Kaleida Health – Global Heart and Vascular Institute
University at Buffalo – CTRC/Incubator

Buffalo, New York

Technical Report #1

William McDevitt
Structural Option
Dr. Richard Behr
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Executive Summary

The following document is the first technical report of senior thesis and includes information regarding the structural concepts and existing conditions of the Kaleida Health Global Heart and Vascular Institute and University at Buffalo CTRC/Incubator, Additions, and Renovations. This project will be referred to throughout this report simply as GHVI. This report includes a structural system overview, codes and references, a list of construction materials, a gravity and lateral load analysis, and various typical floor spot checks.

Wind analysis for this report was performed using ASCE 7-10, with a resulting base shear of 2244.5 kips. This value was then compared to the results from Cannon Design, who performed their analysis using ASCE 7-02. The value obtained from ASCE 7-10 was almost twice that from ASCE 7-02, and it is probable that this large difference can be attributed to the change in basic wind speed in the new code.

Seismic analysis for this report was performed using ASCE 7-10, with a resulting base shear of 1316 kips. This value was then compared to the resulting base shear of 1030 kips from Cannon Design, who performed their analysis using ASCE 7-02. The difference in these two numbers can be attributed to the fact that the design engineers used a slightly smaller total building weight, but more importantly because the response modification factor, R, was 5 in ASCE 7-02, and is now 3.25 in ASCE 7-10. This lower R value results in a more conservative number, and therefore a higher base shear.

Four spot checks were also performed as a part of this report. These included checking the composite metal deck, a typical beam, a typical girder, and a column on a lower level of the building. Each of these checks resulted in a confirmation of the initial designs.

Introduction

GHVI is a state-of-the-art medical facility and a fundamental component in a joint undertaking between Kaleida Health Systems and the University at Buffalo School of Medicine. The building spans ten levels and includes exam rooms, classrooms, offices, a café, a wellness center and library, and a research facility. It is intended to bring patients, surgeons, and researchers together to collaborate in an unprecedented way.

Key themes considered throughout the design were collaboration, flexibility, and comfort. Kaleida Health Systems sought a structure that would link clinical and research work and combine all vascular disciplines. A spirit of collaboration was the driving force behind bringing both Kaleida and the University at Buffalo together in a single structure. Keeping this in mind, the design team developed the facility with a “collaborative core” which enables interaction among those working within the facility. This collaborative learning environment brings together research, ideas, and solutions and results in better patient care.

A universal grid design increases the flexibility of space and achieves measurable advantage in initial capital cost, speed to market, operating economy, and future adaptability. The universal grid is comprised of three 10’-6” building modules and forms a 31’-6” x 31’-6” structural grid capable of integrating the building’s diverse functions. When combined with an 18’ floor-to-floor height, the flexible grid creates an open plan capable of adapting to present and future healthcare needs. The building will be able to incorporate unknown, but rapidly changing technological developments within the industry, also giving it longevity through its adaptability.

With comfort in mind, a separate “hotel” level was designed on the second floor and separated from the procedural floors to provide a calmer environment for patient care. Functionally, the “hotel” is comprised of private patient rooms and a small lounge area. Other family lounges are provided and the perimeter of the building is shaped to bring in as much natural daylight as possible. These architectural details, combined with a philosophical approach akin to a hotel concierge desk, seek to provide the positive first and last impressions that can be so vital to patient and family satisfaction. The vision of GHVI is to create an environment that is more than simply a clinical facility, but instead a building that encompasses a world-class delivery within a first-class patient/family-focused setting.

Refer to Appendix A for a site plan, a typical floor framing plan, and an elevation.

Structural System Overview

Foundation

Based on the recommendations of the October 2008 Geotechnical Report by Empire Geo-Services, Inc., the foundation of GHVI consists of grade beams and pile caps placed on top of steel helical piles, as seen in Figure A.

The helical piles are HP12x74 sections with an allowable axial capacity of 342 kips (171 tons) which are driven to absolute refusal on limestone bedrock 82 to 87 feet below the sub-basement finish level. Grade beams and pile caps have a concrete strength of 4000 psi, and it should be noted that the width of the grade beams equals that of the pile caps at the foundations of the braced frames. The grade beams provide resistance to lateral column base movement, and the pile caps link the steel helical piles and the structural steel columns of the superstructure.

Spanning the grade beams is the sub-basement floor, a 5” slab-on-grade. Due to the slope of the site, part of this sub-basement is below grade, and therefore a one foot thick foundation wall slopes along the west elevation of the sub-basement.

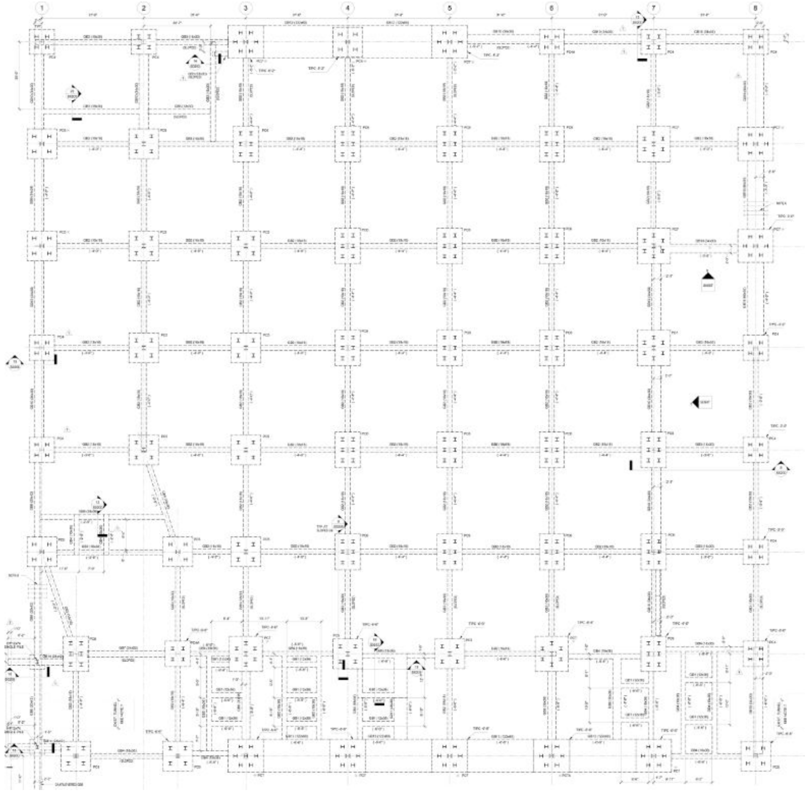


Figure A – Foundation Plan

Floor System

The floors of GHVI consist of 3” composite metal deck with a total slab thickness ranging from 4” to 7½”. The metal deck is 18-gage galvanized steel sheets resting on various different beam and girder sizes. These sizes change throughout the structure because of the various functions of the spaces. The bay sizes through the building are mostly 31’-6” by 31’-6”, with beams spaced at 10’-6”. As was discussed in the introduction, this universal grid design increases the future flexibility of the space. A slight variation in the floor can be seen on Levels 6-8. On these levels, part of the floor structure is left open to provide for the collaborative atrium that was designed to bring the various disciplines together.

Gravity System

Steel columns are used throughout the building to transmit the gravity load to the foundation. All of the columns in the building are W14s, but they range in weight from 68 lb/ft to 370 lb/ft, and they are typically spliced every 36 feet. These columns provide an 18’ floor-to-floor height which also contributes to the universal grid and future flexibility of the space.

Lateral System

The lateral system of GHVI utilizes braced frames located near the perimeter of the building. All of the braced frames are HSS sections and are present from the Basement Level up to Level 3, where some of them are discontinued. A braced frame system is ideal in steel buildings because of its low cost compared to moment connection frames. There are moment connections in some parts of this structure, but they are used to support the small amount of slab overhang that is cantilevered. These moment connections may actually add some stiffness to the lateral system, but they cannot be included in the lateral system design. Figure B depicts the location of the braced frames on the outer part of the structure, and Figure C shows how some of the braced frames continue to the roof level.

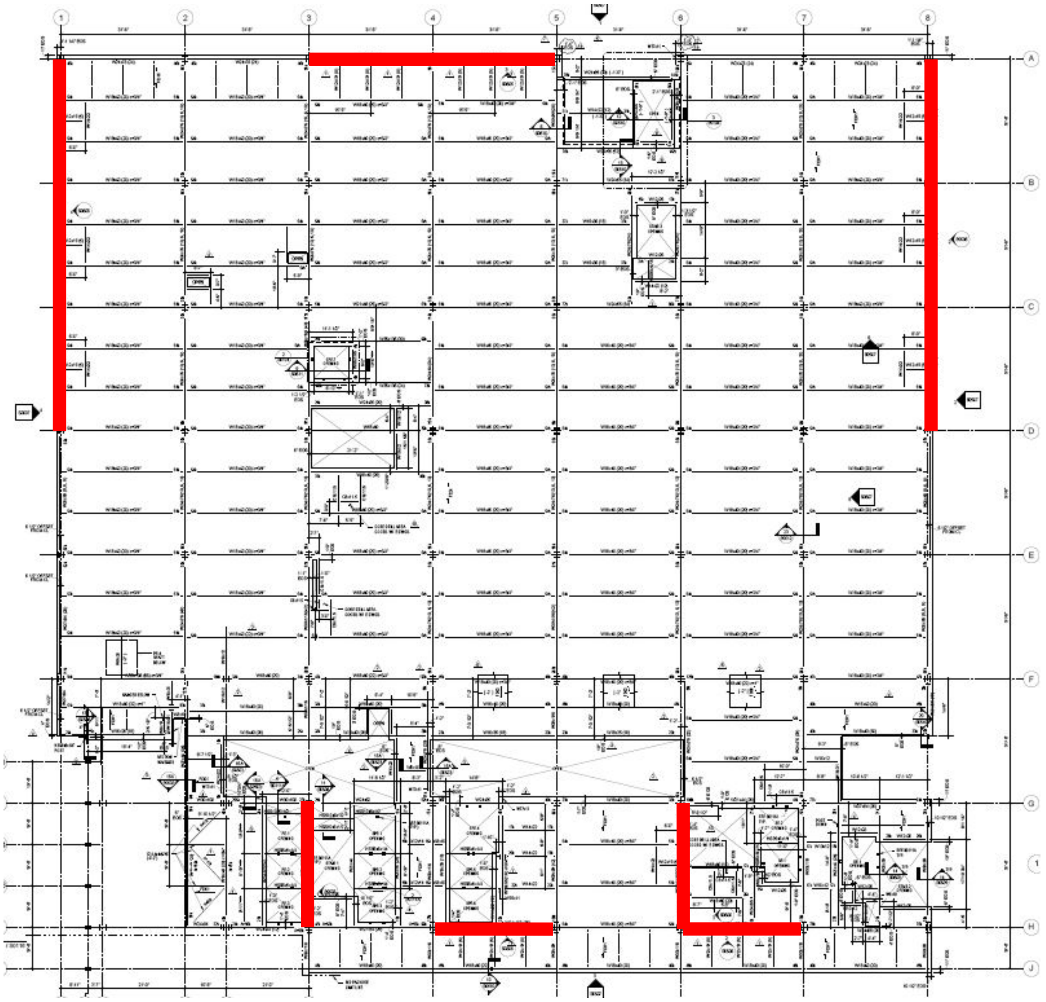


Figure B – Level Two Framing Plan with Braced Frames Highlighted

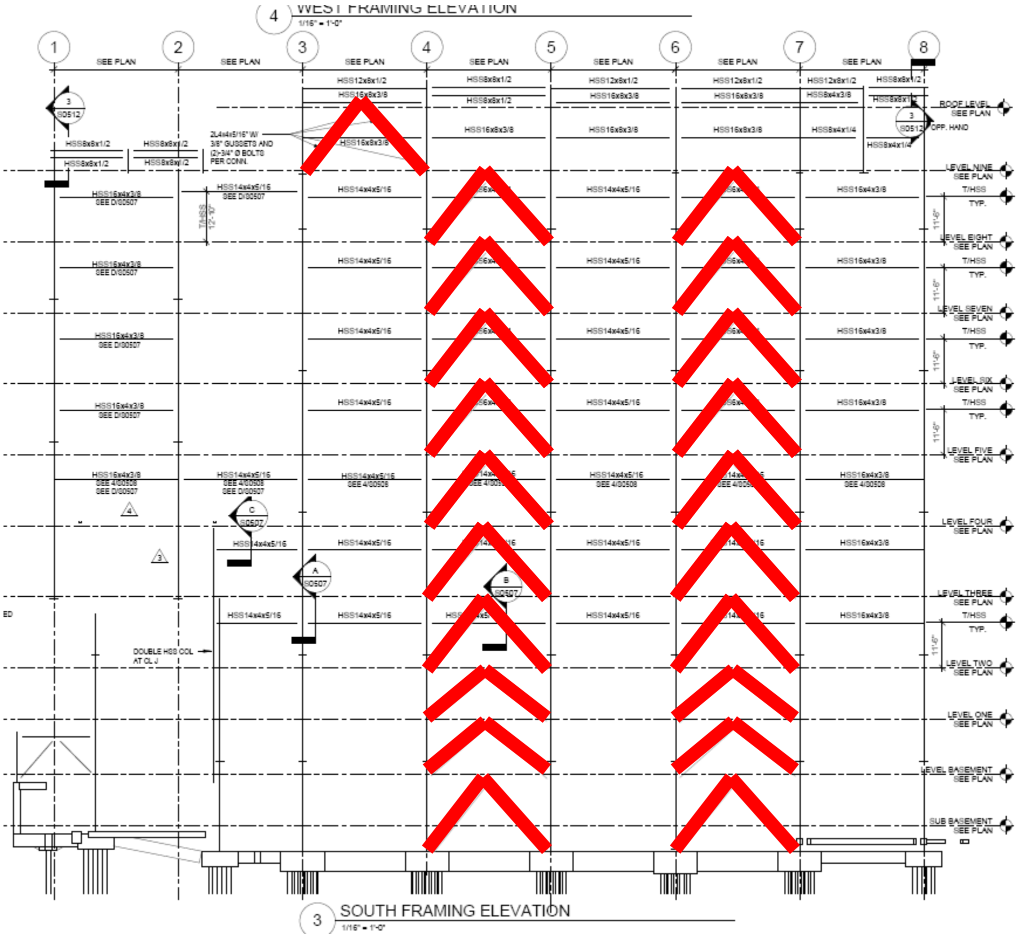


Figure C – South Braced Frame Elevation

Link to Existing Structure

It should be noted that GHVI is connected at four levels to the existing University at Buffalo Hospital. This link is minimal and has very little impact on the structure of the new building. It has been determined that Cannon Design modeled this new building as its own entity, and so for the sake of this thesis this same assumption will be made.

Codes and References

Original Design Codes

- National Model Code:
  Building Code of New York State 2007

- Design Codes:
  "Load and Resistance Factor Design Specification for Structural Steel Buildings," AISC

  "Code of Standard Practice for Steel Buildings and Bridges", AISC

  "Manual of Steel Construction - Load and Resistance Factor Design," AISC

- Structural Standards:
  American Society of Civil Engineers, SEI/ASCE 7-02,
  Minimum Design Loads for Buildings and Other Structures

Thesis Design Codes

- National Model Code:
  2009 International Building Code

- Design Codes:
  Steel Construction Manual 13th edition, AISC

  ACI 318-05, Building Code Requirements for Structural Concrete

- Structural Standards:
  American Society of Civil Engineers, ASCE/SEI 7-10,
  Minimum Design Loads for Buildings and Other Structures

Materials

Structure Steel:

Type	Standard	Grade
Wide Flange Shapes, WT's	ASTM A-992	
Channels & Angles	ASTM A-36	
Pipe	ASTM A-53	Grade B
Hollow Structural Sections (Rectangular & Round)	ASTM A-500	Grade B
Base Plates	ASTM A-572	Grade 42
All Other Steel Members	ASTM A-36	

Concrete:

Type	f'c (psi)	Unit Weight (pcf)
Pile Caps	4000	150
Grade Beams	4000	150
All Other Concrete	4000	150
Slabs-On-Grade	3000	150
Foundation Walls	4000	150

Reinforcing:

Type	Standard	Grade
Typical Bars	ASTM A-615	60
Welded Bars	ASTM A-706	60
Welded Wire Fabric	ASTM A-185	
Steel Fibers	ASTM A-820	Type 1
Bars Noted To Be Field Bent	ASTM A-615	40

Connectors:

Type	Standard
High Strength Bolts, Nuts, & Washers	ASTM A-325 or A-490 (min. 3/4 Diameter)
Anchor Rods	ASTM F1554
Welding Electrode	E70XX
Steel Deck Welding Electrode	E60XX min.

Gravity and Lateral Loads

Design Floor Dead Loads

Typical Floor

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Total	87.0 psf

Typical Roof

3' Steel Deck	4.5 psf
Adhered Membrane	2.0 psf
4" Rigid Insulation	6.0 psf
1/2" Protection Board	2.0 psf
Total	14.5 psf

Electrical and Mechanical Areas

Steel Deck and 7.5" Slab	75.0 psf
Steel Beams	12.0 psf
Concrete Pad	25.0 psf
Total	112.0 psf

Vivarium (Level 7)

Steel Deck and 7.5" Slab	75.0 psf
Membrane and 6" LTWT Topping	65.0 psf
Steel Beams	12.0 psf
Masonry Partitions	73.0 psf
Total	225.0 psf

Superimposed Dead Load

MEP	15.0 psf
Ceiling	5.0 psf
Leveling Concrete for Deflection	5.0 psf
Total	25.0 psf

Exterior Curtain Wall – 15.0 psf

Partitions – 10.0 psf

Floor Live Loads

Occupancy or Use	Design (psf)	ASCE 7-10 (psf)
Vivarium	80	60
Hotel (Patient) Floor	125	40
Procedure and Lab Floors	125	60
Mechanical Floors	150	--
Mechanical Floors with Catwalks below	175	--
Electrical Floors	200	--
Mechanical Mezzanine (Low)	40	40
Storage	--	20
Lobby	--	100
Stairs	--	100
Corrridors	--	100
Roof	--	20

Wind Analysis

The wind loads for GHVI were analyzed using Chapters 26 and 27 of ASCE 7-10. Wind loads for the Main Wind-Force Resisting System were determined using the directional procedure for buildings of all heights. The building was modeled as a square and therefore each direction shared the same inputs and results. Based on an occupancy category of IV, a basic wind speed of 120 mph was used to find the windward and leeward pressures. By code, flexible buildings can be affected by wind gusts and have the potential for resonance response. Because this building is considered flexible, a gust-effect factor also had to be determined. Refer to Appendix B for detailed calculations including the initial parameters, an effective length check, gust-effect factor calculations, wind pressure coefficients, and the calculated wind pressures.

Wind Story Forces

		Load (kips)		Shear (kips)		Moment (ft-kips)	
Level	**Height (ft)**	**N-S**	**E-W**	**N-S**	**E-W**	**N-S**	**E-W**
Roof	184	248.9	248.9	0.0	0.0	45797.6	45797.6
9	166	232.7	232.7	248.9	248.9	38628.2	38628.2
8	148	229.5	229.5	481.6	481.6	33966	33966
7	130	226.0	226.0	711.1	711.1	29380	29380
6	112	221.8	221.8	937.1	937.1	24841.6	24841.6
5	94	217.4	217.4	1158.9	1158.9	20435.6	20435.6
4	76	212.4	212.4	1376.3	1376.3	16142.4	16142.4
3	58	206.3	206.3	1588.7	1588.7	11965.4	11965.4
2	40	171.8	171.8	1795.0	1795.0	6872	6872
1	27	117.1	117.1	1966.8	1966.8	3161.7	3161.7
Basement	18	71.7	71.7	2083.9	2083.9	1290.6	1290.6
Mechanical	13	88.9	88.9	2155.6	2155.6	1155.7	1155.7
	Total	2244.5	2244.5	2244.5	2244.5	233636.8	233636.8

Table 1 – Wind loads, shears, and moments calculated for each story

From Table 1 it can be seen that there is a base shear of 2244.5 kips at the bottom level of the structure. This number is almost double compared to the value determined by the design engineer using the ASCE 7-02. It is probable that this large difference can be attributed to a difference in the two codes. In ASCE 7-02, the basic wind speed for Buffalo, NY is 90 mph, whereas in ASCE 7-10, the basic wind speed is 120 mph. Because the basic wind speed is squared in the equation for wind pressure, this difference is further exacerbated. The conclusion can be made that ASCE 7 is definitely becoming more stringent with its wind design, and so a larger base shear can be expected.

Figure D – Wind pressure diagram for both directions

Figure D shows the wind pressure diagram for both the north-south and east-west directions. The windward loads are on the left, and the leeward loads are on the right. Note that the large increase in load at the top of both the windward and leeward sides is due to the increase in pressure at the parapet, as prescribed by ASCE 7-10 section 27.4.5. Figure E shows the wind force diagram and the base shear the building experiences.

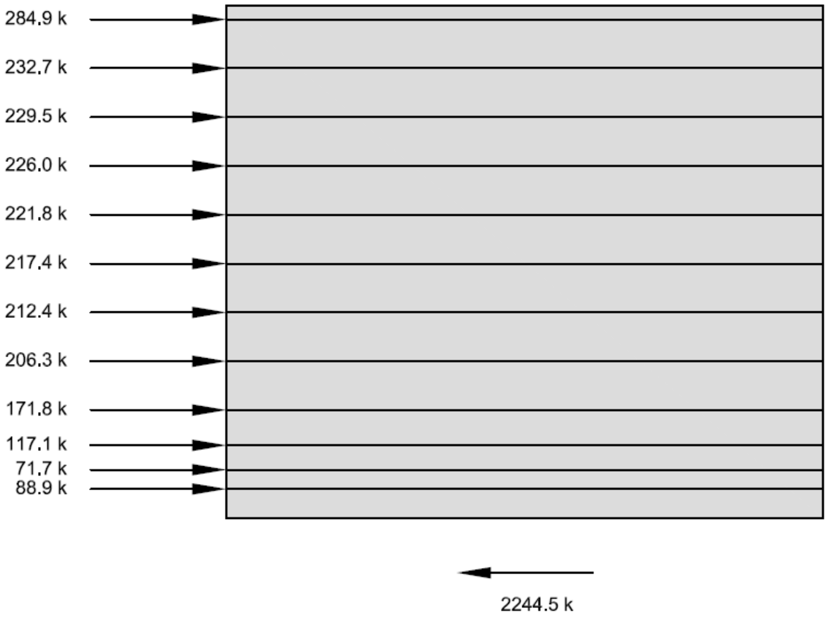


Figure E – Wind force diagram for both directions

Seismic Analysis

Seismic analysis for GHVI was done with reference to Chapters 11 and 12 in ASCE 7-10. Because the building is square, both the north-south and east-west directions are the same. The first step in this analysis was the estimated summation of the entire building weight above grade, which included the beams, columns, composite slab, exterior walls, superimposed dead load, and partitions of each level. An excel spreadsheet was set up to go through the building floor-by-floor and estimate as precisely as possible the building weight. An example of a typical level summation, as well as the total floor and building weights, can be found in Appendix C. The estimated building weight was found to be 52636 kips, which is slightly larger than the number provided by Cannon Design. The Equivalent Lateral Force Procedure was then used to determine the base shear and this base shear was then distributed to the diaphragm of each level as seen in Table 2. A more detailed set of calculations for the seismic analysis can be found in Appendix C.

Level	h_i (ft)	h (ft)	w (k)	$w*h^k$	C_{VX}	f_i (k)	V_i (k)	M_i (ft-k)
Roof	16.5	183.5	1056	4399489	0.050	66	66	12025
9	18	167	4089	14647886	0.166	218	284	36436
8	18	149	6354	18965263	0.215	282	566	42090
7	18	131	6437	15637755	0.177	233	799	30513
6	18	113	6395	12265558	0.139	183	982	20644
5	18	95	6167	8963056	0.101	134	1115	12683
4	18	77	6202	6442027	0.073	96	1211	7388
3	18	59	6433	4365516	0.049	65	1276	3836
2	13	41	6067	2300314	0.026	34	1311	1405
1	14	28	958	197321	0.002	3	1313	82
Base/Mech	14	14	2478	168587	0.002	3	1316	35
		Σ =	52635.7	88352771	1.000	1316		167137

Table 2 – Seismic Design Loads

Table 2 shows a total base shear of 1316 kips, and an overturning moment of 167137 foot-kips. The design engineers calculated a base shear of 1030 kips using ASCE 7-02. The difference in these two numbers can be attributed to the fact that the design engineers used a smaller total building weight, but more importantly because the response modification factor, R, was 5 in ASCE 7-02, and is now 3.25 in ASCE 7-10. A lower R value results in a more conservative number, hence a higher base shear. Refer to Figure F on the next page for the seismic force diagram.

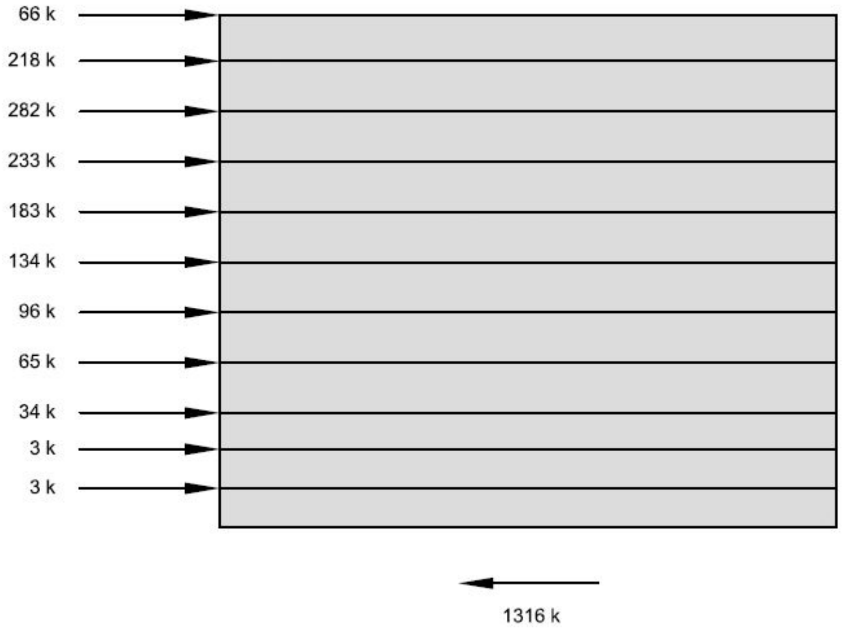


Figure F – Seismic Force Diagram

Snow Loads

Snow loading for GHVI was calculated based on Chapter 7 in ASCE 7-10. A ground snow load of 50 psf was determined from a site-specific case study provided by Cannon Design. The exposure factor, thermal factor, and importance factor were then obtained from the code and used to calculate the flat roof snow load of 42 psf, which matched the value obtained by the design engineers. Because part of the roof is lower than the majority, drift calculations were performed to find the maximum snow loading in these areas. The detailed calculations for snow loading can be found in Appendix D.

Typical Floor Spot Checks

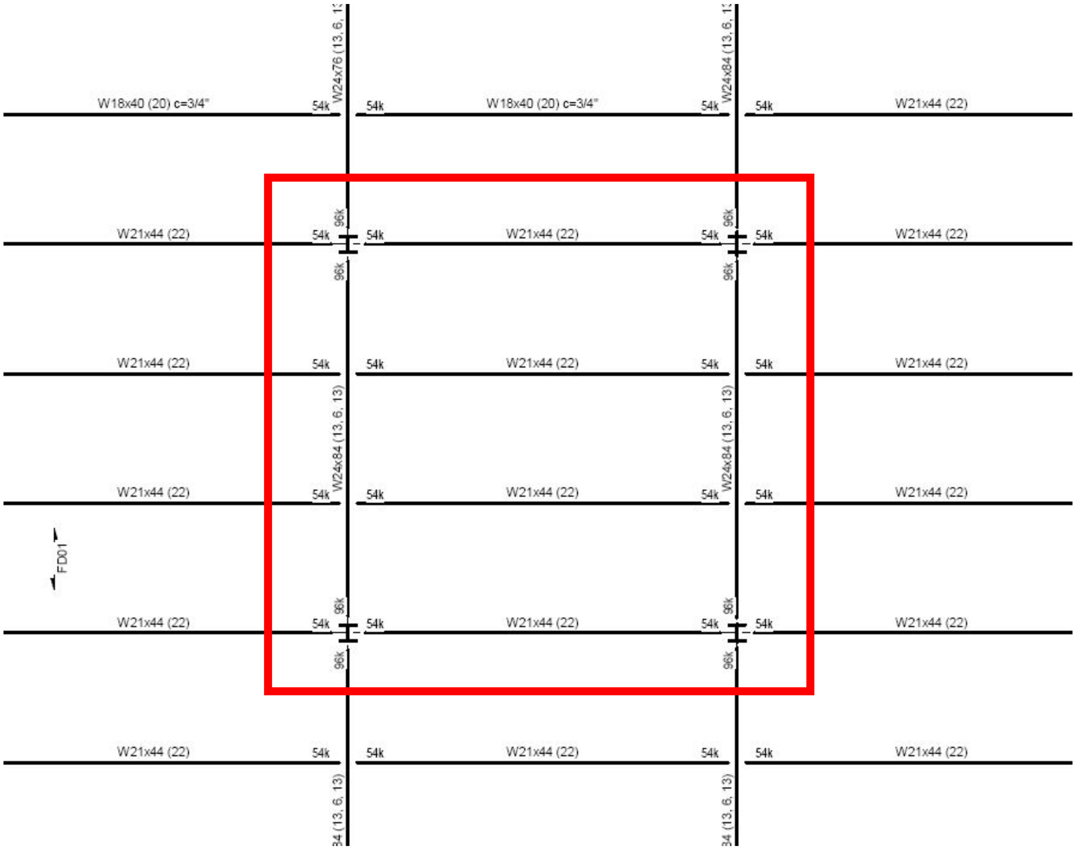


Figure G – Typical Bay on Level 4

Metal Decking Spot Check

A spot check was performed to verify the composite metal decking on the fourth floor. From the structural drawings and specifications, it was determined that the most typical type of composite metal deck was 3", 18-gage deck with 3000 psi normal weight concrete and a total slab thickness of 7½". The structural notes also require the deck be placed over a minimum of three spans, and from Figure G it can be seen that the deck clear span is 10'-6". Referencing the Vulcraft Deck Catalog, it can be determined that the maximum unshored span for 7½" slab over 3 or more spans is 13'-3", and so this deck is sufficient. When checking the superimposed live load it must be noted that level four is a procedural floor with a live load of 125 psf and a superimposed dead load of 25 psf. The maximum allowable superimposed live load is 275 psf, much greater than the 150 psf this deck requires. This may seem excessive, but in order to obtain the required fire rating for this floor assembly, a thicker deck is required. Refer to Appendix E for detailed calculations on the metal decking spot check.

Beam Spot Check

A spot check on the composite W21x44 beam shown in red in Figure H was also performed. This beam is most typical on the fourth floor, spanning 31’-6” from girder to girder, with a tributary width of 10’-6”, and composite metal deck running parallel. The spot check verified that the beam meets all requirements for strength and serviceability. These requirements include shear strength, flexural strength, live load deflection, and wet concrete deflection. From the spot check analysis it was determined that this beam is more than sufficient, and that 20 shear studs are necessary for the beam to act compositely. The calculations for this spot check can be found in Appendix E.

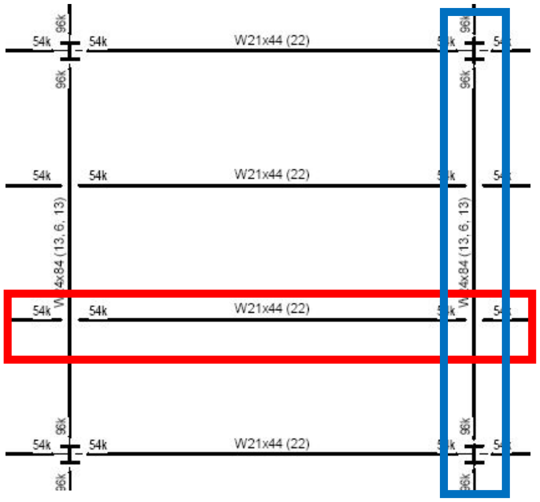


Figure H – Beam Spot Check

Girder Spot Check

The girder analyzed in this spot check was the W24x84 beam shown in blue in Figure H. Also spanning 31’-6”, this is the most typical girder on the fourth floor. The girder was analyzed using two point loads from the previously checked beam, and composite metal deck spanning perpendicular. It was checked for shear strength, flexural strength, live load deflection, and wet concrete deflection and like the beam, it was more than sufficient. This slight overdesign of these members may be contributed to the future variability of this building and its spaces so that the floors can possibly meet higher load requirements. The girder spot check calculations can also be found in Appendix E.

Column Spot Check

The final spot check was performed on the W14x159 column E-6 on the fourth floor. In order to check this column, the axial load was summed from the roof level downward, taking into effect the changes in dead load, superimposed dead load, live load, and reducible live load for each level. Table 11 in Appendix E shows the calculation of this axial load. After the axial load P_u was determined, it was checked against the value of ϕP_n for a W14x159 member with an unbraced length of 18 feet. The maximum allowable axial load from the Steel Manual was greater than the calculated axial load, and therefore the column is sufficient. Refer to Appendix E for more detailed calculations regarding this column spot check.

Conclusion

This first technical report has provided an investigation into the structural concepts and existing conditions of the Kaleida Health Global Heart and Vascular Institute and University at Buffalo CTRC/Incubator, Additions, and Renovations.

The structural system has been described in detail, focusing on the foundation, floor system, gravity system, lateral system, and link to the existing structure. It has been concluded from inquiry for this report that the building should be modeled without regard to this existing link.

The original design codes and the ones used for thesis have been listed for comparison and reference, and the various materials throughout the building have also been incorporated. The building dead loads have been calculated, and the live loads used by the design engineers have been checked with the current version of ASCE 7, so that the most severe live loads were used in calculations.

From the wind and seismic analysis the conclusion can be made that ASCE 7 is definitely becoming more and more stringent with its design. Both the wind analysis and the seismic analysis conducted using ASCE 7-10 resulted in higher base shears and overturning moments. When one searches for the cause of these differences it can be found in basic increases to the design code.

Finally, spot checks throughout the building verified that the existing conditions are adequate. Because of the careful analysis of these existing conditions and structural concepts, it will now be possible to perform the remaining technical reports required by thesis.

Appendix

Appendix A: Typical Floor Plans and Elevations

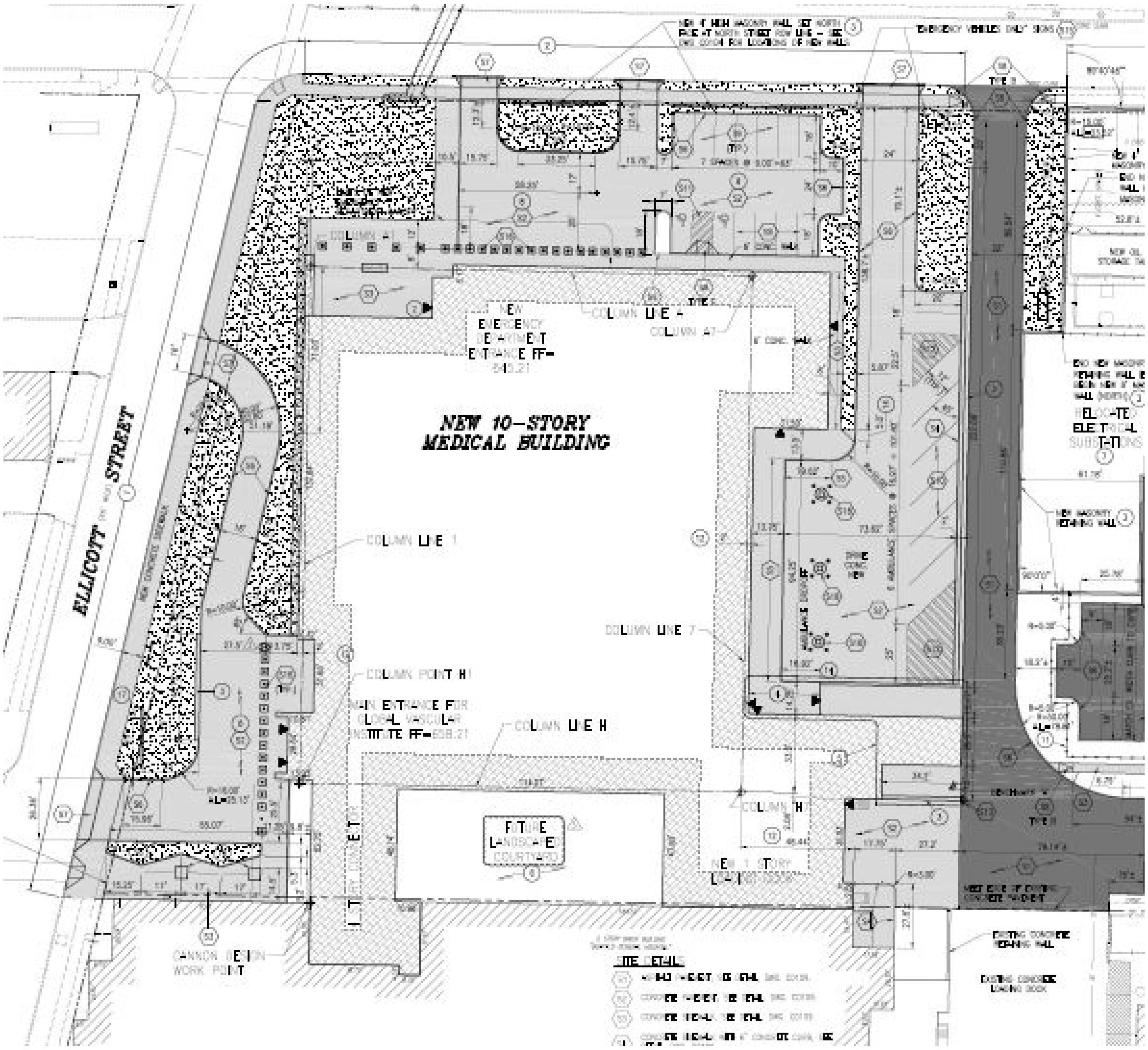


Figure I – Site Plan

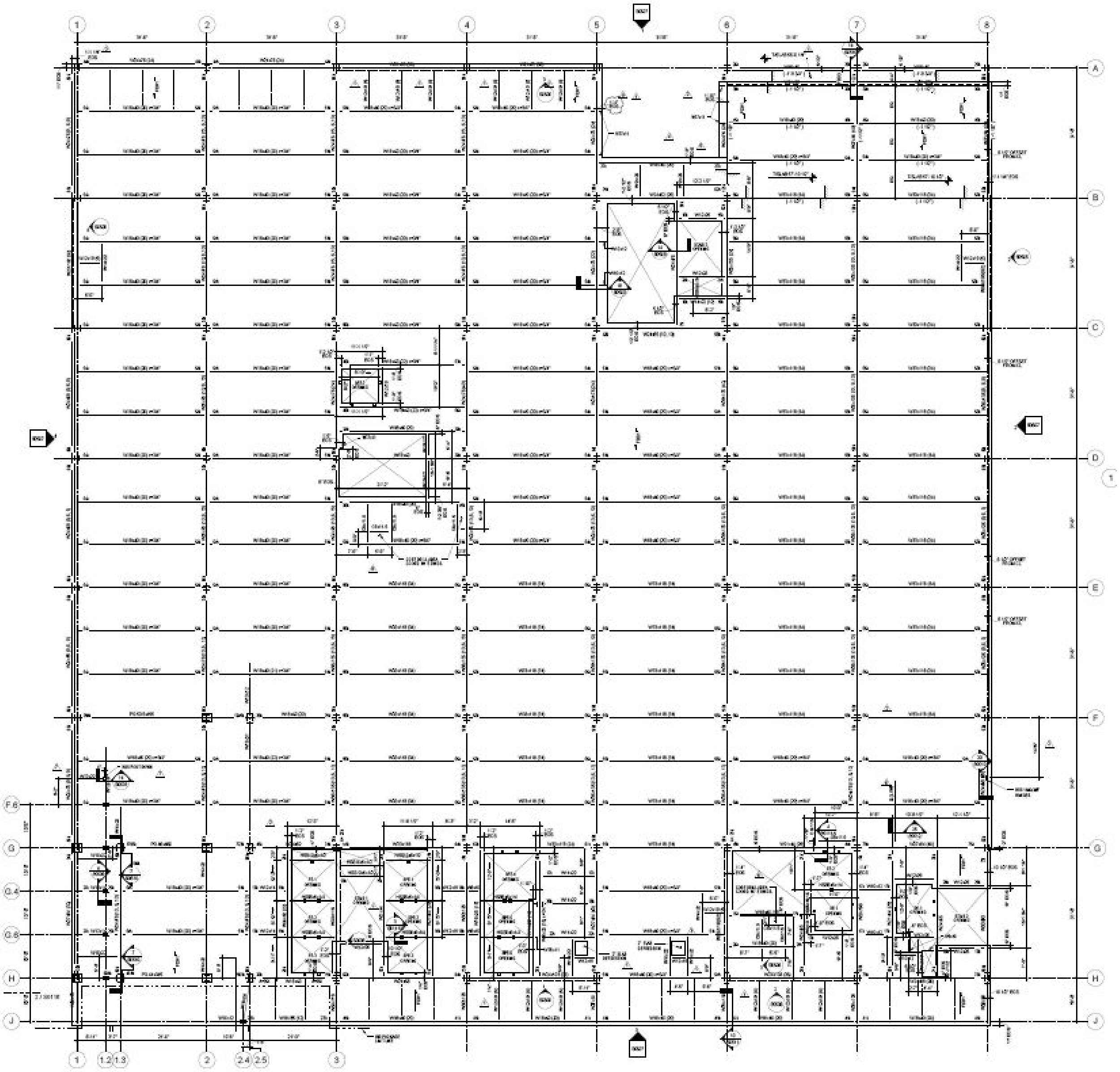


Figure J – Typical floor framing plan

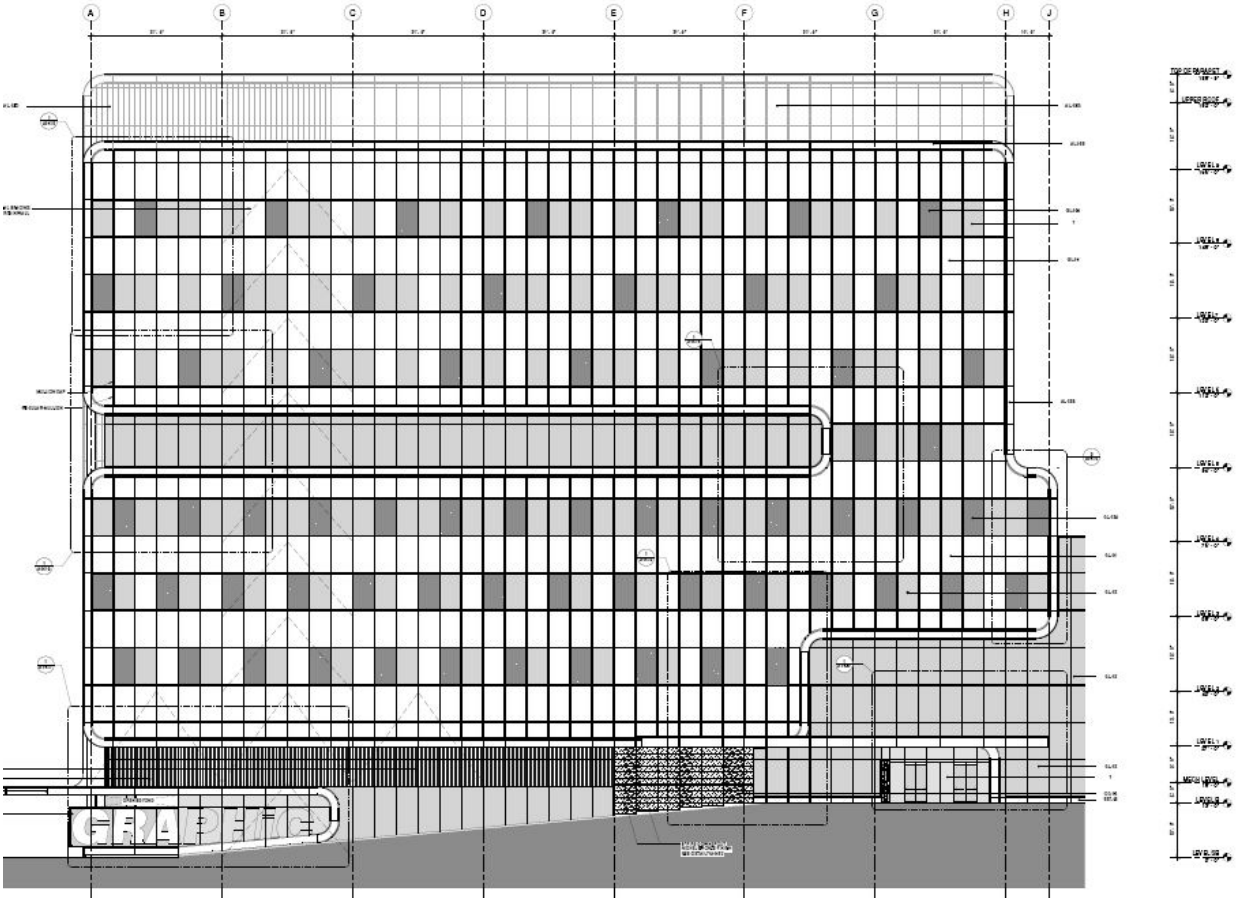


Figure K – West elevation

Appendix B: Wind Analysis

The following table contains the initial parameters used in the wind analysis as determined from ASCE 7-10:

V	120
K_d	0.85
Exposure	B
K_{zt}	1
GC_{pi}	0.18

Table 3 - Parameters

The following table contains the effective length calculations completed to assure that the natural frequency could be approximated:

N-S Direction				**E-S Direction**			
Level	**h_i**	**l_i**	**h_il_i**	**Level**	**h_i**	**l_i**	**h_il_i**
Sub basement	13	221	2873	Sub basement	13	174	2262
Basement	18	221	3978	Basement	18	221	3978
Mechanical	27	221	5967	Mechanical	27	221	5967
1	40	221	8840	1	40	221	8840
2	58	221	12818	2	58	221	12818
3	76	221	16796	3	76	221	16796
4	94	221	20774	4	94	221	20774
5	112	221	24752	5	112	221	24752
6	130	221	28730	6	130	221	28730
7	148	221	32708	7	148	221	32708
8	166	221	36686	8	166	221	36686
9	189	158	29862	9	189	221	41769
Σ =	1071		224784	**Σ =**	1071		236080
L_{eff} =	209.8824			**L_{eff} =**	220.4295		

Table 4 – Effective Length Check Calculations

The following table contains the calculations to determine the gust-effect factor:

Gust Effect Calculation		
	N-S	**E-W**
B	221	221
L	221	221
h	189	189
n_a	0.3968	0.3968
	FLEXIBLE	FLEXIBLE
I_z	0.244	0.244
c	0.30	0.30
z	113.4	113.4
g_Q	3.4	3.4
g_v	3.4	3.4
g_R	3.96	3.96
R	0.575	0.575
R_n	0.0956	0.0956
N_1	1.777	1.777
L_z	482.89	482.89
V_z	107.83	107.83
R_h	0.2638	0.2638
n	3.20	3.20
R_B	0.2316	0.2316
n	3.74	3.74
R_L	0.0767	0.0767
n	12.52	12.52
Q	0.799	0.799
β	0.01	0.01
G_f	0.95	0.95

Table 5 – Gust Effect Calculations

The following table contains the wind pressure coefficients:

Wind Pressure Coefficients			
Surface	**L/B**	**Cp**	**Use With**
Windward	All	0.8	q_z
Leeward	1	-0.5	q_h
Side	All	-0.7	q_h

Table 6 – Wind Pressure Coefficients

The following tables contains the wind pressure in pounds per square feet for both the windward and leeward directions:

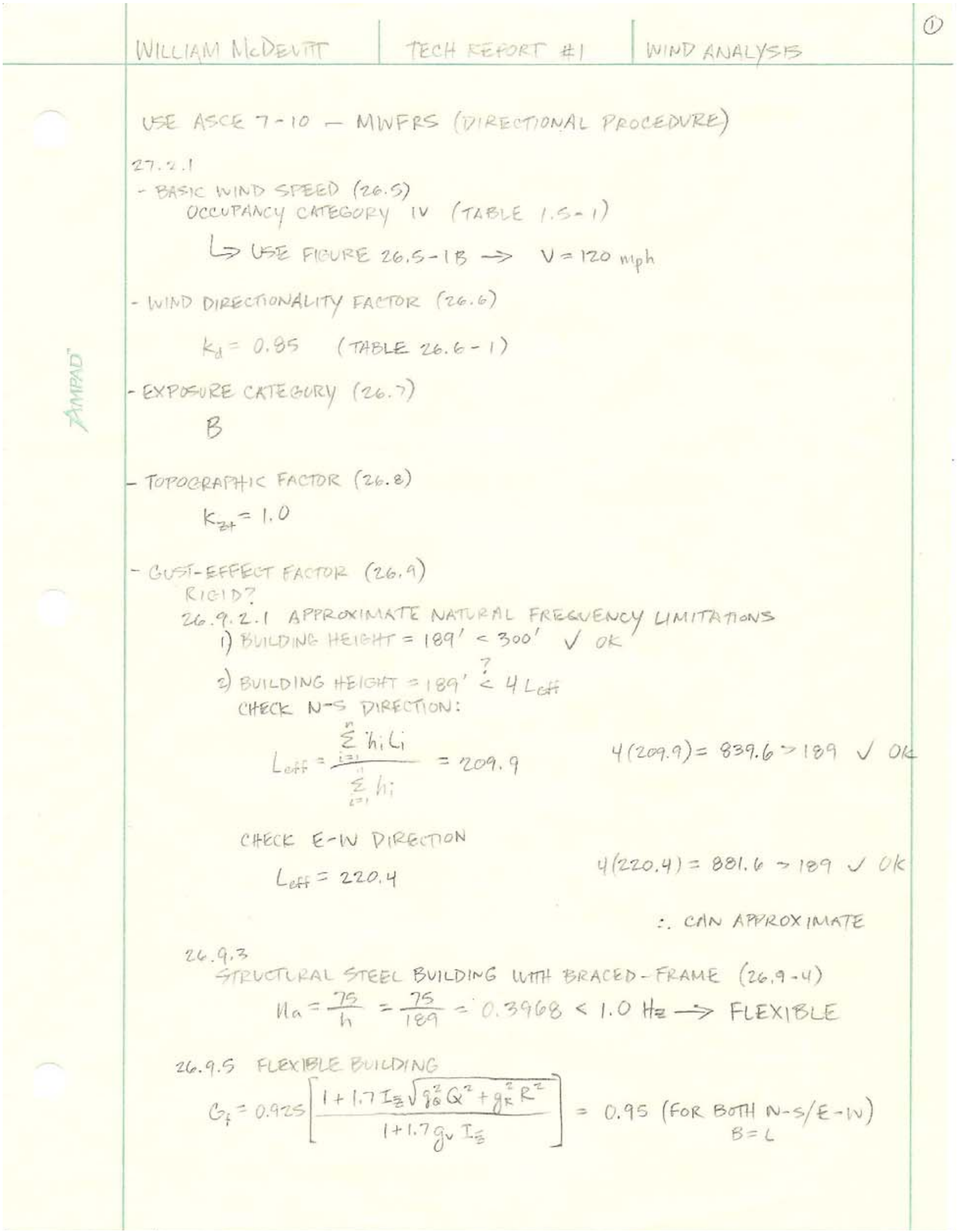
	Level	Height (ft)	K_z	q_z	Wind Pressure	
					N-S	E-W
Windward	Top of Parapet	189	1.18	37.1	90.4	90.4
	Upper Roof	184	1.18	36.8	34.6	34.6
	9	166	1.14	35.8	33.8	33.8
	8	148	1.11	34.7	33.0	33.0
	7	130	1.07	33.4	32.0	32.0
	6	112	1.02	32.0	30.9	30.9
	5	94	0.97	30.5	29.8	29.8
	4	76	0.91	28.6	28.4	28.4
	3	58	0.84	26.4	26.7	26.7
	2	40	0.76	23.8	24.7	24.7
	1	27	0.68	21.2	22.7	22.7
	Mechanical	18	0.60	18.8	20.9	20.9
	Basement	13	0.57	17.9	20.2	20.2

Table 7 – Windward Wind Pressures

	Level	q_h	Wind Pressure	
			N-S	E-W
Leeward	Top of Parapet	37.1	-61.4	-61.4
	Remaining	37.1	-24.3	-24.3

Table 8 – Leeward Wind Pressures

Technical Report #1



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USE ASCE 7-10 — MWFRS (DIRECTIONAL PROCEDURE)

27.2.1

- BASIC WIND SPEED (26.5)
  OCCUPANCY CATEGORY IV (TABLE 1.5-1)
  ↳ USE FIGURE 26.5-1B → V = 120 mph

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

- EXPOSURE CATEGORY (26.7)
  B

- TOPOGRAPHIC FACTOR (26.8)
  $K_{zt} = 1.0$

- GUST-EFFECT FACTOR (26.9)
  RIGID?

26.9.2.1 APPROXIMATE NATURAL FREQUENCY LIMITATIONS

1) BUILDING HEIGHT = 189′ < 300′ ✓ OK

2) BUILDING HEIGHT = 189′ $\overset{?}{<}$ 4 L_{eff}

CHECK N-S DIRECTION:

$$L_{eff} = \frac{\sum_{i=1}^{n} h_i L_i}{\sum_{i=1}^{n} h_i} = 209.9 \qquad 4(209.9) = 839.6 > 189 \checkmark \text{ OK}$$

CHECK E-W DIRECTION

$$L_{eff} = 220.4 \qquad 4(220.4) = 881.6 > 189 \checkmark \text{ OK}$$

∴ CAN APPROXIMATE

26.9.3

STRUCTURAL STEEL BUILDING WITH BRACED-FRAME (26.9-4)

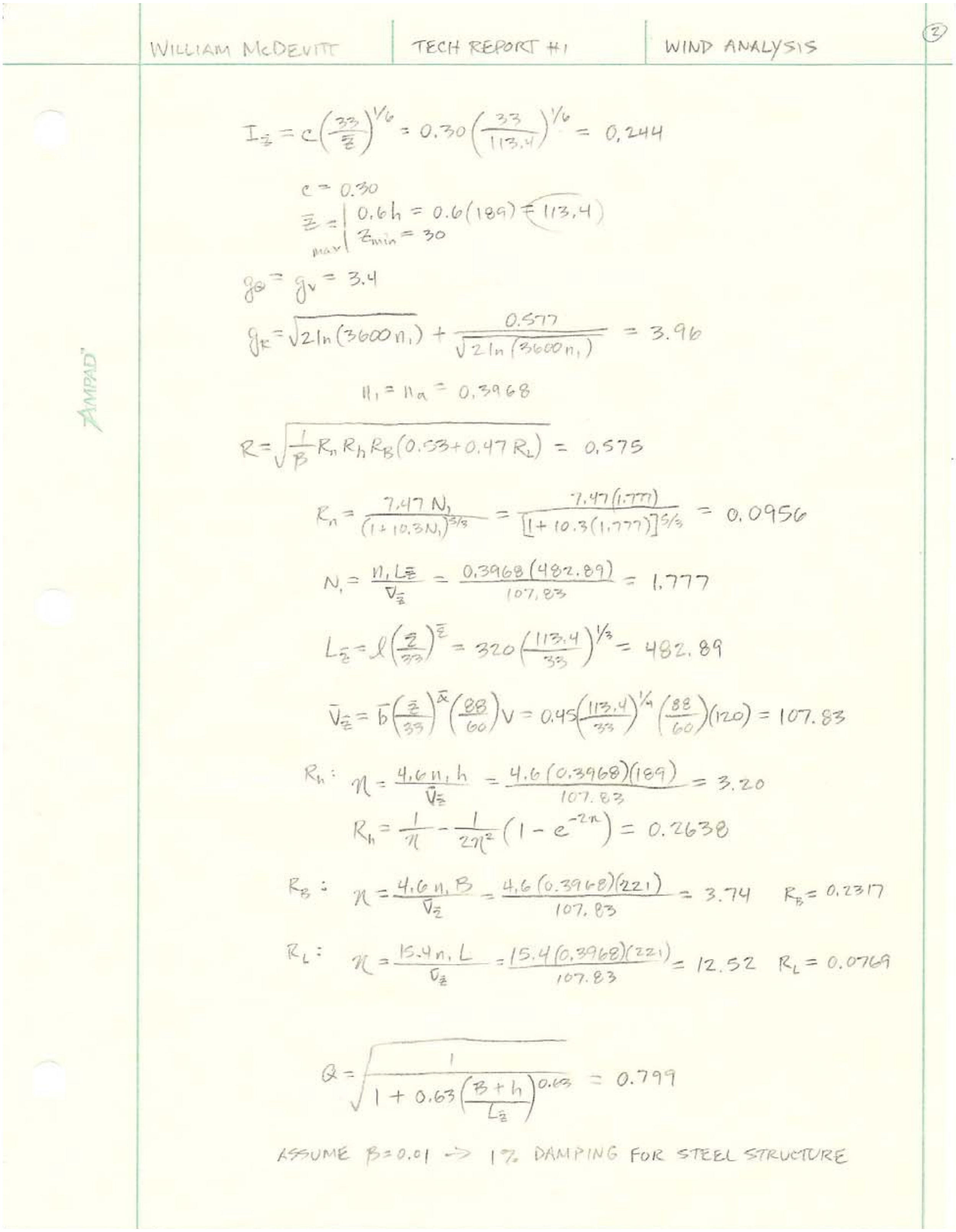
$$n_a = \frac{75}{h} = \frac{75}{189} = 0.3968 < 1.0 \text{ Hz} \rightarrow \text{FLEXIBLE}$$

26.9.5 FLEXIBLE BUILDING

$$G_f = 0.925\left[\frac{1 + 1.7 I_{\bar{z}} \sqrt{g_Q^2 Q^2 + g_R^2 R^2}}{1 + 1.7 g_v I_{\bar{z}}}\right] = 0.95 \text{ (FOR BOTH N-S/E-W)}$$

B = L

Technical Report #1



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$$I_{\bar{z}} = c\left(\frac{33}{\bar{z}}\right)^{1/6} = 0.30\left(\frac{33}{113.4}\right)^{1/6} = 0.244$$

$c = 0.30$

$$\bar{z} = \max \left| \begin{array}{l} 0.6h = 0.6(189) = \boxed{113.4} \\ z_{min} = 30 \end{array} \right.$$

$g_Q = g_v = 3.4$

$$g_R = \sqrt{2\ln(3600 n_1)} + \frac{0.577}{\sqrt{2\ln(3600 n_1)}} = 3.96$$

$n_1 = n_a = 0.3968$

$$R = \sqrt{\frac{1}{\beta} R_n R_h R_B (0.53 + 0.47 R_L)} = 0.575$$

$$R_n = \frac{7.47 N_1}{(1+10.3N_1)^{5/3}} = \frac{7.47(1.777)}{[1+10.3(1.777)]^{5/3}} = 0.0956$$

$$N_1 = \frac{n_1 L_{\bar{z}}}{\bar{V}_{\bar{z}}} = \frac{0.3968(482.89)}{107.83} = 1.777$$

$$L_{\bar{z}} = \ell\left(\frac{\bar{z}}{33}\right)^{\bar{\epsilon}} = 320\left(\frac{113.4}{33}\right)^{1/3} = 482.89$$

$$\bar{V}_{\bar{z}} = \bar{b}\left(\frac{\bar{z}}{33}\right)^{\bar{\alpha}}\left(\frac{88}{60}\right)V = 0.45\left(\frac{113.4}{33}\right)^{1/4}\left(\frac{88}{60}\right)(120) = 107.83$$

R_h: $\eta = \frac{4.6 n_1 h}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3968)(189)}{107.83} = 3.20$

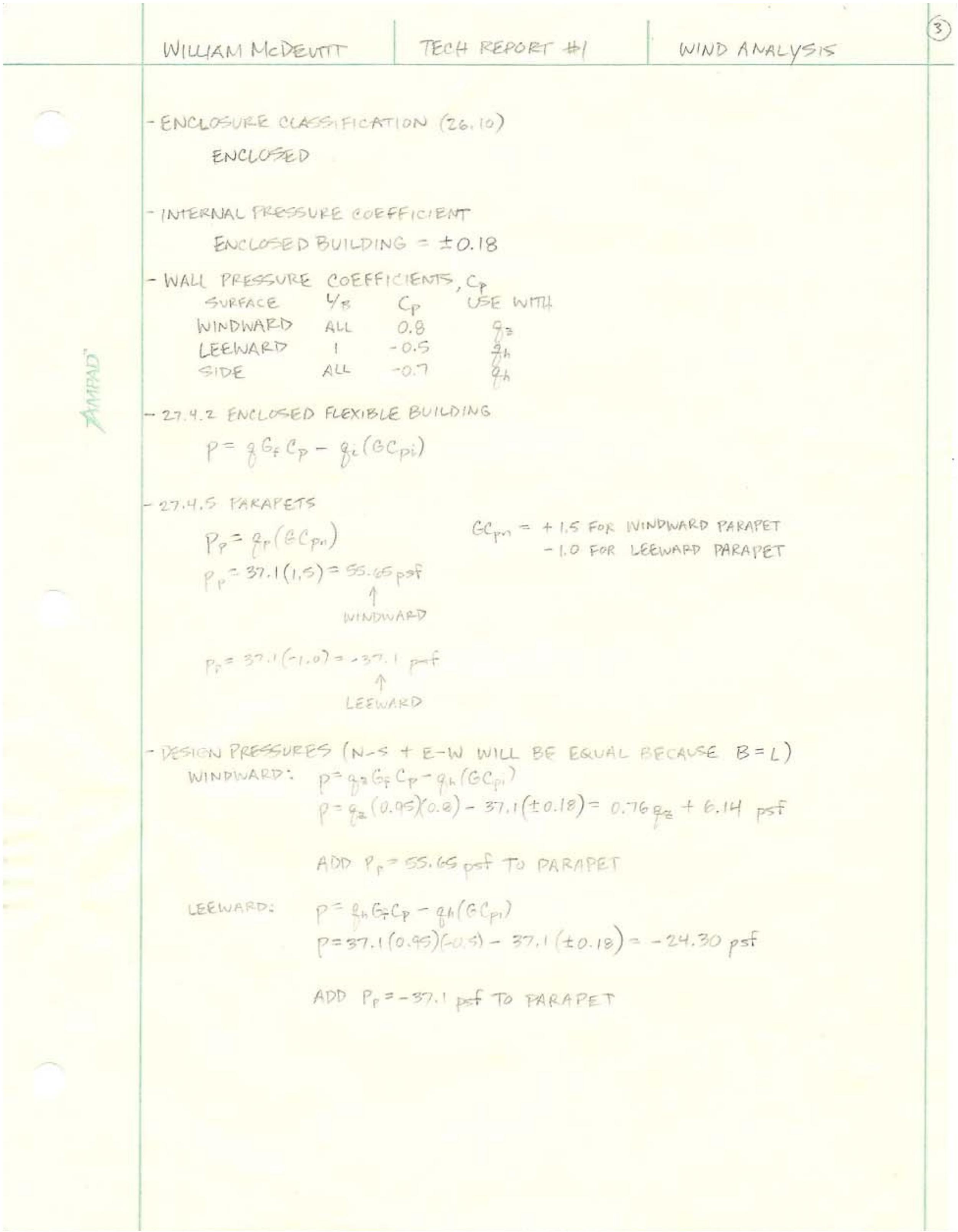
$$R_h = \frac{1}{\eta} - \frac{1}{2\eta^2}\left(1 - e^{-2\eta}\right) = 0.2638$$

R_B: $\eta = \frac{4.6 n_1 B}{\bar{V}_{\bar{z}}} = \frac{4.6(0.3968)(221)}{107.83} = 3.74$ $R_B = 0.2317$

R_L: $\eta = \frac{15.4 n_1 L}{\bar{V}_{\bar{z}}} = \frac{15.4(0.3968)(221)}{107.83} = 12.52$ $R_L = 0.0769$

$$Q = \sqrt{\frac{1}{1 + 0.63\left(\frac{B+h}{L_{\bar{z}}}\right)^{0.63}}} = 0.799$$

ASSUME $\beta = 0.01$ → 1% DAMPING FOR STEEL STRUCTURE



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- ENCLOSURE CLASSIFICATION (26.10)

ENCLOSED

- INTERNAL PRESSURE COEFFICIENT

ENCLOSED BUILDING = ±0.18

- WALL PRESSURE COEFFICIENTS, C_p

SURFACE	L/B	C_p	USE WITH
WINDWARD	ALL	0.8	q_z
LEEWARD	1	-0.5	q_h
SIDE	ALL	-0.7	q_h

- 27.4.2 ENCLOSED FLEXIBLE BUILDING

$$p = q G_f C_p - q_i (GC_{pi})$$

- 27.4.5 PARAPETS

$$P_p = q_p (GC_{pn})$$

GC_{pn} = + 1.5 FOR WINDWARD PARAPET
- 1.0 FOR LEEWARD PARAPET

$$P_p = 37.1(1.5) = 55.65 \text{ psf}$$

↑
WINDWARD

$$P_p = 37.1(-1.0) = -37.1 \text{ psf}$$

↑
LEEWARD

- DESIGN PRESSURES (N-S + E-W WILL BE EQUAL BECAUSE B = L)

WINDWARD:

$$p = q_z G_f C_p - q_h (GC_{pi})$$

$$p = q_z (0.95)(0.8) - 37.1(\pm 0.18) = 0.76 q_z + 6.14 \text{ psf}$$

ADD P_p = 55.65 psf TO PARAPET

LEEWARD:

$$p = q_h G_f C_p - q_h (GC_{pi})$$

$$p = 37.1(0.95)(-0.5) - 37.1(\pm 0.18) = -24.30 \text{ psf}$$

ADD P_p = -37.1 psf TO PARAPET

Appendix C: Seismic Analysis

The following table contains an example summation of the weight of a floor for use in seismic analysis:

Level 6					
Steel-Beams	**Type**	**Number**	**Length(ft)**	**Weight (lb/ft)**	**Weight (lb)**
	W27x94	154	31.5	94	455994
	W30x108	56	31.5	108	190512
				Total Beams	646506.0
Steel-Columns	**Type**	**Number**	**Length(ft)**	**Weight (lb/ft)**	**Weight (lb)**
	W14x68	5	18	68	6120
	W14x74	3	18	74	3996
	W14x90	31	18	90	50220
	W14x109	15	18	109	29430
	W14x120	10	18	120	21600
				Total Columns	111366.0
Deck	**Type**	**Weight (psf)**	**Area (ft2)**	**Weight (lb)**	
	3" (7.5")	75	48841	3663075.0	
			Total Deck	3663075.0	
Total L6 Weight (lb)				4420947.0	

Table 9 – Example Weight Summation

The following table contains the summation of the total building weight above grade:

Level	Weight (k)
Roof	1056
9	4089
8	6354
7	6437
6	6395
5	6167
4	6202
3	6433
2	6067
1	958
Base/Mech	2478
Total	52636

Table 10 – Total Weight

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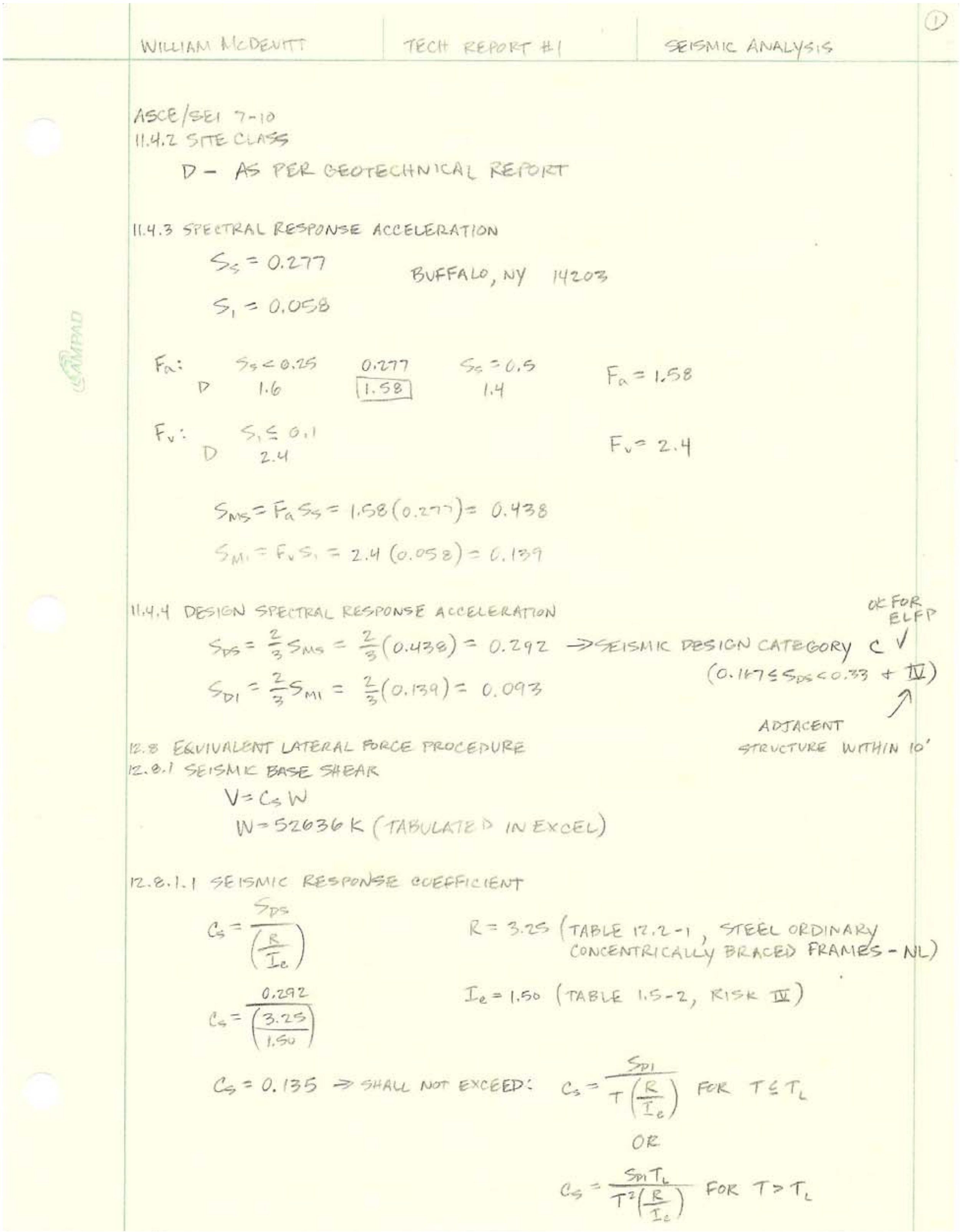

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ASCE/SEI 7-10

11.4.2 SITE CLASS

D - AS PER GEOTECHNICAL REPORT

11.4.3 SPECTRAL RESPONSE ACCELERATION

$S_S = 0.277$ BUFFALO, NY 14203

$S_1 = 0.058$

F_a:		$S_S < 0.25$	0.277	$S_S = 0.5$	$F_a = 1.58$
	D	1.6	$\boxed{1.58}$	1.4	
F_v:		$S_1 \leq 0.1$			$F_v = 2.4$
	D	2.4			

$S_{MS} = F_a S_S = 1.58(0.277) = 0.438$

$S_{M1} = F_v S_1 = 2.4\,(0.058) = 0.139$

11.4.4 DESIGN SPECTRAL RESPONSE ACCELERATION

$S_{DS} = \frac{2}{3} S_{MS} = \frac{2}{3}(0.438) = 0.292 \Rightarrow$ SEISMIC DESIGN CATEGORY C ✓ OK FOR ELFP

$S_{D1} = \frac{2}{3} S_{M1} = \frac{2}{3}(0.139) = 0.093$

$(0.167 \leq S_{DS} < 0.33$ + IV)

↗ ADJACENT STRUCTURE WITHIN 10'

12.8 EQUIVALENT LATERAL FORCE PROCEDURE

12.8.1 SEISMIC BASE SHEAR

$V = C_s W$

$W = 52636$ K (TABULATED IN EXCEL)

12.8.1.1 SEISMIC RESPONSE COEFFICIENT

$$C_s = \frac{S_{DS}}{\left(\frac{R}{I_e}\right)}$$

R = 3.25 (TABLE 12.2-1, STEEL ORDINARY CONCENTRICALLY BRACED FRAMES - NL)

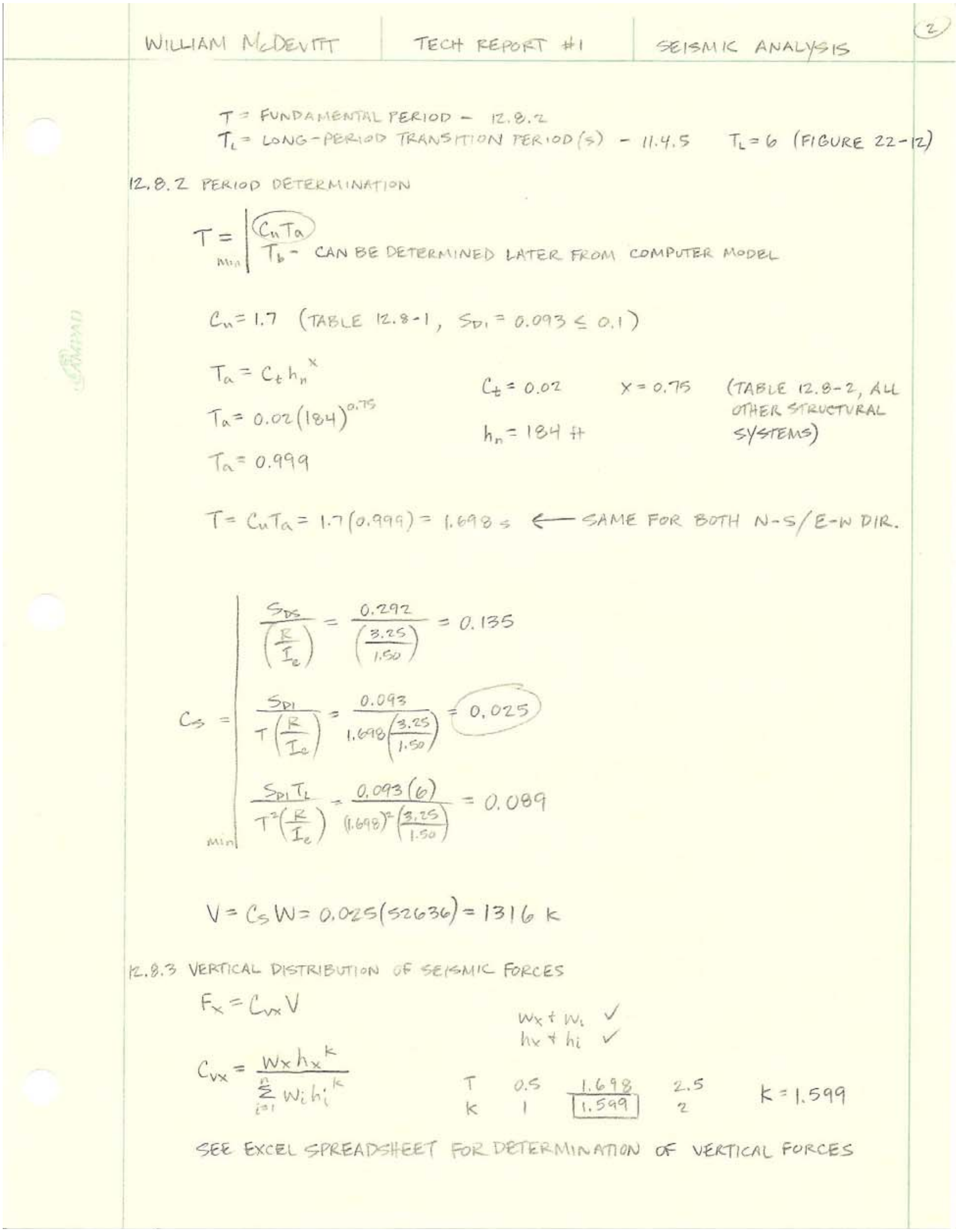
$$C_s = \frac{0.292}{\left(\frac{3.25}{1.50}\right)}$$

$I_e = 1.50$ (TABLE 1.5-2, RISK IV)

$C_s = 0.135 \Rightarrow$ SHALL NOT EXCEED: $C_s = \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)}$ FOR $T \leq T_L$

OR

$$C_s = \frac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} \text{ FOR } T > T_L$$



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T = FUNDAMENTAL PERIOD – 12.8.2
T_L = LONG-PERIOD TRANSITION PERIOD (s) – 11.4.5 $T_L = 6$ (FIGURE 22-12)

12.8.2 PERIOD DETERMINATION

$$T = \min \begin{cases} \boxed{C_u T_a} \\ T_b \text{ - CAN BE DETERMINED LATER FROM COMPUTER MODEL} \end{cases}$$

$C_u = 1.7$ (TABLE 12.8-1, $S_{D1} = 0.093 \leq 0.1$)

$T_a = C_t h_n^{x}$ $C_t = 0.02$ $x = 0.75$ (TABLE 12.8-2, ALL OTHER STRUCTURAL SYSTEMS)

$T_a = 0.02(184)^{0.75}$ $h_n = 184$ ft

$T_a = 0.999$

$T = C_u T_a = 1.7(0.999) = 1.698$ s ← SAME FOR BOTH N-S/E-W DIR.

$$C_s = \min \begin{cases} \dfrac{S_{DS}}{\left(\frac{R}{I_e}\right)} = \dfrac{0.292}{\left(\frac{3.25}{1.50}\right)} = 0.135 \\[2ex] \dfrac{S_{D1}}{T\left(\frac{R}{I_e}\right)} = \dfrac{0.093}{1.698\left(\frac{3.25}{1.50}\right)} = \boxed{0.025} \\[2ex] \dfrac{S_{D1} T_L}{T^2\left(\frac{R}{I_e}\right)} = \dfrac{0.093(6)}{(1.698)^2\left(\frac{3.25}{1.50}\right)} = 0.089 \end{cases}$$

$V = C_s W = 0.025(52636) = 1316$ k

12.8.3 VERTICAL DISTRIBUTION OF SEISMIC FORCES

$F_x = C_{vx} V$

$w_x \neq w_i$ ✓
$h_x \neq h_i$ ✓

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

T	0.5	1.698	2.5
k	1	1.599	2

$k = 1.599$

SEE EXCEL SPREADSHEET FOR DETERMINATION OF VERTICAL FORCES

Appendix D: Snow Loading

7.2 GROUND SNOW LOADS

P_g = CS FROM FIGURE 7-1

AS PER CASE STUDY, $P_g = 50$ psf

7.3 FLAT ROOF SNOW LOADS, P_f

$P_f = 0.7 C_e C_t I_s P_g$

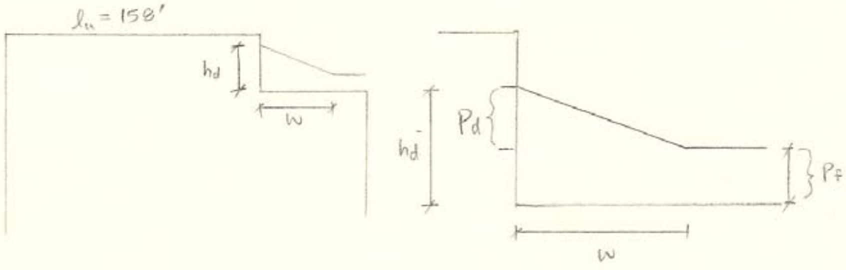
$P_f = 0.7(1.0)(1.0)(1.2)(50) = 42$ psf

7.3.1 $C_e = 1.0$ (EXPOSURE B, PARTIALLY EXPOSED - PARAPET)

7.3.2 $C_t = 1.0$

7.3.3 $I_s = 1.2$ (RISK CATEGORY IV)

7.7 DRIFT



$h_d = 0.43\sqrt[3]{l_u}\sqrt[4]{P_g+10} - 1.5$

$h_d = 0.43(158)^{1/3}(50+10)^{1/4} - 1.5 = 4.97$ ft

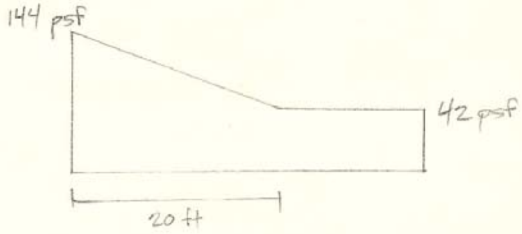
$w = 4h_d = 4(4.97)$

$w = 19.9$ ft

$$\gamma_{\min} = \left|\begin{array}{l} 0.13P_g + 14 = 0.13(50) + 14 \boxed{= 20.5\text{ pcf}} \\ 30\text{ pcf} \end{array}\right.$$

$P_d = h_d\gamma = 4.97(20.5) = 101.9$ psf

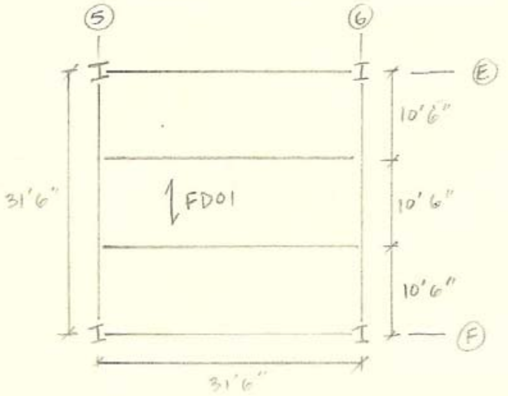
$P_{total} = 42 + 101.9 = 143.9$ psf



Appendix E: Spot Checks

METAL DECKING SPOT CHECK

CHECK MOST COMMON DECK TYPE MARK FD01, ON LEVEL 4
TOTAL SLAB THICKNESS = 7 ½"
DECK TYPE = 3"
GAGE = 18
f'_c = 3000 psi, 145 pcf
PLACE OVER MINIMUM OF THREE SPANS



LEVEL 4 IS PROCEDURE FLOOR
LL = 125 psf
SDL = 25 psf

TOTAL LOAD = 150 psf

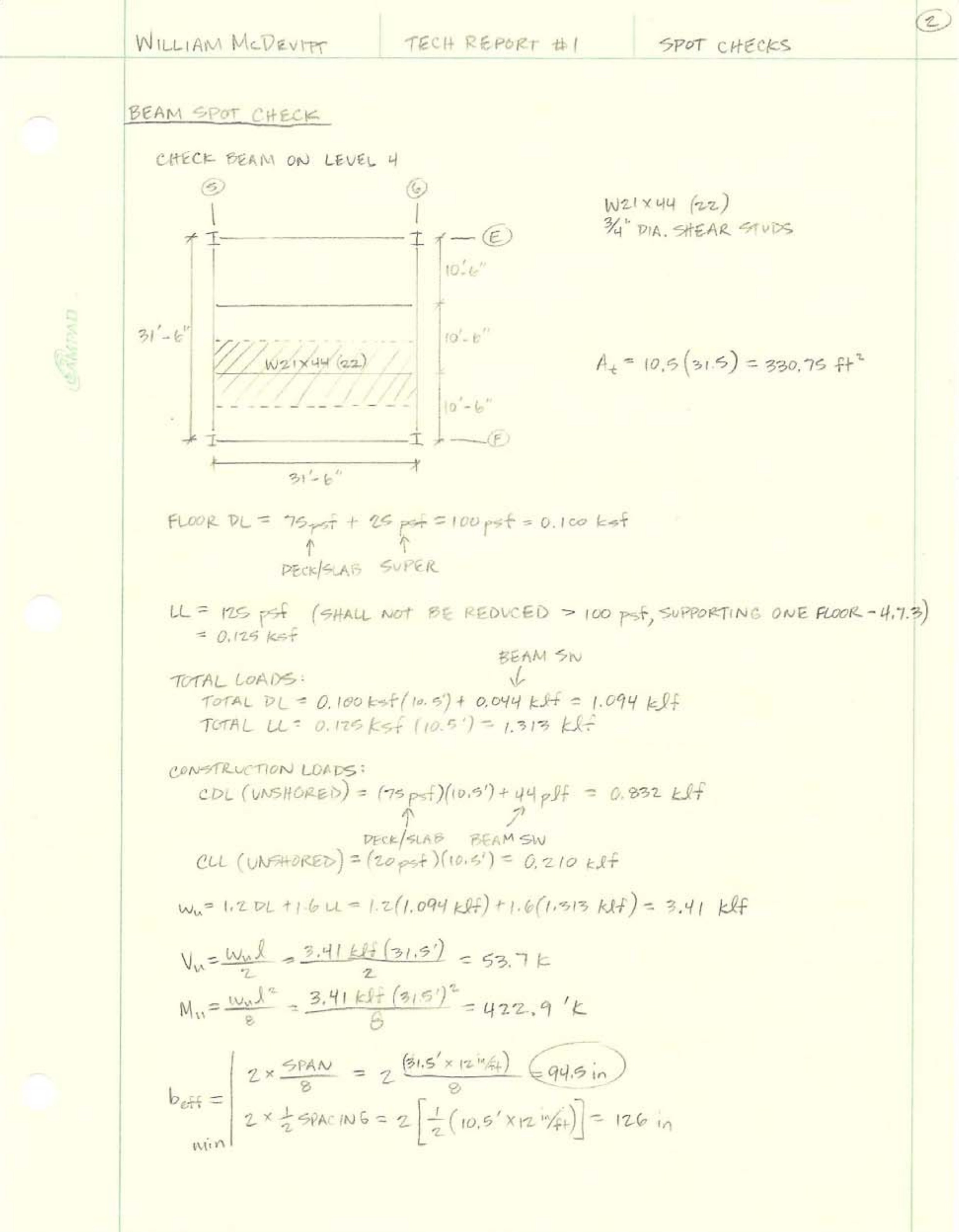
USING VULCRAFT DECK CATALOG:

CHECK MAXIMUM UNSHORED SPAN
$\left.\begin{array}{l}\text{7.5" SLAB}\\ \text{3 OR MORE SPANS}\end{array}\right\}$ 3VLI18 ⟶ 13'-3" > 10'-6" ✓ ∴ OK

CHECK SUPERIMPOSED LL
7.5" SLAB
10'-6" CLEAR SPAN ⟶ 275 psf >> 150 psf ✓ ∴ OK

THIS SEEMS EXCESSIVE BUT
IN ORDER TO OBTAIN THE
REQUIRED FIRE RATING A
THICKER DECK IS NEEDED

Technical Report #1



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BEAM SPOT CHECK

CHECK BEAM ON LEVEL 4

W21X44 (22)
3/4" DIA. SHEAR STUDS

$A_t = 10.5(31.5) = 330.75 \text{ ft}^2$

FLOOR DL = 75 psf + 25 psf = 100 psf = 0.100 ksf

$\uparrow$ DECK/SLAB $\uparrow$ SUPER

LL = 125 psf (SHALL NOT BE REDUCED > 100 psf, SUPPORTING ONE FLOOR - 4.7.3)
= 0.125 ksf

TOTAL LOADS:

TOTAL DL = 0.100 ksf(10.5') + 0.044 klf = 1.094 klf ($\downarrow$ BEAM SW)
TOTAL LL = 0.125 ksf (10.5') = 1.313 klf

CONSTRUCTION LOADS:

CDL (UNSHORED) = (75 psf)(10.5') + 44 plf = 0.832 klf

$\uparrow$ DECK/SLAB $\nearrow$ BEAM SW

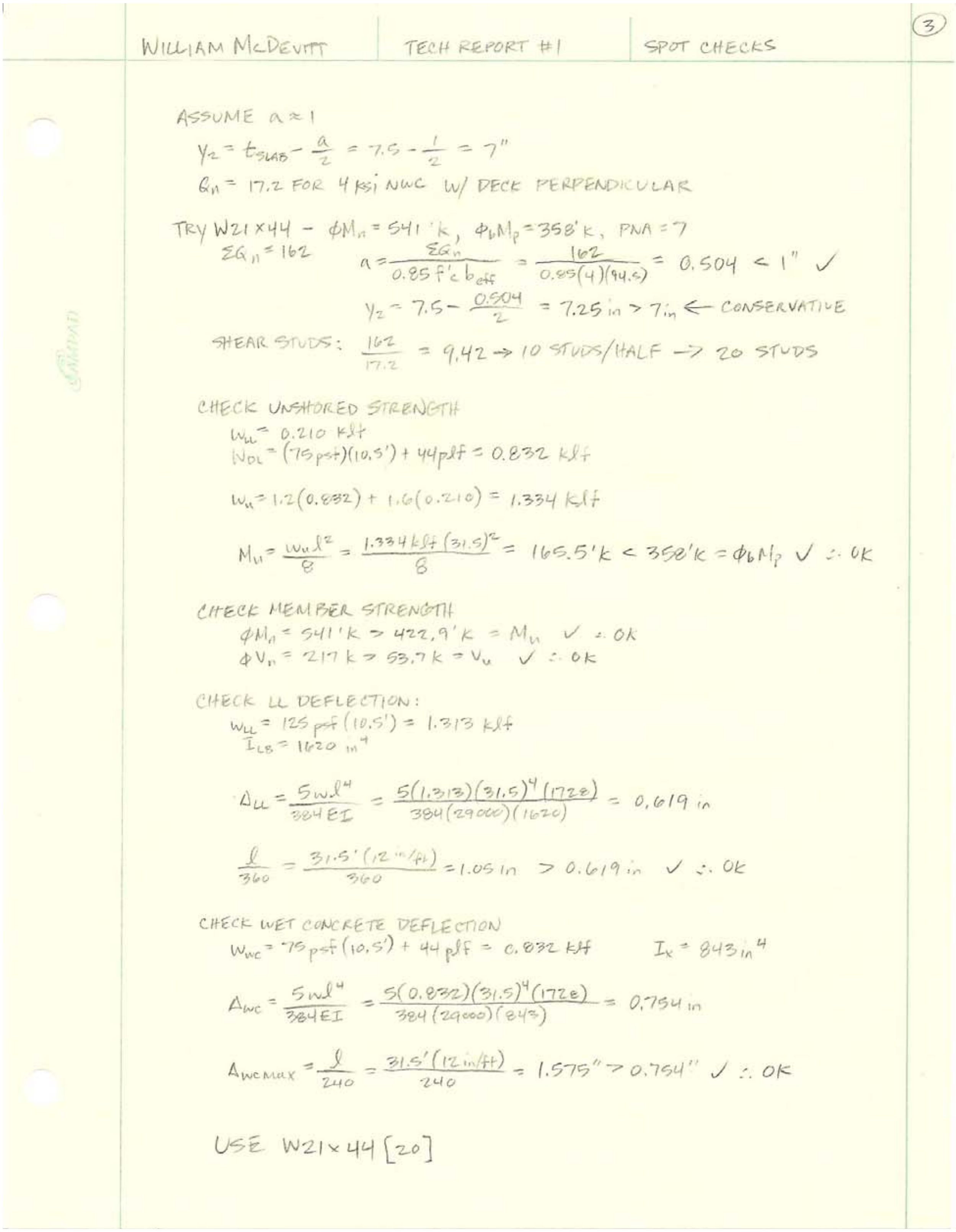
CLL (UNSHORED) = (20 psf)(10.5') = 0.210 klf

$w_u = 1.2DL + 1.6LL = 1.2(1.094 \text{ klf}) + 1.6(1.313 \text{ klf}) = 3.41 \text{ klf}$

$$V_u = \frac{w_u l}{2} = \frac{3.41 \text{ klf}(31.5')}{2} = 53.7 \text{ k}$$

$$M_u = \frac{w_u l^2}{8} = \frac{3.41 \text{ klf}(31.5')^2}{8} = 422.9 \text{ 'k}$$

$$b_{eff} = \min \left| \begin{array}{l} 2 \times \frac{SPAN}{8} = 2 \frac{(31.5' \times 12 \text{ in/ft})}{8} \boxed{= 94.5 \text{ in}} \\ 2 \times \frac{1}{2} SPACING = 2\left[\frac{1}{2}(10.5' \times 12 \text{ in/ft})\right] = 126 \text{ in} \end{array} \right.$$



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ASSUME $a \approx 1$

$$y_2 = t_{SLAB} - \frac{a}{2} = 7.5 - \frac{1}{2} = 7''$$

$Q_n = 17.2$ FOR 4 ksi NWC W/ DECK PERPENDICULAR

TRY W21×44 – $\phi M_n = 541'k$, $\phi_b M_p = 358'k$, PNA = 7

$\Sigma Q_n = 162$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{162}{0.85(4)(94.5)} = 0.504 < 1'' \checkmark$$

$$y_2 = 7.5 - \frac{0.504}{2} = 7.25 \text{ in} > 7_{in} \leftarrow \text{CONSERVATIVE}$$

SHEAR STUDS: $\frac{162}{17.2} = 9.42 \rightarrow$ 10 STUDS/HALF $\rightarrow$ 20 STUDS

CHECK UNSHORED STRENGTH

$w_{LL} = 0.210$ klf

$w_{DL} = (75 \text{psf})(10.5') + 44 \text{plf} = 0.832$ klf

$w_u = 1.2(0.832) + 1.6(0.210) = 1.334$ klf

$$M_u = \frac{w_u l^2}{8} = \frac{1.334 \text{ klf} (31.5)^2}{8} = 165.5'k < 358'k = \phi_b M_p \checkmark \therefore OK$$

CHECK MEMBER STRENGTH

$\phi M_n = 541'k > 422.9'k = M_u \checkmark \therefore OK$

$\phi V_n = 217 k > 53.7 k = V_u \checkmark \therefore OK$

CHECK LL DEFLECTION:

$w_{LL} = 125 \text{ psf} (10.5') = 1.313$ klf

$I_{LB} = 1620 \text{ in}^4$

$$\Delta_{LL} = \frac{5wl^4}{384EI} = \frac{5(1.313)(31.5)^4(1728)}{384(29000)(1620)} = 0.619 \text{ in}$$

$$\frac{l}{360} = \frac{31.5'(12 \text{ in/ft})}{360} = 1.05 \text{ in} > 0.619 \text{ in} \checkmark \therefore OK$$

CHECK WET CONCRETE DEFLECTION

$w_{wc} = 75 \text{ psf}(10.5') + 44 \text{ plf} = 0.832$ klf $\qquad I_x = 843 \text{ in}^4$

$$\Delta_{wc} = \frac{5wl^4}{384EI} = \frac{5(0.832)(31.5)^4(1728)}{384(29000)(843)} = 0.754 \text{ in}$$

$$\Delta_{wc\,max} = \frac{l}{240} = \frac{31.5'(12 \text{ in/ft})}{240} = 1.575'' > 0.754'' \checkmark \therefore OK$$

USE W21×44 [20]

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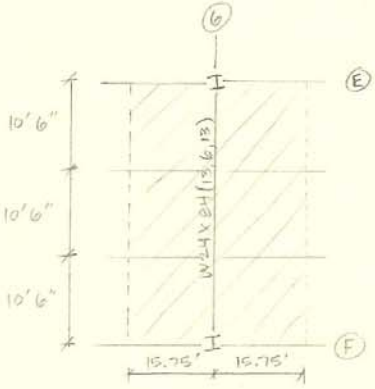
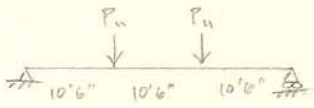
GIRDER SPOT CHECKS

CHECK GIRDER ON LEVEL 4





TOTAL LOADS:

TOTAL DL = 1.094 klf (2 · 15.75') = 34.5 k
TOTAL LL = 1.313 klf (2 · 15.75') = 41.4 k

CONSTRUCTION LOADS

CDL (UNSHORED) = 0.832 klf (2 · 15.75') = 26.2 k + BEAM SW
CLL (UNSHORED) = 0.210 klf (2 · 15.75') = 6.6 k

$P_u = 1.2DL + 1.6LL = 1.2(34.5) + 1.6(41.4) = 107.64\ K$

$V_u = P_u = 107.64\ k$

$M_u = 10.5'\ (107.64\ k) = 1130.2\ 'k$

$$b_{eff} = \left|\begin{array}{l} 2 \times \frac{SPAN}{8} = 2\,\frac{(31.5' \times 12\,in/ft)}{8} = \boxed{94.5\ in} \\ 2 \times \frac{1}{2} SPACING = 2\left[\frac{1}{2}(31.5' \times 12\,in/ft)\right] = 378\ in \end{array}\right.$$
min

ASSUME $a \approx 1$

$Y_2 = t_{SLAB} - \frac{a}{2} = 7.5 - \frac{1}{2} = 7''$

$Q_n = 21.2$ FOR 4 ksi NWC W/ DECK PARALLEL

TRY W24x84 – $\phi M_n = 1210'k$, $\phi_b M_p = 840'k$, PNA = 7

$\Sigma Q_n = 309$

$$a = \frac{\Sigma Q_n}{0.85 f'_c b_{eff}} = \frac{309}{0.85(4)(94.5)} = 0.962 < 1'' \checkmark$$

$$Y_2 = 7.5 - \frac{0.962}{2} = 7.02\ in > 7\ in \leftarrow \text{CONSERVATIVE}$$

SHEAR STUDS: $\frac{309}{21.2} = 14.58 \rightarrow 15$ STUDS/HALF $\rightarrow$ 30 STUDS

CHECK UNSHORED STRENGTH

$w_u = 1.2DL + 1.6LL = 1.2(0.084\text{ klf}) = 0.101\text{ klf}$ ← BEAM SW

$P_u = 1.2DL + 1.6LL = 1.2(26.2k) + 1.6(6.6k) = 42.0k$

$$M_u = \frac{w_u l^2}{8} + \frac{P_u l}{3} = \frac{0.101(31.5)^2}{8} + \frac{42.0(31.5)}{3} = 453.5'k < 840'k = \phi_b M_p$$

✓ ∴ OK

CHECK MEMBER STRENGTH

$\phi M_n = 1210'k > 1130.2'k = M_u$ ✓ ∴ OK

$\phi V_n = 340k > 107.6k = V_u$ ✓ ∴ OK

CHECK LL DEFLECTION

$P_{LL} = 41.4\text{ k}$ $\quad I_{LB} = 2940\text{ in}^4$

$$\Delta_{LL} = \frac{P_{LL} l^3}{28EI} = \frac{41.4(31.5)^3(1728)}{28(29000)(2940)} = 0.937\text{ in}$$

$$\Delta_{LL\,MAX} = \frac{l}{360} = \frac{31.5'(12\text{ in/ft})}{360} = 1.05\text{ in} > 0.937\text{ in}$$ ✓ ∴ OK

CHECK WET CONCRETE DEFLECTION

$P_{wc} = [75\text{ psf}(10.5') + 44\text{ plf}](2 \cdot 15.75') = 52.4\text{ k}$ $\quad I_x = 2370\text{ in}^4$

↖ BEAM

$w_{wc} = 75\text{ psf}(9.02''/12\text{ in/ft}) + 84\text{ plf} = 0.140\text{ klf}$

↖ b_f OF GIRDER

$$\Delta_{wc} = \underset{\text{GIRDER}}{\frac{5wl^4}{384EI}} + \underset{\text{BEAM}}{\frac{Pl^3}{28EI}} = \frac{5(0.140)(31.5)^4(1728)}{384(29000)(2370)} + \frac{52.4(31.5)^3(1728)}{28(29000)(2370)}$$

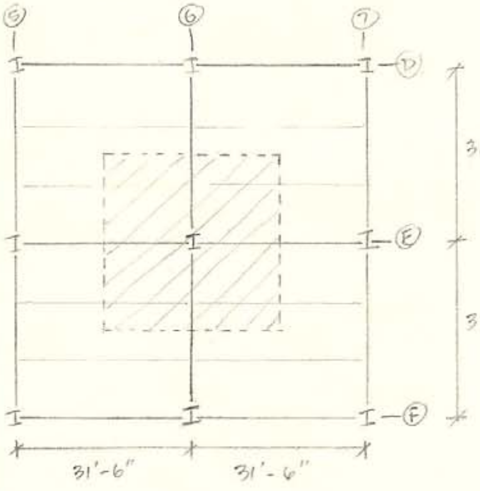
$$\Delta_{wc} = 0.045 + 1.47 = 1.515$$

$$\Delta_{wc\,MAX} = \frac{l}{240} = \frac{31.5'(12\text{ in/ft})}{240} = 1.575\text{ in} > 1.515\text{ in}$$ ✓ ∴ OK

USE W24 × 84 [30]

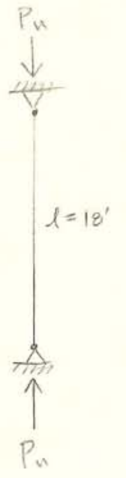
COLUMN SPOT CHECK

CHECK COLUMN E6 ON LEVEL 4 - W14 × 159



$A_t = (31.5)(31.5) = 992.25 \text{ ft}^2$

REFER TO EXCEL SPREADSHEET FOR AREA AND LOAD CALCULATIONS



$P_u = 1672$ K (FROM EXCEL)

$K = 1.0$

$l = 18'$ (UNBRACED LENGTH = LEVEL 4 FLOOR-TO-FLOOR HEIGHT)

FROM TABLE 4-1:

W14 × 159 $\phi P_n = 1700 \text{ k} > 1672 \text{ k} = P_u$ ✓ ∴ OK

USE W14 × 159

The following table contains calculations to determine the factored point load on column E-6 on Level 4:

Level	Area (ft^2)	DL (psf)	SL (psf)	LL (psf)	LL_{red} (psf)*	Column SW (lb)	P_u (k)
Roof	992.25	14.5	42	20	20	1353	100
9	992.25	100	--	125	125	1476	319
8	992.25	100	--	125	125	2160	320
7	992.25	225	--	80	64	2160	372
6	992.25	100	--	125	100	2862	281
5	992.25	100	--	125	100	2862	281
						$P_{u\ Total}$ =	1672

*Live Loads greater than 100 psf cannot be reduced unless they are supporting 2 or more floors; if so, they can be reduced by 20 percent.

Table 11 – Summation of Factored Column Point Load