



# Hershey Research Park Building One

---

## *Technical Report 1*

Jonathan Krepps

Structural Option

Advisor: Dr. Hanagan

9/17/12

# Table of Contents

Executive Summary.....3

Building Introduction.....4

Structural Overview.....5

*Foundation*.....5

*Floor System*.....5

*Lateral System*.....6

*Framing*.....8

Materials.....9

Design Codes.....10

Design Loads.....11

*Dead*.....11

*Live*.....11

*Wind*.....12

*Earthquake*.....13

Conclusion.....13

Appendix A: Wind Tables.....15

Appendix B: Structural Plans.....16

Appendix C: Hand Calculations.....19

## Executive Summary

The following is the first technical report for the Hershey Research Park Building One. The report is an overview of the existing conditions of the site and the building. The drawings, specifications, and pictures have all been provided by Brinjac Engineering with permission given by Wexford Equities, LLC. The building was constructed by Whiting – Turner Construction and all the architectural design work was performed by Ayers/Saint/Gross, Inc.

Located outside Hershey, Pa HRPBO is a pretty standard office/research building. It is the first building of a planned twelve to be part of a research park. The building has over 80,000 square feet of available tenant space, with access to the facilities of the Penn State Milton S. Hershey Medical Center.

The engineers for this project used ASCE 7-02 along with IBC 2003 to determine the design loads, and both were used as a starting point when performing wind, earthquake analysis, and spot checks.

The wind analysis was completed using method two, which is the analytical method. The results revealed an overturning moment of approximately 12450 ft-kip in the east-west direction and overturning moment of approximately 4000 ft-kip in the north-south direction.

The earthquake analysis was performed using two different methods, first was the simplified method used by the buildings engineers which was through ASCE 7-02, and the second method was the equivalent lateral force method from ASCE 7-10. The first method resulted in a base shear of 433 kips and the second method resulted in a base shear of 325 kips.

From all the analysis performed all aspects of the building's structural system came out to be adequate. The spot checks confirmed the floor system deck, beams, and columns used to carry the gravity loads of the building.

## Building Introduction

The Hershey Research Park Building One (HRPBO) is a research facility located in Hershey, Pa., directly across the street from the Penn State Milton S. Hershey Medical Center. It was designed by Ayers/Saint/Gross Inc. with the engineering done by Brinjac Engineering and the construction by Whiting – Turner Constructuion. Building One is the first building to be finished of a twelve building research park known as the Hershey Center for Applied Research or HCAR for short.

Completed in Spring 2007, HRPBO is a state of the art research lab home to various medical and chemical research companies. They include Apeliotus Vision Science, Apogee Biotechnology, and vivoPharm along with some departments of Penn State Hershey’s College of Medicine. The building has 80,867 square feet of rentable space and cost approximately \$10.7 million dollars total to build. It was designed using the 2003 edition of the International Building Code and its supplements along with ASCE 7-02. Building One consists of a steel moment frame with brick, glass, curtain wall and metal panel façade.

The foundation is drilled steel piles system with concrete pile caps. The main superstructure is composite steel floor deck. Also some parts of the first floor and basement levels are just slab on grade. The roof system galvanized roof deck with insulation and water proofing placed on top of the beams. The Hershey Research Park Building One is design to withstand wind gusts up to 90 mph and is seismic use group II along with a seismic site class of “D”. The lateral resisting system is an ordinary steel moment frame which resists both the seismic and wind loads on the building. Even though Building One is not LEED certified there are still multiple forms of sustainability integrated into the building. Regional recycled steel was used in the building which reduces cost as well as waste by reuse. The roof system incorporates an efficient thermoplastic that helps reduce the energy used by the HVAC system, leading to overall reduced costs and emissions. Stones for the excavation of the site were reused for landscaping purposes. Also there is a storm management system integrated with green roof technology. The research center developers, Wexford Science and Technology, are planning on achieving a silver LEED certification on building two of the research park.



Figure 1: Site Master Plan

## Structural Overview

Hershey Research Park Building One sits on a combination of footings and piers. Due to problems with the soil, footings are not enough to support the building. Other than a small portion of the basement, the building is composite steel deck spanning between steel beams. The lateral system utilizes a flexible steel moment frame throughout the entire building.

## Foundation

Testing Service, Inc. preformed geotechnical testing of the soil before the construction of Building One. The test consisted of nine different borings located throughout the footprint of the building with depths ranging from 25 feet to 38 feet. The results of their tests found three types of layers: residual soil with few rock fragments, residual soil with significant rock fragments, and decomposed limestone. In addition, groundwater was observed in seven of the nine borings after drilling was completed.

TSI recommended certain types of foundations to be used for Building One based on the results of their tests. Their recommendation was to use a shallow spread footing to support the building. In the report TSI also found that the proposed area of Building One was prone to sink holes. Keeping this in mind the engineers decide to use piers with concrete caps. Using a deep foundation like this added more support just in case sinkholes began to develop.

## Floor System

The main superstructure is composite steel floor deck which is comprised of 4 ½ inch concrete slab on top of 3 inch deep 18 gage, galvanized composite steel floor deck reinforced with welded wire frame mesh. In addition, ¾ inch diameter, 6 inch steel studs are placed evenly across the beams. Also some parts of the first floor and basement levels are just 4 inch thick slab on grade. The concrete is 4000 psi with the reinforcement being grade 60 steel ( $F_y = 60\text{ksi}$ ). On the structural steel side of things, the wide flange steel is A992 steel. Figure 2 is a typical floor section showing the composite metal deck sitting on top of the steel beam.

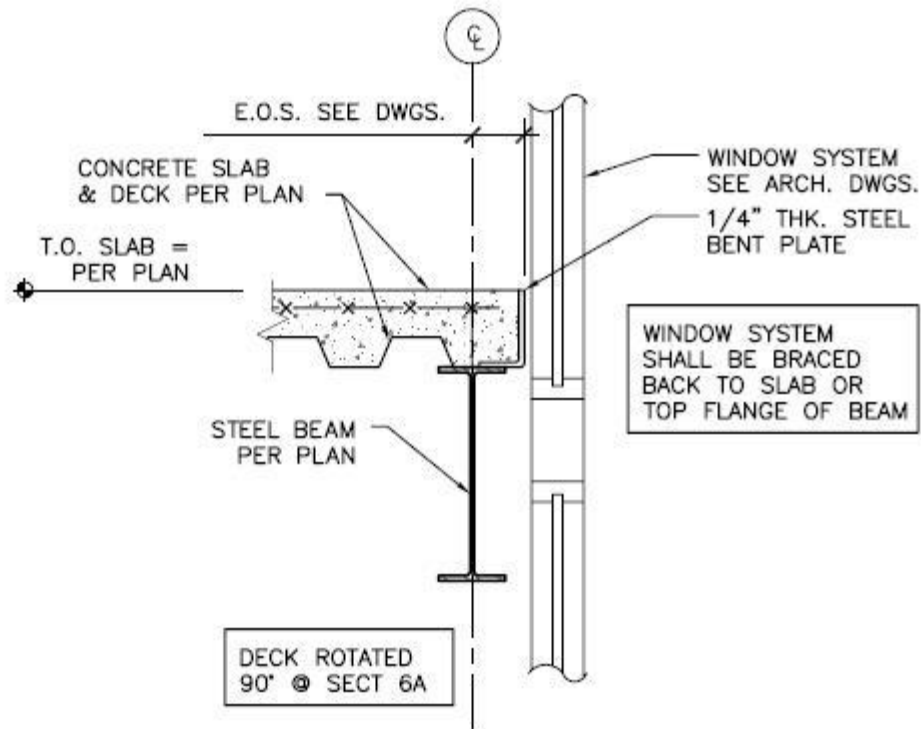
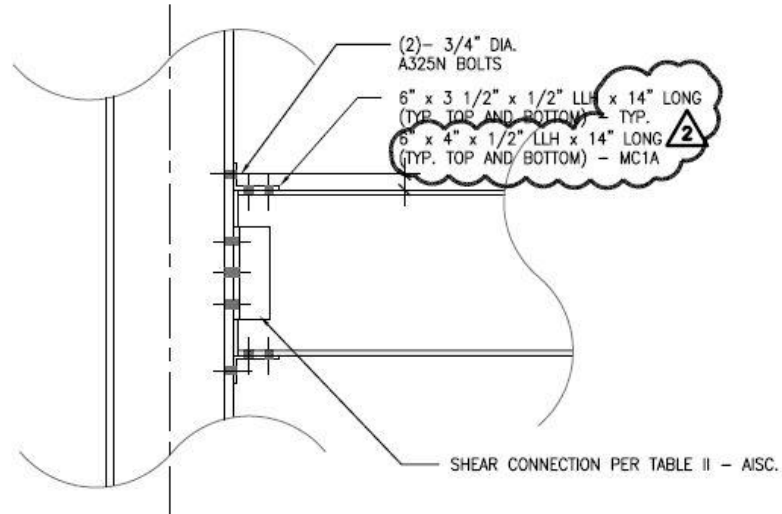


Figure 2: Typical Floor System

## Lateral System

The lateral force resisting system is an ordinary moment frame construction. This type of resisting system transfer the moments in the beams and girders to the columns which then transfer them to the foundation. Building One uses two different types of moment connections between the columns and beams. These two types are shown in figures three and four.



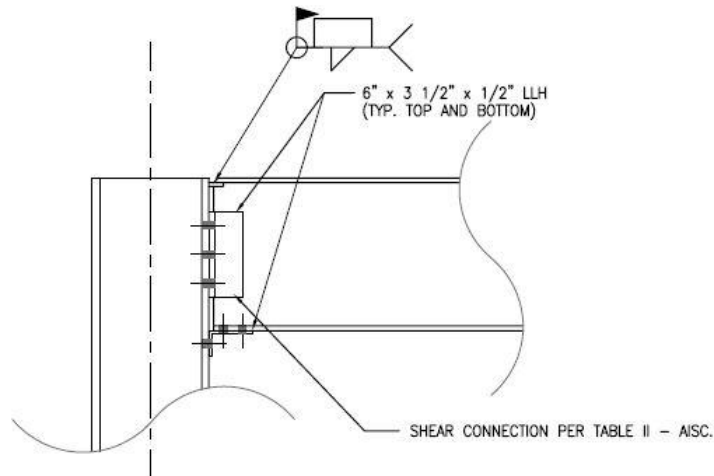


3  
S302

**TYPICAL MOMENT CONNECTION (MC-1) DETAIL**

SCALE: 3/4" = 1'-0"

Figure 3



2  
S302

**TYPICAL MOMENT CONNECTION (MC-2) DETAIL**

SCALE: 3/4" = 1'-0"

Figure 4

## Framing System

The framing system of Hershey Research Park Building One is a very basic one. It has a steel frame with composite metal deck on top. Beams frame into girders while the girders then frame into the columns which then transfer the forces to the foundation, the basic load path for any building. Figure five shows a basic floor framing plan whith a zoomed in view of a typical bay. The numbers within the brackets next to the beam sizes refers to the number of evenly spaces steel studs.

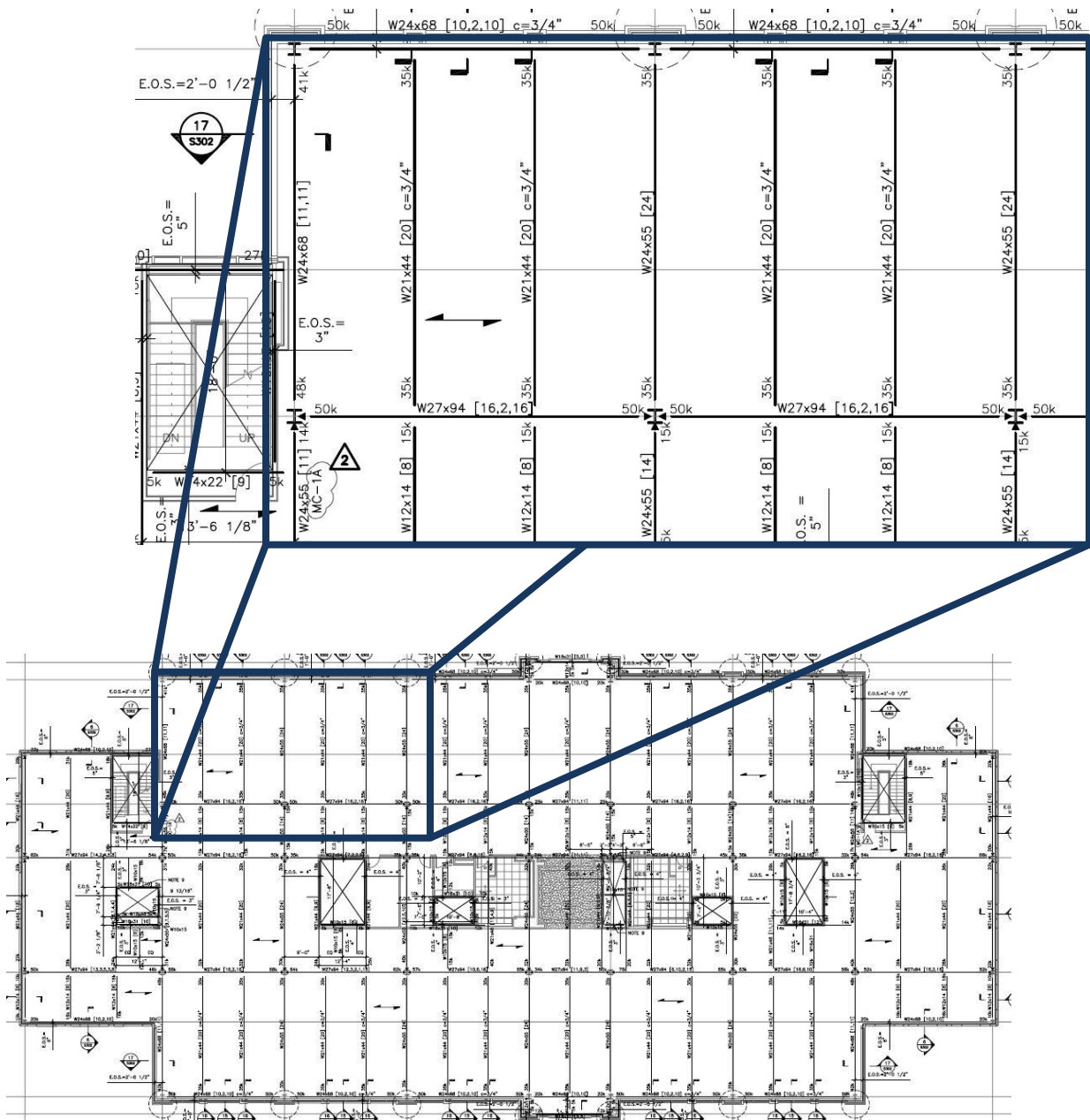


Figure 5



## Structural Materials Used

Here is a list of all the structural materials as noted in the general notes section of the structural specifications.

Structural Steel Properties	
Material Shape	ASTM Standard
Wide Flange	ASTM A992
Tubes	ASTM A500, Grade B
Pipes	ASTM A53
M/S/Channel	ASTM A572, Grade 50
Angles and Plates	ASTM A36
High Strength Bolts	ASTM A325
Reinforcing Steel	ASTM A615, Grade 60
Welded Wire Fabric	ASTM A185
Embedded and Misc.	ASTM A36

Table 1

Structural Concrete Properties	
Type	f' c (psi)
Caissons	3000
Slab on Grade	4000
Elevated Slabs	4000
Stairs	4000
Foundations	4000
Piers	4000
Walls	4000

Table 2 - Note: All exterior exposed concrete is air entrained.

Metal Deck Properties		
Deck Type	Gage	Depth
Roof	22	1 ½ in
Floors (Composite)	18	3 in

Table 3 - Note: Both types are galvanized steel deck.

## Design Codes and Standards

The Hershey Research Park Building One was designed to the following codes.

Design Codes	
Name	Description
IBC 2003	International Business Code – Minimum Design Loads
ASCE 7-02	American Society of Civil Engineers – Minimum Design Loads
ACI 318/301	American Concrete Institution – Reinforced Concrete Construction (318) / Structural Concrete for Buildings (301)
ASTM	American Society for Testing and Materials - Various standard use throughout the building
AISC	American Institute for Steel Construction – Specifications for Steel Buildings
NEC	National Electric Code – Specifications of Electrical Components
IMC 2003	International Mechanical Code – Specifications of HVAC Requirements

Table 4

## Design Loads

### Dead Loads

All the dead loads for the building were designed using IBC 2003 Section 1606. The superimposed dead loads are as shown in the table below.

Dead Loads	
<b>Slab on Grade</b>	50 psf
<b>Floor Framing</b>	85 psf
<b>Stair Framing</b>	85 psf
<b>Roof Framing</b>	15 psf

Table 5

### Live Loads

Live loads determined through IBC 2003 section 1607, which was the version that was used by the engineers on this project. Compared to the values in the IBC, the design live load numbers were more conservative.

Live Loads	
<b>Slab on Grade</b>	100 psf
<b>Lab</b>	100 psf
<b>Office</b>	100 psf
<b>Mechanical</b>	150 psf
<b>Roof Framing</b>	30 psf

Table 6

## Wind Loads

The wind analysis was performed once using ASCE 7-02, since that was used by the original engineers. The hand calculations for the wind design loads can be found in Appendix A. The Hershey Research Park Building One is located in the 90 mph wind velocity section of figure 6-1 of the code, and also the fundamental frequency for the building is greater than one. Since the fundamental frequency is greater than one, that means the building is rigid. Being rigid that leads to a gust factor of 0.85.

In plan view, the building geometry is not exactly a rectangle, but a simplifying assumption was made to change the geometry to a rectangle. A pressure distribution diagram is also present in the hand calculations to show how the wind load increases as the height increases. The figure below shows the distribution of forces throughout the different levels of the building.

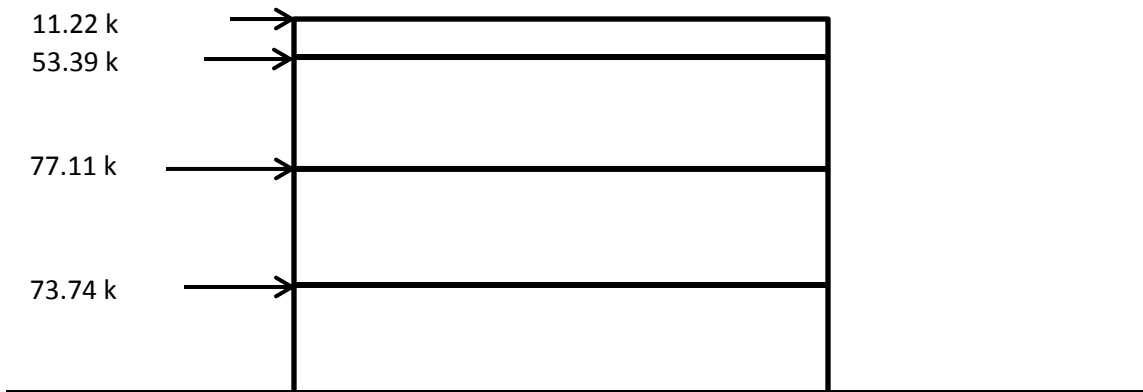


Figure 6 – Wind Force Distribution E-W

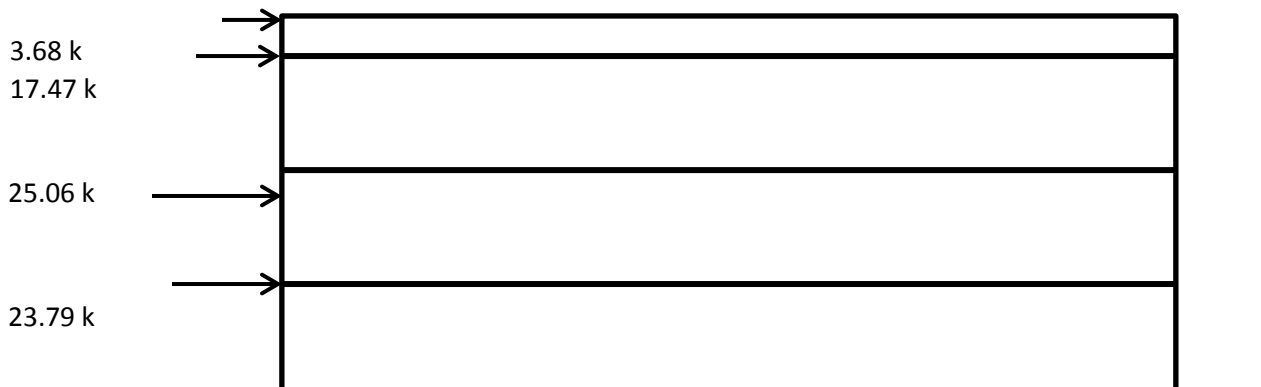


Figure 7 – Wind Force Distribution N-S

## Earthquake Loads

The lateral system of the Hershey Research Park Building One was designed using ASCE 7-02 using the simplified method. The equivalent lateral force method from ASCE 7-10 is the more common method used. Both ways of calculating the earthquake forces were analysis in the calculations. The geotechnical report by TSI, was used to help determine the site classification which came out to be “D”. The resulting base shear from using the simplified method gave an answer closer to that actual value compared to using the equivalent lateral frame method.

## Conclusion

Through the analysis of the Hershey Research Park Building One, the main structural systems of the building have become clearer. The various calculations including the wind, lateral, and spot checks have shown to all be adequate for dealing with all of the forces acting on the structural system of the building.

The composite deck floor system, along with the moment frame, was found to be able to carry the required loads. The beams and column sizes seemed to be overdesigned somewhat from the gravity spot checks. This most likely has to do with combination of gravity and lateral that together would add more stresses to the system. The combination of lateral and gravity loads together is a topic that must be explored further in the future technical reports.

The second and third technical reports will go into more detail about the lateral system of the building as well as alternative approaches to solving the structural demands for the buildings.

# Appendices



## Appendix A: Wind Load Tables

East-West Wind									
Floor	Elevation (ft)	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Trib. Area (ft <sup>2</sup> )	Force (kip)
1	409.25	0	0.7	12.3	14.2	10.96	-8.64	1881.30	36.87
2	423.916	14.6	0.7	12.3	14.2	10.96	-8.64	3762.60	73.74
3	438.58	29.3	0.7	12.3	14.2	10.96	-8.64	3934.6	77.11
Roof	454.6	45.3	0.78	13.8	14.2	11.98	-8.64	2589.7	53.39
High Roof Framing	458.6	49.3	0.81	14.2	14.2	12.28	-8.64	536.4	11.22
								Total	252.32
								Overturn Moment	12451.8

Table A-1

North-South Wind									
Floor	Elevation (ft)	z	kz	qz	qh	Windward (psf)	Leeward (psf)	Trib. Area (ft <sup>2</sup> )	Force (kip)
1	409.25	0	0.7	12.3	14.28	10.96	-6.21	697.8	11.98
2	423.916	14.66	0.7	12.3	14.28	10.96	-6.21	1385.6	23.79
3	438.58	29.33	0.7	12.3	14.28	10.96	-6.21	1459.4	25.06
Roof	454.6	45.35	0.785	13.8	14.28	11.98	-6.21	960.6	17.47
High Roof Framing	458.6	49.35	0.81	14.2	14.28	12.28	-6.21	199	3.68
								Total	81.98
								Overturn Moment	4045.77

Table A-2

## Appendix B: Structural Plans

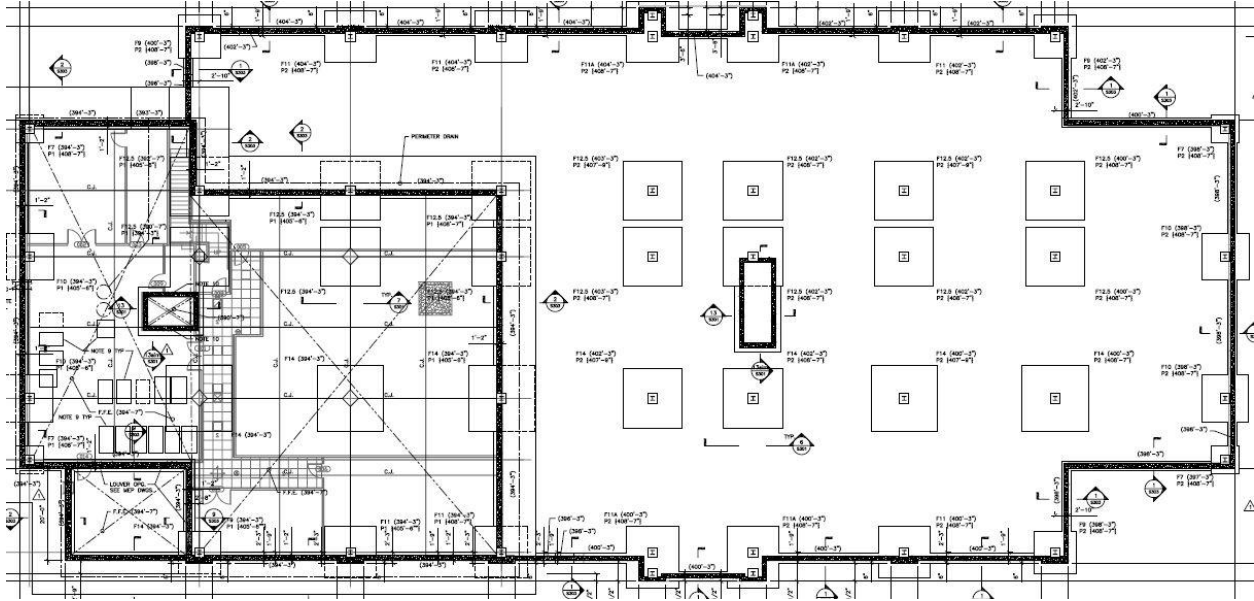


Figure 8 – Basement/Foundation Structural Plan

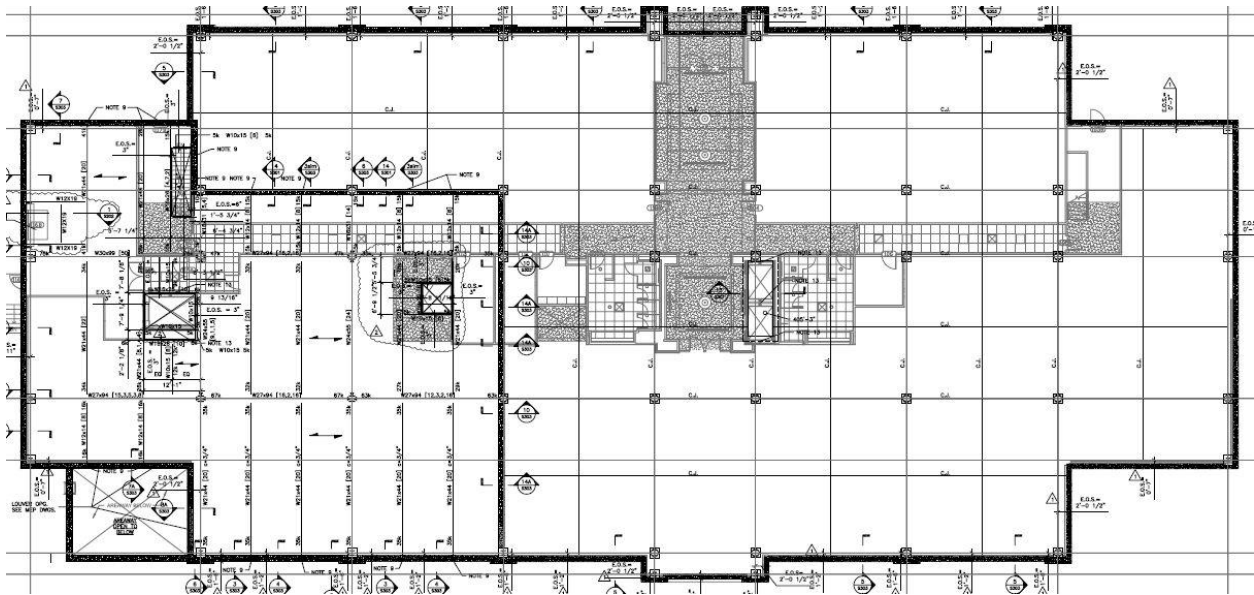


Figure 9 – First Floor Structural Plan

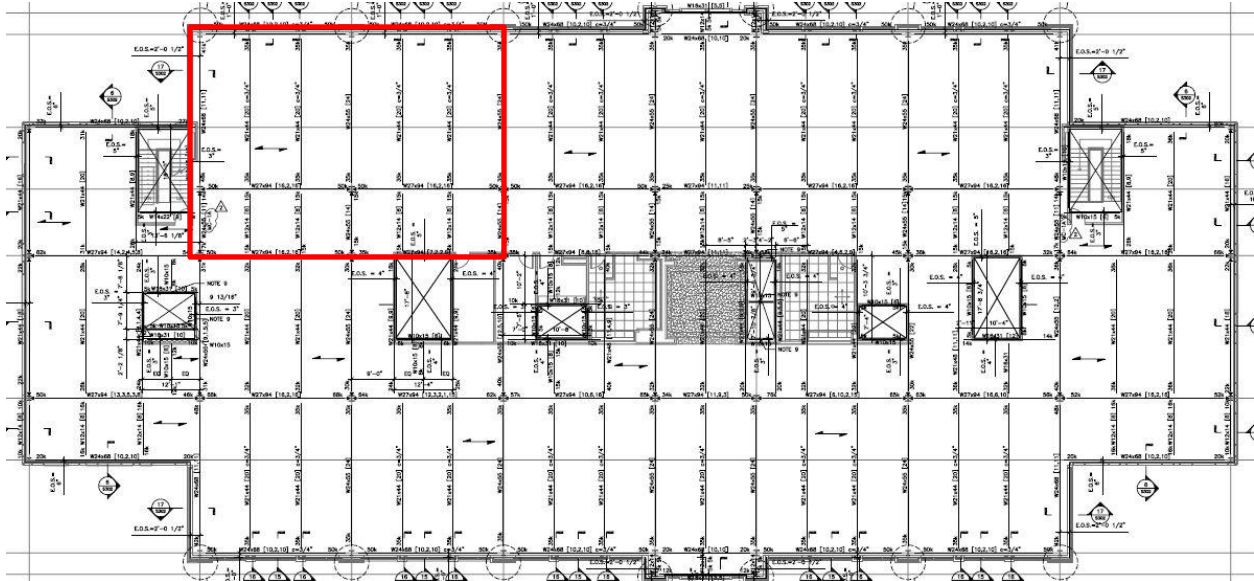


Figure 10 – Second Floor Structural Plan

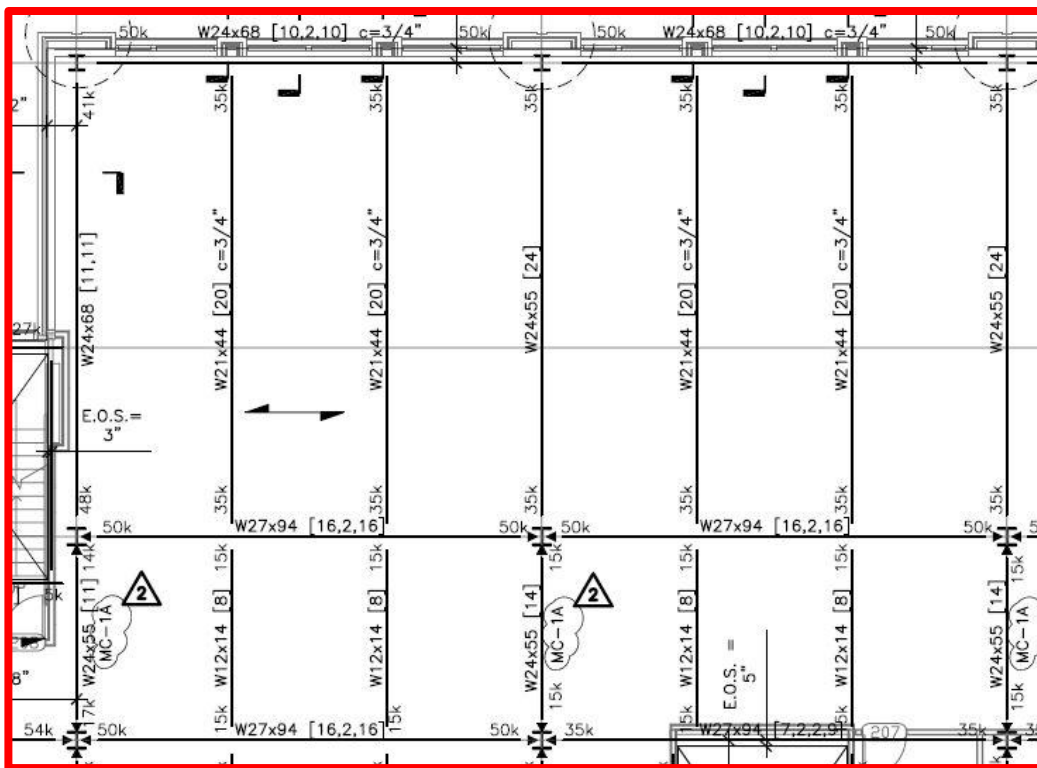


Figure 11 – Spot Check Area





## Appendix C: Hand Calculations

Jon Krepps AE Senior Thesis Wind Calcs - pg 1

Basic wind speed = 90 mph =  $V$  (Figur 6-1)

Occupancy Type II (Table 1-1)

Importance Factor  $I_w = 1.0$  (Table 6-1)

Wind Exposure Factor B

$$q_z = 0.00256 K_z K_{zt} K_d V^2 I$$

 $K_z$  = varies with height

$$K_{zt} = 1.0$$

$$K_d = 0.85$$

$$V = 90 \text{ mph}$$

$$I = 1.0$$

structure is Rigid so  $G = 0.85$ 

$$\hookrightarrow f > 1$$

Internal Pressure Coefficient

$$GC_{pi} = \pm 0.18$$



$\hookrightarrow$  Table 6-3  
Exposure B =  
Case 1

$$0-30 \text{ ft} = 0.7$$

$$45 \text{ ft} = 0.785$$

$$\rightarrow 50 \text{ ft} = 0.81$$

$$q_z = 0.00256 (K_z) (1.0) (0.85) (90)^2 (1.0)$$

$$q_z = 12.34 \text{ psf} \rightarrow 0-30 \text{ ft}$$

$$q_z = 13.84 \text{ psf} \rightarrow 45 \text{ ft}$$

$$q_z = 14.28 \text{ psf} \rightarrow 50 \text{ ft}$$





Lee ward Pressure East-West Wind - Pg 3

$$P = 14.28(0.85)(-0.5) - 14.28(\pm 0.18)$$

$$= -6.07 \pm 2.57$$

$$P = -8.64 \text{ psf} \rightarrow \text{constant throughout height}$$

Lee ward Pressure North-South

$$P = 14.28(0.85)(-0.3) - 14.28(\pm 0.18)$$

$$P = -6.21 \text{ psf} \rightarrow \text{constant}$$

Roof Pressure

$$C_p = -0.9 \quad 0 \text{ to } h$$

$$= -0.5 \quad h \text{ to } 2h$$

$$= -0.3 \quad > 2h$$

$$P = 14.28(0.85)(C_p) - 14.28(\pm 0.18)$$

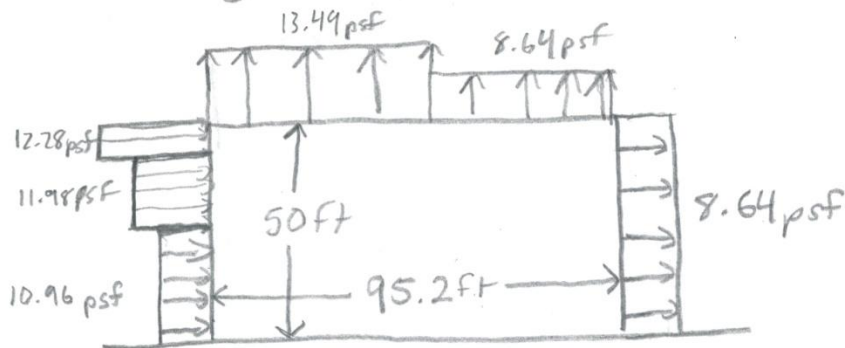
$$= -10.92 \pm 2.57 \rightarrow 0-50 \text{ ft}$$

$$= -6.07 \pm 2.57 \rightarrow 50-100 \text{ ft}$$

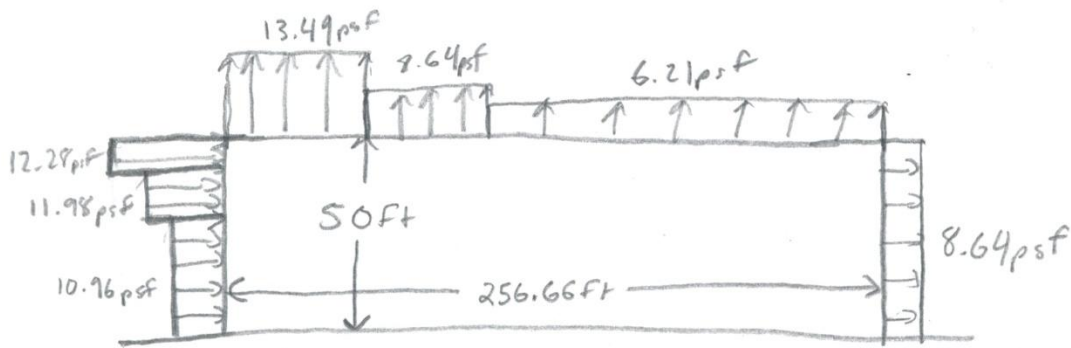
$$= -3.64 \pm 2.57 \rightarrow > 100 \text{ ft}$$

Wind - pg 4

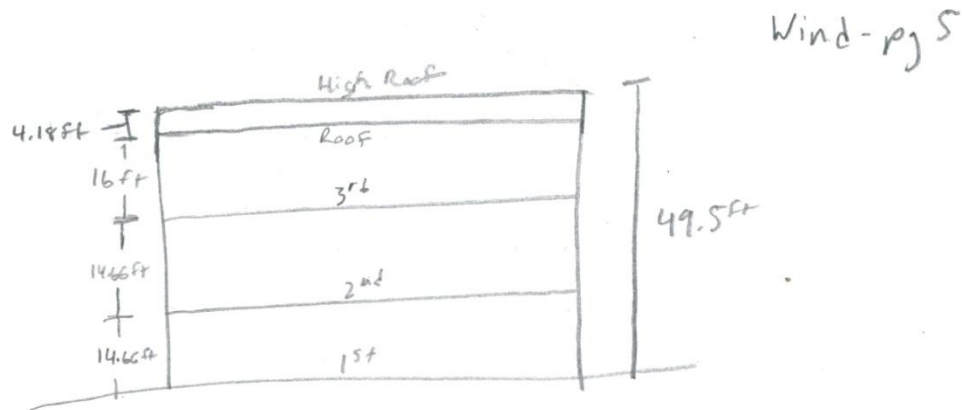
### Loading Diagrams



East-West



North-South



Trib. Area

	East - West	North - South
1 <sup>st</sup>	$\left(\frac{14.66}{2}\right)(256.66)$ $= 1881.3 \text{ ft}^2$	$\left(\frac{14.66}{2}\right)(95.2) = 697.8 \text{ ft}^2$
2 <sup>nd</sup>	$14.66(256.66)$ $= 3762.6 \text{ ft}^2$	$14.66(95.2) = 1395.6 \text{ ft}^2$
3 <sup>rd</sup>	$\left(\frac{14.66+16}{2}\right)(256.66)$ $= 3934.6 \text{ ft}^2$	$\left(\frac{14.66+16}{2}\right)(95.2)$ $= 1459.4 \text{ ft}^2$
Roof	$\left(\frac{16+4.18}{2}\right)(256.66)$ $= 2589.7 \text{ ft}^2$	$\left(\frac{16+4.18}{2}\right)(95.2)$ $= 960.6 \text{ ft}^2$
High Roof	$\left(\frac{4.18}{2}\right)(256.66)$ $= 536.4 \text{ ft}^2$	$\left(\frac{4.18}{2}\right)(95.2) = 199 \text{ ft}^2$

Jon Krepps AE Senior Thesis Seismic - pg 1

Seismic Site Class "D"

↳ Firm Soil

$$S_s = 0.23$$

$$S_1 = 0.07$$

$$F_a = 1.6 \Rightarrow \text{Site Class D, } S_s \leq 0.25$$

$$F_v = 2.4 \Rightarrow \text{Site Class D, } S_1 \leq 0.1$$

$$S_{ms} = F_a S_s$$

$$= 1.6(0.23) = 0.368$$

$$S_{m1} = F_v S_1$$

$$= 2.4(0.07) = 0.168$$

$$S_{DS} = \frac{2}{3} S_{ms}$$

$$= \left(\frac{2}{3}\right)(0.368) = 0.245$$

$$S_{D1} = \frac{2}{3} S_{m1}$$

$$= \left(\frac{2}{3}\right)(0.168) = 0.112$$

Seismic USE Group

$$\text{II} \rightarrow \text{I} = 1.25$$

Ordinary Steel Moment Frame

$$\text{↳ } R = 3.5$$

Seismic - pg 2

$$T_0 = 0.2 \left( \frac{S_{D1}}{S_{DS}} \right)$$

$$= 0.2 \left( \frac{0.112}{0.245} \right) = 0.091$$

$$T_s = \frac{S_{D1}}{S_{DS}} = \frac{0.112}{0.245} = 0.457$$

$$T_L = 6s$$

Simplified Analysis Method  $\Rightarrow$  Used by designers  
ASCE 7-02

$$V = \frac{1.2 S_{DS} W}{R}$$

Total Dead Load

$$W = DL + 20\% \text{ Roof Snow load}$$

$$\text{Roof DL} = 15 \text{ psf}$$

$$\text{Floor DL} = 85 \text{ psf} + 10 \text{ psf for partitions}$$

$$\text{Snow Load} = 0.2(30) = 6 \text{ psf}$$

Roof Load

$$(256.66)(95.2)(15+6) = 513 \text{ k}$$

Floor Load

$$(256.66)(95.2)(95) = 2,321 \text{ k}$$

Total DL

$$= 513 \text{ k} + (2,321 \text{ k})(2 \text{ Floors}) = 5,155 \text{ k}$$

Seismic pg 3

$$V = \frac{1.2 S_{Ds} W}{R}$$

$$V = \frac{1.2(0.245)}{3.5} (5155 \text{ k})$$

$$V = 433 \text{ k}$$

↳ Base Shear using ASCE 7-02

Equivalent Lateral Force Procedure  $\Rightarrow$  ASCE 7-10

$$V = C_s W$$

$$W = 5155 \text{ k} \rightarrow \text{From last method}$$

$$C_s = \frac{S_{Ds}}{(R/I_e)}$$

$$S_{Ds} = 0.245$$

$$R = 3.5 \rightarrow \text{Ordinary Steel Moment Resisting Frame}$$

$$I = 1.25$$

$$C_s = \frac{0.245}{3.5/1.25} = 0.0875$$

$$T = C_T h_n^x \quad C_T = 0.28$$

$$x = 0.8$$

$$T = 0.028(49.5)^{0.8} \quad h_n = 49.5 \text{ ft}$$

$$T = 0.635 < T_L = 0.6$$

$$C_s < \frac{S_{D1}}{(R/I_e)(T)} = \frac{0.112}{\left(\frac{3.5}{1.25}\right)(0.635)} = 0.063 < 0.0875$$

↳ use for  $C_s$

$$V = 0.063(5155 \text{ k}) = 324.7 \text{ k}$$



Jon Krepps AE Senior Thesis Snowload - pg 1

$$P_f = 0.7 C_e C_t I P_g \quad P_g = 30 \text{ psf}$$

$$P_f = 0.7(30)$$

$$C_e = 1.0$$

$$C_t = 1.0$$

$$I = 1.0$$

$$P_f = 21 \text{ psf}$$

Drift

$$r = 0.13 P_g + 14 = 0.13(30) + 14 = 17.9 < 30 \checkmark$$

Find drift height  $\rightarrow$  Roof Projection, use figure 7-9  
ASCE 7-02

$$P_g = 30 \text{ psf}$$

$$\text{Chart value } h_d = 1.7 \text{ ft}$$

$$l_u = 27 \text{ ft}$$

$$\text{From } \S 7.8 \quad h_d = 0.75 h_d \Rightarrow \text{From Figure 7-9}$$

$$h_d = 0.75(1.7) = 1.275 \text{ ft}$$

$$P_d = (1.275)(17.9) = 22.8 \text{ psf} \rightarrow \text{Roof snow load w/drift}$$

$$22.8 \text{ psf} > 21 \text{ psf} \checkmark$$

Jon Krepps AE Senior Thesis SPOT Checks - pg 1

Roof Deck

→ 1½" deep, 22 Gage, Type B

Typical SPAN

= 6.4ft between beams

Total Roof Load

15psf DL  
30psf LL  
22.8psf Snow } 67.8psf

From VULCRAFT Deck Catalog - pg 7

1.5 B, 22 Gage, 3 or more spans

Allowable load @ 6.5ft spans

= 74psf > 67.8psf ok

Max Construction span

= 6'-11" = 6.917' > 6.4' ok

## Floor Deck

Spot Checks - pg 2

→ 3" deep, 18 Gage, Composite Metal Deck

→ Slab thickness = 4 1/2" \*NWC

Total thickness = 7 1/2"

## Floor Loads

85 psf DL	}	185 psf
100 psf LL		

VULCRAFT Deck Catalog pg 54

3 VLI Deck

Typical Span Length = 10.66 ft = 10'-8"

Using 3VLI18 Total thickness = 7.5"

@ 11' Allowable Load = 210 psf > 185 psf ok

Max Unshored span

3 or more spans = 13'-3" &gt; 10'-8"

ok for unshored construction

## Typical Beam - B1

Spot Checks - pg 3

## Loads

85 PSF DL

100 PSF LL

$$f'_c = 4000 \text{ psi}$$

$$d = 20.7 \text{ in}$$

$$W21 \times 44 [20] \quad A_s = 13 \text{ in}^2 \quad \text{Total Thickness} = 7.5$$

$$b_{\text{eff}} = \begin{cases} \frac{32.5(12)}{8} = 48.75'' \checkmark \\ \min \left( \frac{10.67}{2}(12) = 64'' \right) \end{cases} \quad \text{Assume } a = 1$$

$$7.5 - Y_2 = 7 \Rightarrow Y_2 = 3.5$$

$$b_{\text{eff}} = 48.75'' (2) = 97.5''$$

$$w_u = [1.2(85) + 1.6(100)] 10.67 \text{ ft}$$

$$w_u = 2.8 \text{ k/ft}$$

$$M_u = \frac{w_u l^2}{8} = \frac{(2.8)(32.5)^2}{8} = 370 \text{ k-ft}$$

$$V = \frac{2.8(32.5)}{2}$$

$$Q_n = 21.5 \text{ k } 3/4 \phi \text{ studs}$$

$$V = 45.5 \text{ k}$$

$$\Sigma Q_n = 21.5 \text{ k } (10) = 215 \text{ k}$$

$$a = \frac{\Sigma Q_n}{0.85(f'_c)(b_{\text{eff}})} = \frac{215}{0.85(4)(97.5)} = 0.648 \quad a/2 = 0.324$$

$$7.5 - 0.324 = 7.176 \Rightarrow 7 = Y_2$$

$$x = \frac{A_s f_y - \sum Q_n}{2(b_f)(f_y)} = \frac{13(50) - 215}{2(6.5)(50)}$$

$$x = 0.67 > 0.45 \quad Y_1 \text{ in web}$$

Through Interpolation

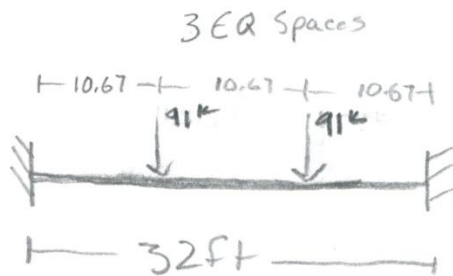
$$\phi M_n = 681 \text{ k}\cdot\text{ft} > 370 \text{ k}\cdot\text{ft} \quad \underline{\text{ok}}$$

Bigger than necessary

Spot Checks - pg 4

Typical Girder - G1

Spot Checks - Pg. 5



$$G1 = W27 \times 94$$

$$\frac{91k + 91k}{32ft} = 5.7k/ft \text{ equivalent uniform load}$$

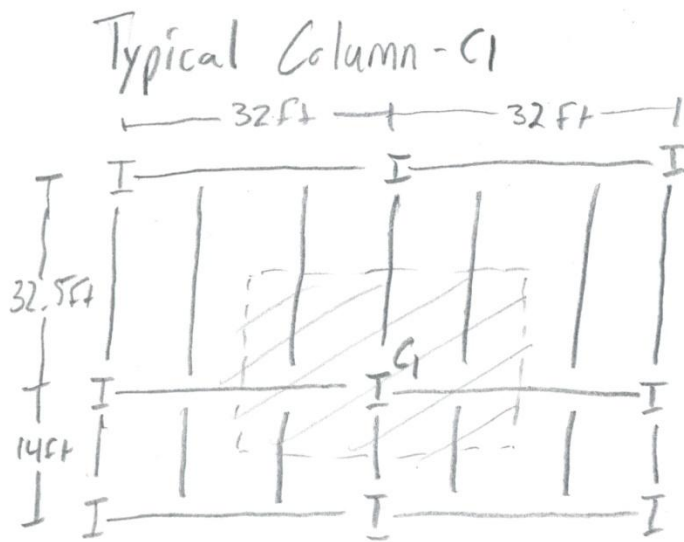
$$M = \frac{wl^2}{12} = \frac{5.7(32)^2}{12} = 486.4k \cdot ft = \text{Max } M.$$

$$\text{Unbraced length} = 10.67ft$$

$$\text{For } W27 \times 94 @ \text{UBL} = 10.67ft$$

$$\phi M_n = 948k \cdot ft > 486.4k \cdot ft \quad \underline{OK}$$

Bigger than necessary



Spot Checks - pg 6

Loads

$$DL = 85 \text{ psf}$$

$$LL = 100 \text{ psf}$$

$$A_e = 32 \times \left( \frac{14}{2} + \frac{32.5}{2} \right)$$

$$A_e = 744 \text{ ft}^2$$

No live Load reduction

$$1.2(85) + 1.6(100) = 262 \text{ psf}$$

$$P_u = (262 \text{ psf})(744) = 195 \text{ k}$$

Check

W14x74 used

$$r_y = 2.48 \text{ in} \quad r_x = 6.04 \text{ in}$$

$$L = 14.67 \text{ ft} \quad K = 0.5 \quad KL = 14.67(0.5) = 7.335$$

$$\frac{KL}{R} = \frac{7.335}{6.04} = 1.21 \quad \phi F_{cr} = 45 \text{ ksi Table (4-22)}$$

$$A = 21.8 \text{ in}^2 \quad \phi F_{cr} A = (45)(21.8) = 981 \text{ k}$$

$$P_u = 195 \text{ k} < 981 \text{ k} \quad \underline{\text{ok}}$$