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Technical Report 1
Bed Tower Addition at Appleton Medical Center

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Senior Thesis Studio

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Executive Summary

Analyzing of the addition and checking of the various areas of construction, results were fair and it was found that building is very well designed. The systems used within this building work well to help transfer the loads to the foundation in an efficient manner.

The braced frames help transfer loads both vertically and laterally to the base. The lateral loads which come into the building are transferred to these braced frames through the decking and slab which acts as a diaphragm to these frames. The bracing are advantageous in helping the building say structurally stable and taking load off the other structural members in the building.

It was easy to see that the wind loads controlled over the seismic loads when designing this building. This is logical because of the building's location away from a seismic region and in a more wind based area. Because of the flat exterior walls which run vertically throughout the entire building, wind loads did not change much throughout the building.

Structural components of the addition were designed conservatively, but these components will be able to hold a great amount of strength over time which makes them very durable and could be able to assist loads in abnormal situations. The large strength capacities of these structural members help make the system stiff ready to take any lateral loads that my occur.

Introduction

The Bed Tower Addition at Appleton Medical Center, owned by ThedaCare is located in Appleton, Wisconsin approximately two hours from Madison, Wisconsin. The building was measured at a height of 107'-3" above grade to the highest occupied floor which entails 9 stories including a basement and the total size is at 152,330 sq. ft. including the renovation which was done on the existing hospital it is attached to.

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The addition of the bed tower was put into place in order to accommodate more patients for the hospital. Because of its size, it stands out amongst the rest of the complex. It has a unique triangular shape layout which is carried throughout all the floors of the building. The horizontal streaks of CMU along the exterior make the addition look very sleek and long. Accommodating the long streaks are large areas of glass. Both materials work together in order to show floor separation and this gives the perspective that the addition is deceptively taller than it looks.

The first floor is the lobby area which consists of the registration and waiting area along with a mini coffee shop. The second floor is the office area which is a very large space and movable partitions. The third floor to the eighth floor consists of the patient rooms, waiting rooms, and floor manager offices. The second through fourth floor connect to

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the original hospital with the fourth floor extended into the original building.

The building façade was very simple and consists of two essential components which are a stone façade and large areas of glazing. Limestone and Cast Stone make up the entire exterior with the limestone making up the crown running along the bottom of building. The cast stone is what is seen throughout the rest of the exterior which makes up the vertical façade.


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Glazing makes up the other half of the exterior. There are three kinds of glazing. They are: 1) Clear Vision Glass 2) Tinted Visual Glass and 3) Spandrel Glass. The clear vision glass is used on the first floor where the lobby is located to allow the most daylight and energy. The tinted visual glass and spandrel glass work together to shade the patient rooms and stairwells and they don’t allow as much sunlight or energy as the clear vision glass.

Structurally, the addition is made up of a system of steel framing and composite deck. The foundation is a mat padding. On top of the roof, there is a large penthouse which holds the mechanical equipment which is all supported by the steel framing of the building. For lateral loads, cross bracing is integrated within the frame.

Code

International Code

- 2006 International Building Code
    - Live load reduction used for typical floor loads and corridors above the first floor.

Design Codes

- ASTM International
    - Concrete and testing of masonry
- ACI 318
    - Reinforced concrete design and construction
- AISC
    - Structural steel - Designed for “in place” loads
- SDI
    - Steel roof decking
    - Steel composite floor deck - Designed as unshored
- OSHA Safety Standards
    - Steel erection
    - Steel joist erection
    - Metal Decking erection
- ASCE 7-05
    - Wind loads

Bracing

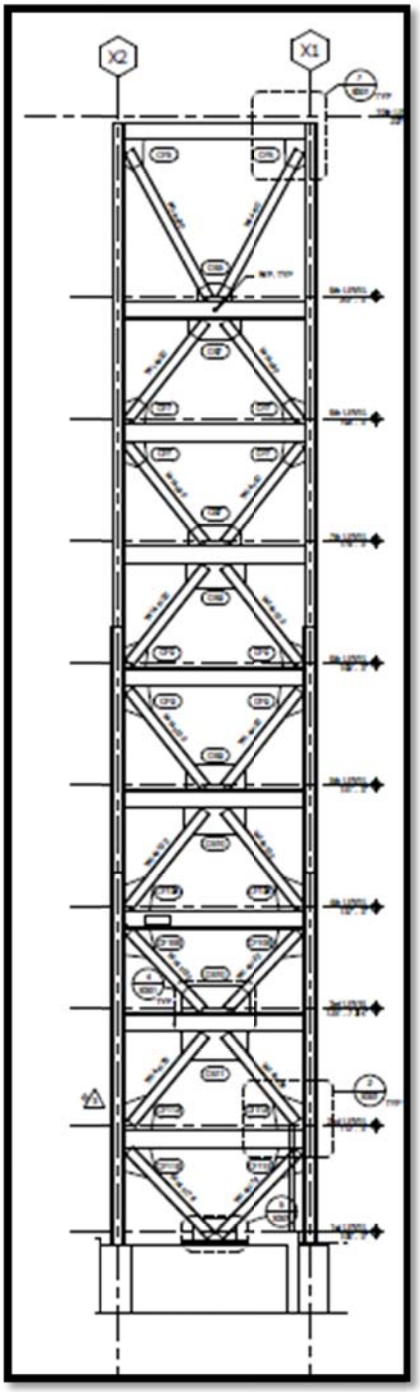


Figure 1

Steel braced frames in each direction resist the lateral loads while the concrete slabs act as the diaphragm which transfers the loads to the braced frames. There are 8 sections where the braced frames run vertically throughout the building. The typical frame runs from the top of the foundation to the top of the 10th level penthouse. Two others run to the top of the 9th level and the last one runs just between the 9th and 10th level. The locations of the braced frames help resist lateral loads from all directions. These locations can be found in figure 6 in the foundation section.

Connection to the bases, explained later in the foundation section, help transfer the lateral loads to the base. The braced beams are connected to the columns and floor beams by gusset plates for ease of construction and transfer of loads. Close-up of the braced frames are picture on the left in figure 2.

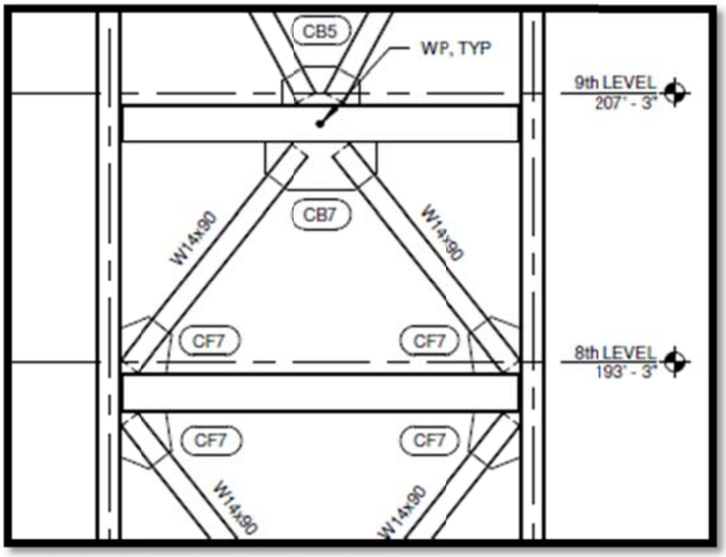


Figure 2

Figure 3

To the right are construction photos of the gusset plates used and connection to the foundation for the braced frames in figures 3 and 4 respectively.

Figure 4

Foundation

The geotechnical report was completed by River Valley Testing Corporation. Originally, the foundation was designed with spread footing in mind but after investigation by RVT, they recommended three alternatives which included the currently used mat foundation. Tests indicated that the natural soils on the site were able to hold bearing pressures ranging from 1,500 psf to more than 6,000 psf. The footings were then designed for a maximum soil bearing pressure of 3500 psf for just gravity loads and 4200 psf for gravity plus lateral loads. Footings range from 6 ft x 6 ft to 9 ft by 9 ft with depths being 1 to 2 ft. Maximum interior column loads were to be 1,500 kips and the maximum perimeter wall load be 3 kips per lineal foot.

Typical reinforcement for the mat slab includes the use of #7, #9, and #11 bars. The thickness of the mat slab is 3’6” throughout the entire foundation under the triangular side of the addition. The area where the addition connects to the original part of the building has various thicknesses with 12” being the typical.

Most importantly, the braced frames are connected at the foundation. The concrete bases. Typical thicknesses of these are 4 ft and stretch as long as the column line width. The columns are connected to the bases by plates which are then connected to the top of the concrete by 6 #6 hooks. The bases are reinforced by 5 #5 bars running horizontally and #5 bars running vertically spaced at 12” O.C. Pictured below is a section and elevation of the braced frame to foundation connection with reinforcement.

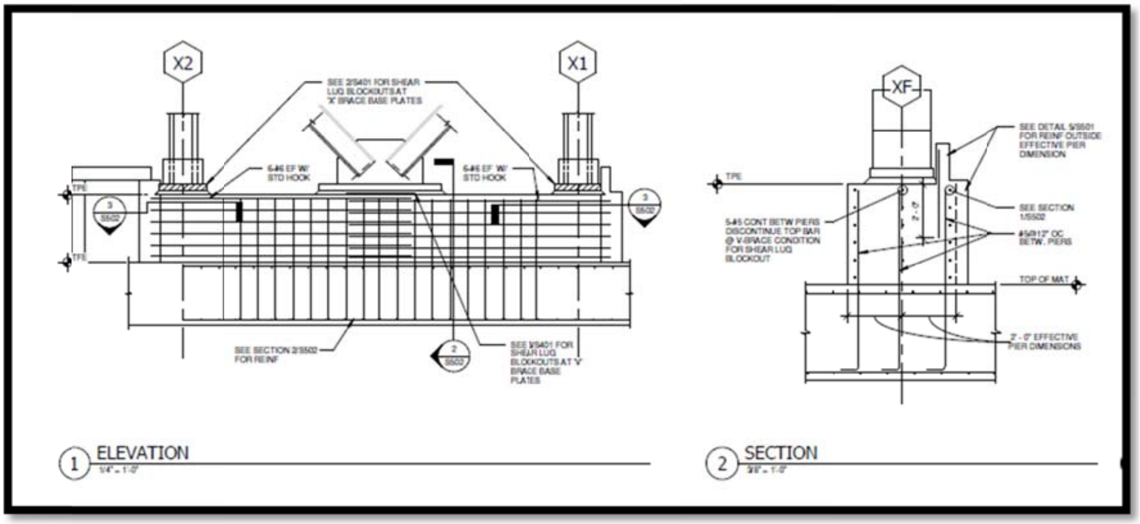


Figure 5

The picture on the next page in figure 6 shows where the braced frames are connected at the foundation level in green. There is one more braced frame but as stated earlier in the bracing section, this one is located on the top level.

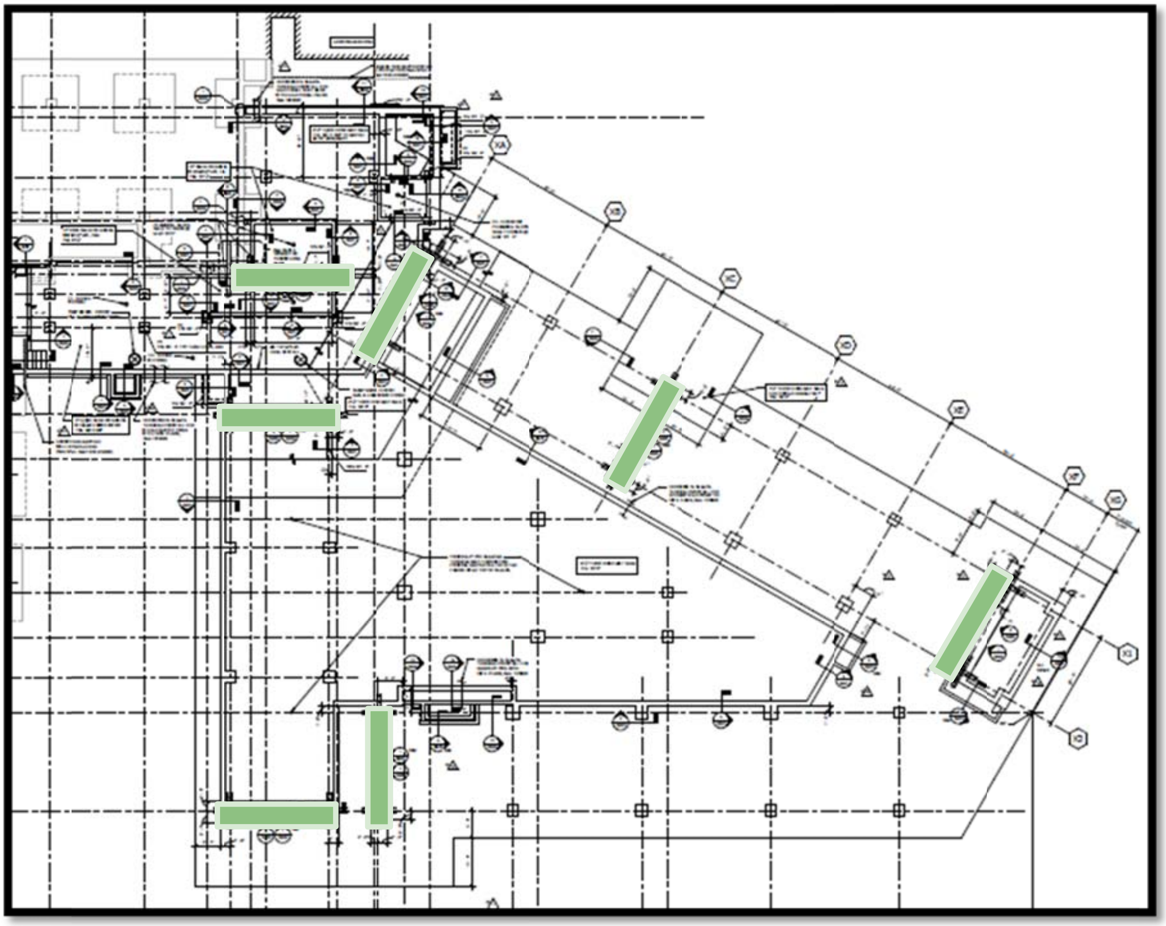


Figure 6

Floor Construction

Typical floor construction for the addition included the use 4 types of "deck." Most floors were constructed of 3" 18 gage galvanized steel deck with a 4 ½" normal weight concrete topping making it a total thickness of 7 ½" reinforced with 6x6 WWF. One floor was a combination of two decks. One "deck" was a 10" light weight concrete slab which was reinforced with #4 @ 18" O.C. running longitudinal. The other deck was a 2" 18 gage galvanized steel deck with a 3 ½" light weight concrete topping making it a total thickness of 5 ½" and reinforced with 6x6 WWF. Both the galvanized decks are composite and require a stud length of 5" for the 7 ½" deck and 4" for the 5 ½" deck. The roof deck was just a 1 ½" 20 gage galvanized steel decking.

Bay sizes were typically set at 30' especially on the outer spans of the building where the patient rooms are located. But due to the irregular shape of the addition, column lines were hard to align so bay sizes within the middle area of the building ranged in various lengths but came to an average of around 30'. Decking typically spanned 10' and were supported by beams ranging from W14's to W21's with the typical being W16's. Lengths of the beams were typically 22' and were supported by girders ranging from W18's to W24's but some exterior girders were W30's. Below in figure 7 is a typical floor plan for floors 4 through 8.

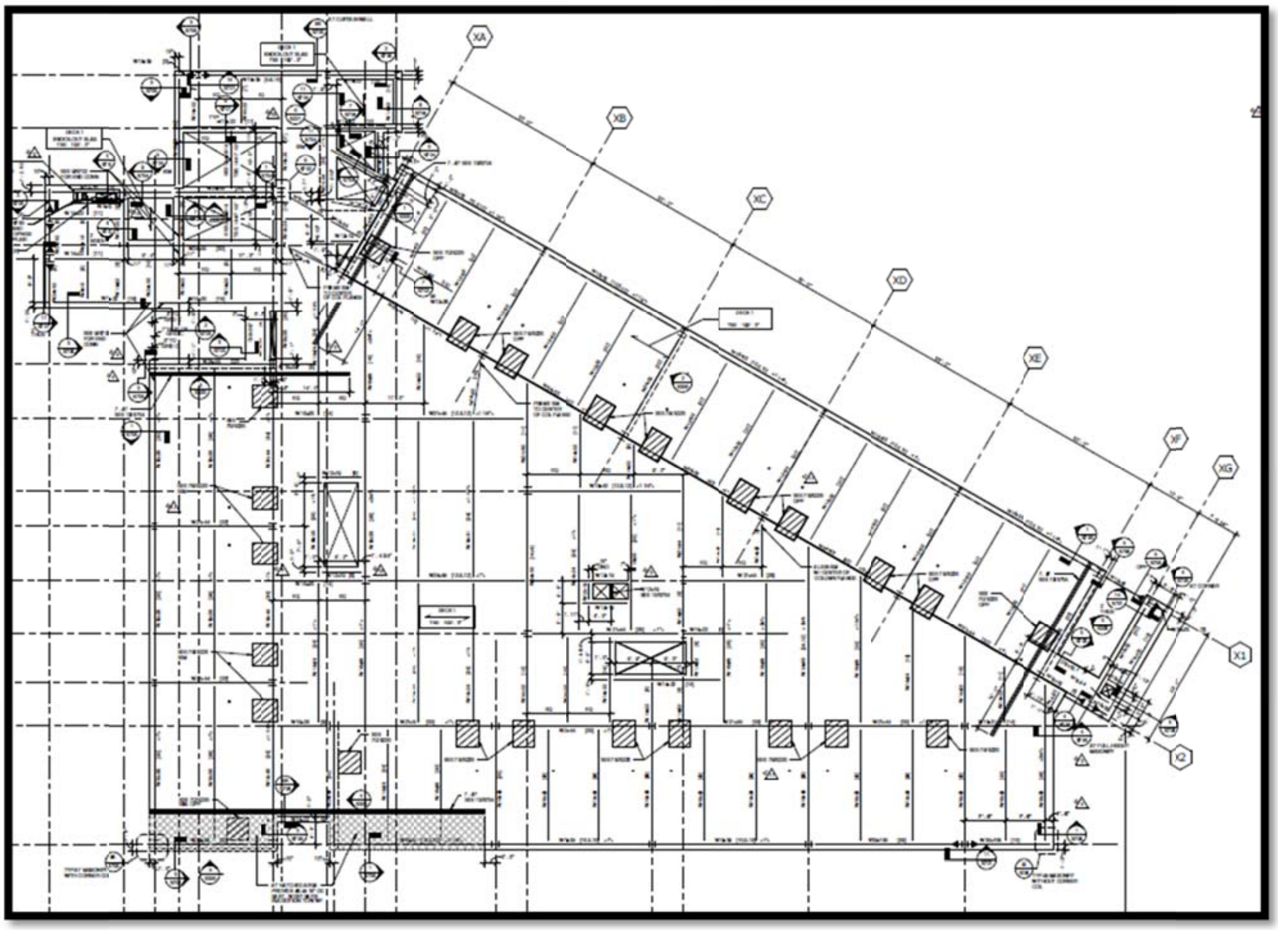


Figure 7

Construction Materials and Building Loads

Materials used in construction were specified in the general structural notes on S001. More information on the materials were found on the floor plans and detailed sections and elevations as well.

Properties of Materials		
Material		*Strength*
Concrete	**Weight**	**f'c (psi)**
Composite Deck	145	3500
All other concrete	145	4000
Slabs	115	4000
Steel	**Grade**	**fy (ksi)**
Reinforcing Bars	A615 - 60	
W Shapes	A992	50
Other Shapes	A36	36
Rectangular HSS	A500 - B	46
Round HSS	A500 - B	42
Bolts	A325/A490	
Studs	A108	

Figure 9

Dead Loads	
Material	Load (psf)
Superimposed	30
Composite Deck	
7.5" Thick 3" Steel	75
5.5" Thick 2" Steel	57
Roof	2.14
10" Slab	120 pcf

Figure 8

Dead loads used for calculations were found in various ways. The composite deck and roof deck were found using the Vulcraft Roof and Steel Deck manual. The weight of the 10” light weight concrete slab was known and it was then assumed a superimposed dead load of 30 psf was used.

Live Loads		
Occupancy	Design (psf)	Thesis (psf)
Typ. Hosp. Floor	80	80
Corridors (Above 1st Floor)	80	80
Corridors (1st Floor)	100	100
Lobby Floor	100	100
Stair and Exits	100	100
Storage	125	125
Mechanical Room	125	125
Snow Load	34	34

Figure 10

Live loads were found using ASCE7-05. Just a quick note on the lives loads. When doing research, typical hospital floors for patient rooms were found to be 40 psf but it is believed that 80 psf was used because corridors (above 1st floor) with a load of 80 psf controlled. Because the patient rooms were found above the 1st floor, 80 psf was used for ease of calculations although it is a conservative approach to the design.

Snow Load and Drift

Snow Load was determined by ASCE7-05 of Section 7 for flat roof snow loads. After looking up variables from the tables and figures of the section, it was found that the roof snow load was 33.6 psf.

Snow drift was checked for between levels 9 and 10 at the 9^{th} level where the mechanical penthouse is set back from the rest of the building on the south side. This set back will cause snow drift with windward winds coming from the south. The snow drift load was then calculated to be 69.7 psf. Snow load and snow drift calculations can be found in Appendix A.

Wind Load

Wind Load calculations were designed using ASCE7-05. Due to the shape of the tower, three directions were evaluated with each of the forces transferring to the Main Wind Force Resisting System (MWFRS). The pressures were calculated in the South-North direction, West-East direction, and Northwest-Southwest direction.

It was assumed that the structure was rigid because of its distinct shape and relatively low height when comparing it to its length. Because the structure was assumed rigid, the equation $T=C_t h_n^x$ (ASCE 12.8.2.1) was used to calculate its approximate period. After calculation, it was determined that the period was 1.3 which is greater than 1 thus supporting the assumption the structure was rigid. Since the structure was proven rigid, the gust factor used was 0.85 for all calculations.

Pictured on the next page in figure 11 are the two rectangular shapes used in simplifying the wind load calculations. The green rectangle and arrows indicate the W-E direction while the blue rectangle and arrow indicate the NE-SW direction. In doing this, windward pressures were found on all three walls of the triangular frame. Leeward pressures were also calculated but they can be considered very conservative because of the simplified shapes. Future analysis of wind load calculations should be determined more accurately taking into account the slope of the wall which could change the leeward pressures from all three directions.

Wind load calculations can be found in Appendix B with the results on the next page. It was determined that the South to North Calculations were to control the wind load because it has the bigger overturning moment as well as the highest base shear.

Figure 11

Northeast to Southwest				
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)
1	0	0.00	0	0
2	12.25	11.65	18.71	229.22
3	25.646	12.97	20.29	520.45
4	37.25	14.05	22.51	838.67
5	51.25	15.03	26.35	1350.39
6	65.25	15.79	27.68	1805.98
7	79.25	16.62	29.13	2308.36
8	93.25	17.17	30.09	2806.26
9	107.25	17.72	38.27	4104.55
10	127.75	18.41	23.63	3018.11
Total		20650.77		16982.0
Base Shear	236.66	**Overturning M**		16982.0

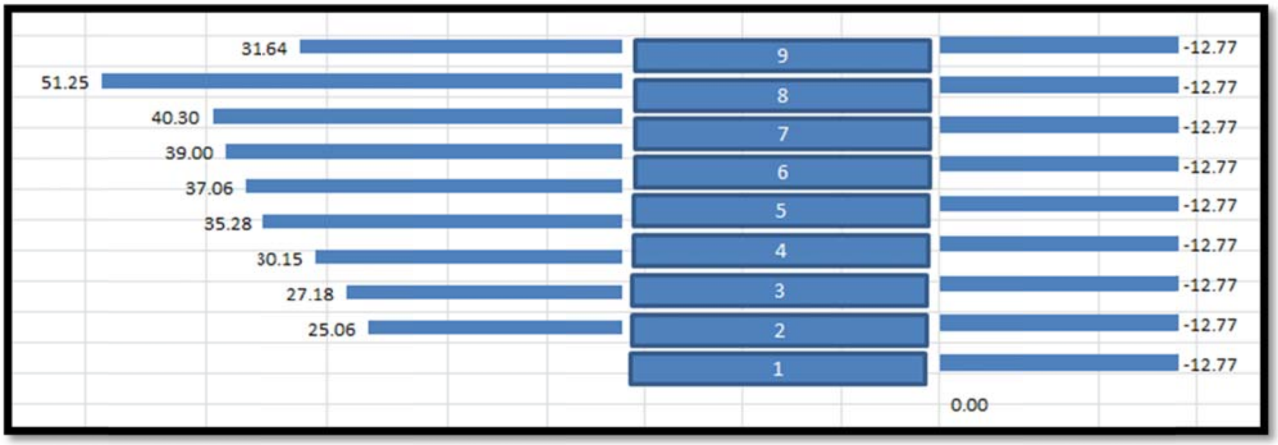
Figure 12

South to North				
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)
1	0	0.00	0	0
2	12.25	11.65	25.06	306.95
3	25.646	12.97	27.18	696.94
4	37.25	14.05	30.15	1123.08
5	51.25	15.03	35.28	1808.32
6	65.25	15.79	37.06	2418.41
7	79.25	16.62	39.00	3091.15
8	93.25	17.17	40.30	3757.90
9	107.25	17.72	51.25	5496.45
10	127.75	18.41	31.64	4041.59
Total		20650.77		22740.8
Base Shear	316.92	**Overturning M**		22740.8

Figure 13

West to East				
Level	Ht. (ft)	Windward (psf)	Windward (k)	M (k-ft)
1	0	0.00	0	0
2	12.25	11.65	18.87	231.12
3	25.646	12.97	20.46	524.77
4	37.25	14.05	22.70	845.64
5	51.25	15.03	26.57	1361.60
6	65.25	15.79	27.91	1820.98
7	79.25	16.62	29.37	2327.53
8	93.25	17.17	30.34	2829.57
9	107.25	17.72	38.59	4138.65
10	127.75	18.41	23.82	3043.18
Total		20650.77		17123.0
Base Shear	238.63	**Overturning M**		17123.0

Figure 14



Figure

Figure 15 – Northeast to Southwest Results

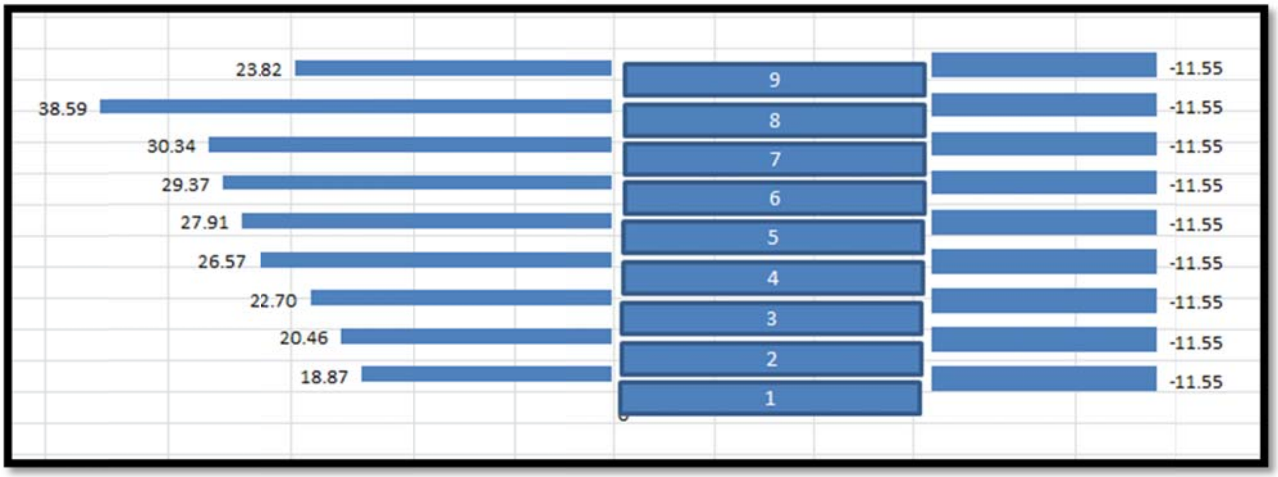


Figure 16 – South to North Results

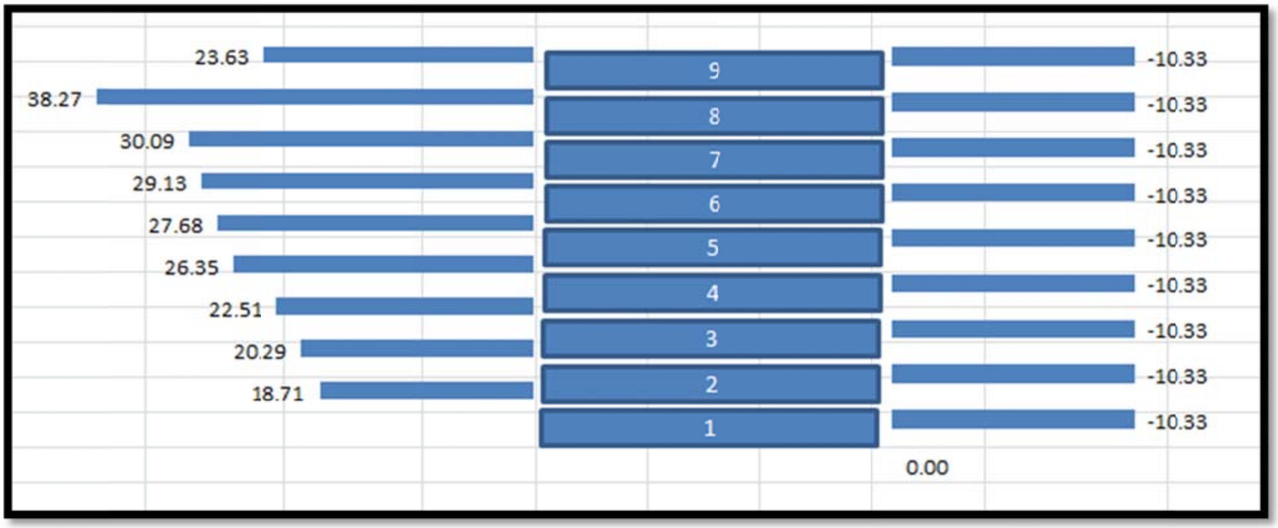


Figure 17 – West to East Results

Seismic Design Calculations

The seismic calculations were simple because it followed the seismic design category A parameters of section 11 of ASCE7-05. The seismic loads were found using the total dead weight of the building and the equation $F_x=0.01w_x$. Loads were then just a portion of the total weight of the floor which was calculated. Seismic design calculations can be found in Appendix C. Below are the results from the calculations.

Seismic Load Calculations				
Level	Ht. (ft)	Weight (k)	Fx	M (k-ft)
1	0	0	0	0
2	12.25	2274.51	22.75	278.63
3	25.646	2401.53	24.02	615.89
4	37.25	2191.80	21.92	816.44
5	51.25	2385.17	23.85	1222.40
6	65.25	2373.11	23.73	1548.45
7	79.25	2328.45	23.28	1845.30
8	93.25	2323.24	23.23	2166.42
9	107.25	2532.50	25.32	2716.10
10	127.75	1840.46	18.40	2351.19
Total		20650.77		13560.8
Base Shear		206.51	**Overturning M**	13560.8

Gravity Spot Load Checks

Spot load checks were checked for a typical beam, girder, column, and deck. Complete hand calculations of the spot checks can be found in Appendix D.

Beam Check

The typical beam analyzed was a W16 x 26. Strength and deflection checks for both the construction dead loads and live loads were evaluated. The member evaluated passed easily for moment and shear checks. The reason for this could be because the beam was designed conservatively because of the repetitive nature of using this beam throughout the building. When it came to live load deflection, the beam passed but when checking wet concrete deflection, it just barely passed.

Girder

The girder, a W18 x 35 used in the spot check was picked because of the location. On the south side of the girder were two typical beams at the same length. On the north side of the girder were two typical beams but of different length due to the slope of the northeast wall. After strength and deflection checks were evaluated it passed with a considerable amount. Live load and wet concrete deflection checks also passed. Both the girder and the beam were designed well but conservative.

Column

The column chosen located on the 3rd floor had an atypical tributary area because of the irregular column lines with in the building. After find the tributary area, a spreadsheet was used to calculate the loads on the column from the floors above. An equivalent axial load was found to be 1060 kips which passed the max load found in the AISC manual of section 4. Below is the results from the column spot check.

Column Analysis									
Floor	DL (psf)	Corr. LL (psf)	Reduced (psf)	Area (ft^2)	Patient LL (psf)	Reduced (psf)	Area (ft^2)	Unred. P (k)	Reduced P (k)
3	110	80	56.83	265.33	80	53.31	324.5	153.357	129.663
4	92	80	56.83	265.33	80	53.31	324.5	140.616	116.922
5	110	80	56.83	265.33	80	53.31	324.5	153.357	129.663
6	110	80	56.83	265.33	80	53.31	324.5	153.357	129.663
7	110	80	56.83	265.33	80	53.31	324.5	153.357	129.663
8	110	80	56.83	265.33	80	53.31	324.5	153.357	129.663
9	110	80	not	265.33	80	not	324.5	153.357	
Roof not included in this because column does not go up to roof level							Total:	1060.76	765.236

Conclusion

After analyzing the addition and checking the various areas of construction, it was found that building is very well designed. The systems used within this building work well to help transfer the loads to the foundation in an efficient manner.

The braced frames help transfer loads both vertically and laterally to the base. The lateral loads which come into the building are transferred to these braced frames through the decking and slab which acts as a diaphragm to these frames. The location of these braces frames are very well placed, but for future analyzing, these could be looked at again to see if these braced are necessary in some spots.

It easy to see that the wind loads controlled over the seismic loads when designing this building. This is logical because of the building's location away from a seismic region and in a more wind based area. Because of the flat exterior walls which run vertically throughout the entire building, wind loads did not change much throughout the building. In future reports, the location of the building could be switched to check if the building would be able to hold up in a more prone seismic region.

Lastly, it was calculated that the structural components of the addition were designed conservatively. Nonetheless, these components will be able to hold a great amount of strength which could assist in abnormal situations such as a unique snow storm which could occur in the area. Also because the components have a large strength capacity, they will be durable over time and should be able to handle any loads that occur to the system.

Appendix A: Snow Load and Drift Calculations

AE SENIOR THESIS | SNOW LOAD | JESSEL ELLIOTT

CHECK ASCE 7-05 FOR
SNOW DESIGN LOADS. GIVEN: USING ASCE 7-05

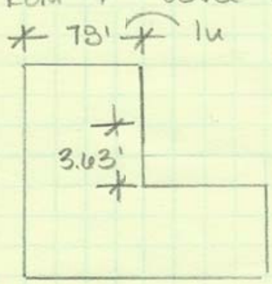
GROUND SNOW LOAD (P_g) (FIGURE 7-1): 40 PSF OK ✓
SNOW EXPOSURE FACTOR (C_e) (TABLE 7-2): 1.0 OK ✓
SNOW IMPORTANCE FACTOR (TABLE 7-4): 1.2 OK ✓
THERMAL FACTOR C_T (TABLE 7-3): 1.0 OK ✓

FLAT ROOF SNOW LOAD

$$p_f = 0.7\, C_e C_t I p_g = 0.7(1.0)(1.0)(1.2)(40) = 33.6 \approx 34 \text{ PSF}$$ OK ✓

THE BUILDING STEPS BACK FROM THE:

FROM 9TH LEVEL ELEVATION TO 10TH LEVEL ELEVATION



FROM FIGURE 7-9
USING 78' AS ROOF LENGTH

$h_d = 3.63$ ft

LEEWARD DRIFT CONTROLS
BECAUSE OF LONGER
ROOF LENGTH.

$$w = 4h_d = 4(3.63) = 14.52 \text{ ft}$$

$P_d = h_d \gamma$

$$\gamma = 0.13 P_g + 14 \leq 30 \text{ PCF}$$
$$= 0.13(40) + 14$$
$$= 19.2 \text{ PCF}$$

$$P_d = (3.63')(19.2 \text{ PCF}) = \boxed{69.7 \text{ PSF}}$$

Appendix B: Wind Load Calculations

AE SENIOR THESIS | WIND LOAD CALC. | JESSEL ELLIOTT

USING ASCE 7-05:

BASIC WIND SPEED V (FIG 6-1) : 90 MPH (40 m/s)

IMPORTANCE FACTOR I (TAB. 6-1) : 1.15

OCCUPANCY CATEGORY (TAB 1-1) : IV

EXPOSURE CATEGORY (F/ PLANS) : B

DIRECTIONALITY FACTOR Kd (TAB 6-4) : 0.85

TOPOGRAPHIC FACTOR Kzt : 1.0

VEL PRESSURE EXP. COEF. Kz (TAB 6-3) : 1.04

VELOCITY PRESSURE

$q_z = 0.00256\, K_z K_{zt} K_d V^2 I$ SAMPLE CALCULATION

$= 0.00256(1.04)(1.0)(0.85)(90)^2(1.15) = 21.1$ psf

$q_h = 21.1$ psf FOR $h = 127'$

GUST FACTOR

RIGID IF $n_1 > 1.0$ HZ ; FLEXIBLE IF $n_1 < 1.0$ HZ

$n_1 = \frac{1}{T}$ T = NATURAL PERIOD $T = C_t h_n^{x}$

CONSIDERED ALL OTHER STRUCTURAL SYSTEMS

$h_n = 127'$ $C_t = .02$, $x = 0.75$

$T = 0.02\,(127)^{0.75} = 0.757$ sec.

$n_1 = 1/0.757 = 1.3 > 1.0$ SO RIGID STRUCTURE $\rightarrow G = 0.85$

DESIGN WIND PRESSURE

$p = q G C_p - q_i (GC_{pi})$ w/ $GC_{pi} = \pm 0.18$

for Kz 25 − 0.66 25.65
30 − 0.70

$$\left(\frac{30-25}{0.70-0.66}\right) = \left(\frac{30-25.65}{0.70-x}\right)$$

$$x = 0.70 - \frac{(30-25.65)(0.70-0.66)}{(30-25)}$$

EXTERNAL PRESSURE COEFFICIENTS

WINDWARD WALL $C_p = 0.8$ USE w/ q_z

LEEWARD WALL USE w/ q_h

W-E DIREC. $L/B = \frac{167.67}{126.25} = 1.34$ $C_p = -0.432$

S-N DIRECT. $L/B = \frac{126.25}{167.67} = 0.75$ $C_p = -0.5$

NE-SW DIRECT. $L/B = \frac{165.167}{98.33} = 1.68$ $C_p = -0.364$

SIDE WALL $C_p = -0.7$ USE w/ q_h

Appendix B: Wind Load Spreadsheets

South to North										
Floor	Elev. (ft)	z	Floor Ht. (ft)	kz	qz	qh	Windward (psf)	Windward (k)	Leeward (psf)	Leeward (k)
1	100.00	0	12.25	0.57	11.55	21.1	11.65	0	-12.77	0
2	112.25	12.25	13.40	0.57	11.55	21.1	11.65	25.06	-12.77	-27.45
3	125.65	25.65	11.60	0.67	13.48	21.1	12.97	27.18	-12.77	-26.75
4	137.25	37.25	14.00	0.74	15.07	21.1	14.05	30.15	-12.77	-27.40
5	151.25	51.25	14.00	0.82	16.52	21.1	15.03	35.28	-12.77	-29.97
6	165.25	65.25	14.00	0.87	17.63	21.1	15.79	37.06	-12.77	-29.97
7	179.25	79.25	14.00	0.93	18.85	21.1	16.62	39.00	-12.77	-29.97
8	193.25	93.25	14.00	0.97	19.66	21.1	17.17	40.30	-12.77	-29.97
9	207.25	107.25	20.50	1.01	20.47	21.1	17.72	51.25	-12.77	-36.92
10	227.75	127.75	0.00	1.06	21.49	21.1	18.41	31.64	-12.77	-21.94

West to East										
Floor	Elev. (ft)	z	Floor Ht. (ft)	kz	qz	qh	Windward (psf)	Windward (k)	Leeward (psf)	Leeward (k)
1	100.00	0	12.25	0.57	11.55	21.1	11.65	0	-11.55	0
2	112.25	12.25	13.40	0.57	11.55	21.1	11.65	18.87	-11.55	-18.69
3	125.65	25.65	11.60	0.67	13.48	21.1	12.97	20.46	-11.55	-18.22
4	137.25	37.25	14.00	0.74	15.07	21.1	14.05	22.70	-11.55	-18.66
5	151.25	51.25	14.00	0.82	16.52	21.1	15.03	26.57	-11.55	-20.41
6	165.25	65.25	14.00	0.87	17.63	21.1	15.79	27.91	-11.55	-20.41
7	179.25	79.25	14.00	0.93	18.85	21.1	16.62	29.37	-11.55	-20.41
8	193.25	93.25	14.00	0.97	19.66	21.1	17.17	30.34	-11.55	-20.41
9	207.25	107.25	20.50	1.01	20.47	21.1	17.72	38.59	-11.55	-25.14
10	227.75	127.75	0.00	1.06	21.49	21.1	18.41	23.82	-11.55	-14.94

Northeast to Southwest										
Floor	Elev. (ft)	z	Floor Ht. (ft)	kz	qz	qh	Windward (psf)	Windward (k)	Leeward (psf)	Leeward (k)
1	100.00	0	12.25	0.57	11.55	21.1	11.65	0	-10.33	0
2	112.25	12.25	13.40	0.57	11.55	21.1	11.65	18.71	-10.33	-16.58
3	125.65	25.65	11.60	0.67	13.48	21.1	12.97	20.29	-10.33	-16.16
4	137.25	37.25	14.00	0.74	15.07	21.1	14.05	22.51	-10.33	-16.55
5	151.25	51.25	14.00	0.82	16.52	21.1	15.03	26.35	-10.33	-18.10
6	165.25	65.25	14.00	0.87	17.63	21.1	15.79	27.68	-10.33	-18.10
7	179.25	79.25	14.00	0.93	18.85	21.1	16.62	29.13	-10.33	-18.10
8	193.25	93.25	14.00	0.97	19.66	21.1	17.17	30.09	-10.33	-18.10
9	207.25	107.25	20.50	1.01	20.47	21.1	17.72	38.27	-10.33	-22.30
10	227.75	127.75	0.00	1.06	21.49	21.1	18.41	23.63	-10.33	-13.25

Appendix C: Seismic Load Calculations

AE SENIOR THESIS | SEISMIC LOAD | JESSEL ELLIOTT

SEISMIC BASE SHEAR

CALCULATE SDs & SDS1

BECAUSE THE ADDITION IS SEISMIC DESIGN CATEGORY A, ANALYSIS SHALL BE DETERMINED BY

$F_x = 0.01\, w_x$

TO BE MORE ACCURATE GO TO USGS

CHECK SEISMIC DESIGN DATA

FIGURES

22-1 SPECTRAL RESPONSE ACCELERATION, S_S = 0.068 OK ✓

22-2 SPECTRAL RESPONSE ACCELERATION, S_1 = 0.033 OK ✓

SEISMIC IMPORTANCE FACTOR I (TABLE 11.5-1) = 1.5 OK ✓

(TABLE 11.4-1) $F_a = 0.8$

(TABLE 11.4-2) $F_v = 0.8$

$S_{MS} = F_a S_S = 0.8(.068) = .0544$

$S_{M1} = F_v S_1 = 0.8(.033) = .0264$

$S_{DS} = 2/3\, S_{MS} = 2/3(.0544) = .0363 < 0.167$ (TABLE 11.6-1)

$S_{D1} = 2/3\, S_{M1} = 2/3(.0264) = .0176 < .067$ (TABLE 11.6-2)

BOTH VALUES FALL IN CATEGORY A FOR DESIGN REQUIREMENTS SO STRUCTURE IS SDC A.

ALSO ACCORDING TO 11.4.1

$S_1 = .033 < .04$ and $S_S = .068 < 0.15$

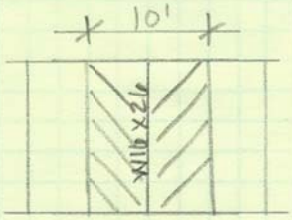
THIS CLARIFIES SDC A

Appendix D: Gravity Spot Checks – Beam 1 of 2

AE SENIOR THESIS | SPOT CHECKS | JESSEL ELLIOTT

COMPOSITE BEAM: W16X26 [20]

FROM AISC STEEL MANUAL: $A_g = 7.68\ in^2$
$I_x = 301\ in^4$
$Z_x = 44.2\ in^3$



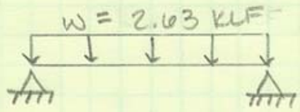
TRIB WIDTH: 10'
SPAN: 22'

DEAD LOAD

S.D = 30 PSF
DECK = 75 PSF
ALLOW. = 5 PSF
SELF = 26 PLF

LIVE LOAD

FLOOR: 80 PSF



$w = 1.2D + 1.6L$
$D = (30+75+5)(10') + 26 = 1126\ PLF$
$L = (80)(10') = 800\ PLF$

$w = 1.2(1126) + 1.6(800) = 2631.2\ PLF = 2.63\ KLF$

$V = \frac{(2.63)(22)}{2} = 28.9\ K$ $M = \frac{wl^2}{8} = \frac{(2.63)(22)^2}{8} = 159.1\ k\cdot ft$

beff : min | $\frac{SPAN}{8} = \frac{22' \times 12''/'}{8} = 33\ in$ ← CONTROLS FOR BOTH SIDES
$\frac{1}{2}(SPACING) = \frac{10' \times 12''}{2} = 60\ in$

$b_{eff} = 2(33\ in) = 66\ in$

FROM TABLE 3-19

$PNA = 7$ $\Sigma Q_n = 96.0$

$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{96.0}{0.85(3.5)(66\ in)} = 0.489 < 1.0$ use $a = 1.0$

$Y_2 = t_{slab} - a/2 = 7.5 - 1/2 = 7.0''$

$\phi M_n = 259\ k\,ft > 159.1\ k\cdot ft$ OK ✓

$Q_n = \frac{96.0}{14.6} = 6.6 \sim 7$ 14 STUDS NEEDED
20 USED OK ✓

SHEAR CHECK: (TABLE 3-2) $\phi V_n = 106 > 28.9$ OK ✓

Appendix D: Gravity Spot Checks – Beam 2 of 2

CHECK ΔLL:

$$\Delta_{LL} = \frac{l}{360} = \frac{(22')(12'')}{360} = 0.73 \text{ in}$$

FROM TABLE 3-20:
W16×26 PNA 7, $I_{LB} = 640 \text{ in}^4$

$$\Delta_{LL} = \frac{5 w_{LL} l^4}{384 EI} = \frac{5\left(\frac{80 \cdot 10}{1000}\right)(22)^4(1728)}{384(29000)(640)} = 0.227$$

$0.227 \text{ in} < 0.73 \text{ in}$ OK ✓

CHECK BEAM DEFLECTIONS
UNDER WET CONCRETE

$$\Delta_{max} = \frac{l}{240} = \frac{(22')(12')}{240} = 1.1''$$

$$I = \frac{5 w l^4}{384 E \Delta_{max}} = \frac{5\left(\frac{(75+5)(22)+26}{1000}\right)(22)^4(1728)}{384(29000)(1.1'')}$$

$$= 295.1 \text{ in}^4 < I_x = 301 \text{ in}^4$$ OK ✓

$\boxed{\text{W16×26 OK TO USE!}}$

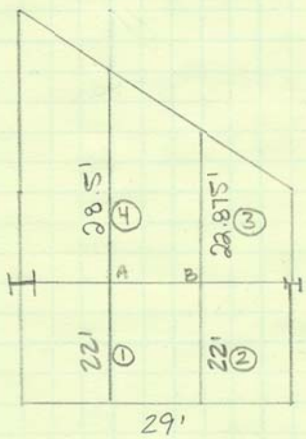
Appendix D: Gravity Spot Checks – Girder 1 of 2

AE SENIOR THESIS | SPOT CHECKS | JESSEL ELLIOTT

COMPOSITE GIRDER: W18 x 35 [28]

A = 10.3 in²
Ix = 510 in⁴
Zx 66.5 in³

LOAD FROM BEAMS: [ALL W16X26]



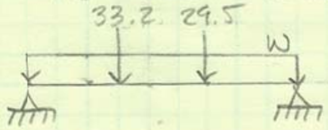
① + ② D = (30+75+5)(9.67) + 26
= 1126 PLF
L = 80 (10) = 800 PLF
W = 1.2(1126) + 1.6(800)
= 2.63 KLF

P = (22')(2.63 KLF) / 2 = 28.9 K
PL = 8.8

③ P = (22.875')(2.63 KLF) / 2 = 30.1 K
PL = 9.15

④ P = (28.5')(2.63 KLF) / 2 = 37.5 K
PL = 11.4

PA = (28.9 + 37.5)/2 = 33.2 K PLA = (11.4 + 8.8)/2 = 10.1
PB = (28.9 + 30.1)/2 = 29.5 K



w = 35/1000 = .035 KLF

$R_1 = V_1 = \frac{P_A(l-a) + P_B b}{l} = \frac{33.2(29-9.67) + 29.5(9.67)}{29}$

$R_1 = 32.0$ K

$R_2 = V_2 = \frac{P_A(a) + P_b(l-b)}{l} = \frac{33.2(9.67) + 29.5(29-9.67)}{29}$

$R_2 = 30.7$ K

$V_u = 32.0$ K

$M_{MAX} = R_1 a = (32.0K)(9.67) = 309.3$ k·ft

$V_u = .035(29')/2 = 0.5$ $V_{u_{TOT}} = 32.5$

$M = \frac{wl^2}{8} = \frac{(.035)(29)^2}{8} = 3.7$ k·ft $M_{u_{TOT}} = 313.0$ k·ft

Appendix D: Gravity Spot Checks – Girder 2 of 2

(TABLE 3-19)

PNA = 7 $\Sigma Q_n = 129$

b_{eff} min $\left| \begin{array}{l} \frac{SPAN}{8} = \frac{29(12)}{8} = 43.5'' \leftarrow \text{CONTROLS} \\ \frac{SPACING}{2} = \frac{22(12)}{2} = 132'' \end{array} \right.$ $b_{eff} = 87$ in

$a = \frac{\Sigma Q_n}{0.85 f'c\, b_{eff}} = \frac{129}{0.85(3.5)(87 \text{ in})} = 0.50 < 1.0$ use $a = 1.0$

$y_2 = t_{SLAB} - a/2 = 7.5 - 1/2 = 7.0$

$\phi M_n = 382.0$ k·ft > 313.0 k-ft OK ✓

(TABLE 3-2) $\phi V_n = 159$ k > 32.5 k OK ✓

DEFLECTION $I_b = 1030$ $\Delta_{LL} = l/360 = 29(12)/360 = 0.967$

$\Delta_{LL} = \frac{Pl^3}{28EI} + \frac{5wl^4}{384EI} = \frac{10.1(29)^3(1728)}{28(29000)(1030)} + \frac{5(.035)(29)^4(1728)}{384(29000)(1030)}$

$\Delta_{LL} = 0.527 < 0.967$ OK ✓

DEFLECTION W/ WET CONCRETE

$\Delta_{MAX} = l/240 = 29(12)/240 = 1.45$

$I = \frac{Pl^3}{28E\Delta} + \frac{5wl^4}{384E\Delta} = \frac{10.1(29)^3(1728)}{28(29000)(1.45)} + \frac{5(.035)(29)^4(1728)}{384(29000)(1.45)}$

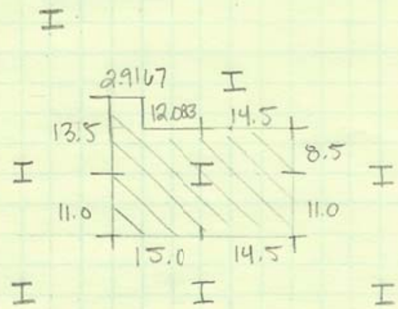
$I = 374.8 \text{ in}^4 < 510 \text{ in}^4$ OK ✓

$\boxed{\text{W18x35 WORKS}}$

Appendix D: Gravity Spot Checks – Column

AE SENIOR THESIS | SPOT CHECKS | JESSEL ELLIOTT

COLUMN SPOT CHECK: COLUMN @ LEVEL 3 W14x132



$A_T = 282.75\ ft^2 + 292.5\ ft^2 + 14.583\ ft^2$

$A_T = 589.83\ ft^2$

$A_{COR} = 265.33\ ft^2$

$A_{PATIENT} = 324.5\ ft^2$

LL REDUCTION (PATIENT)

$L = L_o\left(0.25 + \frac{15}{\sqrt{K_{LL} A_T}}\right)$

$L = 80\left(0.25 + \frac{15}{\sqrt{4 \cdot 324.5}}\right)$

$L = 53.31$ PSF

LL REDUCTION (CORRIDOR)

$L = 80\left(0.25 + \frac{15}{\sqrt{4 \cdot 265.33}}\right)$

$L = 56.83$ PSF

FROM AISC TABLE 4-1

W14x132 w/ Effective length 12'

$\phi P_n = 1570\ K > 1060.76\ K$

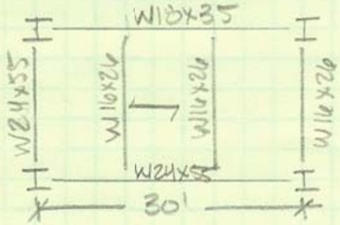
FOUND FROM THE EXCEL SPREADSHEET

$\boxed{\text{W14 x 132 OK ✓}}$

Appendix D: Gravity Spot Checks – Composite Deck

AE SENIOR THESIS | SPOT CHECKS | JESSEL ELLIOTT

DECK CHECK IN TYPICAL 30' BAY FLOOR 7



DECK INFO FROM VULCRAFT

3" DECK
4½" TOPPING
18 GAUGE
NW CONCRETE
75 PSF
f'c = 3500 psi

LL = 80 PSF
DL = 30 PSF ← SUPERIMPOSED

3VL118 w/ UNSHORED LENGTH 13'-3" FOR 3-SPAN CONDITION

13'-3" > 10' OK ✓

SUPERIMPOSED LL (PSF)

CLEAR SPAN - 10'   SLL = 294 PSF > 110 PSF OK

FROM GENERAL NOTES, FLOOR CONSTRUCTION SHOULD BE 2-HR RATING

ACCORDING TO FIRE RATINGS FROM VULCRAFT DECK IS 2HR RATING

$\boxed{\text{3VL118 OK ✓}}$