Becker College Campus Center

Major Qualifying Project

BY

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Abstract

This MQP report presents a structural design for a two-story campus center for Becker Colleges. This project encompasses the Capstone Design requirements. Many aspects of steel design were explored in this project, including the calculation of lateral and horizontal loads, structural analysis, and AISC provisions. This MQP drew upon topics learned in the classroom and applied them to a real world situation; it has broadened my understanding of structural engineering.

Capstone Design Statement

The problem faced in this project is the design of a new campus center for Becker College. The main objective in this project was to design the structural members that will be needed for safe construction and to meet serviceability. Every aspect of the structure was designed in accordance with building codes. Architectural plans have been provided for a general idea of the building's layout. In addition to the design, a cost estimate was prepared in order to provide an overall cost of the building.

The first step in designing the building was to determine the lateral and vertical loads acting on the structure. Floor and roof beams were designed using AISC codes(AISC). Structural analysis software was used to determine the bending moments and axial forces acting on the structure. The girders and columns were then designed for the structural frames. A cost estimate was produced using unit cost data per ton of steel required and square footage estimates for the rest of the building.

Eight realistic constraints were incorporated. The constraints that were explored are economic, manufacturability, health and safety, social and political, and sustainability. The economic constraint was addressed by estimating the cost of the building and considering all cost saving options. Sustainability and manufacturability were explored by looking at the plans of the building and determining if there are common size members and considering the service life of the building. Health and safety constraints were investigated throughout the design of the building by adhering to all ASCE, AISC and local provisions for structural safety. Social and political constraints were captured by discussing how the new campus center will affected Becker College.

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Chapter 1

Introduction

In today's world, jobs can be hard to come by. A college degree offers the promise of more employment opportunities for young adults. With this strong emphasis on education more and more students are continuing their studies after high school, including many other adults who are enrolling in both full and part time college programs. Many students cannot afford the tuition of high-priced colleges and universities and look toward state and community colleges. The enrollment in state and community colleges has increased due to the emphasis on higher education. This higher enrollment has made these colleges expand and try to better student life.

Becker College, a private college in Worcester, MA that offers both two and four-year programs, has experienced enrollment growth as well. Their student body has grown, and they have added many new majors. One of the most noticeable changes is their recent creation of a degree in game design. Beckerôs game design program has gained credibility and has risen to the fourth-ranked school in the nation for game design according to the *Princeton Review*(Princeton Review). This is just one example of how Becker has been growing, and they are looking to upgrade their campus. In an effort to improve on-campus student life, Becker has decided to build a new campus and student center.

This project's focus will be on the structural design of the new campus center at Becker College. The design must be done with many constraints limiting the possibilities of the building. The foremost structural design issue is always safety. The building must be safe for all members of the school who will be using it. The next constraint is affordability and sustainability. These are very important to the members of Becker College because they must be able to afford the project, and they also want a building that will be a part of their campus for a long time.

This project first looks at the architectural design of the building. The design of the structural members is discussed through text in chapters 5 and 6, and is confirmed through calculations and tables in the appendix. A cost estimate of the building is described in chapter 7 and is illustrated in calculations in the tables. Chapter 8 provides a conclusion for the entire project.

Chapter 2

Background

This MQP, like most projects, required some research. The first thing that was required was to refer to ASCE 7 and determine the necessary requirements relating to design load values for snow and live loads. The live load was chosen as a small office building yielding a live load of 50 pounds per square foot. The snow load for Worcester, MA was found to be 40 pounds per square foot. Canadian codes were used for the lateral loads. Although Worcester is not in Canada, the Canadian codes are equivalent to the ASCE and local codes but are more straight forward to apply. Structural analysis is possible to do by hand, however the use of a structural analysis software is much more practical and realistic. Mastan 2 was used for the structural analysis of this building(Mastan 2). A cost estimate is necessary for every project and was conducted by using *Engineering News Record* for average cost per ton of many building materials(ENR).

Chapter 3

Methodology

This project covers topics from architectural layout to a final cost estimate. The layout was learned and a structural analysis of the building was conducted. This project report presents the design of the members in the gravity system of the structure and considers a cost estimate of the shell structure and an estimate of the entire building cost.

This project started with obtaining a copy of the architectural drawings. These are necessary to understand what the building is to entail, giving an idea of the size and scale of the building and the architect's expectations. The building was desired to be a two-story building with column spacing of 24 feet in the transverse direction and a spacing of 30 feet longitudinally. This column spacing was necessary to accommodate the need for open areas that are somewhat large.

The first thing to do in any project is to know the location of the site to be able to determine the necessary conditions to which the building will be exposed to. These conditions play a significant role in the design of the structure of the building, and these conditions include wind, earthquake and snow factors. With these conditions, a structural analysis of the building frame can be performed.

In the case of the Becker Campus center, there were two different moment frames shown on the plans that had to be subjected to these external conditions. Structural analysis can be performed in several different ways, and the use of structural analysis software is now most common. The next step was to design all of the members of the structure. Floor and roof beams were determined using load factors that are available in ASCE 7 (ASCE). Along with these loads, deadweight from concrete slabs and self weight of the beams were included in the analysis to select a sufficient beam size. The deflection of the beam was limited to 1/360 of its span length. Using these beams and the structural analysis, the forces acting on the members of the moment frames were determined. Once the columns and girders were selected a general cost of the building was determined. The cost was estimated by calculating the total weight of steel and concrete to be used and multiplying the weights by the unit cost of steel per ton and cost of concrete per ton.

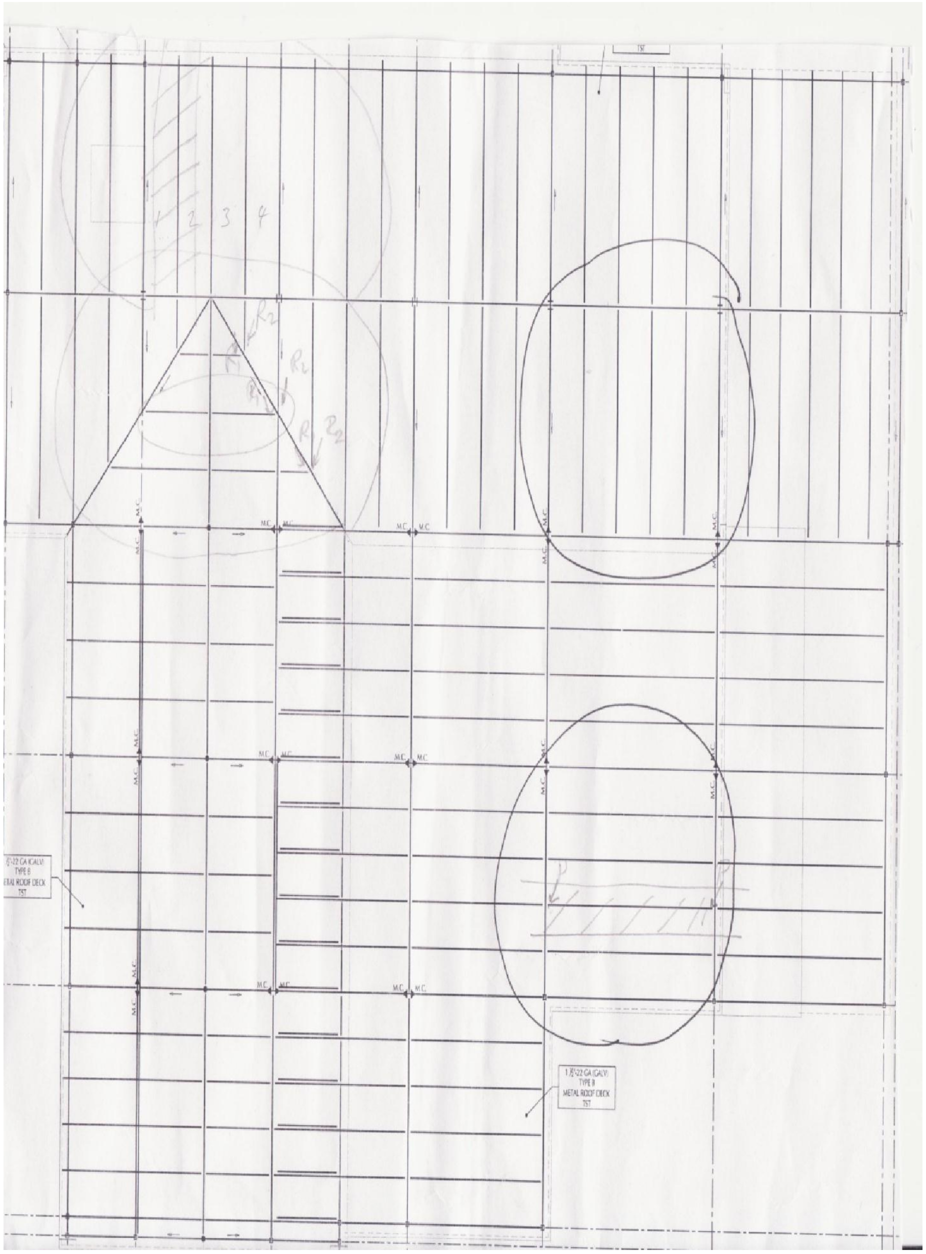
Chapter 4

Architectural Layout

The architectural drawing for the Becker College Campus Center were done and provided by Bolton & D'Martino Inc(BDI). The building is to be built in Worcester, MA. The building is two stories tall with each story 14 feet in height. The building is 144 feet long and 108 feet wide. The building has two different moment frames to stabilize the structure. One frame is 24 feet wide and 28 feet high. The girders in this frame have four filler beams on both the roof and floor. The second moment frame is 30 feet wide and 28 feet tall. The girders in this frame support four filler beams. A concrete slab, with a thickness of 4.5 inches, will be placed on both the floor and roof beams. A section of the roof is sloped and has three filler beams supported by 24' girders.

Architectural Plan

Framing Plan



Chapter 5

Determination of Structural Design Loads

The first major process in any engineering project is to determine the loads and forces acting on a structure. These loads can vary by region and must be determined specifically for each project. The obvious load that is first considered is the dead load. These loads are made up by the weight of the members of the structure as well as the weight of permanent fixtures in the building. These dead loads cause bending moments and shear forces throughout the structure. The live loads are loads attributed to movement of people or objects in the building. Suggested live loads are list in ASCE 7(ASCE). The forces due to moments and shearing forces were determined by using a structural analysis software. The remaining loads are all location specific loads which includes wind, snow and earthquake. This chapter discusses the steps taken to determine all of the loads acting on the Becker College Campus Center.

The dead loads was the first calculated load. To determine dead loads the architectural drawings were evaluated to determine beam and girder spans. First the tributary area or each member was determined by dividing the span of the girder by the number of spans between beams. In the campus center, there were two different girder spans one of 30ô and one of 24ô. With a girder span of 30ô and 4 filler beams the tributary area acting on each beam is 6ô and 4ô 10ò for the girder span of 24ô. To calculate the dead load, the tributary area was multiplied by the distributed weight of the concrete. The Campus center drawings call for 4.5ò of concrete slab on both the floor and roof, an assumed beam weight was added to determine the entire dead load. The required live load was determined by ASCE 7 which defines live loads for each different type of building(ASCE). For the campus center a live load of 70 PSF was used for the floor beams and a snow load of 40 PSF was used for the roof beams. Once both the dead and live loads were determined the bending moment for the beams was calculated.

The remaining loads acting on the structure are horizontal loads. The two horizontal loads are seismic and wind loads. The horizontal load that applies the most force to the structure is the governing load and provides the basis for the structural analysis of the building frames. In order to determine the governing load both the seismic and wind loads were calculated. For this project Canadian Codes were used to determine both the seismic and wind loads(Canadian Code). To determine the wind loads the fastest wind speed must be determined. For this project a value of 90 MPH was used(Canadian Code). This value is divided by a reduction factor of 1.25. The resulting value is known as the mean wind speed. The mean wind speed is then used to determine the mean wind pressure. The mean wind pressure is determined by squaring the mean wind speed and multiplying it by a constant of .0027. The mean wind pressure is then multiplied by several factors to obtain the wind pressure on the building. The three factors are gust, exposure, and pressure. All three factors are determined by referencing the Canadian Codes.

The wind pressure on a building is assumed to act at each level of the building. The pressure is multiplied by the tributary area for each level of the building. These calculations must be done for each side of the building. The roof level will have a tributary height of 7 feet on either side and a depth of either 48 or 60 feet, depending on whether the transverse or longitudinal perspective is considered. For the Campus Center the transverse of the building was subject to 12.23 kips at roof level and a force of 24.46 kips at the second floor level. The total force on this side of the building was 36.69 kips. The force on the longitudinal side of the building were larger than the transverse side. These forces were 15.29 kips at the roof level and

30.58 kips at the second floor level. The total force due to wind on the longitudinal side of the building was 45.87 kips.

The final considered lateral load acting on the building was the earthquake or seismic load. This load, like the wind load, was calculated by using the Canadian code(Canadian Code). The equation for the earthquake force is V=ASKIFW. A is the acceleration rate which is determined through a table in the Canadian code and the value used for this building is .1g. S is the seismic coefficient which is equal to .05 divided by the square root of the period. The period is equal to .05 times the height of the building divided by the square root of the depth of the building. The seismic coefficient and period must be determined for each side of the building. K is a constant that is determined by referencing the Canadian code table. I is an importance factor and is also determined by referencing a table in the Canadian code. F is the foundation factor is also determined by a table in the Canadian code. W is the weight of the building. The weight has to be estimated since the member sizes are not known at this point in a design.

Seismic forces like wind forces were applied to each level of the building. To determine the force at each level, the height and estimated weight of each level was defined. The height and weight were then multiplied and divided by the sum of the height times weight. This fraction shows the percent of the total weight that each level contributes. These percents were multiplied by the total seismic force determining the seismic force at each level. These calculations were performed for each side of the building. Seismic and wind loads are both horizontal loads and can act at the same time, however the loads with the highest force on the building were used in the calculations, not the combination of the loads. All of the loads that were determined were used in the structural analysis to determine the required member sizes.

Chapter 6

Structural Design and Analysis

The purpose of this chapter is to discuss how each member of the building was selected. The first members that were determined were the floor and roof beams. The floor beams were subject to both dead and live loads. The dead load acting on each beam is its self weight and the weight of the concrete slab on top of it. The dead load due to concrete was calculated by multiplying the thickness of the concrete by the tributary area acting on the beam and the weight of concrete. An assumed beam weight was added to the weight due to the concrete. For an office building a live load of 50 psf was used along with a load of 20 psf for partitions. The dead load was multiplied by the length of the beam squared and divided by 8 to determine the moment of the beam due to dead load. The same was done for the live load which was then added to the moment due to the dead load to obtain the total moment acting on the beam. This process was done for each frame in the building. Once the total moment was obtain, the maximum moments were compared to the maximum moments listed in the AISC manual to determine the smallest allowable beam size(AISC). A similar process was used in the design of the roof beams for a section of the building. The only change with the roof beams was the live load. The live load acting on the roof was 40 psf due to snow.

The next step in the design process was the girder and column design. An efficient way to determine the moments acting throughout the structure is to use a structural analysis software. In order to use this software, the weight of each floor was determined. The sheer force on each beam was used to determine the force acting on the girders. The equation to determine the total moment on the girder is 2P(L/2)-P(L/5)-P(L/10)= (wL^2)/8. Once the total moments were calculated, the equation was rearranged to solve for the weight. This process was done for each frame on both the roof and floor levels. A structural analysis software, Mastan 2, was used to determine the moments on the structure(Mastan 2). The first step was to define the frame of the structure. The properties of steel were then added to the frame. The dead weight, live loads, and seismic loads were then added to the frame. A first order elastic analysis was then performed on the structure. The resulting moment and shear forces were displayed for each member of the frame.

The moments acting on the floor are roof girders were displayed in inch kips and had to be converted to foot kips in order to compare the maximum moment with the beams listed in the AISC manual(AISC). Once the maximum moment was converted, the required beam size was found by comparing the moment on the girder to the maximum allowed moments in the AISC manual(AISC). Once a girder was selected, the deflection was check to ensure that the girder would not sag. For the section of sloped roof, a different approach was required to determine the moment acting on those girders. The shear forces for each filler beam connect to the girder were calculated. The sum of the moments about a point on the girder gave the moment on the girder due to the filler beams. An assumed girder weight was used to calculate the moment due to the girders self weight. Both of these moments added together gave the total moment acting on the sloped girders. This value was compared to the given values in the AISC manual and a proper girder size was determined(AISC). The deflection also had to be checked to ensure the girder did not bend excessively.

Girder Loading	
Transverse	**Bending (ft*kips)**
Left	393.25
Right	369.67
Longitudinal	**Bending (ft*kips)**
Left	383.92
Right	365.58

The final stage of design was to determine the column sizes for the building. The forces acting on each column were determined using Mastan 2 as well(Mastan 2). The forces acting on the building caused both axial and bending moments on the columns. The column, for each frame of the building, with the largest forces acting on them were designed. The combined axial and bending moments acting on the column was determined using the information from Mastan 2 and the combined axial and bending moment equation(Mastan 2). The combined axial and bending moments were compared to the values in the AISC manual and a column size was chosen(AISC).

	Column Loading	
Transverse	**Axial (kips)**	**Bending (ft*kips)**
Left	117.27	236.80
Center	127.30	285.30
Right	88.82	264.00
Longitudinal	**Axial (Kips)**	**Bending (ft*kips)**
Left	102.30	235.83
Center	116.60	289.25
Right	92.60	268.90

Building Member Requirements

Member Sizes	Length	Amount
Floor Beams		
W14 X 43	24'	42
W1 14X74	30'	42
Floor Girder		
W 27X194	30'	30
W 27x161	24'	31
Roof Beams		
W 14X43	24'	42
W14X68	30'	42
Roof Girders		
W 27x129	30'	30
W 27x94	24'	31
Sloped Roof Girders		
W 14X193	24'	8
Columns		
W 14X90	14'	80

Chapter 7

Cost Estimate

The main material that composes the structure was structural steel. The cost of steel has been rising over the past few years according to *Engineering News Record*(ENR). In order to calculate the shell cost estimate of the building, the total weight of steel and concrete were calculated. To calculate the total weight of steel in the building, the total number of beams, girders, and columns were totaled. The weight per linear foot of each member was multiplied by the length of each respective member and multiplied by the number of each member. Totaling up all of the weights of each individual member yielded the total weight of the building. The total weight was changed into tons to calculate the cost of steel. The total weight in tons was multiplied by the cost per ton to determine the cost of the steel structure. The current cost of steel per ton is $902/ton. The total weight of the steel structure was 442 tons. The total cost of steel in the structure was $398,631.

Weight and Cost of Steel Structure

	Member PLF	Length	Number of Members	Total Weight of Members	
Floor Beams					
	43	24	42	43344	
	74	30	42	93240	
Floor Girder				0	
	194	30	30	174600	
	161	24	31	119784	
Roof Beams				0	
	43	24	42	43344	
	68	30	42	85680	
Roof Girders				0	
	129	30	30	116100	
	94	24	31	69936	
Sloped Roof Girders				0	
	193	24	8	37056	
Columns				0	
	90	14	80	100800	
				Total Weight of Structure	
		$/ton		883884	pounds
			902	441.942	tons
				Cost of Steel	
				$398,631.68	

The cost of the structure due to concrete was computed in a similar way to steel. First the total amount of concrete used in the structure was determined. The total number of cubic feet of concrete was then multiplied by the weight of concrete per cubic yard of 150 lbs/cf. The total weight was then converted into tons and multiplied by the cost of concrete per ton of $105.24/ ton. The total weight of concrete in tons was 1134 tons and the total cost of concrete was $119342. Adding the total cost of steel and the total cost of concrete, a total cost of the building structure was determined. The total cost of the structure was $517,973.84

Cost of Concrete

Area	Thickness	Cubic Feet
20160	0.375	7560
20160	0.375	7560
	Total	15120

Total Weight

Cubic Feet	lbs/cf	Weight	
15120	150	2268000	lbs
		1134	Tons

Cost of Concrete

Weight	$/ton	Cost
1134	$105.24	$119,342.16

Chapter 8

Conclusion

This project highlights and defines all necessary steps required to design a structure to be used as a campus center for Becker College. The explanation of each step in the process has been defined to let the reader understand how the results were generated. The Tables and chapters listed in this report define the required member sizes for each aspect of the building. The estimated cost of the building structure based on material costs for structural steel and concrete was $518,000. This project was intended to design a building to be used as a campus center, and it has displayed the required members and cost needed to build a new campus center.

References

AISC *Steel Construction Manual* June 2008

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The Princeton Review Received on 10/23/10 from www.princetonreview.com

Appendices

Floor Beams

Lateral Loads

Roof Beams

Structural Analysis Outputs

Column Design

Girder Design

Floor Beams

$w_{LL} = 50$ psf

$w_{LL} = 20$ psf (Partition)

$w_d = w_0 + w_{slab}$

$w_{slab} = \frac{4.5}{12} \times \frac{150}{1000} \times 6 = .3375$

$w_0 = \frac{80}{1000} = .08$ kip

$w_d = .08 + .3375 = .4175$ kip (1.2)

$M_{DL} = \frac{.6 \cancel{}5(24)^2}{8} = 30.06$ ft.k

$w_L = \frac{70}{1000} \times 6 = .42 (1.6)$

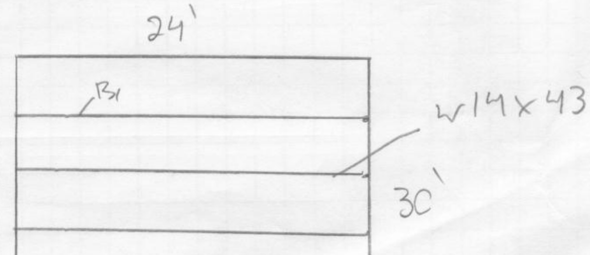
$M_{LL} = \frac{.42(24)^2}{8} = 30.24$

$M_{D+L} = 30.24 + 30.06 = 60.3$ ft.kips

Try 14x48

$\Delta_{LL} = \frac{5(\frac{.42}{12})(12 \times 24)^4}{384(29 \times 10^6)(484)} = .0017$ in $< \frac{288}{360}$ ✓

$\Delta_{DL} = \frac{5(\frac{.4175}{12})(12 \times 24)^4}{384(29 \times 10^6)(484)} = 2.22 \times 10^{-4}$ in $< \frac{288}{360}$ ✓



Floor Beams

$w_{LL} = 50$ psf

$w_{LL} = 20$ psf (Partitions

$w_d =$ self weight + w_{slab}

$w_{slab} = \frac{4.5}{12} \times \frac{150}{1000} = .05625\ K/ft^2 \times A_T$

$.05625 \times \frac{24}{5} = .27\ K/ft$

$w_{LL} = 50 + 20 = \frac{70}{1000} \times \frac{24}{5} = .336\ K/ft\ (1.6)$

$M_{LL} = \frac{.537 \times 30^2}{8} = 37.8$ K ft

$w_b = 80$ plf

$w_d = \frac{80}{1000} + .27 = .35$ klf (1.2)

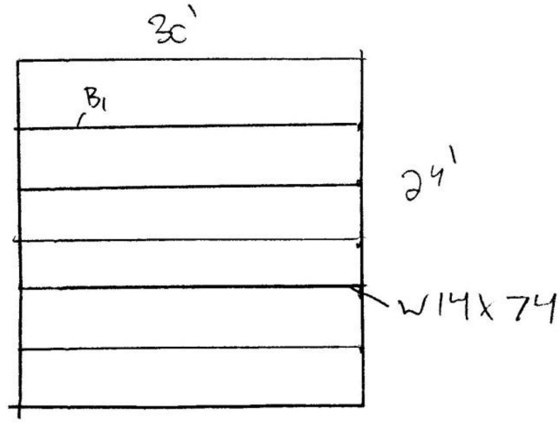
$M_d = \frac{.422(30)^2}{8} = 39.4$ ft · kips

$M_{d+l} = 39.4 + 37.8 = 77.2$ K·ft

TA, W14X74 $M = 83.8$

$\Delta_L = \frac{5}{384}\frac{wL^4}{EI} = \frac{5\ \frac{.336}{12}(12\times30)}{384(29\times10^6)(794)} = 2.66\times10^{-4}$ in $< \frac{12(30)}{360}$ ✓

$\Delta_{DL} = \frac{5\left(\frac{.35}{12}\right)(12\times30)^4}{384(29\times10^6)(794)} = 2.77\times10^{-4}$ in $< \frac{12\times30}{360}$ ✓



Wind loads

$\overline{V_{30}} = \frac{90}{1.25} = 72$ mph

$q_{30} = .00257(72)^2 = 13.997 = 14$ psf

$P = C_e C_g C_p q$

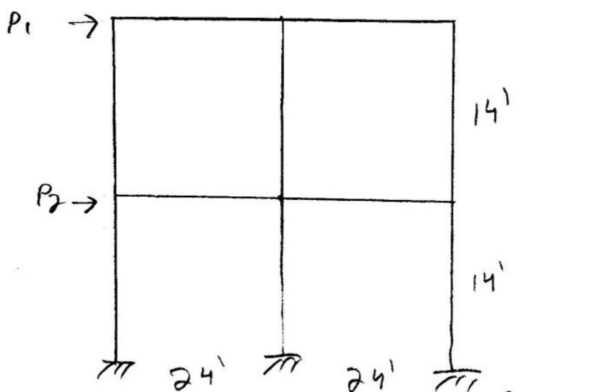
$C_e = 1$

$C_g = 2$

$C_p = .9 + .5 = 1.3$

$P = (1)(2)(1.3)(14) = 36.4$

Short side



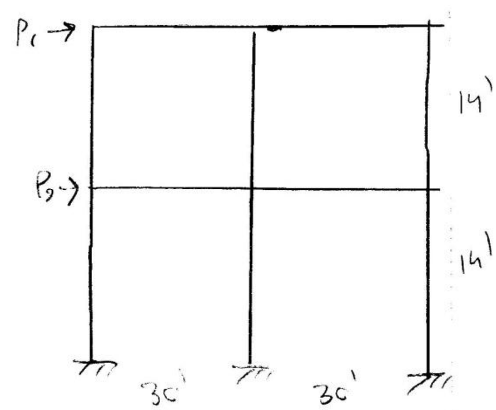
	P	A	P/1000
1	36.4	48(7)	12.23 k
2	36.4	48(14)	24.46 k

$\Sigma P = 12.23 + 24.46 = 36.69$ k

Long side

1	36.4	60(7)	15.29 k
2	36.4	60(14)	30.58 k

$\Sigma P = 15.29 + 30.58 = 45.87$ k



$V = ASKIFW$

$A = .1g$

$S = .5/\sqrt{T}$

$K = 1.3$

$I = 1$

$F = 1$

$W = ?$

$$T = \frac{.05H}{\sqrt{D}} = \frac{.05(28)}{\sqrt{60}} = .1807$$

$$S = \frac{.5}{\sqrt{.1807}} = 1.18$$

$$V = (.1)(1.18)(1.3)(1)(1)W = .1529\,W$$

$W_R = 316.8$

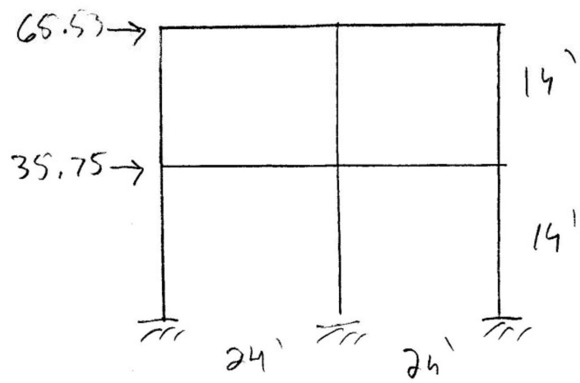
$W_F = 345.6$

$W = 662.4\ k$

$V = .1529(662.4) = 101.29\ k$

$F_x = (V - F_t)\frac{w_i h_i}{\Sigma w_i h_i}$ $\quad \frac{H}{D} < 3 \rightarrow \frac{28}{60} < 3 \rightarrow F_t = 0$

	w_i	h_i	$w_i h_i$	$\frac{w_i h_i}{\Sigma w_i h_i}$	F_x
1	316.8	28	8870.4	.647	65.53 k
2	345.6	14	4838.4	.353	35.75 k
			13708.8		



Earthquake loads

$V = ASKIFW$

$A = .1g$

$S = .5\sqrt{T}$

$K = 1.3$

$I = 1$

$F = 1$

$W = ?$

$$T = \frac{.05(28)}{\sqrt{48}} = .202$$

$$S = \frac{.5}{\sqrt{.202}} = 1.11$$

$V = .1(1.11)(1.3)(1)(1)\,W = .1446 \quad V = 14.46\%$ of W

$W_R = \frac{110}{1000}(60 \times 48) = 316.8^k$

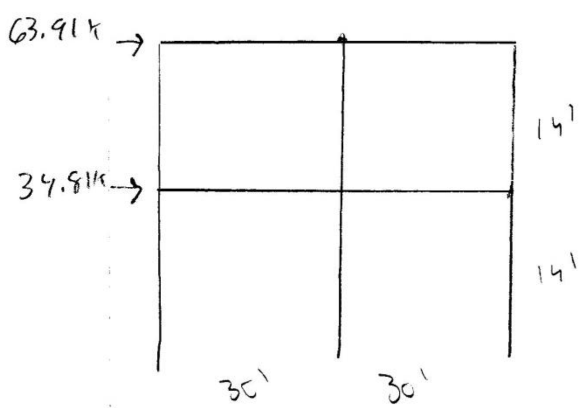
$W_A = \frac{120}{1000}(60 \times 48) = 345.6$ K

$W = W_R + W_A = 316.8 + 345.6 = 662.4$

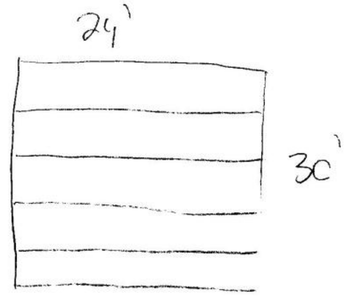
$V = .1446(662.4) = 95.78 >$ wind

$F_x = (V - F_t)\frac{w_i h_i}{\Sigma w_i h_i} \quad F_t = 0 \quad \frac{H}{D} < 3 \quad \frac{28}{45} < 3 \checkmark \rightarrow F_t = 0$

Level	w_i	h_i	$w_i h_i$	$\frac{w_i h_i}{\Sigma w_i h_i}$	Fx
1	316.8	28	8870.4	.647	63.91
2	345.6	14	4838.4	.353	34.87
			13708.8		



Roof Beam



W_{LL} = 40 PSF
W_{LL} = 20 PSF (partitions)

WDL: Self weight Slab

W_{slab} $\frac{4.5}{12} \times \frac{150}{1000} \times 6 = .3375$ kip/ft

W_{LL} = 40+20 = 60 PSF × 6 = .36 kip (1.6)

$M_{LL} = \frac{.576 (24)^2}{8} = 25.92$ ft·kips

W_D = .08 + .3375 = .4175 (1.2)

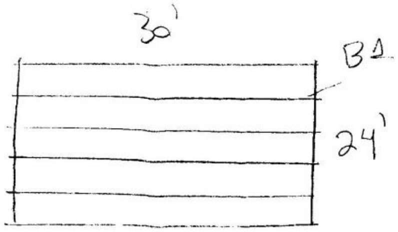
$M_{DL} = \frac{.6205 (24)^2}{8} = 30.06$

$M_D + M_L$ = 55.98 ft·kips < 57.9 ft·kip W14x43 24'

Try W14 x 43

$\Delta_L = \frac{5 W_L L^4}{384 EI} = \frac{5(\frac{.36}{12})(12 \times 24)^4}{384(29 \times 10^6)(428)}$

Roof Beams



$w_{LL} = 40$ psf

$w_{LL} = 20$ PSF (Partitions)

w_d: Self weight + w_{slab}

$w_{slab} = \frac{4.5}{12} \times \frac{150}{1000} = .05625$ k/ft^2 × Tributary Area

$.05625 \times \frac{24}{5} = .27$ kip/ft

$w_{LL} = 40 + 20 = \frac{60\ psf}{1000} \times \frac{24}{5} = .288\,(1.6)$

$M_{LL} = \frac{.4608 \times 30^2}{8} = 32.4$ ft-kips

$w_c = 80$ plf

$w_d = \frac{80}{1000} + .27 = .35$ klf (1.2)

$M_d = \frac{.42\,(30)^2}{8} = 39.4$ ft-kips

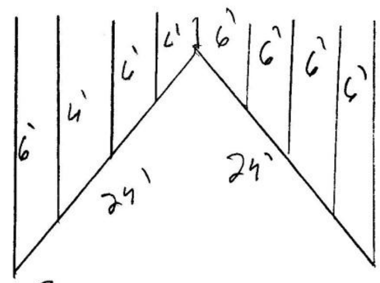
W14x68 30'

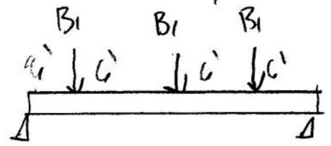
$M_{ult} = 71.8$ ft-kips < 76.5 ft-kip

Try W14x68

$\Delta_{LL} = \frac{5}{384}\frac{wL^4}{EI} = \frac{5(\frac{.288}{12})(12\times30)}{384(29\times10^6)(722)} = 2.50\times10^{-?}$ in ✓ $< \frac{12\times30}{360}$

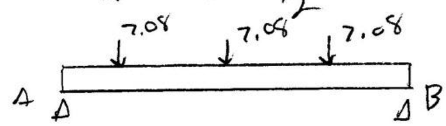
$\Delta_D = \frac{5wL^4}{384EI} = \frac{5(\frac{.35}{12})(12\times30)^4}{384(29\times10^6)(722)} = 3.0?\times10^{-?}$ in $< \frac{360}{360} = 1"$ ✓





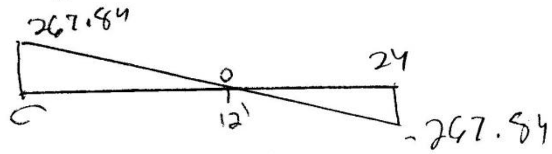
$M_{B1} = \frac{.59 \times 24^2}{8} = 42.48$

$V_{B1} = \frac{.59 \times 24}{2} = 7.08\ K$



$M_A = 7.08(6) + 7.08(12) + 7.08(18) = 267.84$

$M_B = 7.08(6) + 7.08(12) + 7.08(18) = 267.84$



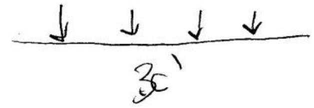
$M = 267.84k +$ Self weight

$w_a = 200\ lb/ft \quad w_a = \frac{.2 \times 24^2}{8} = 14.4\ k$

$M = 267.84 + 14.4 = 282.24\ k$

Use **W 14 x 143** $M = 295 > 282.24$ ✓

w_{roof}

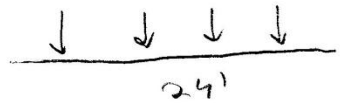


$P = 8.88$

$M_C = 2P\frac{L}{2} - P\frac{L}{5} - P\frac{L}{10}$

$= 2(8.88)(15) - 8.88(6) - 8.88(3) = 186.48$

$M_C = \frac{wL^2}{8} \quad w = \frac{8M_C}{L^2} = \frac{8(186.48)}{30^2} = 1.66$ ft·kips



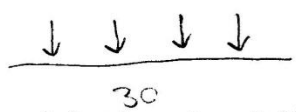
$P = 8.16$

$M_C = 2(8.16)(12) - 8.16\left(\frac{24}{5}\right) - 8.16\left(\frac{24}{10}\right) = 137.69$

$M_C = \frac{wL^2}{8} \quad w = \frac{8(137.09)}{24^2} = 1.904$ ft·kips

w_{floor}

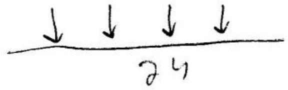
$P = 10.68$



$M_C = 2(10.68)(15) - 10.68(6) - 10.68(3) = 211.68$

$w = \frac{211.68(8)}{30^2} = 1.88$ ft·kips

$P = 9.36$

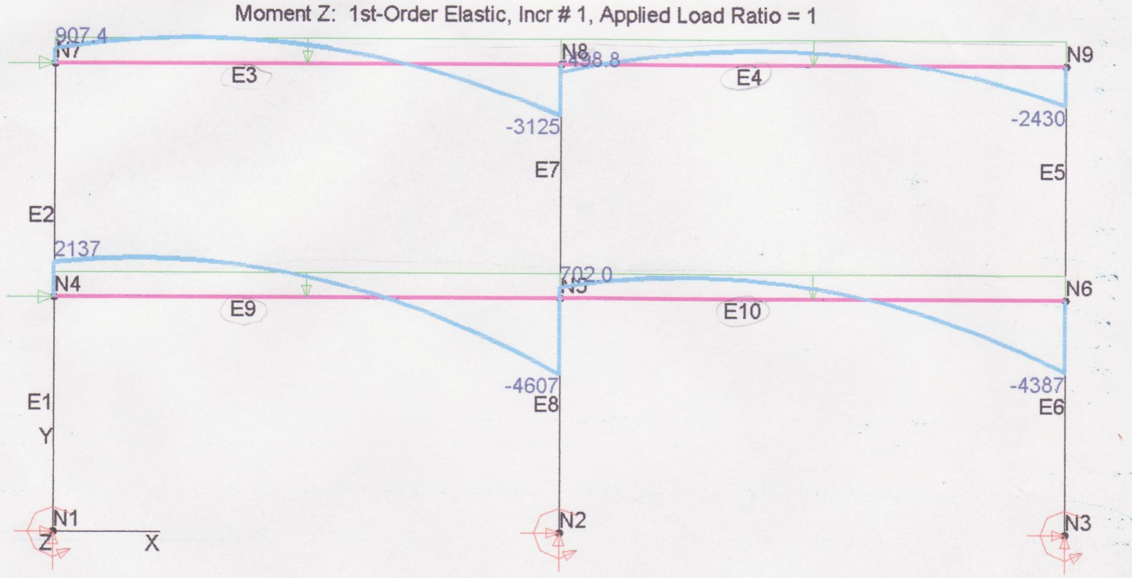


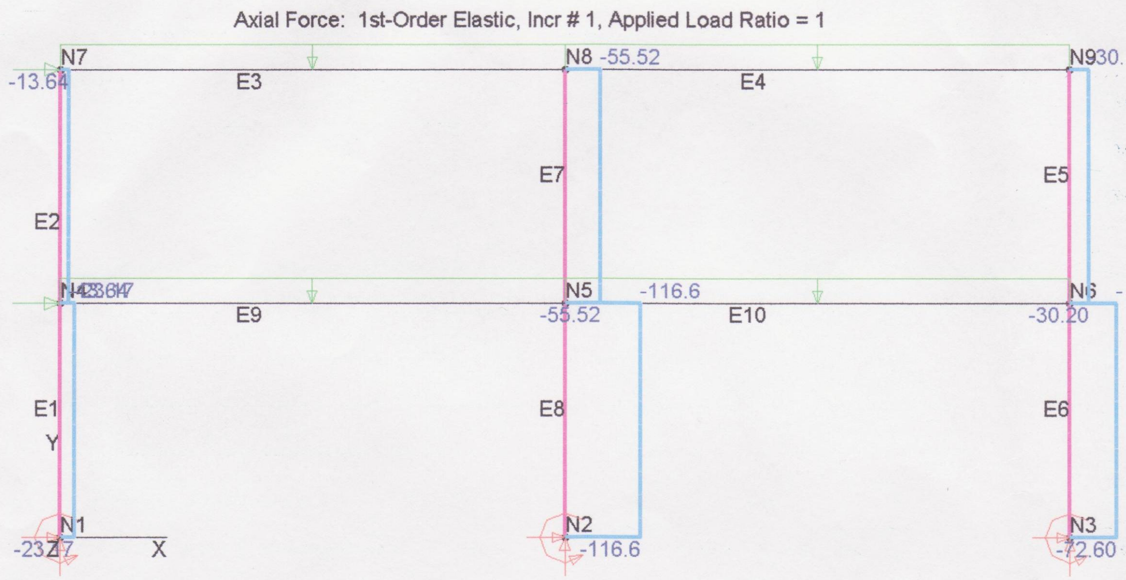
$M_C = 2(9.36)(12) - 9.36\left(\frac{24}{5}\right) - 9.36\left(\frac{24}{10}\right) = 202.18$

$w = \frac{202.18(8)}{24^2} = 2.81$ ft·kips

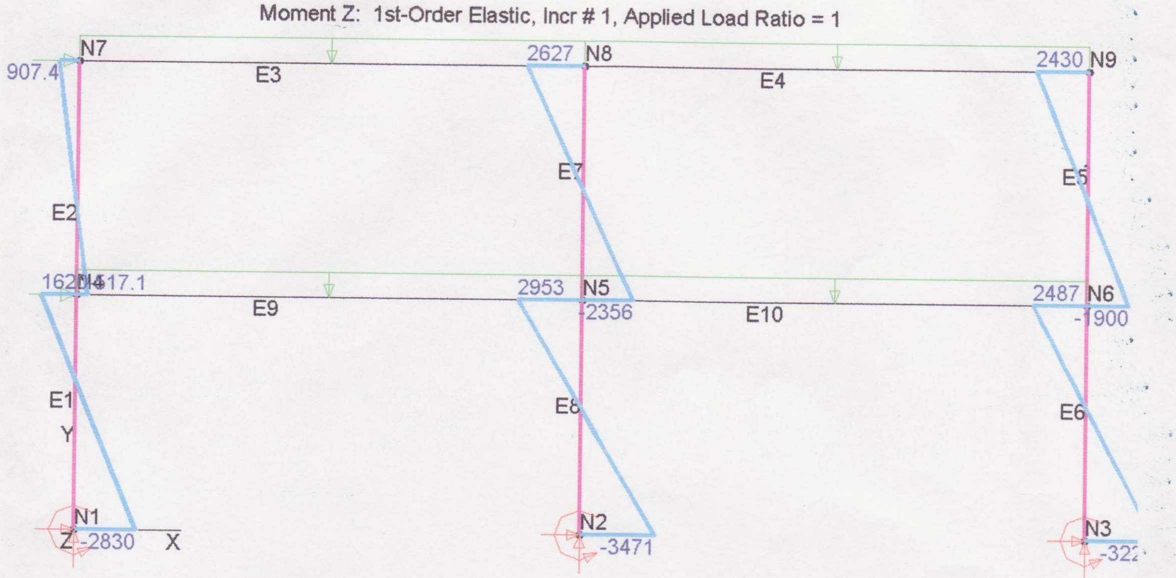
Long Side

Moment Z: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1

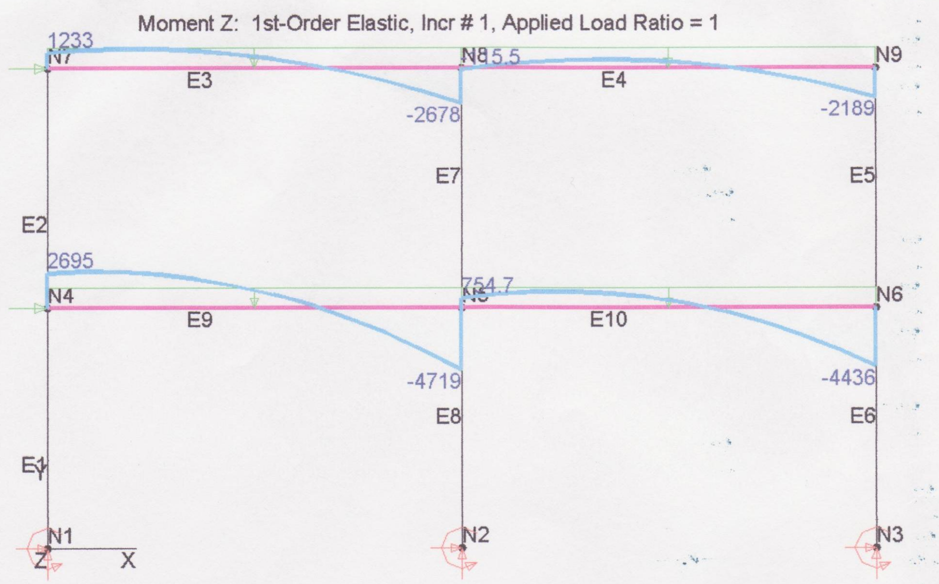



Axial Force: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1
N7
-13.64
E3
N8 -55.52
E4
N930.
E7
E5
E2
N5
-116.6
N6
-55.52
E9
E10
-30.20
E1
E8
E6
Y
N1
N2
N3
X
-116.6
-72.60

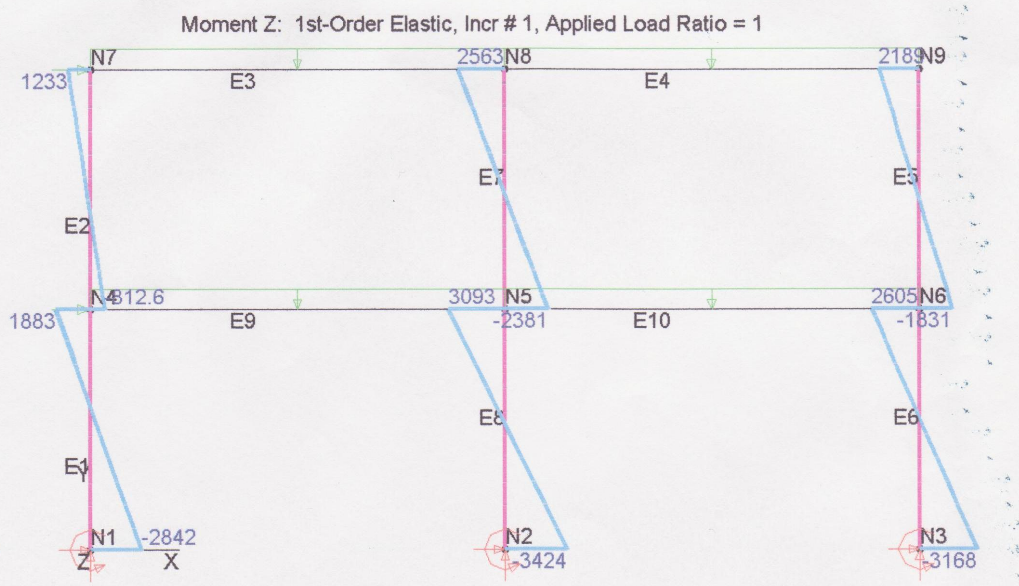
Moment Z: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1

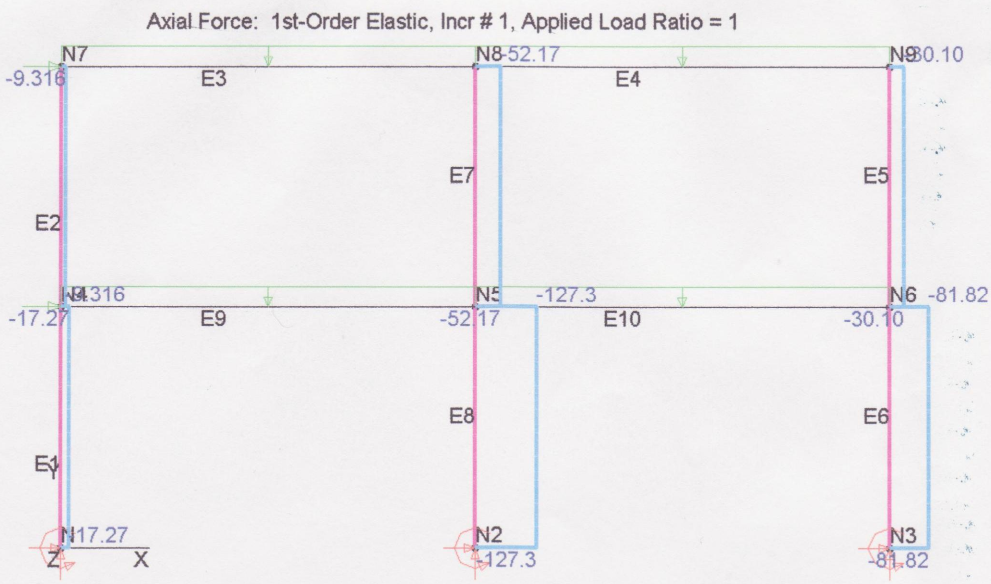


Short Side



Moment Z: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1
1233
N7
N8
5.5
N9
E3
E4
-2678
-2189
E7
E5
E2
2695
754.7
N4
N5
N6
E9
E10
-4719
-4436
E8
E6
E1
Y
N1
N2
N3
Z
X


Moment Z: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1
N7
2563
N8
2189
N9
1233
E3
E4
E7
E5
E2
N4
312.6
3093
N5
2605
N6
1883
E9
-2381
E10
-1831
E8
E6
E1
N1
-2842
N2
N3
X
-3424
-3168


Axial Force: 1st-Order Elastic, Incr # 1, Applied Load Ratio = 1
N7
-9.316
E3
N8
-52.17
E4
N9
-30.10
E7
E5
E2
-17.27
E9
N5
-127.3
E10
N6
-81.82
-52.17
-30.10
E8
E6
N2
N3
17.27
-127.3
-81.82
X
Z

Long side collums

$P_{com} = P_u + M_{ux} m + M_{uy} m$

HL = 14ft m = 1.8

P_{com} = 116.6 + 289.25(1.8) + 0 = 637.25

Try W14x90 P = 682 kips > 637.25 ✓

HL = 14ft m = 1.8

P_{com} = 55.52 + 218.9(1.8) + 0 = 449.57 kips

Try W14x74 $\frac{P_n}{\Omega_n}$ = 468 > 449.57 ✓

Short side

HL = 14ft m = 1.8

P_{com} = 127.3 + 285.33(1.8) + 0 = 640.9 kips

Try W14x90 P = 682 > 640.9 ✓

HL = 14ft m = 1.8

P_{com} = 52.17 + 213.58(1.8) + 0 = 436.62

Try W14x74 P = 468 > 436.62 ✓

Roof Girder Long Side

$M_{max} = 3126$ in·kips $= 260.5$ ft·kips

Use W 27 x 129 $M_{max} = 283 > 260.5$ ✓

Floor Girder

$M_{max} = 4607$ in·kip $= 383.9$ ft·kips

Use W 27 x 194

$M_{max} = 420 > 383.9$ ✓

Floor Girder Short Side

$M_{max} = 4719$ in·kips $= 393.26$

Use W 27 x 161

$M_{max} = 428 > 393.26$ ✓

Roof Girder Short Side

$M_{max} = 2678$ in·kips $= 223.17$ ft·kips

Use W 27 x 94

$M_{max} = 231 > 223.17$ ✓