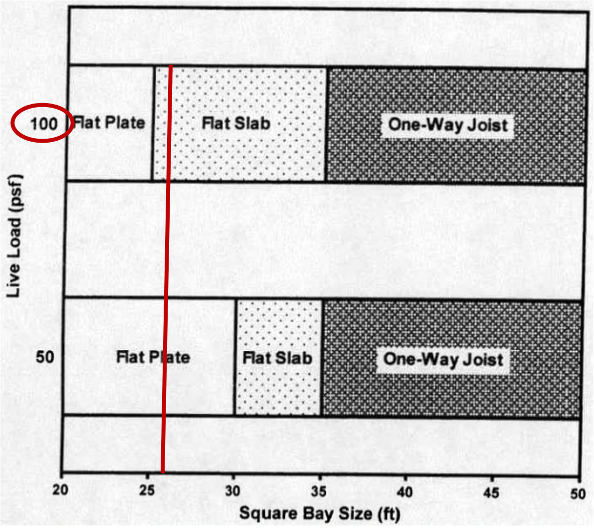


***Figure 15**: System Determination for Span and Loading*

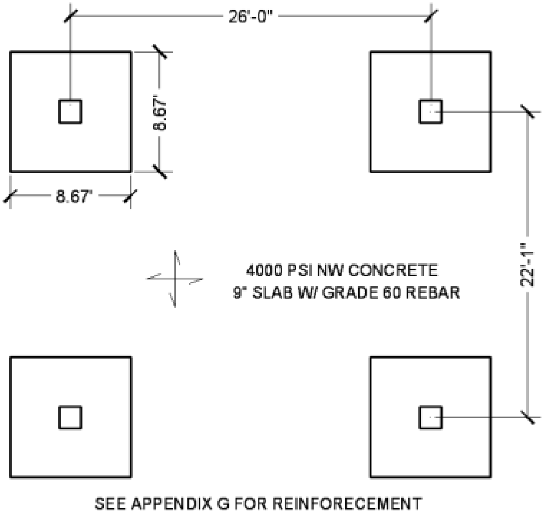


***Figure 16**: Typical Flat Slab Bay*

Advantages

At 15.25", the flat slab system is 7.7" inches less in structural depth than the existing system. This leaves much more room for plenum space, which is always needed in a hospital for the other disciplines to work with. Also, this 7.7" could be used to drop the floor to floor height and save money on the façade. The concrete system is also the lightest out of those analyzed in this report, despite using normal weight concrete, giving a soil pressure of 225 psi. This takes a huge weight off of the foundation, allowing for smaller footings. Now, because Orange Regional Medical Center is only six stories, it is not expected that this lighter structural weight will cause any issues with overturning moment, but it may be an area worth checking for reassurance. In addition to the lighter structure, the flat slab system is also the cheapest of those analyzed, totaling in at roughly $9.77 million (about $1.5 million less than the existing structure). A flat slab system also does not need any additional fireproofing. The nine inches of slab is sufficient to provide a two hour fire rating, and since no fireproofing is needed, a flat slab still appears aesthetically pleasing and may be painted as is and used as the finished ceiling.

Disadvantages

The biggest disadvantage with a concrete structure is the increased construction time to allow for formwork placement and concrete curing. Construction may also be slowed down for rebar placement and inspections. This system also produces larger columns than that of the steel frames. The existing structure used W12's where this system calls for 19" square columns. This may put a strain on the architectural layout, and would need to be coordinated with the architect. It would also be difficult to tie into the existing lateral steel braced frames. This would have to be explored to find effective lateral resistance in a concrete frame by either using shear walls or some other means. One final drawback is the possibility of vibrational problems with such a light system, but in the end, the advantages definitely outweigh the disadvantages. The designer may be able to pitch $1.5 million in savings to the owner in exchange for the longer construction schedule. Therefore, the two-way flat slab system with drop plates is still a viable alternative.

Systems Summary

Floor System Summary Comparison				
	Existing	Alternative Systems		
Criteria	One-Way Composite	Precast Planks	One-Way Non-Composite	Two-Way Flat Slab
Cost	$11.4 million	$11.5 million	$11.6 million	$9.8 million
Weight Ratio	1.0	1.36	1.12	0.74
Vibration Dampening	Unknown	Fine	Fine	Potential Issues
Structural Depth	22.95"	28.4"	19.45"	15.25"
Bay Size Flexibility	Yes	No	Yes	Yes
Lateral System Altered	No	Yes	No	Yes
Constructability	Moderate	Easy	Easy	Tedious
Additional Fireproofing	Yes	Some	Yes	No
Viable Option	Yes	No	No	Yes

***Table 9**: Pros and Cons Summary*

Conclusions

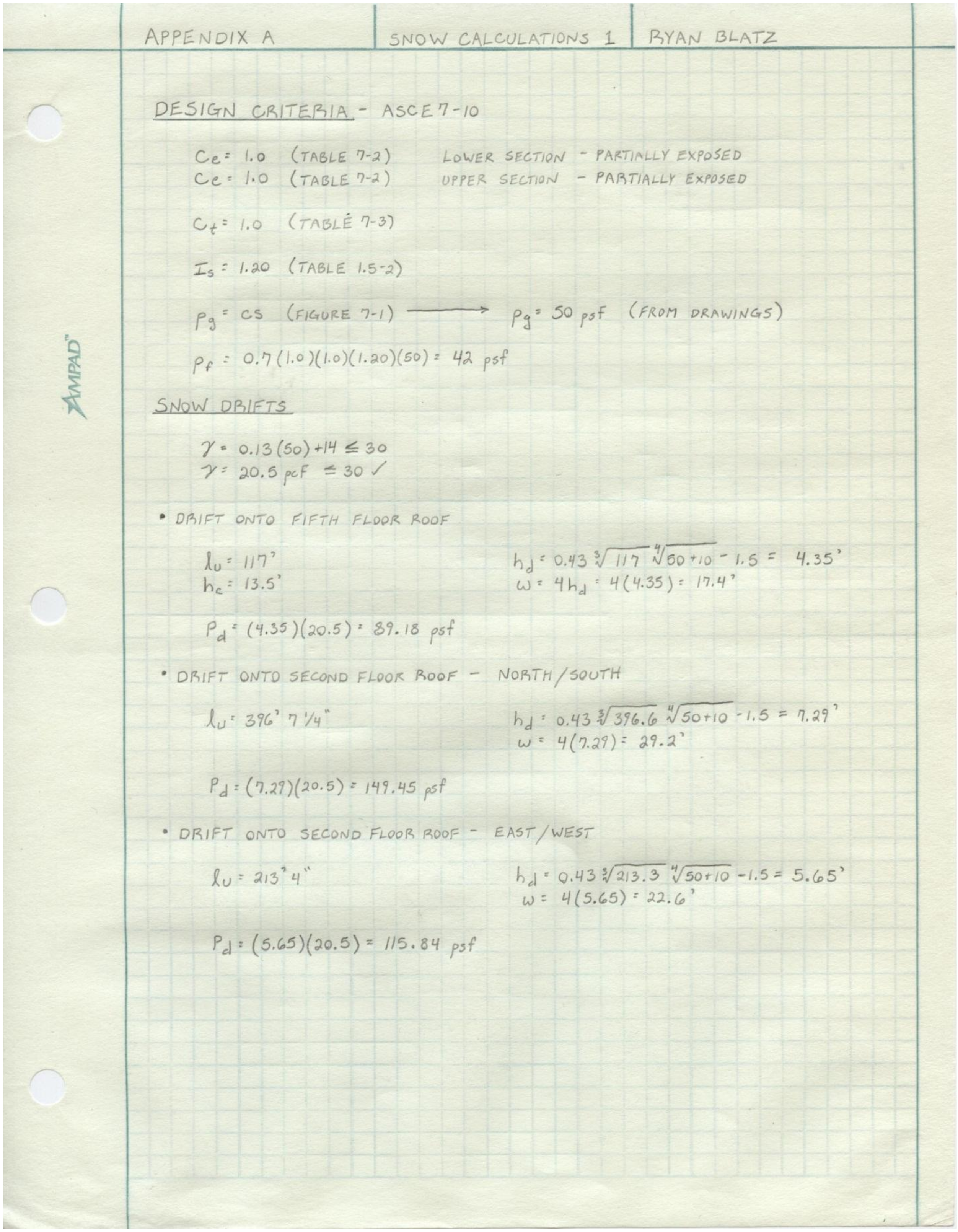
From the calculations performed in this report, we have achieved a greater understanding of Orange Regional Medical Center and its structural components. Although the actual building was designed to a different set of codes, by using ASCE7-10 and AISC we were able to find areas of discrepancy and determine 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.

The lateral system was explored in the second piece of this report. From the use of an ETABS model and hand calculations, it was determined that story drifts were acceptable in both wind and seismic. Additionally, spot checks of a column and brace at a critical location showed that these elements were more than adequate to withstand the loading. The difference in load capacity was credited to the desire for redundancy in seismic loading. After all, seismic loading in the North and South direction proved to be the dominant load at every story level. Now, that greater knowledge of the lateral system and its load distribution has been achieved, exploration of alternative lateral resistance may be an area of interest for a spring proposal.

In the third part of this report, preliminary calculations showed that changes in floor system can have a rather dramatic effect on the structure. Each system had its set of advantages and disadvantages, but it was how those offset each other that really determined whether a system was a viable alternative. In the end, the two-way flat slab with drop panels was the only viable alternative to the existing system. For a much cheaper cost, the flat slab system offers a lighter, shallower design that requires no additional fireproofing. Construction timeline may be extended, but this may be something worth considering, given the benefits. At this point, because this was only a preliminary design, further investigation into this system will be required in order to determine if this is a realistic option. For example, little is known about its vibration characteristics and how the lateral system will work. Additionally, the cost comparison in this report is a very rough estimate and would need to be calculated. However, this system has definitely become a point of interest and may be explored as a thesis proposal in the future.

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

$l_u = 396'\ 7\tfrac{1}{4}''$

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

$w = 4(7.29) = 29.2'$

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

- DRIFT ONTO SECOND FLOOR ROOF - EAST/WEST

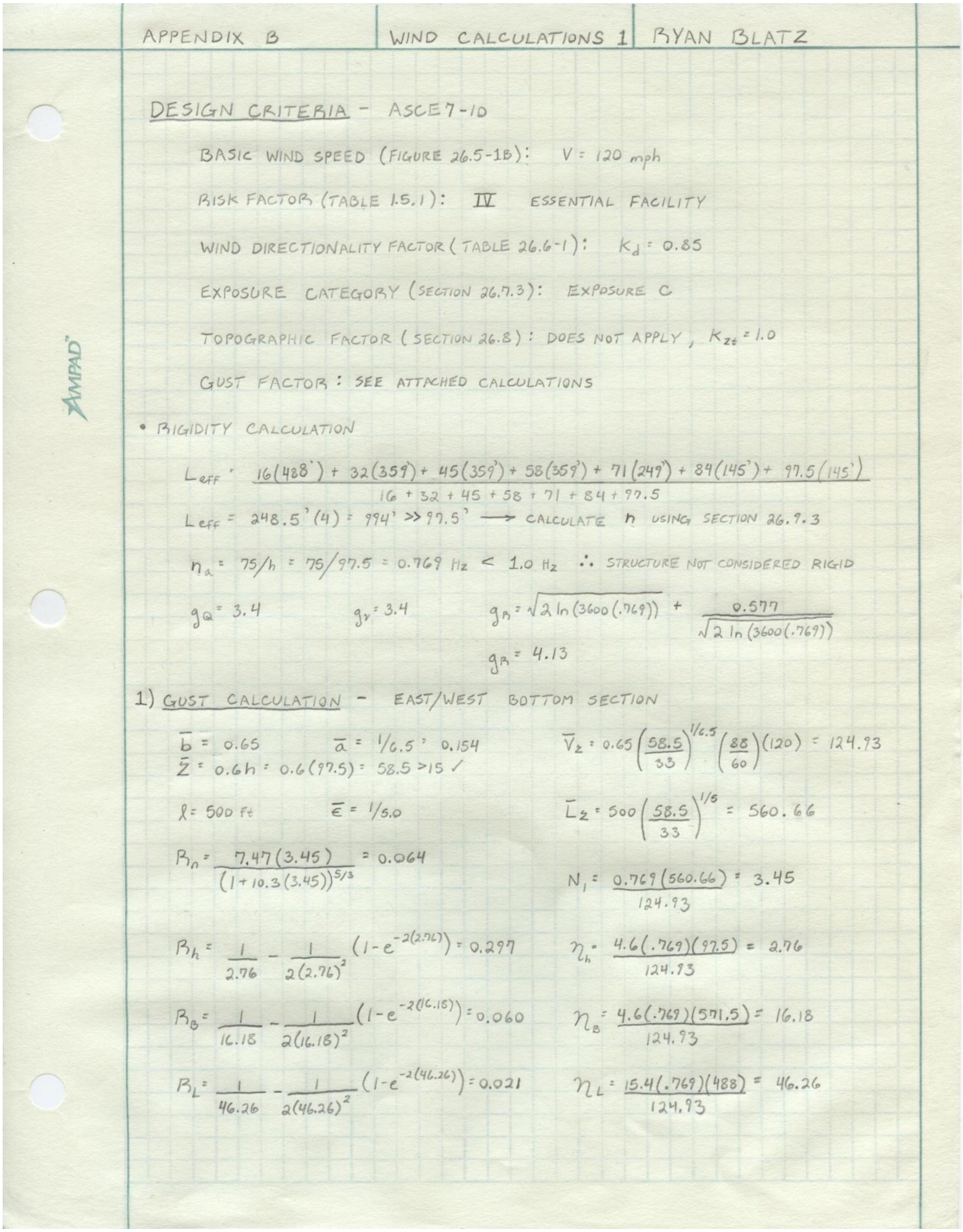
$l_u = 213'\ 4''$

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

$w = 4(5.65) = 22.6'$

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

Appendix B: Wind Calculations



APPENDIX B | WIND CALCULATIONS 1 | RYAN BLATZ

DESIGN CRITERIA – ASCE7-10

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

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

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

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

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

$g_R = 4.13$

1) GUST CALCULATION – EAST/WEST BOTTOM SECTION

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

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

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

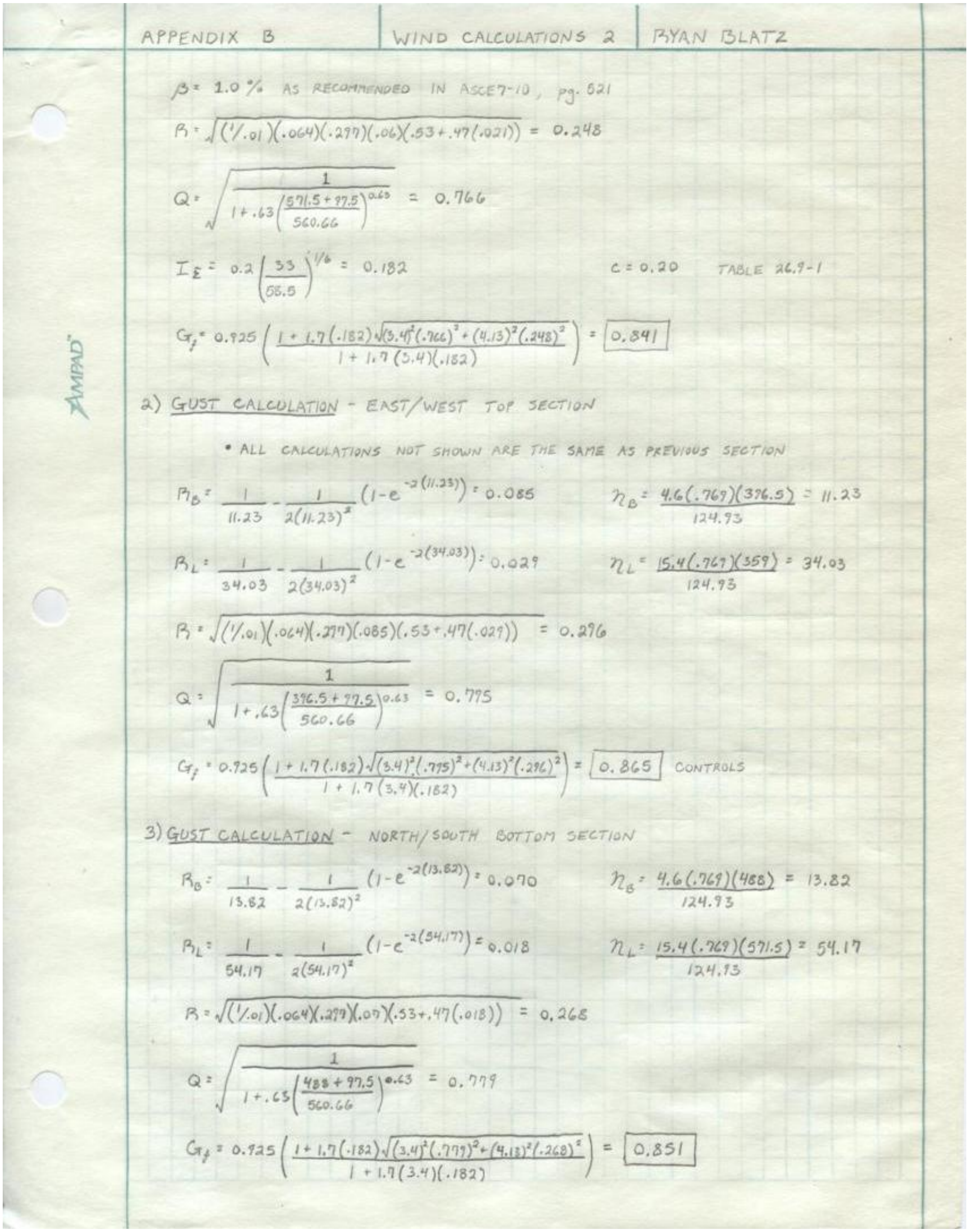
$R_n = \frac{7.47(3.45)}{(1 + 10.3(3.45))^{5/3}} = 0.064$ $\quad N_1 = \frac{0.769(560.66)}{124.93} = 3.45$

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

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

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

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) <u>GUST CALCULATION</u> - EAST/WEST TOP SECTION

- ALL CALCULATIONS NOT SHOWN ARE THE SAME AS PREVIOUS SECTION

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

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

$$R = \sqrt{(1/.01)(.064)(.277)(.085)(.53+.47(.029))} = 0.296$$

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

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

3) <u>GUST CALCULATION</u> - NORTH/SOUTH BOTTOM SECTION

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

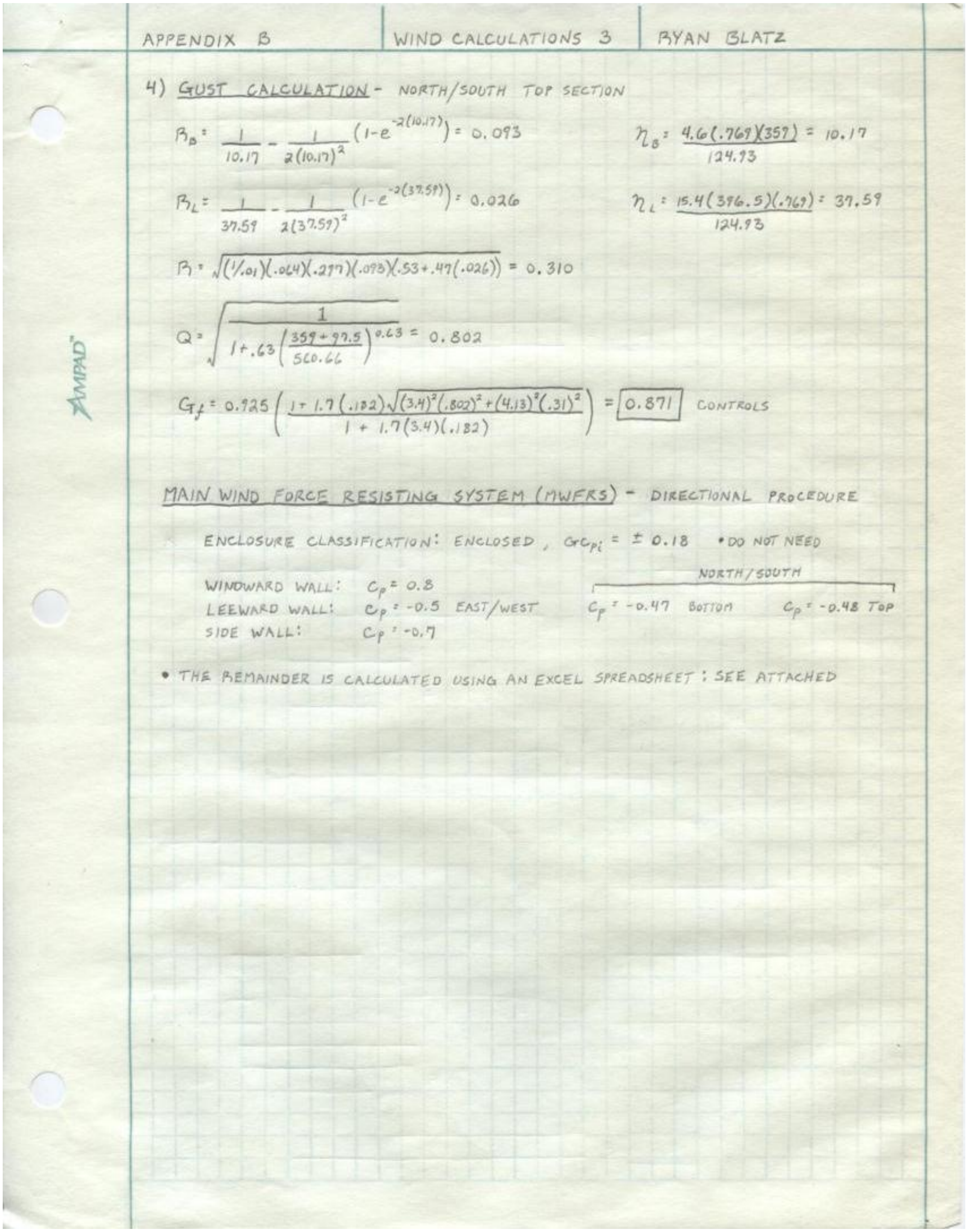
$$R_L = \frac{1}{54.17} - \frac{1}{2(54.17)^2}\left(1-e^{-2(54.17)}\right) = 0.018 \qquad \eta_L = \frac{15.4(.769)(571.5)}{124.93} = 54.17$$

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

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

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

Appendix B: Wind Calculations



APPENDIX B | WIND CALCULATIONS 3 | RYAN BLATZ

4) GUST CALCULATION - NORTH/SOUTH TOP SECTION

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

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

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

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

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

MAIN WIND FORCE RESISTING SYSTEM (MWFRS) - DIRECTIONAL PROCEDURE

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

NORTH/SOUTH

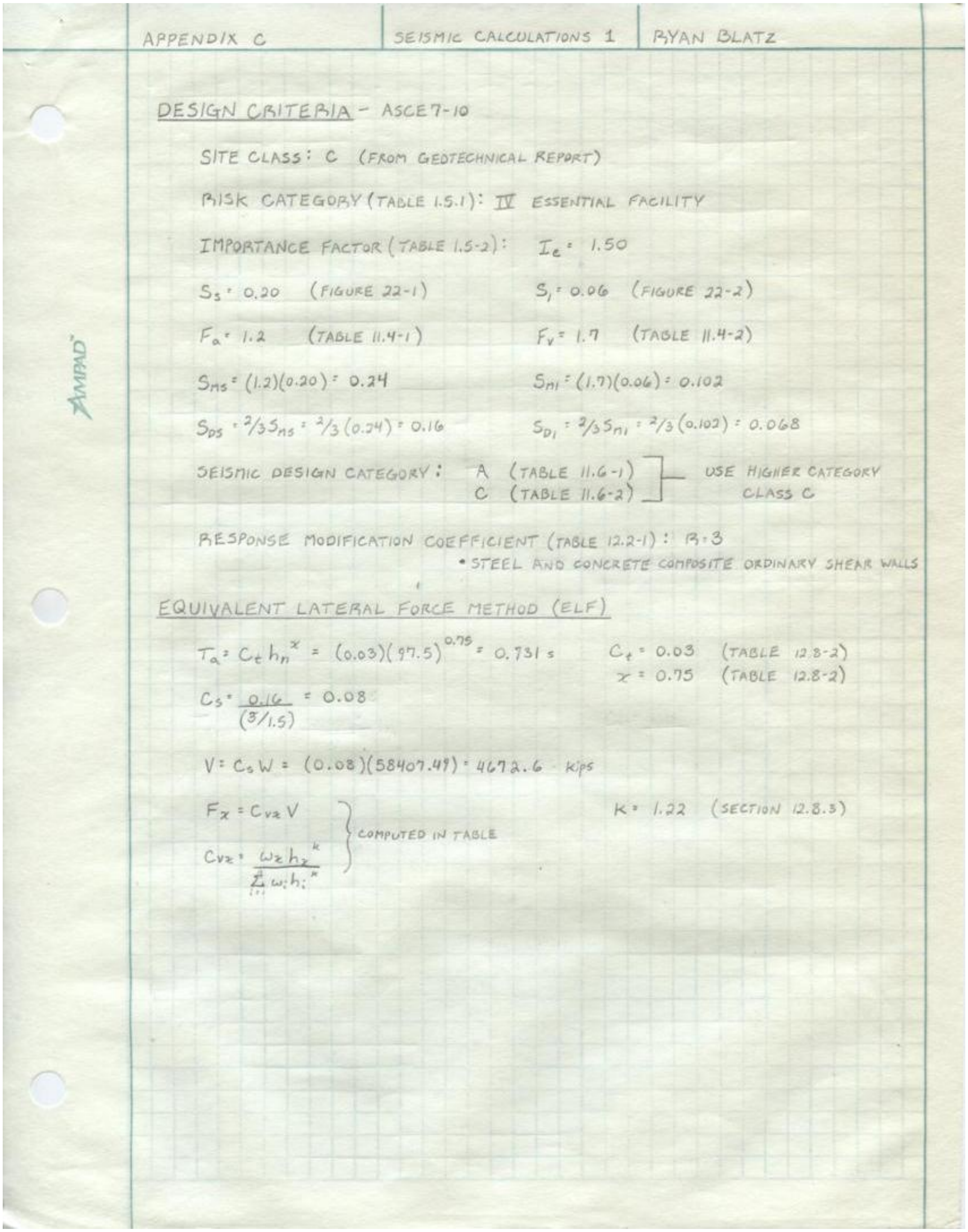
WINDWARD WALL: $C_p = 0.8$

LEEWARD WALL: $C_p = -0.5$ EAST/WEST $C_p = -0.47$ 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 – ASCE 7-10

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): $R = 3$

- 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)

$$C_s = \frac{0.16}{(3/1.5)} = 0.08$$

$V = C_s W = (0.08)(58407.49) = 4672.6$ kips

$$\left.\begin{aligned} F_x &= C_{vx} V \\ C_{vx} &= \frac{w_x h_x^{k}}{\sum_{i=1}^{n} w_i h_i^{k}} \end{aligned}\right\}\ \text{COMPUTED IN TABLE}$$

$K = 1.22$ (SECTION 12.8.3)

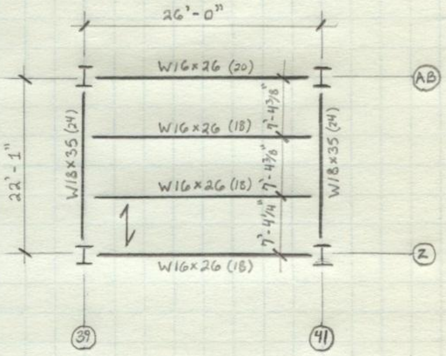
Appendix C: Seismic Calculations

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.)

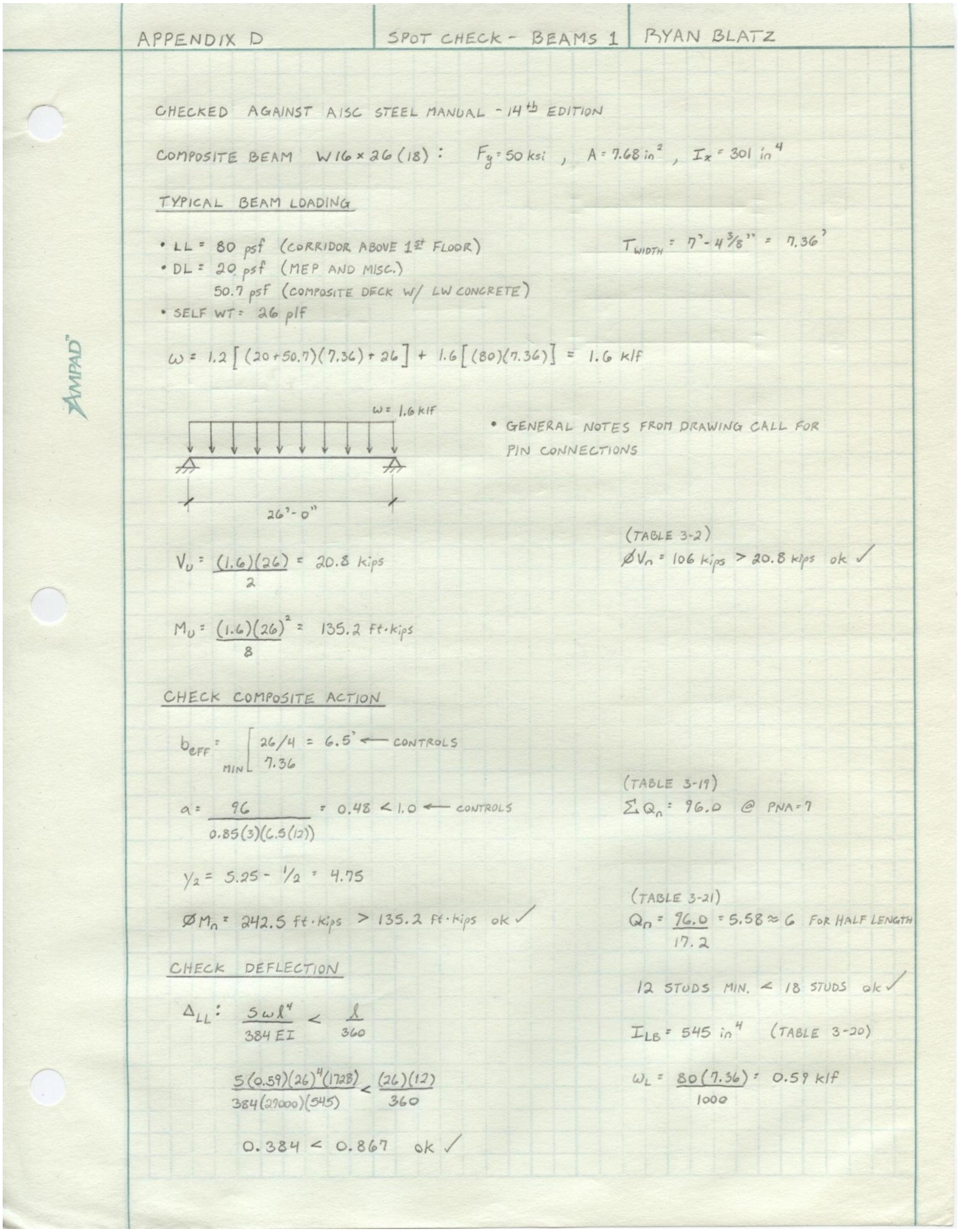
100 psf

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 – 14th 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 1st FLOOR)
- DL = 20 psf (MEP AND MISC.)
  50.7 psf (COMPOSITE DECK W/ LW CONCRETE)
- SELF WT = 26 plf

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

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

- 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 ✓

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

CHECK COMPOSITE ACTION

$$b_{EFF} = \min\left[\begin{array}{l} 26/4 = 6.5' \leftarrow \text{CONTROLS} \\ 7.36 \end{array}\right.$$

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

(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)
$Q_n = \frac{96.0}{17.2} = 5.58 \approx 6$ FOR HALF LENGTH

12 STUDS MIN. < 18 STUDS ok ✓

CHECK DEFLECTION

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

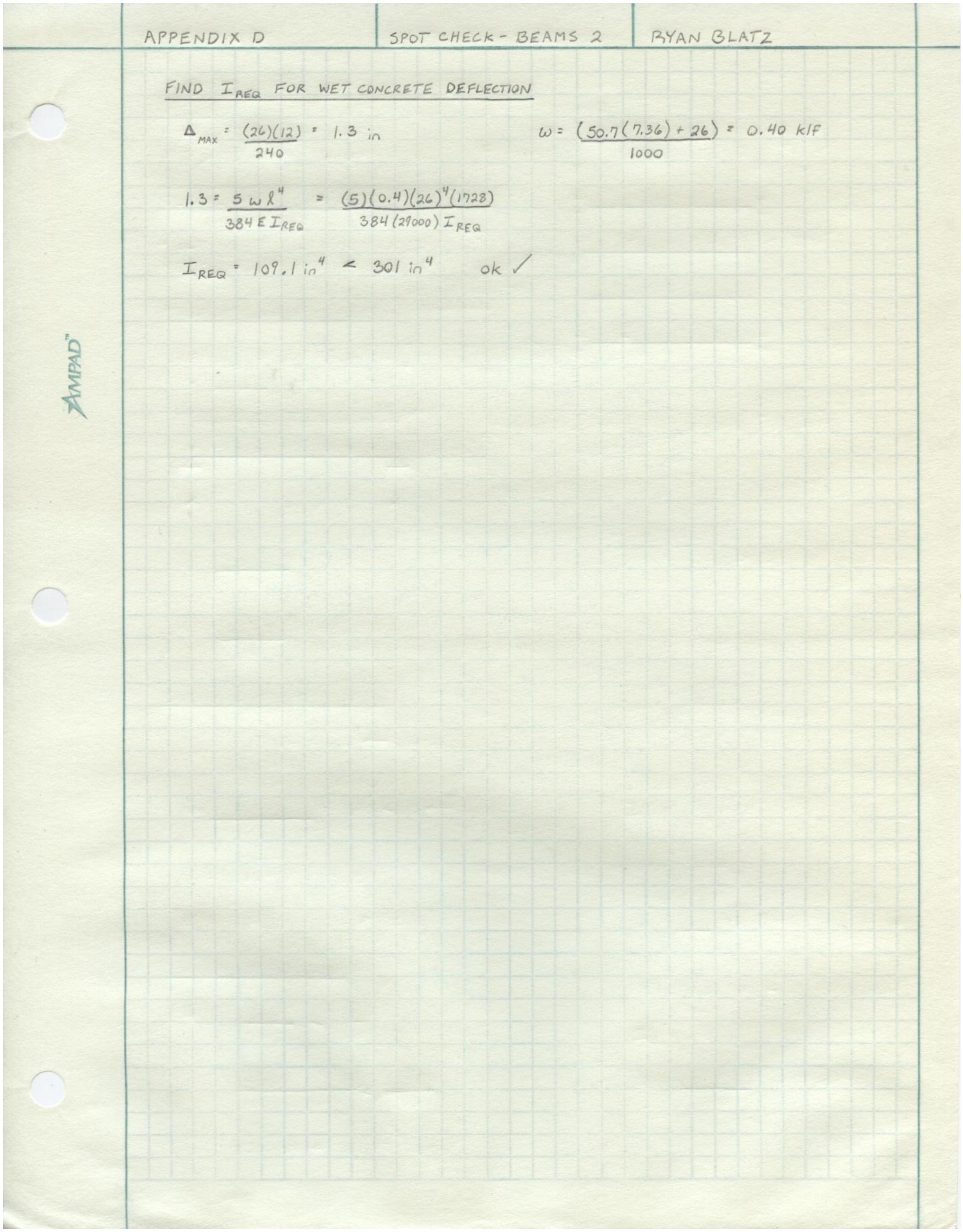
$$\frac{5(0.59)(26)^4(1728)}{384(29000)(545)} < \frac{(26)(12)}{360}$$

$$0.384 < 0.867 \quad \text{ok} ✓$$

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

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

Appendix D: Spot Check – Beams



APPENDIX D | SPOT CHECK - BEAMS 2 | RYAN BLATZ

FIND I_{REQ} FOR WET CONCRETE DEFLECTION

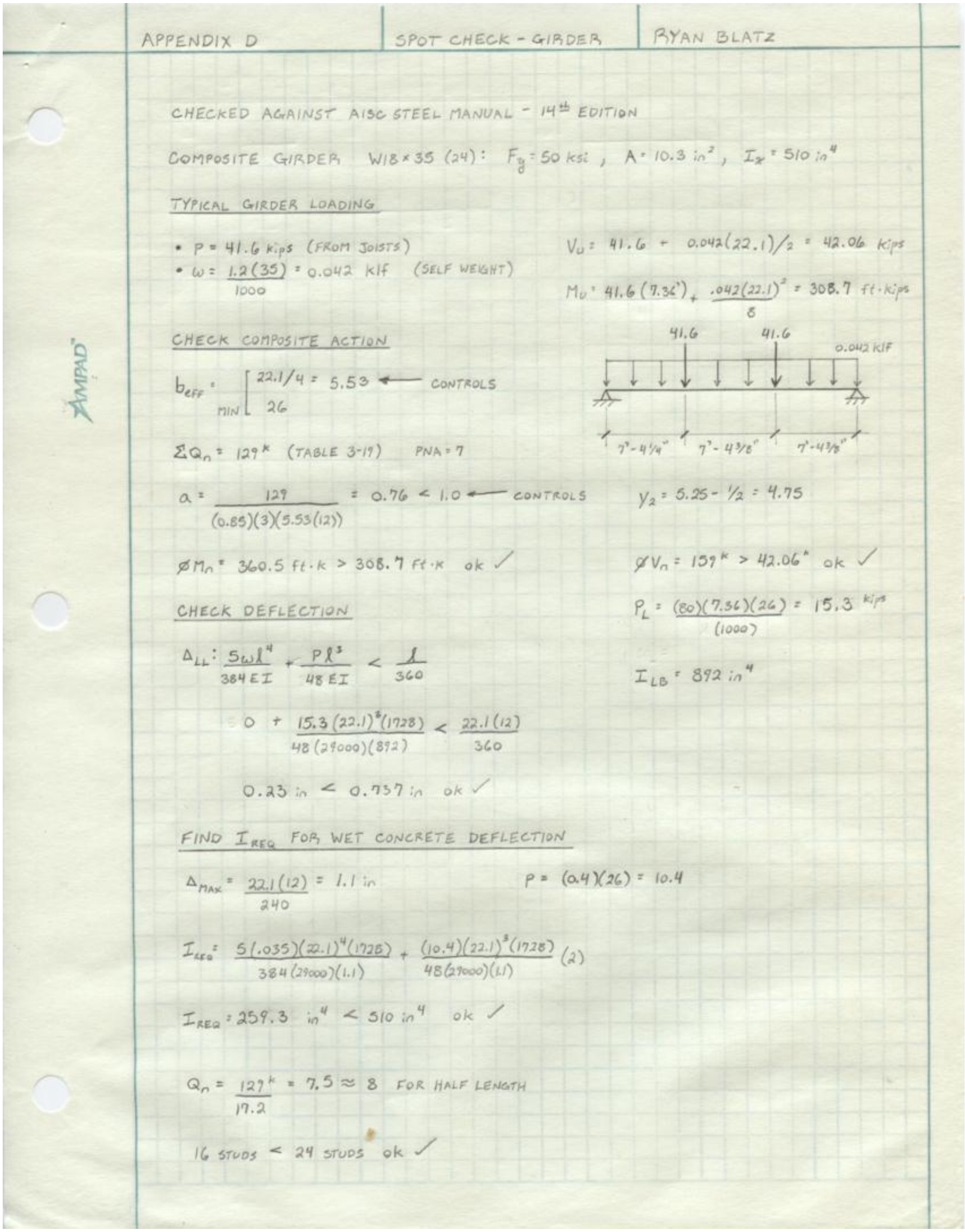
$$\Delta_{MAX} = \frac{(26)(12)}{240} = 1.3 \text{ in}$$

$$\omega = \frac{(50.7(7.36) + 26)}{1000} = 0.40 \text{ klf}$$

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

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

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 = 41.6 kips (FROM JOISTS)
- $\omega = \frac{1.2(35)}{1000} = 0.042$ klf (SELF WEIGHT)

$V_u = 41.6 + 0.042(22.1)/2 = 42.06$ kips

$M_u = 41.6(7.36') + \frac{.042(22.1)^2}{8} = 308.7$ ft·kips

CHECK COMPOSITE ACTION

$b_{eff} = \min \left[\begin{array}{l} 22.1/4 = 5.53 \longleftarrow \text{CONTROLS} \\ 26 \end{array} \right.$

$\Sigma Q_n = 129^k$ (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 > 308.7 ft·k ok ✓

$\phi V_n = 159^k > 42.06^k$ ok ✓

CHECK DEFLECTION

$P_L = \frac{(80)(7.36)(26)}{(1000)} = 15.3$ kips

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

$I_{LB} = 892$ in^4

$0 + \frac{15.3(22.1)^3(1728)}{48(29000)(892)} < \frac{22.1(12)}{360}$

0.23 in < 0.737 in ok ✓

FIND I_{REQ} FOR WET CONCRETE DEFLECTION

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

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

$I_{REQ} = \frac{5(.035)(22.1)^4(1728)}{384(29000)(1.1)} + \frac{(10.4)(22.1)^3(1728)}{48(29000)(1.1)}(2)$

$I_{REQ} = 259.3$ in^4 < 510 in^4 ok ✓

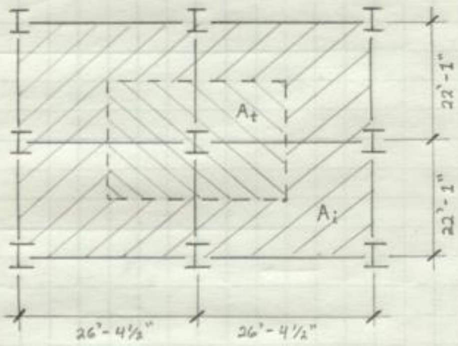
$Q_n = \frac{129^k}{17.2} = 7.5 \approx 8$ FOR HALF LENGTH

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

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

$L = 44.86 > .4L_o$ ✓

$$DL + COL.\ WT = 324.4 + \frac{53(39.5)}{1000} + \frac{58(26)}{1000} = 328.0\ kips$$

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 ✓

SYSTEM WEIGHT CALCULATION

$A_t = (26)(22.1) = 574.2\ ft^2$

DECK w/ CONC. : $1.97 + (110)(.271) = 31.78\ psf$

$$1.2\left[(6)(31.78+20)(574.2) + (6)(35(22.1) + 26(26)(4))\right] + 1.6\left[100(574.2) + 45(574.2)(5)\right]$$

$$\text{WEIGHT ON FOOTING} : \frac{537.7\ kips}{1764\ in^2} = 305\ psi \text{ ON SOIL} < 4{,}000\ psi$$

FROM GEOTECHNICAL REPORT,
FOUNDATION RATED FOR 4,000 psi

$$L = 80\left(0.25 + \frac{15}{\sqrt{2298.4}}\right)$$

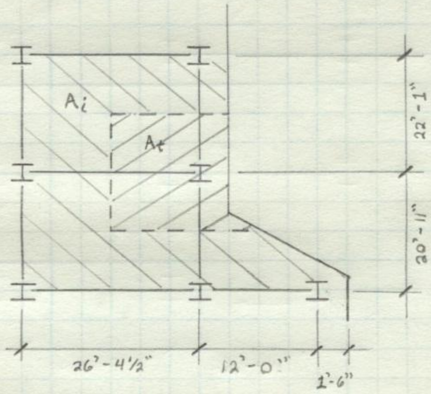
$L = 45\ psf$

$A_{FOOTING} = 1764\ in^2$

Appendix D: Spot Check – Column

APPENDIX D | SPOT CHECK - COLUMN 2 | RYAN BLATZ

COLUMN Z43: W12×53 ANALYZED AT FIRST FLOOR



EXTERIOR COLUMN

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

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

$DL = (88.83)(316.1)(5) + (24)(316.1) + (38)(21.5)(73.5) = 208.0 \text{ kips}$

$S = (42)(316.1) = 13.3 \text{ kips}$

$LL = (26.9)(316.1) + (40.3)(316.1)(4) = 59.5 \text{ kips}$

BRICK FAÇADE: 38 psf

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

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

$L = 40.3 \text{ psf} > 0.4L_o$ ✓

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

$DL + COL. WT = 210.8 \text{ kips}$

- 3rd FLOOR – PATIENT ROOMS

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

$L = 26.9 \text{ psf} > 0.4L_o$ ✓

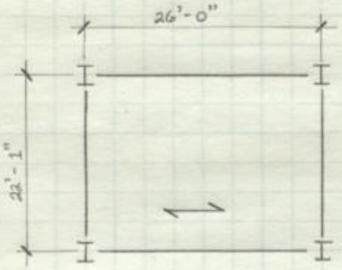
LOAD COMBINATIONS

$1.4D = 1.4(210.8) = 295.1 \text{ kips}$
$1.2D + 1.6L + 0.5S = 1.2(210.8) + 1.6(59.5) + 0.5(13.3) = 354.8 \text{ kips}$ ← CONTROLS

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

Appendix E: Precast Plank System

APPENDIX E | PRECAST HOLLOWCORE PLANK | RYAN BLATZ



PLANK SIZING

USE 10" × 4'-0" HOLLOW CORE PLANK (UNTOPPED)

- TO ACHIEVE 2 HOUR FIRE RATING

LOADING

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

100 psf

- SELF WT ALREADY ACCOUNTED FOR IN TABLES

$1.2D + 1.6L = 1.2(20) + 1.6(80) = 152$ psf

ACCORDING TO NITTERHOUSE HOLLOW CORE SPEC SHEET (SEE NEXT PAGE) FOR A 26'-0" SPAN:

$\boxed{\text{10" × 4'-0" UNTOPPED HOLLOW CORE PLANK WITH 6-1/2" Ø STRAND PATTERN}}$

176 psf (FROM TABLES) > 152 psf ok ✓

NOTE: 4 ft WIDTHS WILL EITHER PUSH BAY SIZES TO BE 1'-11" LONGER OR 2'-1" SHORTER IN THE PLAN EAST AND WEST DIRECTIONS

NEW BAY SIZES: 26'-0" × 24'-0" OR 26'-0" × 20'-0"

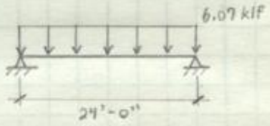
STEEL GIRDER SIZING USE WORST CASE BAY: 26'-0" × 24'-0"

LOADING: LL = 80 psf
DL = 20 psf
PLANK = 68.0 psf
168.0 psf

TRIB WIDTH: 26'-0"

$w = (233.6)(26) = 6.07$ klf

$1.2D + 1.6L = 1.2(88.0) + 1.6(80) = 233.6$ psf



$$V_u = \frac{(6.07)(24)}{2} = 72.8^k$$

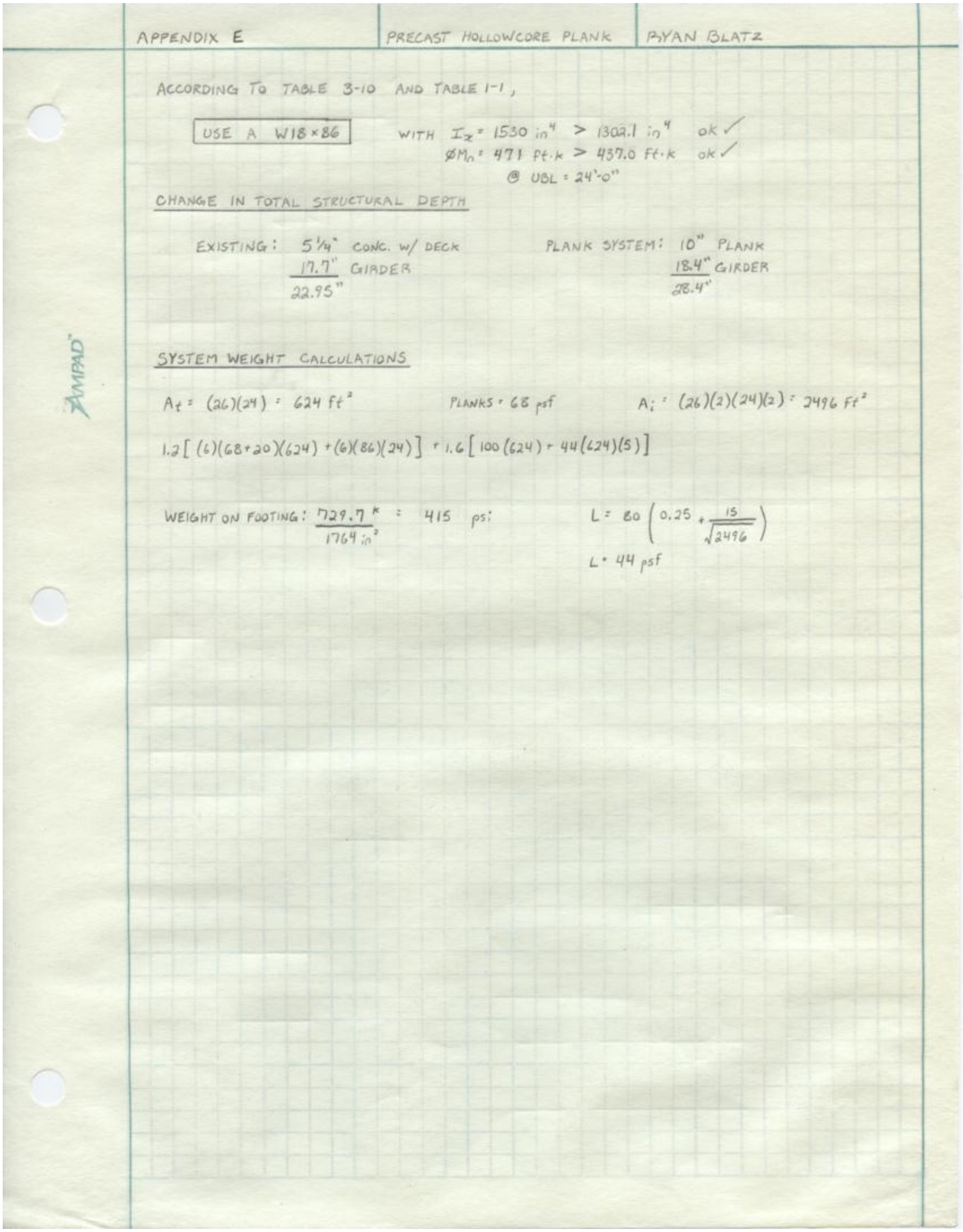
CHECK DEFLECTION

$$M_u = \frac{(6.07)(24)^2}{8} = 437.0 \text{ ft·k}$$

$$\Delta_{LL}: \frac{(24)(12)}{360} = \frac{5(2.08)(24)^4(1728)}{384(29000)I} \qquad I_{REQ} = 669.3 \text{ in}^4$$

$$\Delta_{TL}: \frac{(24)(12)}{240} = \frac{5(6.07)(24)^4(1728)}{384(29000)I} \qquad I_{REQ} = 1302.1 \text{ in}^4$$

Appendix E: Precast Plank System



APPENDIX E | PRECAST HOLLOWCORE PLANK | RYAN BLATZ

ACCORDING TO TABLE 3-10 AND TABLE 1-1,

$\boxed{\text{USE A W18×86}}$ WITH $I_x = 1530 \text{ in}^4 > 1302.1 \text{ in}^4$ ok ✓

$\phi M_n = 471 \text{ ft·k} > 437.0 \text{ ft·k}$ ok ✓

@ UBL = 24'-0"

CHANGE IN TOTAL STRUCTURAL DEPTH

EXISTING: 5 1/4" CONC. w/ DECK
17.7" GIRDER
22.95"

PLANK SYSTEM: 10" PLANK
18.4" GIRDER
28.4"

SYSTEM WEIGHT CALCULATIONS

$A_t = (26)(24) = 624 \text{ ft}^2$

PLANKS = 68 psf

$A_i = (26)(2)(24)(2) = 2496 \text{ ft}^2$

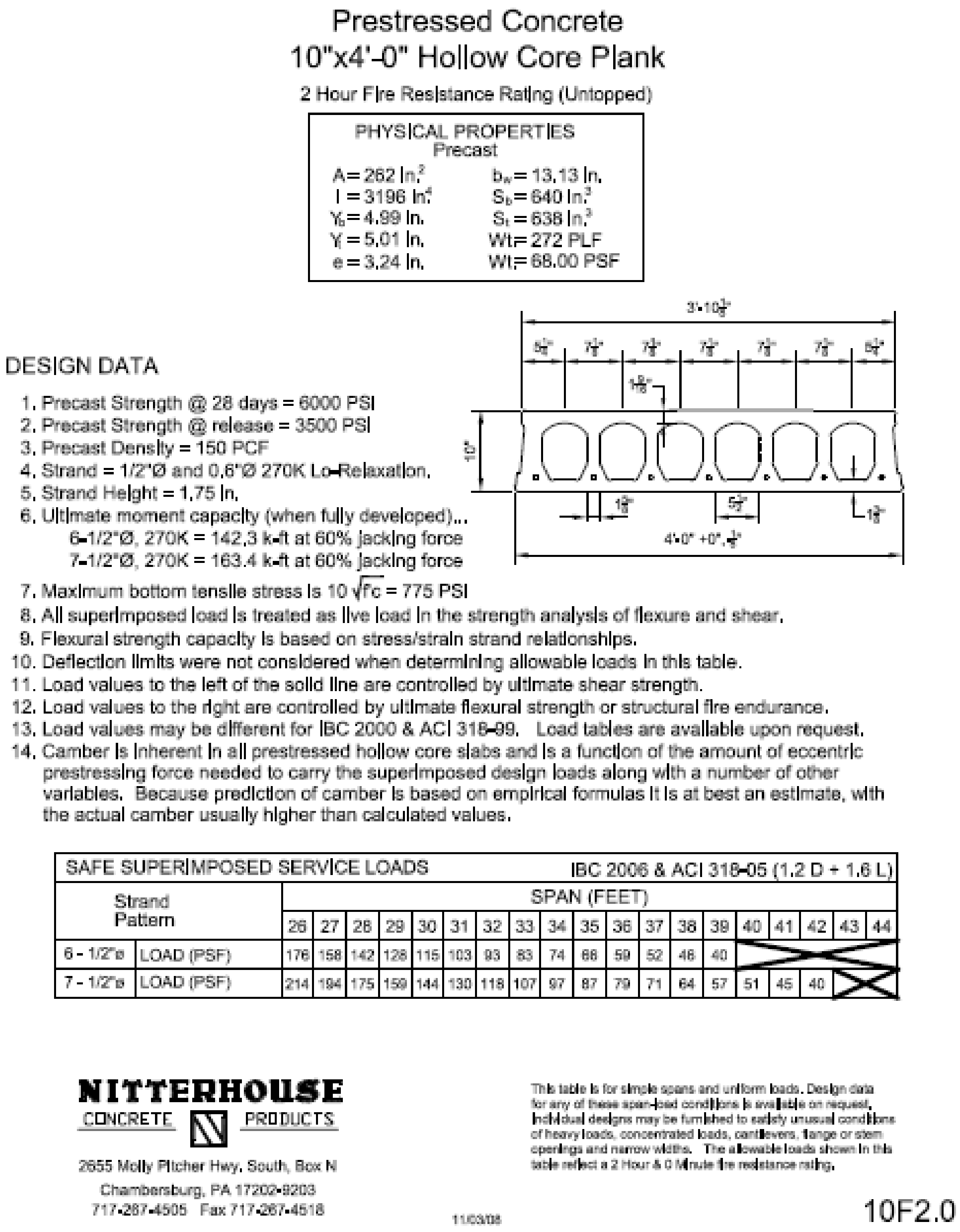
$$1.2\left[(6)(68+20)(624)+(6)(86)(24)\right]+1.6\left[100(624)+44(624)(5)\right]$$

WEIGHT ON FOOTING: $\dfrac{729.7^k}{1764 \text{ in}^2} = 415$ psi

$$L = 80\left(0.25+\frac{15}{\sqrt{2496}}\right)$$

$L = 44$ psf

Appendix E: Precast Plank System



Prestressed Concrete 10"x4'-0" Hollow Core Plank

2 Hour Fire Resistance Rating (Untopped)

PHYSICAL PROPERTIES
Precast

$A = 262$ in.^{2}
$I = 3196$ in.^{4}
$Y_b = 4.99$ in.
$Y_t = 5.01$ in.
$e = 3.24$ in.
$b_w = 13.13$ in.
$S_b = 640$ in.^{3}
$S_t = 638$ in.^{3}
Wt = 272 PLF
Wt = 68.00 PSF

DESIGN DATA

1. Precast Strength @ 28 days = 6000 PSI
2. Precast Strength @ release = 3500 PSI
3. Precast Density = 150 PCF
4. Strand = 1/2"Ø and 0.6"Ø 270K Lo-Relaxation.
5. Strand Height = 1.75 in.
6. Ultimate moment capacity (when fully developed)...
   6-1/2"Ø, 270K = 142.3 k-ft at 60% jacking force
   7-1/2"Ø, 270K = 163.4 k-ft at 60% jacking force
7. Maximum bottom tensile stress is $10\sqrt{f'c} = 775$ PSI
8. All superimposed load is treated as live load in the strength analysis of flexure and shear.
9. Flexural strength capacity is based on stress/strain strand relationships.
10. Deflection limits were not considered when determining allowable loads in this table.
11. Load values to the left of the solid line are controlled by ultimate shear strength.
12. Load values to the right are controlled by ultimate flexural strength or structural fire endurance.
13. Load values may be different for IBC 2000 & ACI 318-99. Load tables are available upon request.
14. Camber is inherent in all prestressed hollow core slabs and is a function of the amount of eccentric prestressing force needed to carry the superimposed design loads along with a number of other variables. Because prediction of camber is based on empirical formulas it is at best an estimate, with the actual camber usually higher than calculated values.

SAFE SUPERIMPOSED SERVICE LOADS — IBC 2006 & ACI 318-05 (1.2 D + 1.6 L)

Strand Pattern		SPAN (FEET)																		
		26	27	28	29	30	31	32	33	34	35	36	37	38	39	40	41	42	43	44
6 - 1/2"ø	LOAD (PSF)	176	158	142	128	115	103	93	83	74	66	59	52	46	40					
7 - 1/2"ø	LOAD (PSF)	214	194	175	159	144	130	118	107	97	87	79	71	64	57	51	45	40		

NITTERHOUSE
CONCRETE N PRODUCTS

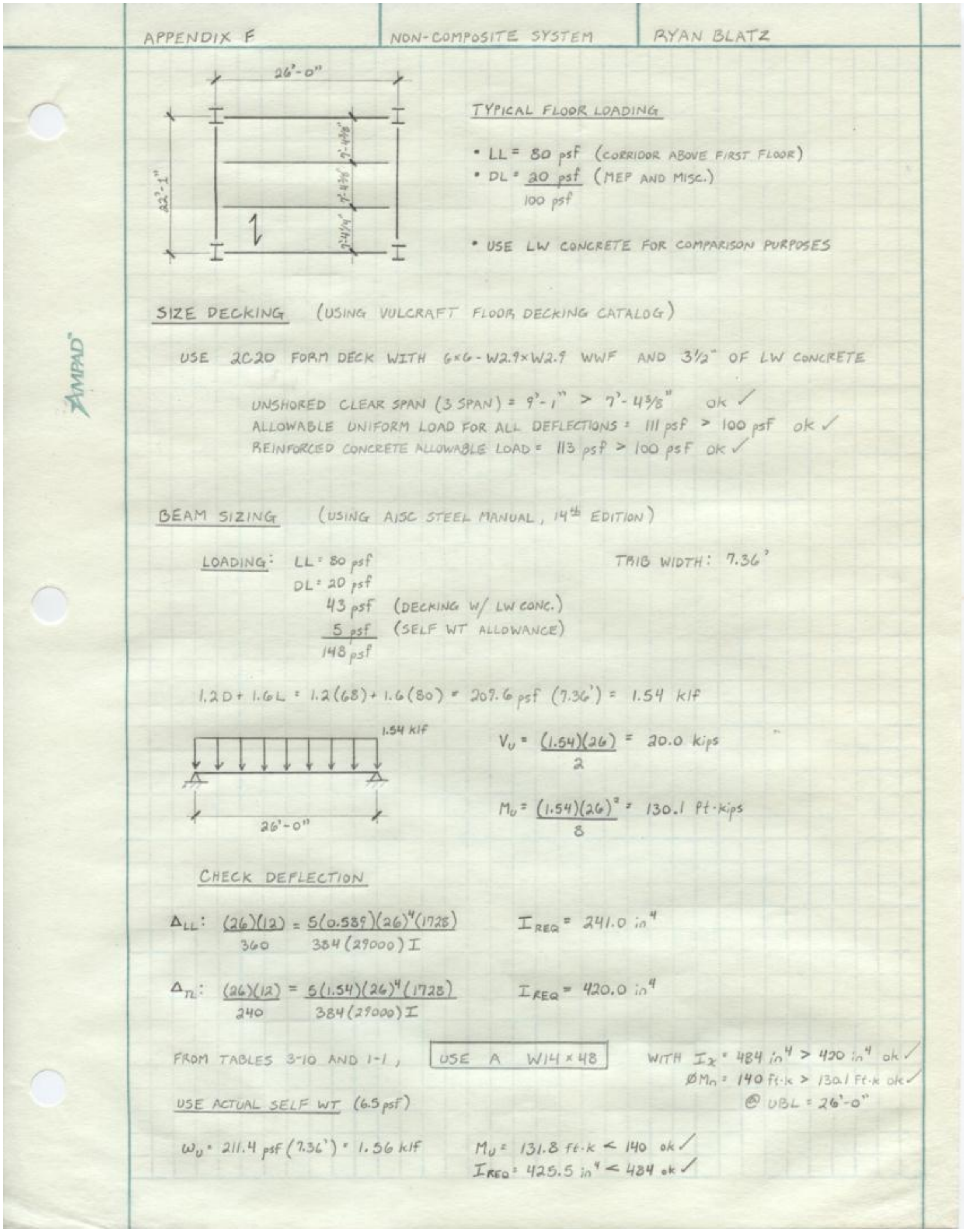
2655 Molly Pitcher Hwy. South, Box N
Chambersburg, PA 17202-9203
717-267-4505 Fax 717-267-4518

This table is for simple spans and uniform loads. Design data for any of these span-load conditions is available on request. Individual designs may be furnished to satisfy unusual conditions of heavy loads, concentrated loads, cantilevers, flange or stem openings and narrow widths. The allowable loads shown in this table reflect a 2 Hour & 0 Minute fire resistance rating.

11/03/08

10F2.0

Appendix F: Non-Composite System



APPENDIX F | NON-COMPOSITE SYSTEM | RYAN BLATZ

TYPICAL FLOOR LOADING

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

  100 psf

- USE LW CONCRETE FOR COMPARISON PURPOSES

SIZE DECKING (USING VULCRAFT FLOOR DECKING CATALOG)

USE 2C20 FORM DECK WITH 6×6-W2.9×W2.9 WWF AND 3½" OF LW CONCRETE

UNSHORED CLEAR SPAN (3 SPAN) = 9'-1" > 7'-4 3/8" ok ✓
ALLOWABLE UNIFORM LOAD FOR ALL DEFLECTIONS = 111 psf > 100 psf ok ✓
REINFORCED CONCRETE ALLOWABLE LOAD = 113 psf > 100 psf ok ✓

BEAM SIZING (USING AISC STEEL MANUAL, 14th EDITION)

LOADING: LL = 80 psf
DL = 20 psf
43 psf (DECKING w/ LW CONC.)
5 psf (SELF WT ALLOWANCE)
148 psf

TRIB WIDTH: 7.36'

$1.2D + 1.6L = 1.2(68) + 1.6(80) = 207.6\text{ psf}\,(7.36') = 1.54\text{ klf}$

$V_u = \frac{(1.54)(26)}{2} = 20.0\text{ kips}$

$M_u = \frac{(1.54)(26)^2}{8} = 130.1\text{ ft-kips}$

CHECK DEFLECTION

$\Delta_{LL}: \frac{(26)(12)}{360} = \frac{5(0.589)(26)^4(1728)}{384(29000)I}$ $I_{REQ} = 241.0\text{ in}^4$

$\Delta_{TL}: \frac{(26)(12)}{240} = \frac{5(1.54)(26)^4(1728)}{384(29000)I}$ $I_{REQ} = 420.0\text{ in}^4$

FROM TABLES 3-10 AND 1-1, $\boxed{\text{USE A W14×48}}$ WITH $I_x = 484\text{ in}^4 > 420\text{ in}^4$ ok ✓
$\varnothing M_n = 140\text{ ft·k} > 130.1\text{ ft·k}$ ok ✓
@ UBL = 26'-0"

USE ACTUAL SELF WT (6.5 psf)

$w_u = 211.4\text{ psf}\,(7.36') = 1.56\text{ klf}$ $M_u = 131.8\text{ ft·k} < 140$ ok ✓
$I_{REQ} = 425.5\text{ in}^4 < 484$ ok ✓

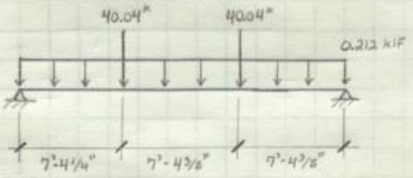
Appendix F: Non-Composite System

APPENDIX F | NON-COMPOSITE SYSTEM | RYAN BLATZ

GIRDER SIZING

LOADING: $P = 40.04$ kips (FROM JOISTS)

$\omega = \frac{1.2(8(22.1))}{1000} = 0.212$ kif (SELF WT ALLOWANCE)



$V_U = 40.04 + (.212)(22.1)/2 = 42.4$ kips

$M_U = 40.04(7.36') + \frac{(.212)(22.1)^2}{8} = 307.6$ ft·k

CHECK DEFLECTION

$\Delta_{LL}: \frac{(22.1)(12)}{360} = \frac{15.3(22.1)^3(1728)}{48(29000)I}$ $\quad I_{REQ} = 278.3 \text{ in}^4$

$\Delta_{TL}: \frac{(22.1)(12)}{240} = \frac{5(.212)(22.1)^4(1728)}{384(29000)I} + \frac{(40.04)(22.1)^3(1728)}{48(29000)I}$

$\frac{(22.1)(12)}{240} = \frac{5(.212)(22.1)^4(1728)}{384(29000)I} + \frac{(8)(40.04)(22.1)^3(1728)}{(8)(48)(29000)I}$ $\quad I_{REQ} = 521.0 \text{ in}^4$

FROM TABLES 3-10 AND 1-1, $\boxed{\text{USE A W14×68}}$ WITH $I_x = 722 \text{ in}^4 > 521.0 \text{ in}^4$ ok ✓

$\phi M_n = 323$ ft·k $> 307.6 \text{ in}^4$ ok ✓

@ UBL = 22'-0"

COST COMPARISON

EXISTING:
W16×26 (4)(26) = 2.7 kips
W18×35 (2)(22.1) = 1.55 kips
122 STUDS (10 lbs/ea.) = 1.22 kips
5.47 kips

NON-COMPOSITE:
W14×48 (4)(26) = 4.99 kips
W14×68 (2)(22.1) = 3.01 kips
8.0 kips

BASED SOLEY ON MATERIAL COSTS, THE NON-COMPOSITE SYSTEM WILL COST MORE

SYSTEM WEIGHT CALCULATIONS

DECK w/ CONC = 43 psf

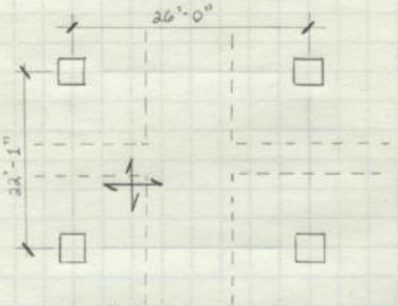
A_t AND L_{RED} FROM COMPOSITE WEIGHT

$1.2[(6)(43+20)(574.2) + (6)(68)(22.1) + (6)(48)(26)(4)] + 1.6[100(574.2) + 45(574.2)(5)]$

WEIGHT ON FOOTING: $\frac{605.8^k}{1764 \text{ in}^2} = 343$ psi ON SOIL

Appendix G: Two-Way Flat Slab

APPENDIX G | TWO-WAY CONC. FLAT SLAB | RYAN BLATZ



FROM CRSI MANUAL FOR PRELIMINARY FLAT SLAB DESIGN

$\ell_2/\ell_1 = 22.1/26 = 0.85$

FOR ℓ_2/ℓ_1 CLOSE TO 1.0, DESIGN USING TABLES FOR LONGER SPAN (26'-0")

LOADING

- LL = 80 psf (CORRIDOR ABOVE FIRST FLOOR)
- DL = $\underline{20 \text{ psf}}$ (MEP AND MISC.)

  100 psf

$h = \ell_n/36 > h_{min}$ (TABLE 10-1)

$h = \frac{[(26)(12)]}{36} > 4"$

$h = 8.7" \approx 9" > 4"$ ok ✓

$1.2D + 1.6L = 1.2(20) + 1.6(80) = 152$ psf

FROM TABLES FOR h = 9" AT 26'-0" SPAN AND 200psf (CONSERVATIVELY)

DROP PANEL: WIDTH - 8.67 ft
DEPTH - 6.25 in

COLUMN SIZE: 19" × 19"

REINFORCING: COLUMN STRIP
TOP: (12) #6's
BOTTOM: (21) #4's
MIDDLE STRIP
TOP: (16) #4's
BOTTOM: (9) #5's

$f'_c = 4{,}000$ psi
GRADE 60 REBAR
NW CONCRETE

CHANGE IN STRUCTURAL DEPTH:

EXISTING: 22.95" (FROM APPENDIX B)

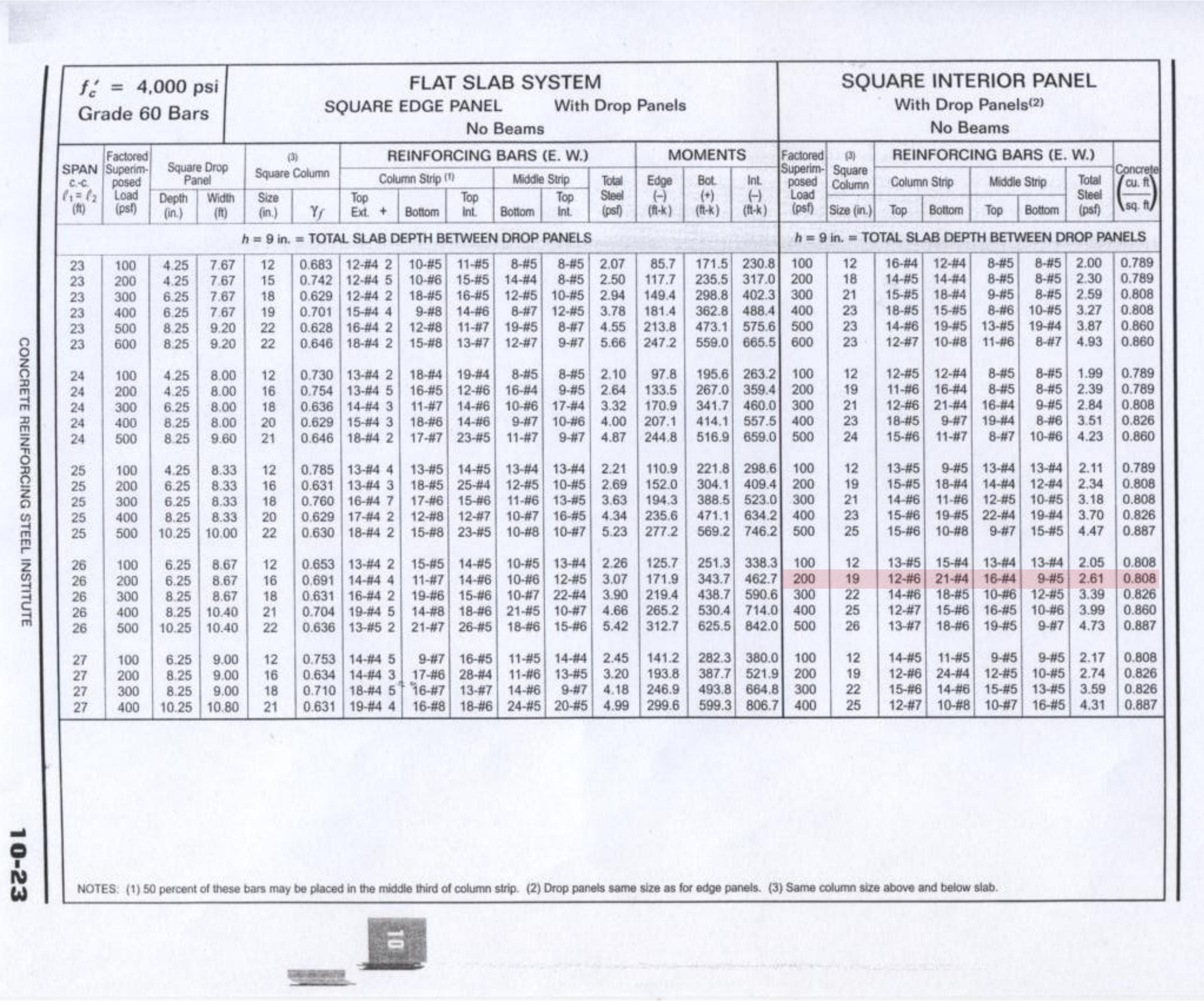
FLAT SLAB: 9" + 6.25" = 15.25"

SYSTEM WEIGHT CALCULATION $A_t = 574.2\ ft^2$

$$1.2\left[574.2\left(150\left(9/12\right) + 20\right) + 8.67^2\left(150\left(6.25/12\right)\right)\right] + 1.6\left[100(574.2) + 45(574.2)(5)\right]$$

WEIGHT ON FOOTING: $\frac{396.9^k}{1764\ in^2} = 225$ psi ON SOIL

Appendix G: Two-Way Flat Slab



f'_c = 4,000 psi
Grade 60 Bars

FLAT SLAB SYSTEM
SQUARE EDGE PANEL With Drop Panels
No Beams

SQUARE INTERIOR PANEL
With Drop Panels(2)
No Beams

SPAN c.-c. $\ell_1 = \ell_2$ (ft)	Factored Superimposed Load (psf)	Square Drop Panel		(3) Square Column		REINFORCING BARS (E. W.)							MOMENTS			Factored Superimposed Load (psf)	(3) Square Column	REINFORCING BARS (E. W.)					Concrete (cu. ft / sq. ft)
						Column Strip (1)				Middle Strip								Column Strip		Middle Strip			
		Depth (in.)	Width (ft)	Size (in.)	γ_f	Top Ext.	+	Bottom	Top Int.	Bottom	Top Int.	Total Steel (psf)	Edge (−) (ft-k)	Bot. (+) (ft-k)	Int. (−) (ft-k)		Size (in.)	Top	Bottom	Top	Bottom	Total Steel (psf)	
h = 9 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS																h = 9 in. = TOTAL SLAB DEPTH BETWEEN DROP PANELS							
23	100	4.25	7.67	12	0.683	12-#4	2	10-#5	11-#5	8-#5	8-#5	2.07	85.7	171.5	230.8	100	12	16-#4	12-#4	8-#5	8-#5	2.00	0.789
23	200	4.25	7.67	15	0.742	12-#4	5	10-#6	15-#5	14-#4	8-#5	2.50	117.7	235.5	317.0	200	18	14-#5	14-#4	8-#5	8-#5	2.30	0.789
23	300	6.25	7.67	18	0.629	12-#4	2	18-#5	16-#5	12-#5	10-#5	2.94	149.4	298.8	402.3	300	21	15-#5	18-#4	9-#5	8-#5	2.59	0.808
23	400	6.25	7.67	19	0.701	15-#4	4	9-#8	14-#6	8-#7	12-#5	3.78	181.4	362.8	488.4	400	23	18-#5	15-#5	8-#6	10-#5	3.27	0.808
23	500	8.25	9.20	22	0.628	16-#4	2	12-#8	11-#7	19-#5	8-#7	4.55	213.8	473.1	575.6	500	23	14-#6	19-#5	13-#5	19-#4	3.87	0.860
23	600	8.25	9.20	22	0.646	18-#4	2	15-#8	13-#7	12-#7	9-#7	5.66	247.2	559.0	665.5	600	23	12-#7	10-#8	11-#6	8-#7	4.93	0.860
24	100	4.25	8.00	12	0.730	13-#4	2	18-#4	19-#4	8-#5	8-#5	2.10	97.8	195.6	263.2	100	12	12-#5	12-#4	8-#5	8-#5	1.99	0.789
24	200	4.25	8.00	16	0.754	13-#4	5	16-#5	12-#6	16-#4	9-#5	2.64	133.5	267.0	359.4	200	19	11-#6	16-#4	8-#5	8-#5	2.39	0.789
24	300	6.25	8.00	18	0.636	14-#4	3	11-#7	14-#6	10-#6	17-#4	3.32	170.9	341.7	460.0	300	21	12-#6	21-#4	16-#4	9-#5	2.84	0.808
24	400	8.25	8.00	20	0.629	15-#4	3	18-#6	14-#6	9-#7	10-#6	4.00	207.1	414.1	557.5	400	23	18-#5	9-#7	19-#4	8-#6	3.51	0.826
24	500	8.25	9.60	21	0.646	18-#4	2	17-#7	23-#5	11-#7	9-#7	4.87	244.8	516.9	659.0	500	24	15-#6	11-#7	8-#7	10-#6	4.23	0.860
25	100	4.25	8.33	12	0.785	13-#4	4	13-#5	14-#5	13-#4	13-#4	2.21	110.9	221.8	298.6	100	12	13-#5	9-#5	13-#4	13-#4	2.11	0.789
25	200	6.25	8.33	16	0.631	13-#4	3	18-#5	25-#4	12-#5	10-#5	2.69	152.0	304.1	409.4	200	19	15-#5	18-#4	14-#4	12-#4	2.34	0.808
25	300	6.25	8.33	18	0.760	16-#4	7	17-#6	15-#6	11-#6	13-#5	3.63	194.3	388.5	523.0	300	21	14-#6	11-#6	12-#5	10-#5	3.18	0.808
25	400	8.25	8.33	20	0.629	17-#4	2	12-#8	12-#7	10-#7	16-#5	4.34	235.6	471.1	634.2	400	23	15-#6	19-#5	22-#4	19-#4	3.70	0.826
25	500	10.25	10.00	22	0.630	18-#4	2	15-#8	23-#5	10-#8	10-#7	5.23	277.2	569.2	746.2	500	25	15-#6	10-#8	9-#7	15-#5	4.47	0.887
26	100	6.25	8.67	12	0.653	13-#4	2	15-#5	14-#5	10-#5	13-#4	2.26	125.7	251.3	338.3	100	12	13-#5	15-#4	13-#4	13-#4	2.05	0.808
26	200	6.25	8.67	16	0.691	14-#4	4	11-#7	14-#6	10-#6	12-#5	3.07	171.9	343.7	462.7	200	19	12-#6	21-#4	16-#4	9-#5	2.61	0.808
26	300	8.25	8.67	18	0.631	16-#4	2	19-#6	15-#6	10-#7	22-#4	3.90	219.4	438.7	590.6	300	22	14-#6	18-#5	10-#6	12-#5	3.39	0.826
26	400	8.25	10.40	21	0.704	19-#4	5	14-#8	18-#6	21-#5	10-#7	4.66	265.2	530.4	714.0	400	25	12-#7	15-#6	16-#5	10-#6	3.99	0.860
26	500	10.25	10.40	22	0.636	13-#5	2	21-#7	26-#5	18-#6	15-#6	5.42	312.7	625.5	842.0	500	26	13-#7	18-#6	19-#5	9-#7	4.73	0.887
27	100	6.25	9.00	12	0.753	14-#4	5	9-#7	16-#5	11-#5	14-#4	2.45	141.2	282.3	380.0	100	12	14-#5	11-#5	9-#5	9-#5	2.17	0.808
27	200	8.25	9.00	16	0.634	14-#4	3	17-#6	28-#4	11-#6	13-#5	3.20	193.8	387.7	521.9	200	19	12-#6	24-#4	12-#5	10-#5	2.74	0.826
27	300	8.25	9.00	18	0.710	18-#4	5	16-#7	13-#7	14-#6	9-#7	4.18	246.9	493.8	664.8	300	22	15-#6	14-#6	15-#5	13-#5	3.59	0.826
27	400	10.25	10.80	21	0.631	19-#4	4	16-#8	18-#6	24-#5	20-#5	4.99	299.6	599.3	806.7	400	25	12-#7	10-#8	10-#7	16-#5	4.31	0.887

NOTES: (1) 50 percent of these bars may be placed in the middle third of column strip. (2) Drop panels same size as for edge panels. (3) Same column size above and below slab.

CONCRETE REINFORCING STEEL INSTITUTE

10-23

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Appendix H: Lateral Stiffness

STORY 2 RELATIVE STIFFNESS

EAST/WEST 100k APPLIED @ CR W/ DEFL OF 0.0064

Frame	Shear	Stiffness	Relative Stiffness
VB-28	2.88	449.3	0.029
VB-29	2.91	454.2	0.029
VB-14	10.79	1685.2	0.109
VB-13	3.02	472.2	0.031
VB-17	10.74	1678.7	0.109
VB-18	9.29	1452.0	0.094
VB-22	4.31	673.1	0.044
VB-21	4.60	718.7	0.047
VB-23	3.02	472.4	0.031
VB-24	2.74	427.3	0.028
VB-25	3.00	468.4	0.030
VB-26	2.86	446.2	0.029
VB-27	2.85	446.0	0.029
VB-38	2.51	392.5	0.025
VB-39	3.26	510.0	0.033
VB-40	1.91	298.7	0.019
VB-41	0.04	6.2	0.000
VB-43	4.69	732.5	0.047
VB-44	5.52	862.2	0.056
VB-49	5.95	929.1	0.060
VB-50	5.02	784.6	0.051
VB-36	6.86	1072.5	0.070
	Sum =	15432.0	

NORTH/SOUTH 100k APPLIED @ CR W/ DEFL OF 0.0085

Frame	Shear	Stiffness	Relative Stiffness
VB-1	0.20	23.7	0.002
VB-2	0.18	21.2	0.002
VB-3	5.33	626.9	0.054
VB-4	5.35	629.6	0.054
VB-5	5.05	594.4	0.051
VB-6	6.33	745.1	0.064
VB-8	10.25	1205.6	0.104
VB-9	5.34	627.7	0.054
VB-10	6.21	730.7	0.063
VB-11	4.39	516.5	0.045
VB-12	4.50	529.8	0.046
VB-30	4.58	538.5	0.047
VB-31	4.58	538.4	0.047
VB-33	9.16	1077.4	0.093
VB-34	5.05	594.4	0.051
VB-35	4.68	551.1	0.048
VB-48	3.70	435.6	0.038
VB-46	3.70	435.5	0.038
VB-45	3.68	432.4	0.037
VB-42	2.28	268.1	0.023
VB-47	3.68	432.5	0.037
	Sum =	11554.86	

Appendix H: Lateral Stiffness

STORY 7 RELATIVE STIFFNESS			
EAST/WEST 100k APPLIED @ CR W/ DEFL OF 0.071			
Frame	Shear	Stiffness	Relative Stiffness
VB-28	4.16	58.6	0.042
VB-29	4.39	61.8	0.044
VB-14	20.71	291.8	0.207
VB-13	3.28	46.2	0.033
VB-16	19.95	280.9	0.199
VB-18	14.05	197.9	0.141
VB-22	7.06	99.5	0.071
VB-21	7.03	99.0	0.070
VB-23	4.30	60.5	0.043
VB-24	3.84	54.1	0.038
VB-25	3.78	53.2	0.038
VB-26	3.68	51.9	0.037
VB-27	3.76	52.9	0.038
	Sum=	1408.2	
NORTH/SOUTH 100k APPLIED @ CR W/ DEFL OF 0.1023			
Frame	Shear	Stiffness	Relative Stiffness
VB-1	6.24	61.0	0.062
VB-2	6.24	61.0	0.062
VB-3	8.19	80.1	0.082
VB-4	9.29	90.8	0.093
VB-5	7.91	77.3	0.079
VB-6	8.37	81.8	0.084
VB-8	22.79	222.7	0.228
VB-9	8.83	86.3	0.088
VB-10	9.47	92.5	0.095
VB-11	6.26	61.2	0.063
VB-12	6.30	61.6	0.063
	Sum =	976.3	

Appendix I: Lateral Loadings

CASE I (P_{WX} + P_{LX})				
1)	*E/W-DIRECTION*		F (k)	
	ROOF		116.56	
	STORY 6		225.03	
	STORY 5		215.44	
	STORY 4		209.64	
	STORY 3		285.48	
	STORY 2		313.55	
2)	*N/S-DIRECTION*			
	ROOF		106.24	
	STORY 6		203.65	
	STORY 5		196.29	
	STORY 4		191.02	
	STORY 3		185.20	
	STORY 2		269.57	
DEFLECTIONS				
	Story	Load	UX	UY
	ROOF	XCASE1	0.4117	-0.0032
	ROOF	YCASE1	0	0.5179
	STORY6	XCASE1	0.3724	-0.0019
	STORY6	YCASE1	0.0012	0.4628
	STORY5	XCASE1	0.3189	-0.0007
	STORY5	YCASE1	0.0023	0.3899
	STORY4	XCASE1	0.2514	0.0004
	STORY4	YCASE1	0.0034	0.3
	STORY3	XCASE1	0.1664	0.0009
	STORY3	YCASE1	-0.0025	0.1977
	STORY2	XCASE1	0.0936	0.0001
	STORY2	YCASE1	0.0007	0.1046

CASE II (.75(P_{WX}+P_{LX})B_X(±.15B_X))				
3)	*E/W-DIRECTION*	F (k)	B_x (in)	M_T (k-in)
	ROOF	87.4	4759.25	-62409
	STORY 6	168.8	4759.25	120486
	STORY 5	161.6	4759.25	115347
	STORY 4	157.2	4759.25	112247
	STORY 3	214.1	6684.375	-214677
	STORY 2	235.2	6857.375	241892
4)	*N/S-DIRECTION*	F (k)	B_x (in)	M_T (k-in)
	ROOF	79.7	4307	51477
	STORY 6	152.7	4307	98676
	STORY 5	147.2	4307	95111
	STORY 4	143.3	4307	92554
	STORY 3	138.9	4307	89737
	STORY 2	202.2	5855	177564
DEFLECTIONS				
	Story	Load	UX	UY
	ROOF	YCASE2	0.0059	0.3955
	ROOF	XCASE2	0.3103	0.0005
	STORY6	YCASE2	0.0078	0.3532
	STORY6	XCASE2	0.2812	0.0012
	STORY5	YCASE2	0.0096	0.2976
	STORY5	XCASE2	0.2415	0.0017
	STORY4	YCASE2	0.0112	0.2292
	STORY4	XCASE2	0.1914	0.002
	STORY3	YCASE2	-0.0063	0.1507
	STORY3	XCASE2	0.1231	0.0015
	STORY2	YCASE2	0.0026	0.0786
	STORY2	XCASE2	0.0713	0.0001

Appendix I: Lateral Loadings

CASE III (.75PWX + .75PLX)

5)	E/W-DIRECTION	N/S-DIRECTION
ROOF	87.4	79.7
STORY 6	168.8	152.7
STORY 5	161.6	147.2
STORY 4	157.2	143.3
STORY 3	214.1	138.9
STORY 2	235.2	202.2

DEFLECTIONS

Story	Load	UX	UY
ROOF	CASE3	0.3088	0.3861
ROOF	CASE3	0.2802	0.3457
STORY5	CASE3	0.2409	0.2919
STORY4	CASE3	0.191	0.2253
STORY3	CASE3	0.1229	0.149
STORY2	CASE3	0.0708	0.0785

CASE IV

6) E/W-DIRECTION	F (k)	B_X (in)	N/S-DIRECTION	F (k)	B_Y (in)	M_T (k-in)
ROOF	65.6	4759.25	ROOF	-59.8	4307	-85490.3
STORY 6	126.7	4759.25	STORY 6	114.7	4307	16372.5
STORY 5	121.3	4759.25	STORY 5	110.5	4307	157983.9
STORY 4	118.0	4759.25	STORY 4	107.5	4307	153737
STORY 3	160.7	6684.375	STORY 3	-104.3	4307	-93788.6
STORY 2	176.5	6857.375	STORY 2	151.8	5855	314871.9

DEFLECTIONS

Story	Load	UX	UY
ROOF	CASE4	0.2377	0.1302
STORY6	CASE4	0.2167	0.1295
STORY5	CASE4	0.1879	0.1152
STORY4	CASE4	0.1511	0.0881
STORY3	CASE4	0.0884	0.0514
STORY2	CASE4	0.0553	0.0288

EARTHQUAKE

7) E/W-DIRECTION	F (k)
ROOF	744.3
STORY 6	895.3
STORY 5	741.4
STORY 4	781.6
STORY 3	573.5
STORY 2	541.1

8) N/S-DIRECTION	F (k)
ROOF	744.3
STORY 6	895.3
STORY 5	741.4
STORY 4	781.6
STORY 3	573.5
STORY 2	541.1

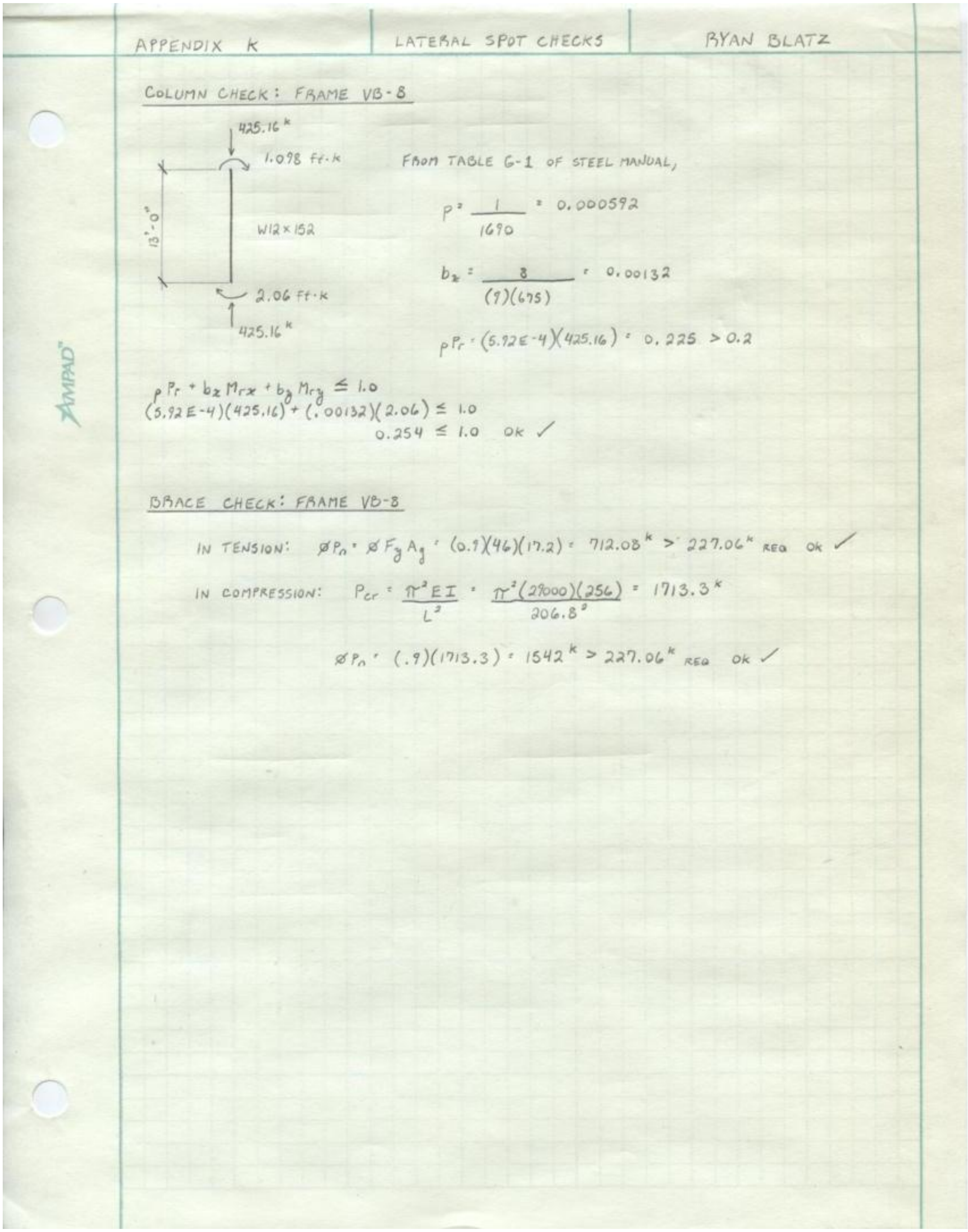
DEFLECTIONS

Story	Load	UX	UY
ROOF	EWXQUAKE	1.6427	-0.0259
ROOF	NSYQUAKE	0.0001	2.3692
STORY6	EWXQUAKE	1.4456	-0.018
STORY6	NSYQUAKE	0.0082	2.0666
STORY5	EWXQUAKE	1.1968	-0.0105
STORY5	NSYQUAKE	0.0159	1.6939
STORY4	EWXQUAKE	0.9043	-0.0035
STORY4	NSYQUAKE	0.0226	1.2654
STORY3	EWXQUAKE	0.5614	0.0014
STORY3	NSYQUAKE	-0.0158	0.7939
STORY2	EWXQUAKE	0.2962	0.0004
STORY2	NSYQUAKE	0.0043	0.3928

Appendix J: Torsional Shear

TORSIONAL SHEAR ON 2ND STORY FRAMES									
X-DIRECTION									
Frame	R	d_i	di2	Rdi2	k*d_i		Seismic $V_{TORSION}$	Seismic VDIRECT	Toatal Shear
VB-28	0.015494	105.854	11205.069	173.612165	2	0.001 F_x	0.27	15.75	16.02
VB-29	0.015663	738.854	545905.23	8550.652499	12	0.004 F_x	1.92	15.92	17.84
VB-14	0.05811	1315.896	1731582.3	100623.0301	76	0.023 F_x	12.69	59.08	71.77
VB-13	0.016283	2811.396	7903947.5	128698.9205	46	0.014 F_x	7.60	16.55	24.15
VB-17	0.057885	1315.896	1731582.3	100233.0509	76	0.023 F_x	12.64	58.85	71.49
VB-18	0.050067	179.354	32167.857	1610.559343	9	0.003 F_x	1.49	50.90	52.39
VB-22	0.023211	179.354	32167.857	746.6374598	4	0.001 F_x	0.69	23.60	24.29
VB-21	0.024781	179.354	32167.857	797.1597141	4	0.001 F_x	0.74	25.19	25.93
VB-23	0.016291	1683.854	2835364.3	46190.77305	27	0.008 F_x	4.55	16.56	21.11
VB-24	0.014736	1554.396	2416146.9	35604.32026	23	0.007 F_x	3.80	14.98	18.78
VB-25	0.016151	952.271	906820.06	14646.41426	15	0.005 F_x	2.55	16.42	18.97
VB-26	0.015387	736.271	542094.99	8341.428173	11	0.003 F_x	1.88	15.64	17.52
VB-27	0.015378	400.271	160216.87	2463.852372	6	0.002 F_x	1.02	15.63	16.66
VB-38	0.013533	3098.396	9600057.8	129916.2991	42	0.013 F_x	6.96	13.76	20.72
VB-39	0.017588	1683.854	2835364.3	49867.88611	30	0.009 F_x	4.91	17.88	22.79
VB-40	0.010301	1683.854	2835364.3	29206.08543	17	0.005 F_x	2.88	10.47	13.35
VB-41	0.000212	1055.354	1113772.1	236.4365268	0	0.000 F_x	0.04	0.22	0.25
VB-43	0.025259	3722.854	13859642	350075.4378	94	0.029 F_x	15.60	25.68	41.28
VB-44	0.02973	3722.854	13859642	412040.5825	111	0.034 F_x	18.36	30.22	48.59
VB-49	0.032037	1827.854	3341050.2	107036.017	59	0.018 F_x	9.72	32.57	42.29
VB-50	0.027054	1708.854	2920182	79003.82459	46	0.014 F_x	7.67	27.51	35.18
VB-36	0.036984	3098.396	9600057.8	355052.1367	115	0.035 F_x	19.01	37.60	56.61
SUM R=	0.532136		J =	1961115.117					

Appendix K: Lateral Spot Checks



APPENDIX K | LATERAL SPOT CHECKS | RYAN BLATZ

COLUMN CHECK: FRAME VB-8

425.16^k

1.098 ft·k

13'-0"

W12×152

2.06 ft·k

425.16^k

FROM TABLE 6-1 OF STEEL MANUAL,

$$p = \frac{1}{1690} = 0.000592$$

$$b_x = \frac{8}{(9)(675)} = 0.00132$$

$$pP_r = (5.92E-4)(425.16) = 0.225 > 0.2$$

$$pP_r + b_x M_{rx} + b_y M_{ry} \leq 1.0$$

$$(5.92E-4)(425.16) + (.00132)(2.06) \leq 1.0$$

$$0.254 \leq 1.0 \quad \text{OK} \checkmark$$

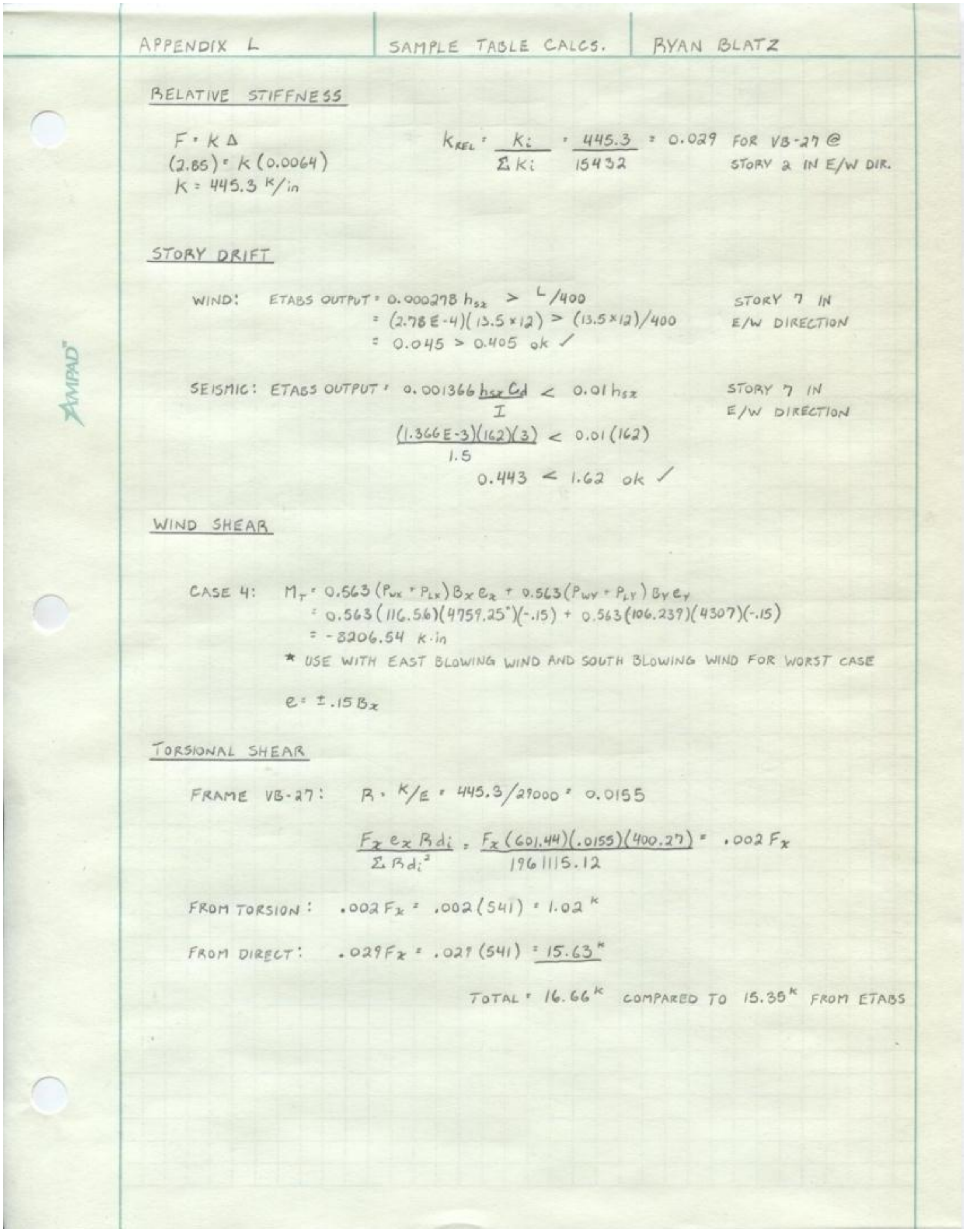
BRACE CHECK: FRAME VB-8

IN TENSION: $\phi P_n = \phi F_y A_g = (0.9)(46)(17.2) = 712.08^k > 227.06^k$ REQ OK ✓

IN COMPRESSION: $P_{cr} = \frac{\pi^2 EI}{L^2} = \frac{\pi^2(29000)(256)}{206.8^2} = 1713.3^k$

$$\phi P_n = (.9)(1713.3) = 1542^k > 227.06^k \text{ REQ} \quad \text{OK} \checkmark$$

Appendix L: Sample Calculations



APPENDIX L | SAMPLE TABLE CALCS. | RYAN BLATZ

RELATIVE STIFFNESS

$F = k\Delta$

$(2.85) = k(0.0064)$

$k = 445.3\ ^{k}/in$

$k_{REL} = \frac{k_i}{\Sigma k_i} = \frac{445.3}{15432} = 0.029$ FOR VB-27 @ STORY 2 IN E/W DIR.

STORY DRIFT

WIND: ETABS OUTPUT $= 0.000278\, h_{sx} > {}^{L}/400$ STORY 7 IN E/W DIRECTION

$= (2.78E{-4})(13.5 \times 12) > (13.5 \times 12)/400$

$= 0.045 > 0.405$ ok ✓

SEISMIC: ETABS OUTPUT $= 0.001366 \frac{h_{sx} C_d}{I} < 0.01 h_{sx}$ STORY 7 IN E/W DIRECTION

$\frac{(1.366E{-3})(162)(3)}{1.5} < 0.01(162)$

$0.443 < 1.62$ ok ✓

WIND SHEAR

CASE 4: $M_T = 0.563(P_{wx} + P_{Lx}) B_x e_x + 0.563(P_{wy} + P_{Ly}) B_y e_y$

$= 0.563(116.56)(4759.25'')(-.15) + 0.563(106.237)(4307)(-.15)$

$= -8206.54$ k·in

* USE WITH EAST BLOWING WIND AND SOUTH BLOWING WIND FOR WORST CASE

$e = \pm .15 B_x$

TORSIONAL SHEAR

FRAME VB-27: $R = {}^{K}/_{E} = 445.3/29000 = 0.0155$

$\frac{F_x e_x R d_i}{\Sigma R d_i^2} = \frac{F_x (601.44)(.0155)(400.27)}{1961115.12} = .002 F_x$

FROM TORSION: $.002 F_x = .002(541) = 1.02^{k}$

FROM DIRECT: $.029 F_x = .029(541) = \underline{15.63^{k}}$

TOTAL $= 16.66^{k}$ COMPARED TO 15.35^{k} FROM ETABS