Kalkin Building Design Project

University of Vermont

CEE4720 Structural Steel Design - Fall 2024

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Introduction

For the final project of CEE4720 Structural Steel Design we were asked to analyze the structural design of Kalkin Hall on the University of Vermont Campus given architectural and structural engineering drawings. We used the computer program SAP2000 to model and analyze the existing structure. Using practices taught to us in class we then designed our own steel members for the building. Both processes were completed using loads we calculated based off industry standards and codes.

Summary Sheet

Table 1: Summary table showing the calculated live and dead loads

Summary Table	
Load	psf
Roof Dead	15
Floor Dead	94
Wall Dead	90
Live (Corridor)	100
Live Roof	20
Snow (Roof)	40

Table 2: Summary table showing the chosen members from the design calculations

Summary Table		
Member	Design Member	As Built Member
Joist	14K3	N/A
Typical Interior Floor Beam	W12x22	W12x19
Typical Exterior Floor Beam	W14x22	W14x22
Typical Interior Floor Girder	W21x44	W18x65
Typical Interior Column	W8x31	W10x49
Typical Exterior Column	W8x31	W10x60/68

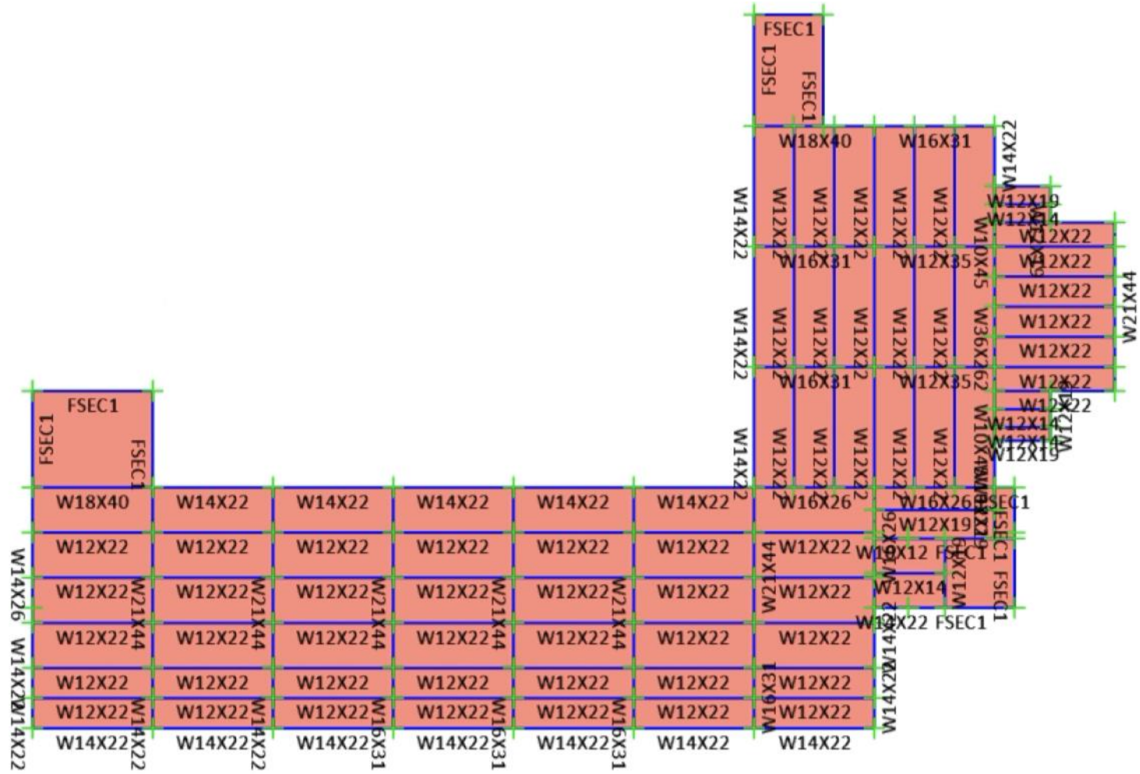


Figure 1. Floor 1, 2, and 3 with Typical Interior Floor Beams, Typical Exterior Floor Beams, and Typical Interior Floor girders assigned.

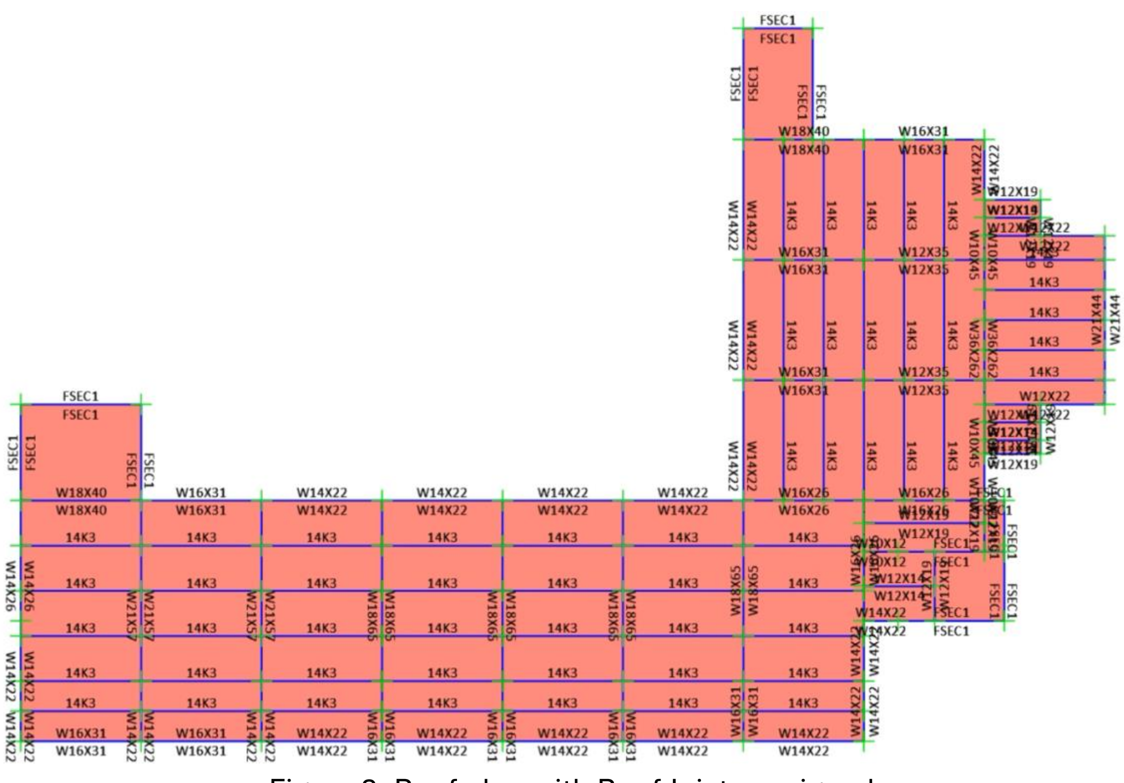


Figure 2. Roof plan with Roof Joists assigned.

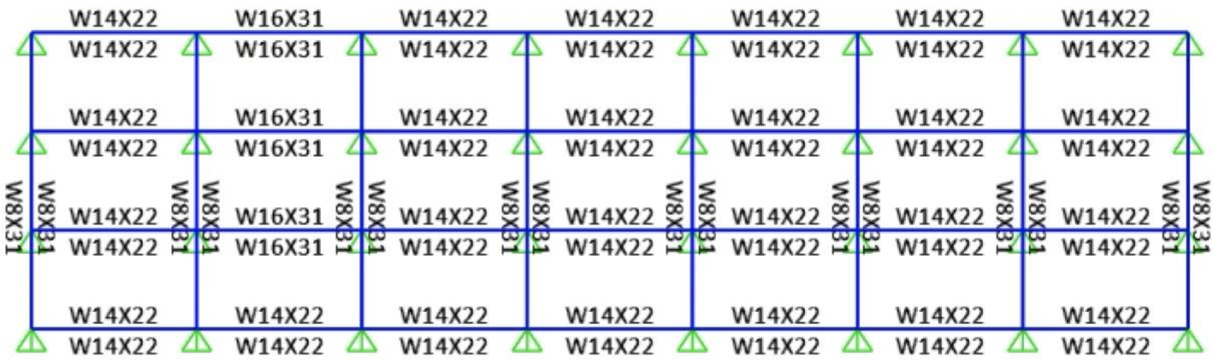


Figure 3. Elevation view with assigned members.

Summary of Analysis Results

The members we designed using the AISC manual and current building codes were close to the existing members utilized for the construction of Kalkin Hall. In most cases the members we designed were lighter in weight than those of the existing members. The one exception is the floor beams. The typical interior floor beam was designed to weigh 3 lb/ft more despite still being a W12 section. The typical exterior floor beam, like the interior, is as large as the current section. The exterior floor beam designed to be the same W14x22 as the members utilized in Kalkin Hall. The typical interior floor girder, while not the same, was designed to be W21 section weighing 21 lb/ft less compared to the utilized W18x65.

The largest difference in our designs compared to that of the existing structure can be observed in the column designs. The columns we selected are the smallest standardized sections which are much smaller than the members in the building. The exterior column we designed had a compressive strength approximately half that of the as the built column and weigh 18 lb/ft less. Additionally, the minimum radius of gyration was only 80% as large as the built column. The designed interior column had greater difference as we selected the same designed member as the exterior column, but the current members are even larger for interior columns compared to the exterior. Our designed interior column weighed 37 lb/ft less than the current interior columns.

Another significant difference between our design and existing structure is the roof section. We simplified the existing sloped roof as a flat roof supported by a 14K3 joist that rests on interior and exterior beams.

Conclusion

To conclude, it was found that most of the members calculated by our group were of equal size or slightly smaller than the original members provided in the plans. The exception to this being found in the column design and joist design. With our group's designed columns and joists being significantly smaller than that of the existing design, we have concluded that this is most likely due to the simplified method in which we designed our members. This means that we were most likely using much less conservative load assumptions than the engineers that designed the actual building, as they have a lot more at stake if their design fails. In addition to this another reason we calculated much lighter members in some cases is that fact that we didn't incorporate seismic loading in our design either, while the company that designed this building did require seismic loading.

Overall, we feel confident in our designed members as we utilized the AISC guidelines as learned in class. We have attributed our nuances compared to the existing Kalkin building to our more simplified design process than that of the professional engineer who designed the building.

References

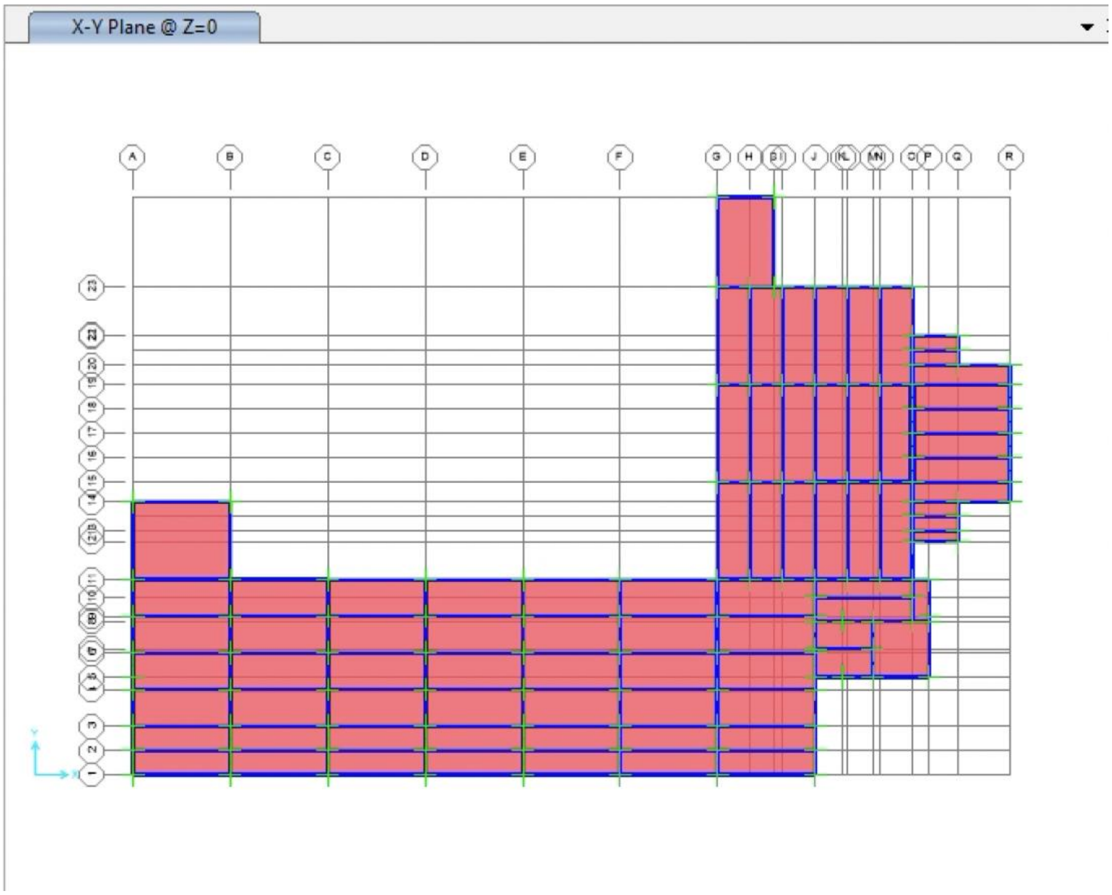
1. American Institute of Steel Construction, *Manual of Steel Construction*, 16th Edition. Chicago: AISC, 2023.

2. International Code Council, *International Building Code*, 2018. Illinois: International Code Council Publications, September 2018.

3. ASCE/SEI 7-05, -10, -16, -22. *Minimum Design Loads for Buildings and Other Structures*. ASCE, 2005, 2010, 2015, 2022.

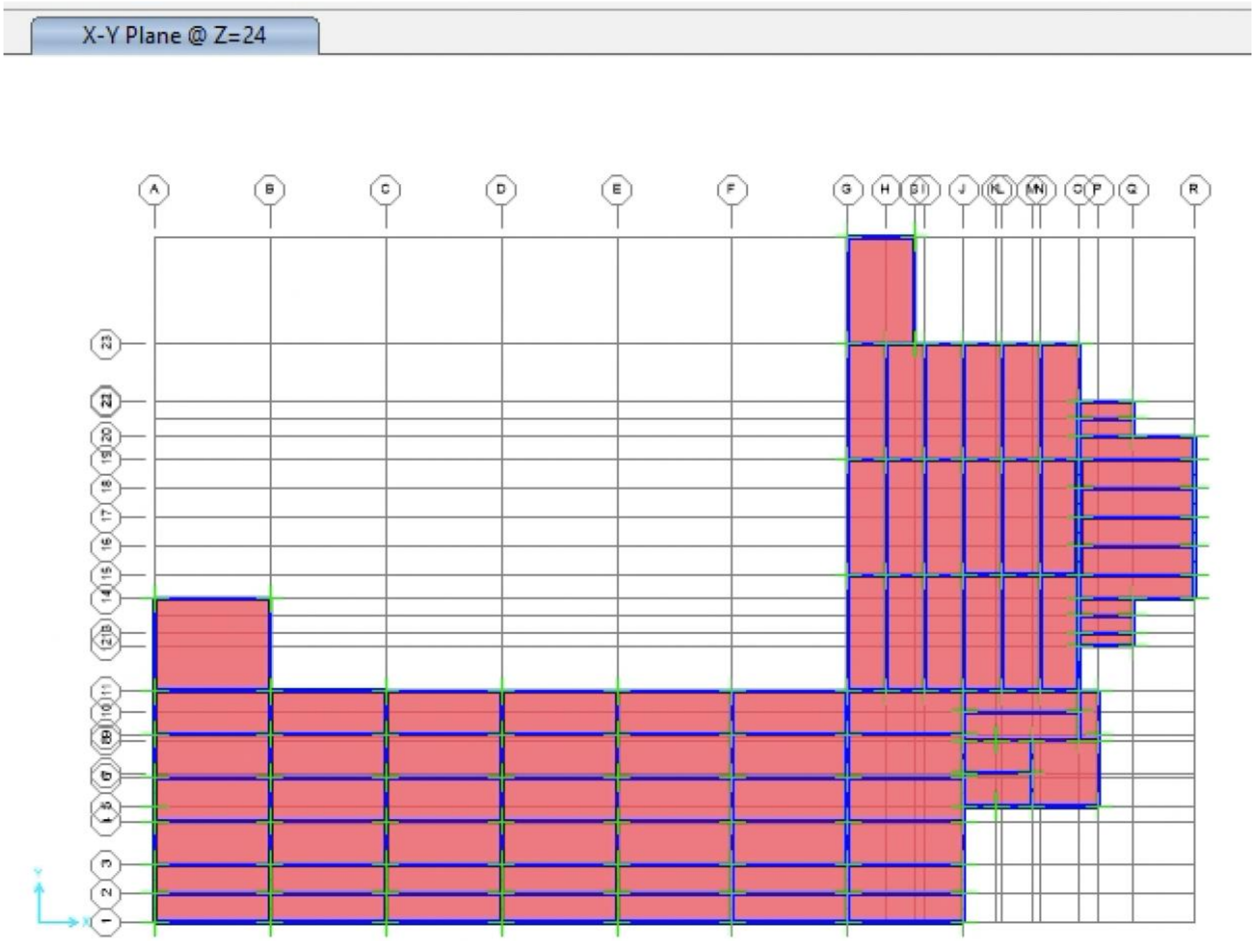
Appendix A: SAP2000 Geometry

First Floor Plan

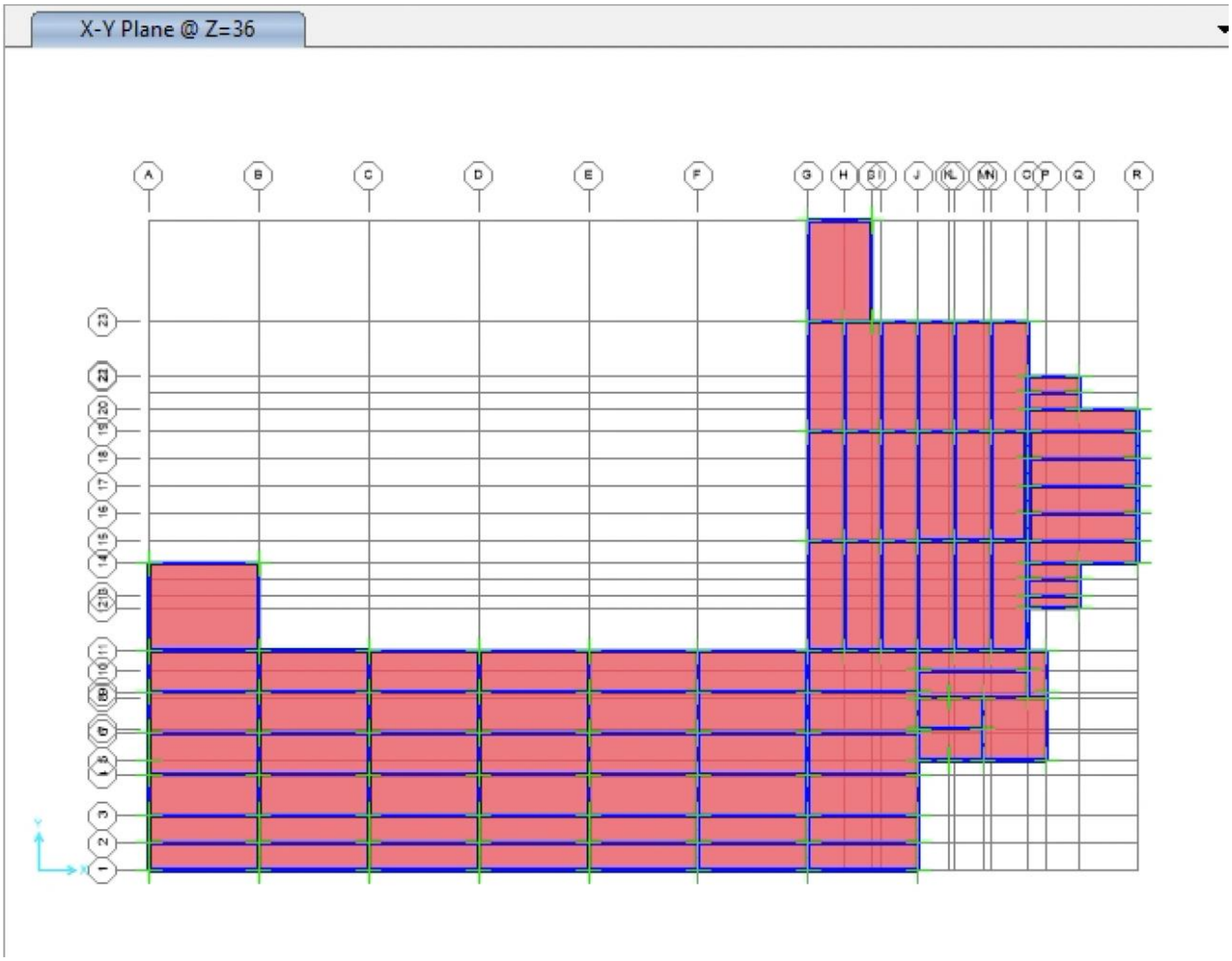


Second Floor Plan

Third Floor Plan

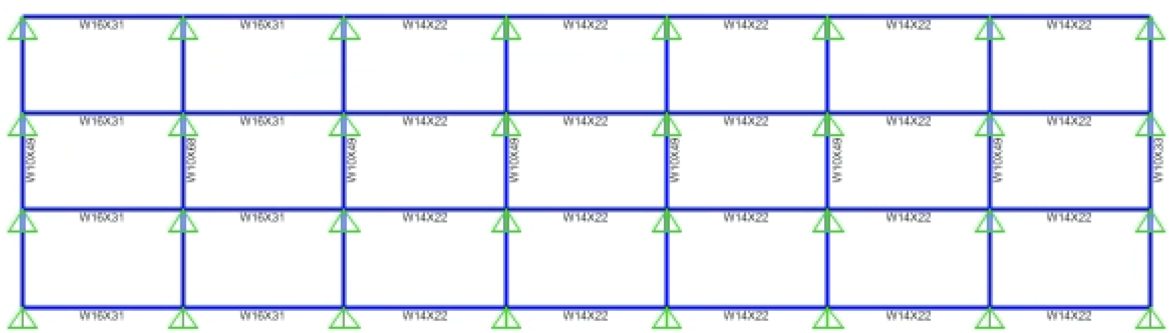


Roof

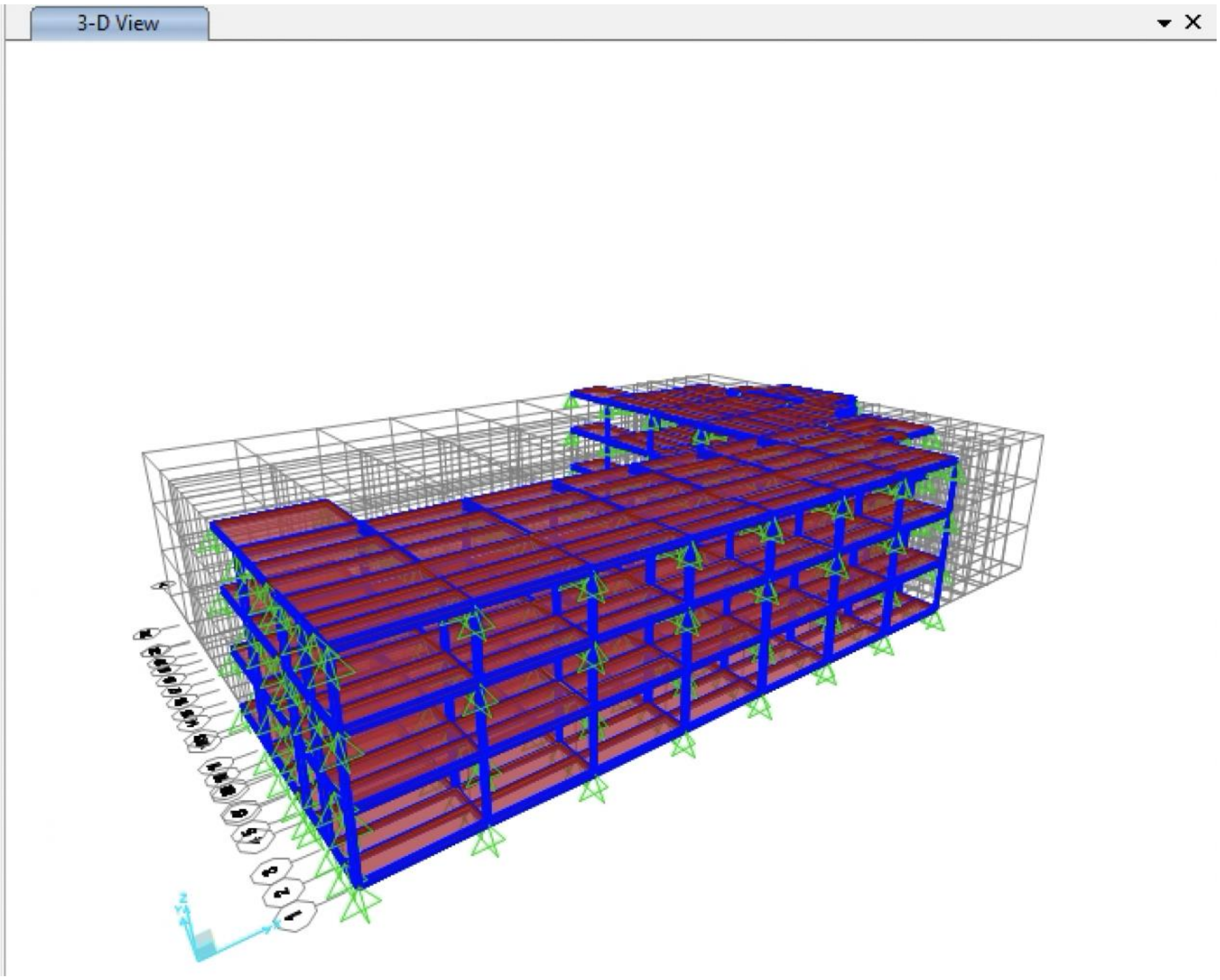


Elevation


Steel Design Sections (AISC 360-10)


W16X31 W16X31 W14X22 W14X22 W14X22 W14X22 W14X22
W16X31 W16X31 W14X22 W14X22 W14X22 W14X22 W14X22
W10X49 W10X68 W10X49 W10X49 W10X49 W10X49 W10X49 W10X33
W16X31 W16X31 W14X22 W14X22 W14X22 W14X22 W14X22
W16X31 W16X31 W14X22 W14X22 W14X22 W14X22 W14X22

3D Frame



Shear and Moment Diagram of Exterior Beam

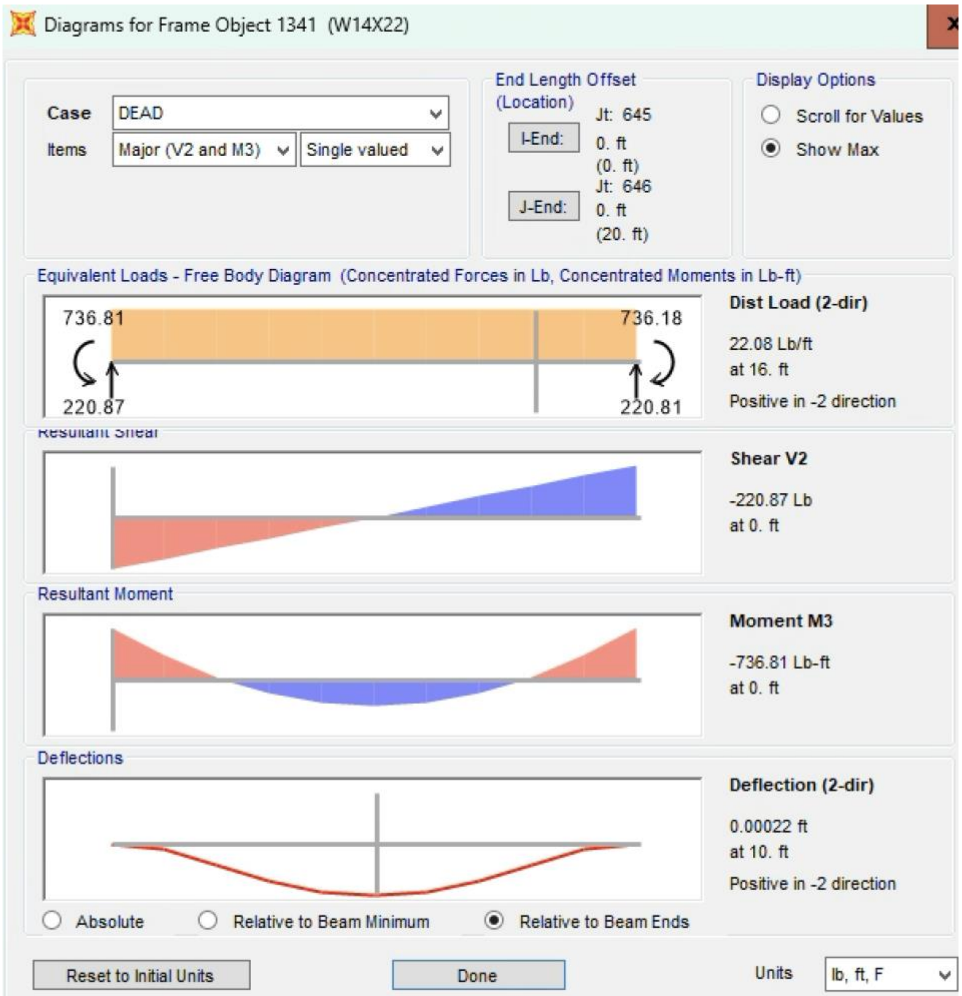


Figure 4. Moment and Shear Diagram of Exterior Beam on Roof

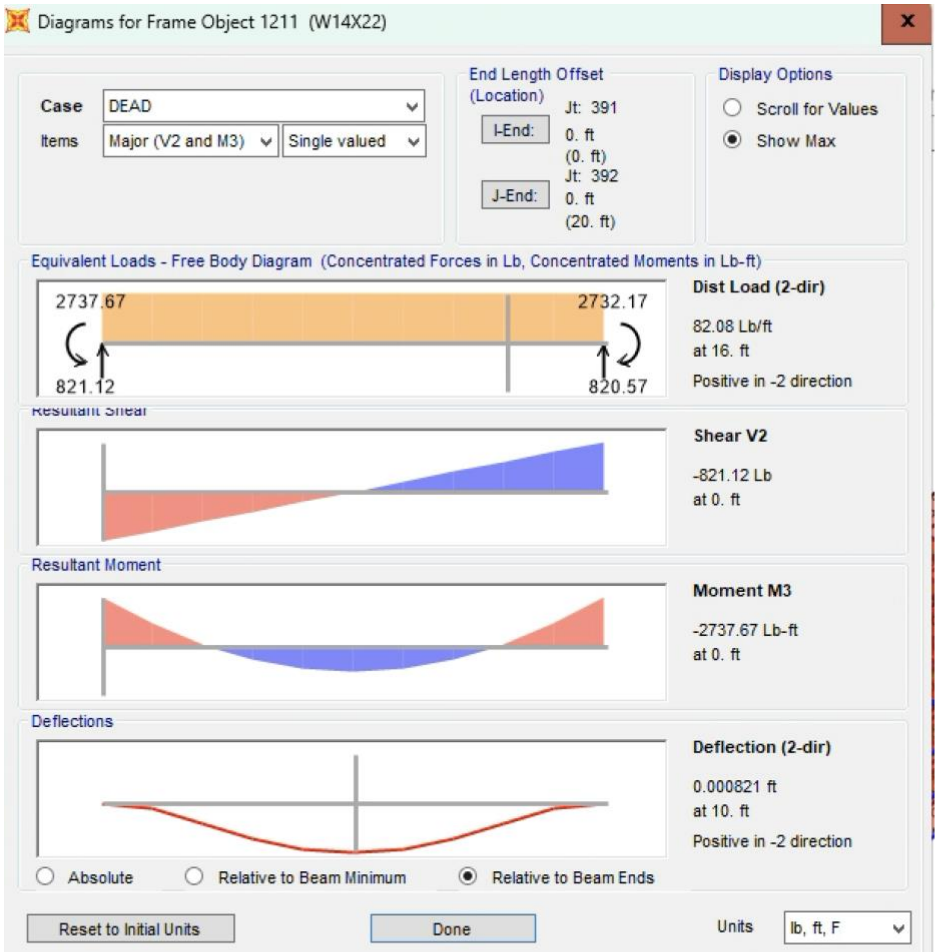


Figure 5. Moment and Shear Diagram of Exterior Beam on Roof on Third Floor

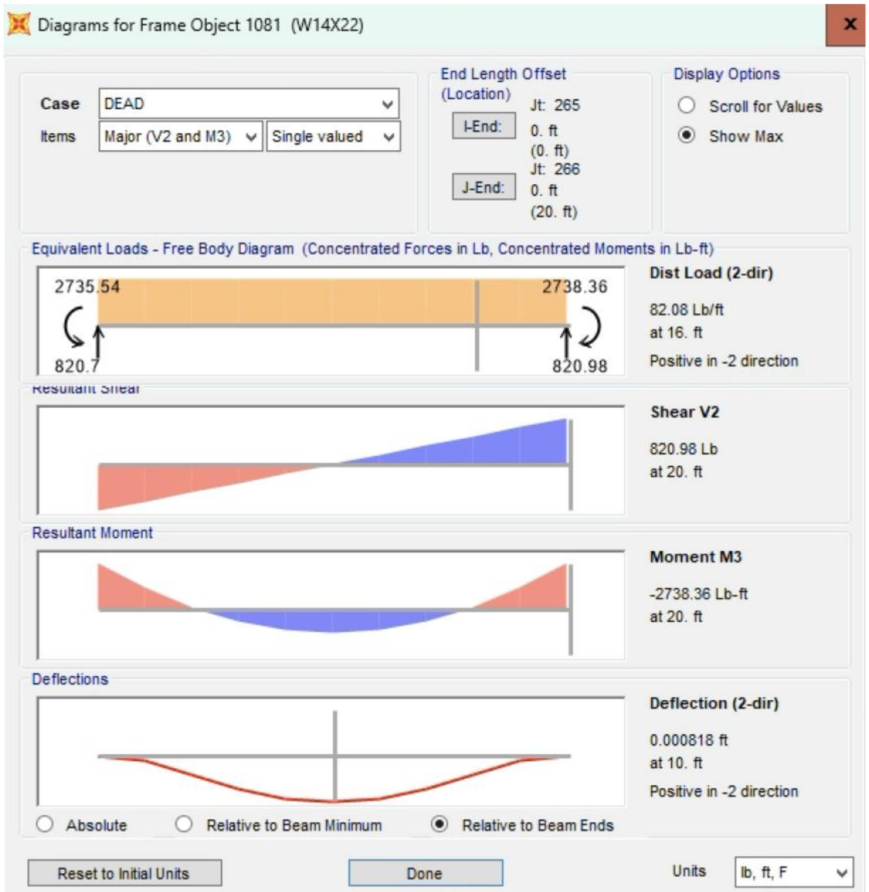


Figure 6. Moment and Shear Diagram of Exterior Beam on Roof on Second Floor

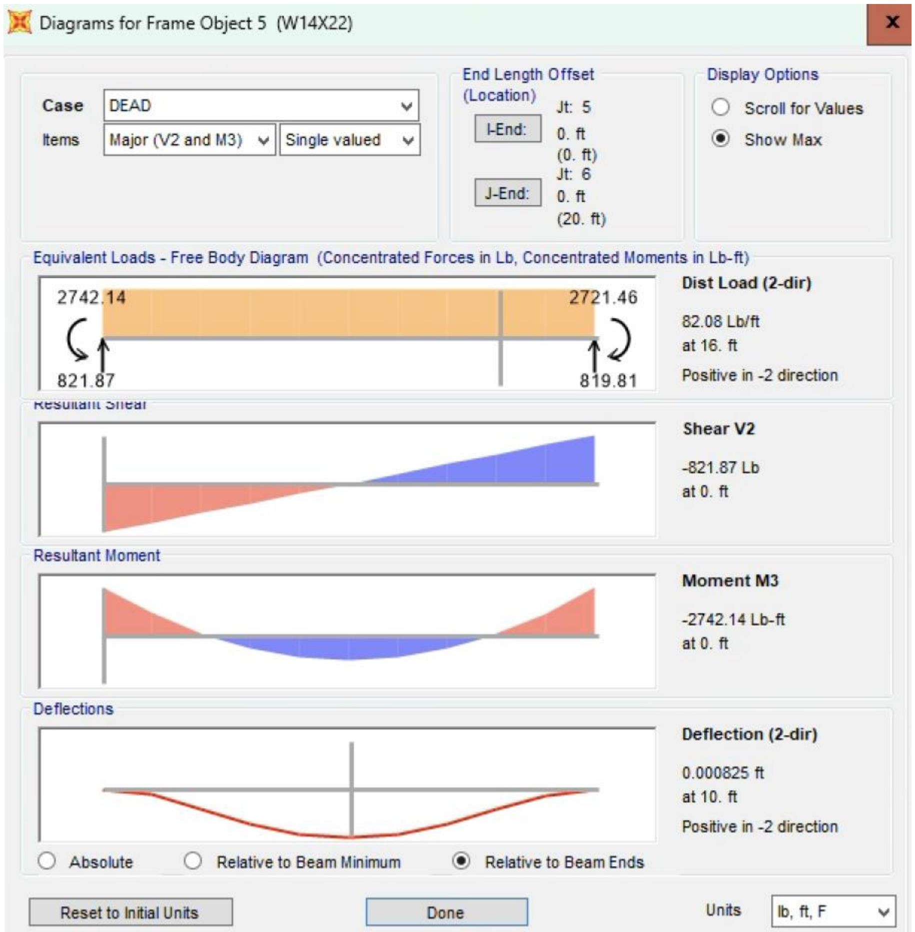


Figure 7. Moment and Shear Diagram of Exterior Beam on Roof on First Floor

Appendix B: Dead and Live Loads

Table 3: Roof, floor, and wall dead load compositions

Dead Loads	
Roof	psf
Joist Framing DL	1
15# BLDG Felt	1
5/8” APA EXP 1 PLYWD Sheath	2
2.8” Rigid Insul W/Intrgl	0.75
Vapor Retarder	2
1 ½" MET Deck	2.5
7/8” Met Furring	3
5/8” GYP BD	2.75
Total	15
Floor	psf
5 ½" CONC Slab	66
Direct Glue Down Carpet	3
4” Roll Length Vinyl	5
Base Board – Typ. All Rooms	5
VCT, Mechanical, Electric, Etc.	15
Total	94
Wall	psf
4” Rigid Fiberglass Insulation	4.4
4” Met Stud	6.6
1” Thick Slate Sills	40
4” Face Brick	39
Total	90

Table 4: Roof and floor live loads

Live Loads	
Load	psf

Floor (Classroom)	40
Floor (Corridor)	100
Roof	20

Appendix C: Hand Calculations and Sketches

1. Roof Steel Joist

11/15/2024	CEE 4720	Group 2

1. Roof Steel Joist

$L = 20$ ft

$$\frac{L}{D} = \frac{20 \cdot 12}{D} = 16$$

$D = 15$

$$\frac{L}{D} = \frac{20 \cdot 12}{D} = 20$$

$D = 12$

Roof Dead = 15
Roof live = 20
Snow = 40

$1.2D + 0.5L_r = 28$
$1.2D + 0.3S = 30$
$1.2D + 1.6L_r = 50$
$1.2D + 1.0S = 58$ psf (7.5ft) = 435 lb/ft

Unfactored live load = 20(7.5) = 150 lb/ft

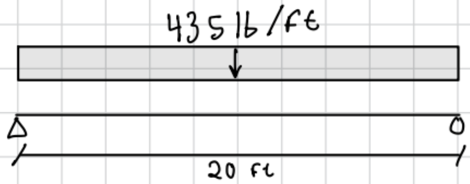
$\boxed{\text{Use 14K3}}$

Red = 246 lb/ft > 150 lb/ft
Black = 534 lb/ft > 435 lb/ft

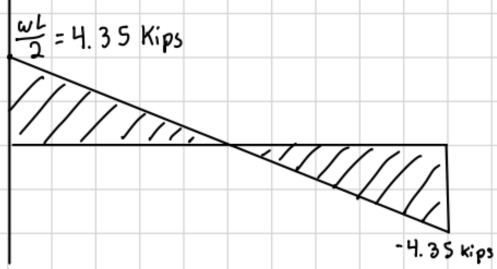
Wt $\simeq$ 6.0 lbs/ft

11/15/2024 | CEE 4720 | Group 2

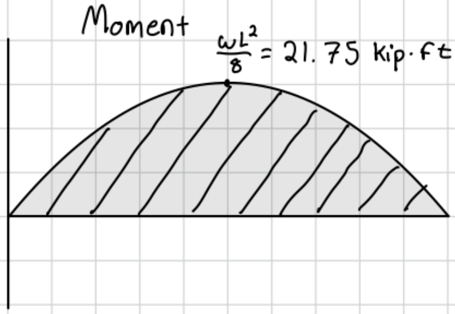
Factored load



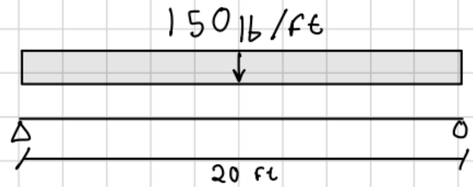
Shear



Moment

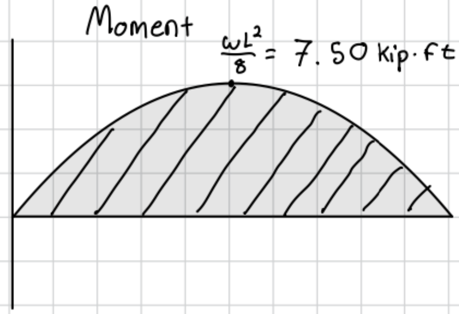


Unfactored Live Load



Shear

Moment



2. Typical Interior Floor Beam

11/15/2024 | CEE 4720 | Group 2

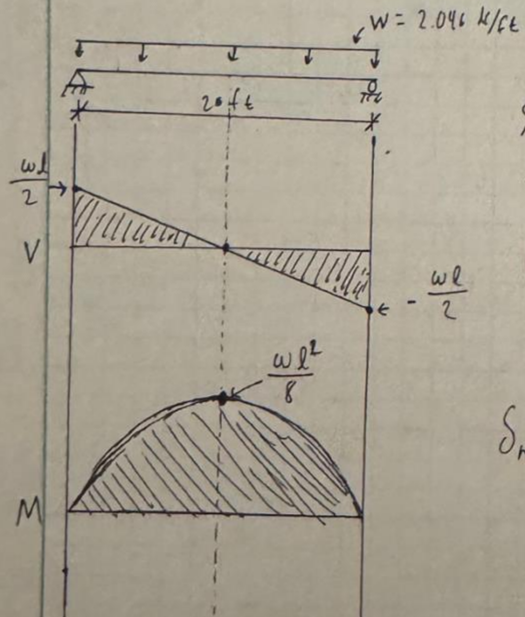
2) Typical Interior Floor beam

Floor dead load: 94 psf   Floor live load: 100 psf

normiliz loads to lb/ft by multiplying by tributary width

Floor dead: 94 psf · 7.5 ft = 705 lb/ft   Floor live: 100 · 7.5 = 750 lb/ft

factor load: 1.2(705) + 1.6(750) = 2.046 Kip/ft



$M_u = \frac{wl^2}{8} = \frac{2.046\,k/ft \cdot 20ft^2}{8} = 102.3$ kip-ft

Select W12x22 from table 3-2

Since $\phi_b M_{px} = 110$ kip·ft

Check self weight by adding (.022)1.2 to udL for new M = 103.4

Still less than 110 kip·ft so OKAY

Check live load deflection

$\delta_{max} = \frac{5wl^4}{384EI}$ E = 29000 ksi, I = 156 in^4, l = 20 ft, w = 1.6(750) = 1.2 k/ft

$\delta_{Live} = \frac{5(1.2)(20)^4}{384(29000)(156)} \cdot 144 = .0795$ ft (unit conversion)

limit = l/240 => 20/240 = .0833

Since .0795 ft < .0833 ft OKAY by live deflection

Select W12x22

3. Typical Exterior Floor Beam

11/15/2024 | CEE 4720 | Group 2

3) Typical exterior Floor beam

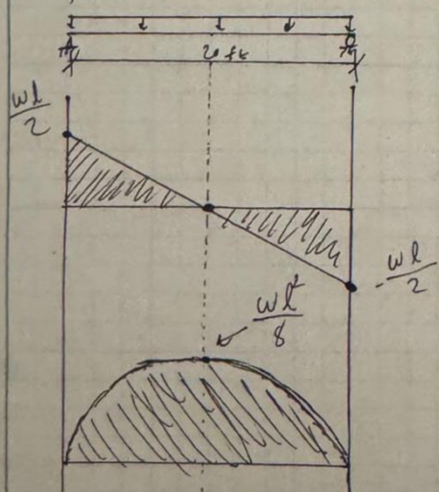
Floor dead: 94 psf, live floor: 100 psf, dead wall: 90 psf

normalize to lb/ft on beam

dead: 94 psf · 3.75 + 90 · 12 = 1432.5 lb/ft

live: 100 psf · 3.75 = 375 lb/ft

load combo action: 1.2(1432.5) + 1.6(375) = 2.319 k/ft



$$M_u = \frac{wl^2}{8} = \frac{2.319 \cdot 20^2}{8} = 115.95 \text{ k·ft}$$

Select W14x22

Since $\phi_b M_{px} = 125$ k·ft

Check self weight

$M_{New} = 117.27$ k·ft

Still greater than 125 k·ft

Check max live deflection

$$\delta_{max} = \frac{5wl^4}{384EI}$$

E = 29000 ksi
I = 199 in^4
L = 20 ft
w = .6 k·ft

$$\delta_{live} = \frac{5(.6)(20)^4}{384(29000)(199)} \cdot 144 = .03119 \text{ ft}$$

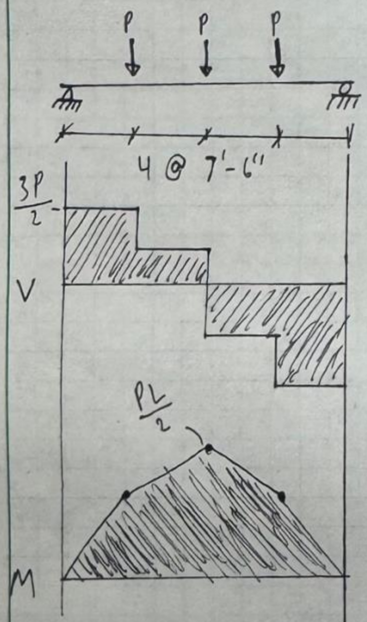
Limit = 20/240 = .0833 > .03119 ft

So W14x22 is OKAY

4. Typical Interior Floor Girder

12/02/2024 | CEE 4720 A | Group 2

4) Girder



P = reaction force of typical interior beam

P = 20.46 kips

$$M_{max} = \frac{PL}{2} = \frac{30 \cdot 20.46}{2} = 306.9 \text{ kip·ft}$$

Select W21x44 from table 3-2 of AISCM

Since $\phi_b M_{px} = 358$ kip·ft

check self weight by adding UDL of .044 lb/ft new M = 312 kip·ft

Still less than 358 kip·ft so OK//

live load deflection

$$\delta_{max} = \frac{7PL^3}{384(E \cdot I)}$$

P = 18 kip
L = 30 ft
E = 29000 ksi
I = 843 in^4

$$\delta_{max} = \frac{7(12 \text{ kips})(30^3)}{384(29000)(843)}$$

$$\delta_{max} = .03478 \text{ ft}$$

limit deflection: (L/240)

30/240 = .125 ft

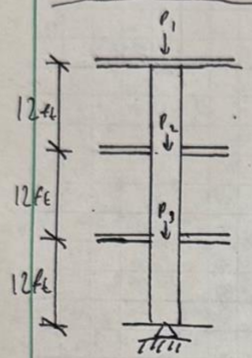
.03478 ft < .125 ft so OK// by live load deflection

So Select W21x44
for typical interior Girder

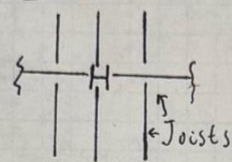
5. Typical Interior Column

12/02/2024 | CEE 4720A | Group 2

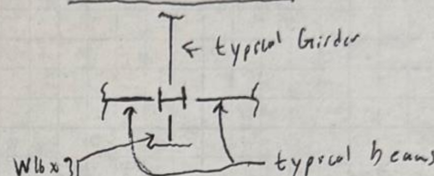
5) Typical Interior Column



Roof



1st + 2nd floor



Roof axial load:

Joist $= 435\ \text{lb/ft} \cdot 10\text{ft}$

$= 4{,}350$ ft

$P_1 = 6 \cdot (4.35\ \text{kips}) = 26.1$ kips

floors Additional load

$P_{additional} = 30.69 + 40.92 + 6.8$

$= 78.41$ kips

$P_2 = P_1 + P_{add}$ $\quad P_3 = P_1 + 2(P_{add})$

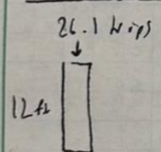
$P_2 = 104.51$ kips $\quad P_3 = 182.92$ kips

try W8x31 from table 4-1 AISCM:

Slenderness ratio: each colom $L = 12$ft

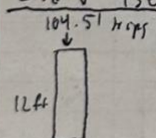
$$\frac{L}{r_{min}} = \frac{12\text{ft} \cdot 12\frac{\text{in}}{\text{ft}}}{2.02} = 71.29 < 200 \quad \text{Ok// by Slenderness}$$

roof column

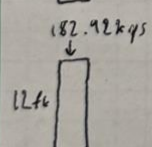


$\phi_c P_n = 283\text{kips} > 26.1$ kips Ok//

2nd + 1st floor Column



2nd: $104.51 < 283$ Ok//



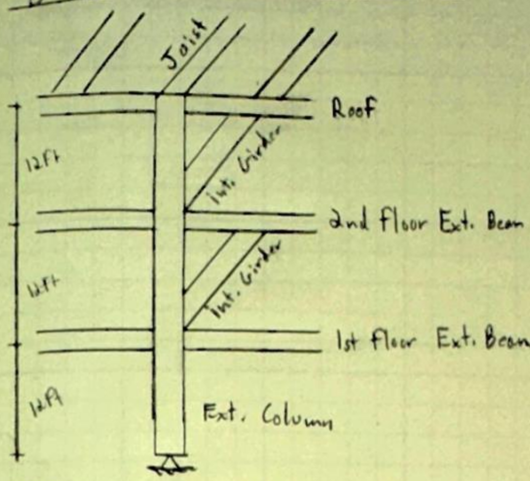
1st: $182.92 < 283$ Ok//

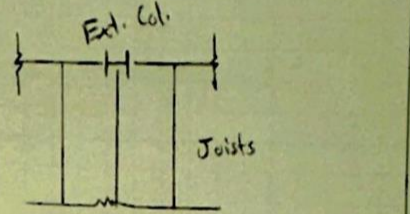
$\therefore$ Select W8x31 for each typical interior column

6. Typical Exterior Column

12/02/2024	CEE 4720A	Group 2 Coles	1/1

Typical Exterior Column:





Joist Loading = 435 lb/ft · 20ft
= 8,700 lb

Rxn's = 4,350 lb

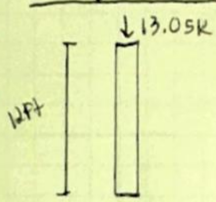
Roof Axial Loading =
3 · (Rxn Joist) = 3(4,350 lb)

Roof Axial = 13.05 Kips

Exterior Beam Rxn on Column = $2.319\ \text{k/ft} \cdot \frac{20\text{ft}}{2}$
= 23.19 K

Interior Girder Rxn on Ex. Column = 30.69 K

Roof Column



K = 1 for pin-pin Connection => L_c = 12ft

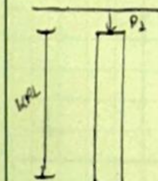
Table 4-1 of AISCM:

W8 x 31 $\quad \phi_c P_n$ = 283 Kips > 13.05K // ok

Table 1-1: r_x = 3.47 $\quad r_y$ = 2.02

$\frac{L}{r} = \frac{12\text{ft} \cdot 12\frac{\text{in}}{\text{ft}}}{2.02} = 71.29 < 300$ // ok

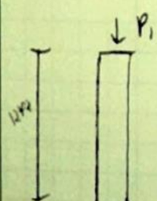
2nd + 1st Floor Columns:



$P_2 = 13.05\text{K} + \underbrace{2 \cdot (23.19\text{K})}_{\text{2 Ext. Beams}} + \underbrace{30.69\text{K}}_{\text{Girder}}$

P_2 = 90.12 K

W8 x 31 $\quad \phi_c P_n$ = 283 K > 90.12 K // ok



$P_1 = 90.12\text{K} + \underbrace{2(23.19\text{K})}_{\text{2 Ext. Beams}} + \underbrace{30.69\text{K}}_{\text{Girder}}$

P_1 = 167.19 K

W8 x 31 $\quad \phi_c P_n$ = 283 K > 167.19 K // ok

Slenderness Check the Same as Roof Column

Appendix D: Extra Credit (Assumptions)

Many simplifications and assumptions were required to make this modeling and design of the Kalkin Building less onerous and able to be completed within the scope of a course. The largest of these assumptions was to ignore lateral loads; designing members or frames to resist these loads was outside of the scope of this class. The Kalkin building does, however, employ multiple methods of resisting lateral loads including moment resisting connections and shear walls. The type of rigid connections used allows these loads to be resisted by the flexural stiffness of the columns. The shear walls present in the Kalkin buildings stairwells and elevator shaft resist lateral loads in a similar manner, by acting as rigid, vertical cantilevers. The other assumptions made include simplifying the sloped roof structure to a simple, horizontal joist system, treating all members as simply supported, and applying uniform member selection to all design members of the same type. These assumptions greatly reduce the complexity of our design calculations and time required to model the as built structure.