# FINAL DESIGN REPORT

FALL 2014

Design Team: Ana Gouveia, Matthew Laskey, Zach Zavalianos

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# PROJECT SUMMARY

## Project Summary

Project Riverside is a two-story, steel-framed, office building located in Lowell, MA. GLZ Design has assigned the design team composed by Structural Engineers: Matthew Laskey, Ana Gouveia and Zachary Zavalianos to perform the structural analysis, evaluation and design of this project.

Guided by local regulations, the design team used of the most up to date reference codes, associated to outstanding optimization analysis, along efficient construction practices as the project's design criteria. A more accurate overview of these values can be found in page 7 of this report. Building classifications and initial information can be found in page 8 of this report.

Riverside's design process started through a complete identification of vertical and lateral loads, which were further analyzed by the Load & Resistance Factor Design (LRFD). These initial values were used in the preliminary-sizing of project horizontal and vertical members.

In the second phase of the project, these preliminary members were further evaluated and analyzed. Final framing plans for both first and roof floor were obtained. Two different lateral load resisting systems (LLRS) were designed: a moment-frame resists longitudinal loads, and braced-frames are responsible for transverse loads. Given building risk categories, per code used, ordinary moment-frame and braced-frame were allowed in the design, reducing the project's final budget.

In the project's final phase, composite floor deck, roof joists, girders and diaphragms, and connections were designed. For quality assurance and control, all of the project's information, calculations, analysis and drawings were further back checked. The final results of this project can be found in this report. Calculations, Analysis and Drawings are referred in its respective section on the report and can also be found at this report's appendixes.

GLZ Design appreciates your business and hope to continue serving your Structural Engineering needs in the future.

Sincerely,

| | | |
|---|---|---|
| Ana Gouveia, EIT | Matt Laskey, EIT | Zac Zavalianos, EIT |

**GLZ Design**

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## Design Team

We are pleased to introduce you Riverside's design team:

### MATTHEW LASKEY, EIT
### STRUCTURAL ENGINEER

m.laskey86@gmail.com

Matt continued working as a Structural Engineer at CDM Smith after his summer internship at the firm

Matt has a bachelor's degree in Environmental and Resources Economics from UNH and a Civil Engineering degree from UMass Lowell.

### ANA GOUVEIA, EIT
### STRUCTURAL ENGINEER

acgouveia@live.com

Ana was born in Brazil where she started her college education in Civil Engineering.

As a challenge-seeker, she immigrated to Massachusetts in 2009. Since then studied at Harvard University Extension School, acquired an associate's degree in Engineering from Bunker Hill Community College, a bachelor's degree in Civil Engineering from the University of Massachusetts Lowell.

### ZAC ZAVALIANOS, EIT
### STRUCTURAL ENGINEER

Zachary_Zavalianos@student.uml.edu

Hardworking and enthusiastic structural engineer with experience in design and inspection of bridge structures. Certified in bridge inspection and proficient in inspection databases such as PONTIS and 4-D, as well as MDX and CSI-bridge analysis programs. Seeking to progress career by obtaining a masters degree and working towards a PE license through the upcoming years.

## Company Information

**GLZ Design**
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GLZ Design, LLC

1 University Ave, Lowell, MA

www.glzdesign.com

# Design Criteria

## Design Criteria and Constraints

The following figure represents the design criteria which guided the team:

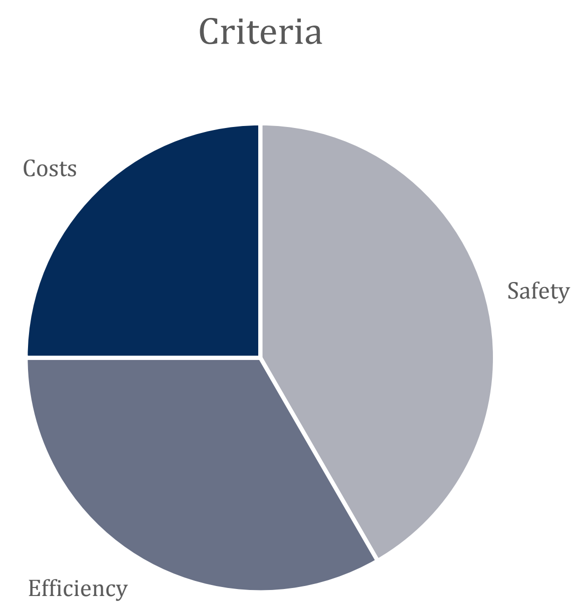



**Figure A – Riverside's guiding criteria.**

In order to satisfy the above criteria and constraints the following items were used:

## References and Regulations

The following literature was used in reference and support of the work done in this project. Please note that names in parentheses are abbreviations commonly cited throughout this project.

- Massachusetts Building Code (CMR 780)
- 2012 International Building Code (2012 IBC)
- Minimum Design Loads for Buildings and Other Structures, (ASCE/SEI 7-10)
- $14^{th}$ Edition of the Steel Construction Manual (AISC 14th)
- American Concrete Institute Manual (ACI)

# DESIGN CRITERIA

- 2007 Steel Deck Institute Design Manual (2007 SDI)
- Vulcraft Joists and Girders (VJG)
- 5th Edition Structural Steel Design, McCormac & Csernak (SSD 5th)

## Methods of Design and Analysis

The following methods of design and analysis were thoroughly applied to the different parts of this project.

- Loads: Load & Resistance Factor Design (LRFD) and ASCE 7-10 guidelines.
- Wind Loads: Directional Method (ASCE 7-10)
- Seismic Loads: (ASCE 7-10)
- Preliminary-Sizing: Principles of Structural Engineering Analysis
- Lateral Load Resisting Systems: Direct Analysis Method (DAM) and Effective Length Method (ELM)
- Composite Deck
- Roof Elements
- Connections

## Building Information and Requirements

### Classifications

- Occupancy: Group B, Office
- Construction: Type IIB
- Risk Category: II
- Seismic Site Class: C
- Environmental Exposure: C

### Building Layout

- Total Area = Approx. 17,000 sf (gross)
- Floor Plan: 110 ft. x 74ft
- Column Grid: 36 ft. x 24 ft.

### Loads

- The building shall be designed in accordance with 2012 IBC structural provisions and as modified by the Massachusetts Building Code (CMR780).
- Snow, wind, and seismic loads shall be calculated for the Lowell, MA location.

# DESIGN CRITERIA

- Minimum Uniform Live Load shall be 100 psf.

### Lateral Load Resisting Systems

- Braced Frame in one direction
- Moment Frame in opposite direction
- Rigid Floor and Roof Diaphragms

### Floor Systems

- 4,000 psi concrete Floor
- Metal deck
- Composite Steel W-shaped A992 beams
- Steel Girders (W)

### Roof Systems

- Single-ply 60 mil EPDM membrane
- 4” rigid insulation
- Metal deck
- Option 1: Open-web steel joists w/ steel girders (W)

### Exterior Wall Systems

- Primary Support: 6” Metal stud
- Exterior cladding: 2” Exterior Insulated Finishing Systems
- Fiberglass Batt. Insulation (6”)

# SCOPE OF WORK

## Scope of Work

### Overview

In order to perform this project, GLZ Design determined the criteria and team mentioned in the previous pages.

During Phase 1 of the design process (30% Submittal), members were given the naming convention mentioned in Figures 1-4 below, load evaluations and analysis were done. Calculations can be found within appendix A, Section A-1 to A-10. Preliminary sizes were determined and are catalogued in Calculations Section A-11.

During Phase 2, preliminary sizes were used to determine the building's lateral load resisting system. New member sections were obtained, results catalogued in Calculations Section A-11, and building framing plans and elevations were drafted. Drawings S-2 to S-4 can be found within appendix B. Further, the building composite deck and roof open-web joists and girders were also designed. Calculations can be found in Section A-21 and A-22, respectively.

During Phase 3, connections and other details were designed. Calculations can be found in Sections A-23 to A-24, while final drawings can be found within appendix B.

### Members

The members in the project were labeled as shown in the figures below:

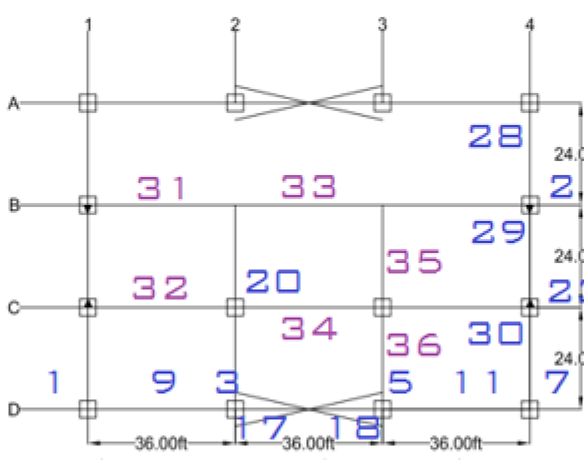



**Figure 1** - First Floor Plan

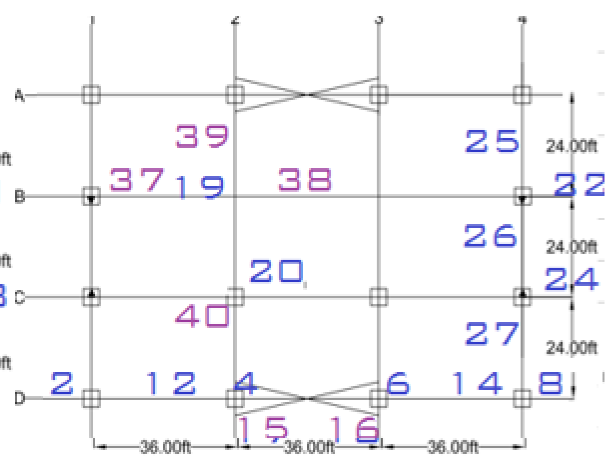



**Figure 2** - Roof Floor Plan

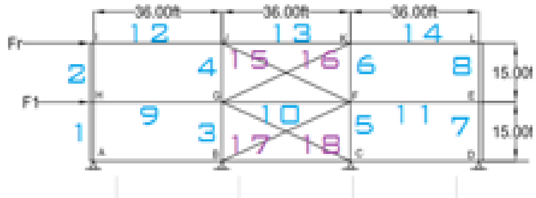



**Figure 3** - Braced-Elevation

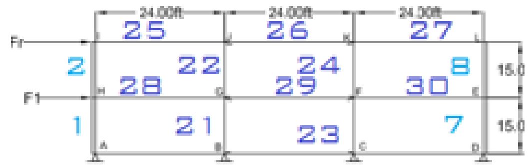



**Figure 4** - Moment Frame - Elevation

## Lateral Load Resisting Systems

The design team designed two different lateral frame systems. Longitudinally a braced frame can be found, while transversely the project uses of a moment frame. The designed connections calculations can be found within Appendix A, section A-30.

## Connections

The designed connections calculations can be found within Appendix A, section A-40.

## Composite Deck & Roof Design

Using a parallel panels to the girders the buildings composite floor consists of 6" of concrete poured in 18" g.a. steel sheets. Calculations are found in appendix A. Further, GLZ Design team determined and used of open-web joists and girders for the roof. Calculations are found in appendix A, section A-50 Additional Details.

## Schedule

The project was completed within schedule.

## Cost Estimate

Attached in Appendix C.

# APPENDIX A: CALCULATIONS

## Appendix A: Calculations

## 1. BUILDING SPECIFICATIONS:

### A. CLASSIFICATIONS:

| | | | | | |
|---|---|---|---|---|---|
| Occupancy: | B | Office | | | |
| Construction Type: | II | B | | | |
| Risk Category: | II | | | | |
| Seismic Site Class: | C | | | Importance Factor: | 1 |
| Environmental Exposure: | C | | | | |

### B. BUILDING LAYOUT

| | | | | | | |
|---|---|---|---|---|---|---|
| Number Columns: | 3 | | | | End Clearance: | 2 in |
| Total Area: | 8140 | $ft^2$ | | | | |
| Floor Plan: | 110 | ft | x | 74 | ft | |
| Column Grid: | 36 | ft | x | 24 | ft | |
| Total Height: | 30 | ft | | | | |
| Finish Floor to Fin. Floor Height: | 15 | ft | | | # Stories: | 2 |
| Structural Allowance: | 4 | ft | | | | |

### C. LATERAL LOAD RESISTING SYSTEMS:

### D. FLOOR SYSTEMS:

| | | | | | |
|---|---|---|---|---|---|
| Concrete Floor Strength: | 4 | ksi | $\gamma =$ | 0.15 | kcf |
| Concrete Floor Thickness: | 6 | in | | | |
| Metal Deck | | | | | |
| Composite Steel Beams: | W | | Type: | A992 | |
| Steel Girders: | W | | | | |

### E. ROOF SYSTEM

| | | |
|---|---|---|
| Single-ply 60 mil EPDM membrane | | |
| Rigid Insulation Thickness: | 4 | in |
| Metal Deck | | |
| Option 1: | Open-web steel joists w/ steel girders (W) | |

### F. EXTERNAL WALL SYSTEM

| | | | | |
|---|---|---|---|---|
| Primary Suport: | Metal Stud | Thickness: | 4 | in |
| Insulation: | Fiberglass Batt. Insulation | Thickness: | 6 | in |
| Exterior Cladding: | Ex. Insulated Finishing Systems | Thickness: | 2 | in |

## 1. BUILDING LOADS AS REQUIRED BY CODE:

| | | | |
|---|---|---|---|
| Loads are in accordance to: | 2012 | IBC | |
| Modified by: | Massachusetts Building Code | | (CMR780) |
| Snow | Lowell | MA | |
| Wind | Lowell | MA | |
| Seismic | Lowell | MA | |
| Construction Live Load: | 20 | psf | |
| Uniform Live Load: | 100 | psf | |

## 2.1 BUILDING VERTICAL LOADS:

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 2.1. Dead Load (D) | 26.0 | psf | 128 | psf |
| 2.2. Live Load (L) | | psf | 100 | psf |
| 2.3. Roof Live Load (Lr) | 20.0 | psf | | psf |
| 2.4. Snow Load (S) | 41.0 | psf | | psf |
| 2.5. Rain Load (R) | | psf | | psf |
| 2.6. Seismic Load (E ) | | psf | | psf |
| 2.7. Wind Load (W) | -23.0 | psf | | psf |
| **Total / Service Load:** | 64.0 | psf | 228 | psf |

## 2.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 1. 1.4D | 36.4 | psf | 179.2 | psf |
| 2. 1.2D + 1.6L + .5(Lr or S or R) | 51.7 | psf | 313.6 | psf |
| 3. 1.2D + 1.6(Lr or S or R) + (L or .5W) | 85.2 | psf | 253.6 | psf |
| 4. 1.2D + 1.0W + L + .5(Lr or S or R) | 28.7 | psf | 153.6 | psf |
| 5. 1.2D + 1.0E + L + .2S | 39.4 | psf | 253.6 | psf |
| 6. 0.9D + 1.0W | 0.4 | psf | 115.2 | psf |
| 7. 0.9D + 1.0E | 23.4 | psf | 115.2 | psf |
| **Controlling Load:** | 85.2 | psf | 313.6 | psf |

## 3.1 BUILDING LATERAL LOAD ON LONGITUDINAL DIRECTION: BRACED-FRAME

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 2.1. Dead Load (D) | | psf | | psf |
| 2.2. Live Load (L) | | psf | | psf |
| 2.3. Roof Live Load (Lr) | | psf | | psf |
| 2.4. Snow Load (S) | | psf | | psf |
| 2.5. Rain Load (R) | | psf | | psf |
| 2.6. Seismic Load (E ) | 20.8 | psf | 20.4 | psf |
| 2.7. Wind Load (W) | 6.4 | psf | 12.9 | psf |
| **Total / Service Load:** | 27.2 | psf | 33.3 | psf |

**3.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:**

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 1. 1.4D | 0.0 | psf | 0.0 | psf |
| 2. 1.2D + 1.6L + .5(Lr or S or R) | 0.0 | psf | 0.0 | psf |
| 3. 1.2D + 1.6(Lr or S or R) + (L or .5W) | 3.2 | psf | 6.4 | psf |
| 4. 1.2D + 1.0W + L + .5(Lr or S or R) | 6.4 | psf | 12.9 | psf |
| 5. 1.2D + 1.0E + L + .2S | 20.8 | psf | 20.4 | psf |
| 6. 0.9D + 1.0W | 6.4 | psf | 12.9 | psf |
| 7. 0.9D + 1.0E | 20.8 | psf | 20.4 | psf |
| **Controlling Load:** | 20.8 | psf | 20.4 | psf |

**4.1 BUILDING LATERAL LOAD ON TRANSVERSE DIRECTION: MOMENT-FRAME**

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 2.1. Dead Load (D) | | psf | | psf |
| 2.2. Live Load (L) | | psf | | psf |
| 2.3. Roof Live Load (Lr) | | psf | | psf |
| 2.4. Snow Load (S) | | psf | | psf |
| 2.5. Rain Load (R) | | psf | | psf |
| 2.6. Seismic Load (E ) | 17.6 | psf | 17.0 | psf |
| 2.7. Wind Load (W) | 9.8 | psf | 19.6 | psf |
| **Total / Service Load:** | 27.3 | psf | 36.6 | psf |

**4.2 LOAD COMBINATIONS PER LRFD SPECIFICATIONS:**

| | Roof | | 1st Floor | |
|---|---|---|---|---|
| 1. 1.4D | 0.0 | psf | 0.0 | psf |
| 2. 1.2D + 1.6L + .5(Lr or S or R) | 0.0 | psf | 0.0 | psf |
| 3. 1.2D + 1.6(Lr or S or R) + (L or .5W) | 4.9 | psf | 9.8 | psf |
| 4. 1.2D + 1.0W + L + .5(Lr or S or R) | 9.8 | psf | 19.6 | psf |
| 5. 1.2D + 1.0E + L + .2S | 17.6 | psf | 17.0 | psf |
| 6. 0.9D + 1.0W | 9.8 | psf | 19.6 | psf |
| 7. 0.9D + 1.0E | 17.6 | psf | 17.0 | psf |
| **Controlling Load:** | 17.6 | psf | 19.6 | psf |

| Reference: | ASCE | 7-10 |
|---|---|---|
| Section | *Eq/Fig/Table/Notes* | |

## DEAD LOAD

3

| Floor: | Roof | | | | | 2nd | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Item** | **Quantity** *(Area)* | **Units** | **Unit Weight** *(ksf or klf)* | **Weight** *(kip)* | **Item** | **Quantity** *(Area)* | **Units** | **Unit Weight** *(ksf or klf)* | **Weight** *(kip)* |
| | | | | | | **Concrete Slab** | 7776 | *sf* | 0.075 | 583 |
| | **Metal Deck** | 7776 | *sf* | 0.014 | 109 | **Metal Deck 18 g.a** | 7776 | *sf* | 0.014 | 109 |
| | **EPDM Membrane** | 7776 | *sf* | 0.001 | 8 | **Cladding** | 7776 | *sf* | 0.01 | 78 |
| | **Insulation** | 7776 | *sf* | 0.006 | 47 | **Partitions** | 7776 | *sf* | 0.01 | 78 |
| | **Mechanical Equipment** | 7776 | *sf* | 0.005 | 39 | **Mechanical Equipment** | 7776 | *sf* | 0.007 | 54 |
| | | | | | | **Steel Structure** | 7776 | *sf* | 0.012 | 93 |
| **Subtotal** | *202* | | | 0.026 | 202 | *995* | | | 0.128 | 995 |
| **Cummulative** | *202* | | | | 202 | *1198* | | | | **1198** |

***Unit Weights per ASCE 7-10**

| Reference: | ASCE | 7-10 |
|---|---|---|
| Section | *Eq/Fig/Table/Notes* | |

## SNOW LOAD

7

| | | | | | |
|---|---|---|---|---|---|
| Exposure Factor | $C_e$ = | 0.90 | | Table | 7-2 |
| Thermal Factor | $C_t$ = | 1.00 | | Table | 7-3 |
| Importance Factor | $I_s$ = | 1.00 | | Table | 1.5-2 |
| Ground Snow Load | $\rho_g$ = | 65.00 | psf | CMR | 780 |
| **Flat Roof Snow Load** | $\rho_f$ = | 40.95 | psf | Eq | 7.3-1 |

| Reference: | ASCE | 7-10 |
|---|---|---|

| Section | *Eq/Fig/Table/Notes* |
|---|---|
| 12 | |

**SEISMIC LOAD**

**Number of Floors:** 2

| **Floor:** | **Roof** | | | | | **2nd** | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| | **Item** | **Quantity** *(Area)* | **Units** | **Unit Weight** *(ksf or klf)* | **Weight** *(kip)* | **Item** | **Quantity** *(Area)* | **Units** | **Unit Weight** *(ksf or klf)* | **Weight** *(kip)* |
| | | | | | | **Concrete Slab** | 7776 | *sf* | 0.075 | 583 |
| | **Metal Deck** | 7776 | *sf* | 0.014 | 109 | **Metal Deck** | 7776 | *sf* | 0.014 | 109 |
| | **EPDM Membrane** | 7776 | *sf* | 0.001 | 8 | **Cladding** | 7776 | *sf* | 0.01 | 78 |
| | **Insulation** | 7776 | *sf* | 0.006 | 47 | **Partitions** | 7776 | *sf* | 0.01 | 78 |
| | **Mechanical Equipment** | 7776 | *sf* | 0.005 | 39 | **Mechanical Equipment** | 7776 | *sf* | 0.007 | 54 |
| | **Snow** | 7776 | *sf* | 0.04095 | 318 | **Steel Structure** | 7776 | *sf* | 0.012 | 93 |
| **Subtotal** | *521* | | | | **521** | *995* | | | | **995** |
| **Cummulative** | *521* | | | | **521** | *1516* | | | | **1516** |
| Mass (kip*s^2/ft) | | | | | 107 | | | | | 58 |

## WIND LOAD ANALYSIS

Reference: ASCE 7-10

| | | | | | | |
|---|---|---|---|---|---|---|
| | | | | Section | Eq/Fig/Table/Notes | |
| **1. BUILDING INFORMATION RELATED TO WIND LOAD ANALYSIS** | | | | 26/27/28 | | |
| Mean roof height | $H_{roof}$ = | 30 | ft | *Height of heighest level of structure* | | |
| Floor-Floor Height | $h_n$ = | 15 | ft | | | |
| Building Length | L = | 112 | ft | | | |
| Building Width | W = | 76 | ft | | | |
| Number of Braces/Level | | 2 | | | | |
| Number of Moment Frames/ Level | | 2 | | | | |
| **2. WIND EXPOSURE, ROUGHNESS AND OCCUPANCY CATEGORY** | | | | 26.4 | | |
| Occupancy Category: | | B | | | Table | 1-1 |
| Ground Surface Roughness: | | B | | | | 26.7.2 |
| Exposure Category: | | C | | | | 26.7.2 |
| **3. ENVIRONMENTAL CHARACTERISTICS AND FACTORS** | | | | 26.5 | | |
| Wind Speed | V = | 120 | mph | 26.5 | Figure | 26.5-1A |
| Zone A | $P_{s30}$ = | 22.8 | psf | | Figure | 28.6-1 |
| Zone C | $P_{s30}$ = | 15.1 | psf | | Figure | 28.6-1 |
| | $a_1$ = | 7.6 | ft | *.1*W* | | |
| | $a_2$ = | 12 | ft | *.4*$H_{roof}$* | | |
| | a = | 7.6 | ft | *Min Value* | | |
| | 2.a = | 15.2 | ft | | | |
| **Weighted Average for $P_{s30}$:** | | | | | | |
| Longitudinal | | 16.1 | psf | | | |
| Transverse | | 16.6 | psf | | | |
| **4. DESIGN WIND PRESSURE** | | | | 26.8 | | |
| Adjustment Factor | $\lambda$ = | 1.4 | | | Figure | 28.6-1 |
| | $K_{zt}$ = | 1 | | *26.8* | | |
| Design wind pressure, | $P_{s\text{-longitudinal}}$ = | 22.6 | kip | | | |
| Design wind pressure, | $P_{s\text{-transverse}}$ = | 23.3 | kip | | | |
| **5. LOAD APPLIED TO EACH LEVEL** | | | | 26.8 | | |
| **Roof** | $F_{u\text{-longitudinal}}$ = | 12.9 | kip | | Figure | 28.6-1 |
| | $F_{u\text{-transverse}}$ = | 19.6 | kip | *26.8* | | |

| | | | |
|---|---|---|---|
| **Level 1** | $F_{u\text{-longitudinal}}$ = | 25.8 | kip |
| | $F_{u\text{-transverse}}$ = | 39.1 | kip |

## 6. LATERAL LOAD APPLIED TO BRACED AND MOMENT FRAME

| | | Roof | Level 1 | |
|---|---|---|---|---|
| **Braced Frame** | (Longitudinal) | 6.44 | 12.88 | kip |
| **Moment Frame** | (Transverse) | 9.78 | 19.57 | kip |

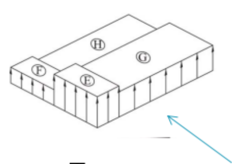
## 7. VERTICAL UPLIFT PRESSURES ON ROOF

| | | | | |
|---|---|---|---|---|
| Zone E $P_{s30}$ | -27.4 | psf | Figure | 28.6-1 |
| Zone F $P_{s30}$ | -15.6 | psf | Figure | 28.6-1 |
| Zone G $P_{s30}$ | -19.1 | psf | Figure | 28.6-1 |
| Zone H $P_{s30}$ | -12.1 | psf | Figure | 28.6-1 |

| | | | |
|---|---|---|---|
| Design wind pressure Zone E, | $P_s$ | -38.36 | psf |
| Design wind pressure Zone F, | $P_s$ | -21.84 | psf |
| Design wind pressure Zone G, | $P_s$ | -26.74 | psf |
| Design wind pressure Zone H, | $P_s$ | -16.94 | psf |

## 7. UPLIFT PRESSURE (TRANSVERSE LOADING)

| | | |
|---|---|---|
| Area, Zone E | 577.6 | $ft^2$ |
| Area, Zone F | 577.6 | $ft^2$ |
| Area, Zone G | 3678.4 | $ft^2$ |
| Area, Zone H | 3678.4 | $ft^2$ |
| Total Roof Area | 8512 | $ft^2$ |

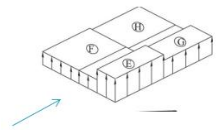


Transverse

| | | |
|---|---|---|
| Weighted Uplift Pressure from Transverse Wind Load | -22.96 | psf |

## 8. UPLIFT PRESSURE (LONGITUDINAL LOADING)

| | | |
|---|---|---|
| Area, Zone E | 851.2 | $ft^2$ |
| Area, Zone F | 851.2 | $ft^2$ |
| Area, Zone G | 3404.8 | $ft^2$ |
| Area, Zone H | 3404.8 | $ft^2$ |
| Total Roof Area | 8512 | $ft^2$ |



Longitudinal

| | | |
|---|---|---|
| Weighted Uplift Pressure from Longitudinal Wind Load | -23.49 | psf |

## 9. MAXIMUM UPLIFT PRESSURE

| | | | |
|---|---|---|---|
| Controlling Uplift Pressure | -23.49 | psf | Largest Absolute Value |

**ASSUMPTIONS:**

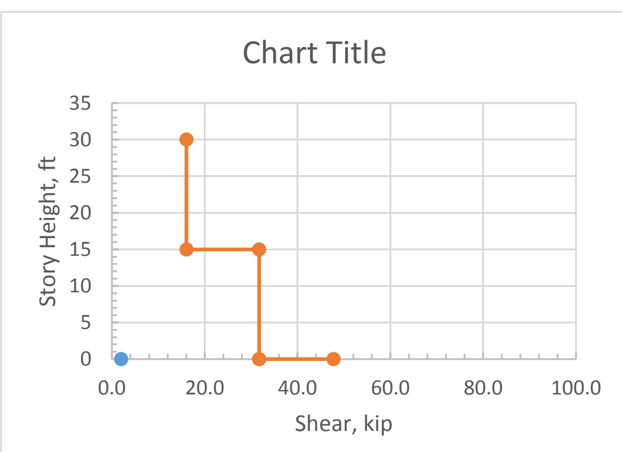
Building Frame System: Steel moment-resisting frame

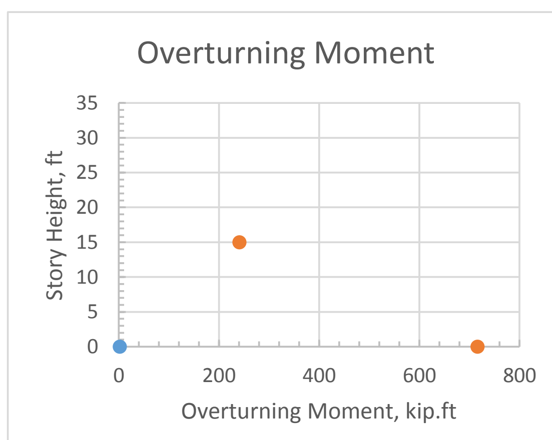
| | | | Reference: Section | ASCE 7-10 Eq/Fig/Table/Notes | |
|---|---|---|---|---|---|
| **1. SEISMIC GROUD MOTION VALUES** | | | 11.4 | | |
| **Seismic Site Class:** | C | | 11.4.2 | *Soil Properties / Ch. 20* | |
| **Maximum Considered Earthquake Spectral Response:** | | | | | |
| $S_s$ = | 0.250 | | 11.4.1 | Fig | 22-1 / 22-4 |
| $S_1$ = | 0.077 | | 11.4.1 | Fig | 22-1 / 22-4 |
| **Adjusted MCE Spectral Response:** | | | | | |
| $F_a$ = | 1.200 | | 11.4.3 | Table | 11.4-1 |
| $F_v$ = | 1.700 | | 11.4.3 | Table | 11.4-2 |
| $S_{MS} = F_a S_s$ = | 0.3 | | 11.4.3 | Eq | 11.4-1 |
| $S_{M1} = F_v S_1$ = | 0.131 | | 11.4.3 | Eq | 11.4-2 |
| **Design Spectral Response Acceleration Parameters:** | | | | | |
| $S_{DS} = 2/3\ S_{MS}$ = | 0.2 | | 11.4.4 | Eq | 11.4-3 |
| $S_{D1} = 2/3\ S_{M1}$ = | 0.087 | | 11.4.4 | Eq | 11.4-4 |
| **Design Response Spectrum:** | | | | | |
| $T_O = 0.2\ S_{D1}/S_{DS}$ = | | s | 11.4.5 | | |
| $T_S = S_{D1}/S_{DS}$ = | 0.436 | s | 11.4.5 | | |
| Long Period Transition $T_L$= | 6 | s | 11.4.5 | Fig | 22-15 |
| T = | 0.60 | s | *Fundamental Period of Structure* | | |
| $S_a$ = if $T < T_O : S_{DS}(0.4+0.6T/T_O)$ = | | | 11.4.5 | Eq | 11.4-5 |
| if $T_O < T < T_S : S_{DS}$ = | | | 11.4.5 | | |
| if $T_S < T < T_L : S_{D1}/T$ = | 0.147 | | 11.4.5 | Eq | 11.4-6 |
| if $T > T_L : S_{D1} * T_L / T^2$= | | | 11.4.5 | Eq | 11.4-7 |

| | | | | |
|---|---|---|---|---|
| **2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY** | | 11.5 | | |
| Occupancy Category: | II | | Table | 1-1 |
| Importance Factor: | 1 | | Table | 11.5-1 |
| **3. SEISMIC DESIGN CATEGORY** | | 11.6 | | |
| SDC based on short period: | B | | Table | 11.6-1 |
| SDS based on 1-s period: | B | | Table | 11.6-2 |
| SDC = | **B** | *Maximum from values above* | | |
| **4. EQUIVALENT LATERAL FORCE PROCEDURE** | | 12.8 | | |
| R = | 8 | 12.8.1 | Table | 12.2-1 |
| $\Omega_O$ = | 3 | 12.8.1 | Table | 12.2-1 |
| $C_D$ = | 3 | 12.8.1 | Table | 12.2-1 |

| | | | | | |
|---|---|---|---|---|---|
| **Approximate Fundamental Period, $T_a$:** | | | 12.8.2.1 | | |
| $C_t$ = | 0.028 | | *Dependent on structur* | Table | 12.8-2 |
| x = | 0.8 | | | Table | 12.8-2 |
| $h_n$ = | 30 | ft | *Height of heighest level of structure* | | |
| $T_a = C_t h_n^{\ x}$ = | 0.425 | s | 12.8.2.1 | Eq | 12.8-7 |
| **Seismic Response Coefficient:** | | | 12.8.1.1 | | |
| $C_{Scalc} = S_{DS}/(R/I)$ = | 0.025 | | | Eq | 12.8-2 |
| $C_{Smax}$ = if T <= $T_L$ : $S_{D1}/(T*(R/I)$ = | 0.018 | | | Eq | 12.8-3 |
| if T > $T_L$ : $S_{D1*}T_L/(T^2*(R/I)$ = | | | | Eq | 12.8-4 |
| $C_{Smin}$ = | 0.01 | | *Revised Sup. 2* | Eq | 12.8-5/12.8-6 |
| $C_S$ = | 0.025 | | *Beware of mins and max* | | |
| **Seismic Base Shear:** | | | 12.8.1 | | |
| Seismic Weight W = | 1268 | kip | 12.7.2 | | Table Below |
| Seismic Base Shear V = | 31.7 | kip | 12.8.1 | Eq | 12.8-1 |
| **Vertical Distribution of Seismic Forces:** | | | 12.8.3 | | |
| Lateral force per level | $F_x = C_{vx}V$ | | | Eq | 12.8-11 |
| | $C_{vx} = (w_x h_x^{\ k})/(\Sigma\ w_i h_i^{\ k})$ | | *Vertical Distribution Factor* | | |
| k = | 1.00 | 0.96 | 12.8.3 | | |
| **Horizontal Distribution of Seismic Forces:** | | | 12.8.4 | | |
| $V_x = \Sigma\ F_i$ = | | | | Eq | 12.8-13 |

| Floor | Height (ft) | Weight (kip) | $w_x h_x^{\ k}$ | $C_{vx}$ | $F_x$ (kip) | $V_x$ (kip) | Overturning Moment (kip.ft) | Total Height (ft) |
|---|---|---|---|---|---|---|---|---|
| **Roof** | 15 | 430 | 12894 | 0.506 | 16.1 | 16.1 | | 30 |
| | | | | | | 16.1 | | 15 |
| **2nd** | 15 | 838 | 12576 | 0.494 | 15.7 | 31.7 | 241 | 15 |
| | | | | | | 31.7 | | 0 |
| **Podium** | 0 | 0 | 0 | 0 | 0 | 31.7 | 716 | 0 |
| **SUM** | 30 | 1268 | 25470 | 1 | 31.7 | 47.8 | | 0 |



**ASSUMPTIONS:**

Building Frame System: Eccentrically braced steel frame

| | | | Reference: Section | ASCE 7-10 Eq/Fig/Table/Notes | |
|---|---|---|---|---|---|
| **1. SEISMIC GROUD MOTION VALUES** | | | 11.4 | | |
| **Seismic Site Class:** | C | | 11.4.2 | *Soil Properties / Ch. 20* | |
| **Maximum Considered Earthquake Spectral Response:** | | | | | |
| $S_s$ = | 0.250 | | 11.4.1 | Fig | 22-1 / 22-4 |
| $S_1$ = | 0.077 | | 11.4.1 | Fig | 22-1 / 22-4 |
| **Adjusted MCE Spectral Response:** | | | | | |
| $F_a$ = | 1.200 | | 11.4.3 | Table | 11.4-1 |
| $F_v$ = | 1.700 | | 11.4.3 | Table | 11.4-2 |
| $S_{MS} = F_aS_s$ = | 0.300 | | 11.4.3 | Eq | 11.4-1 |
| $S_{M1} = F_vS_1$ = | 0.131 | | 11.4.3 | Eq | 11.4-2 |
| **Design Spectral Response Acceleration Parameters:** | | | | | |
| $S_{DS} = 2/3\ S_{MS}$ = | 0.2 | | 11.4.4 | Eq | 11.4-3 |
| $S_{D1} = 2/3\ S_{M1}$ = | 0.087 | | 11.4.4 | Eq | 11.4-4 |
| **Design Response Spectrum:** | | | | | |
| $T_O = 0.2\ S_{D1}/S_{DS}$ = | | s | 11.4.5 | | |
| $T_S = S_{D1}/S_{DS}$ = | 0.436 | s | 11.4.5 | | |
| Long Period Transition $T_L$= | 6 | s | 11.4.5 | Fig | 22-15 |
| T = | 0.54 | s | *Fundamental Period of Structure* | | |
| $S_a$ = if $T < T_O : S_{DS}(0.4+0.6T/T_O)$ = | | | 11.4.5 | Eq | 11.4-5 |
| if $T_O < T < T_S : S_{DS}$ = | | | 11.4.5 | | |
| if $T_S < T < T_L : S_{D1}/T$ = | 0.162 | | 11.4.5 | Eq | 11.4-6 |
| if $T > T_L : S_{D1}{*}T_L/T^2$= | | | 11.4.5 | Eq | 11.4-7 |

| | | | | | |
|---|---|---|---|---|---|
| **2. IMPORTANCE FACTOR AND OCCUPANCY CATEGORY** | | 11.5 | | | |
| Occupancy Category: | II | | Table | 1-1 | |
| Importance Factor: | 1 | | Table | 11.5-1 | |
| **3. SEISMIC DESIGN CATEGORY** | | 11.6 | | | |
| SDC based on short period: | B | | Table | 11.6-1 | |
| SDS based on 1-s period: | B | | Table | 11.6-2 | |
| SDC = | **B** | *Maximum from values above* | | | |
| **4. EQUIVALENT LATERAL FORCE PROCEDURE** | | 12.8 | | | |
| R = | 6 | 12.8.1 | Table | 12.2-1 | |
| $\Omega_O$ = | 2 | 12.8.1 | Table | 12.2-1 | |
| $C_D$ = | 3.25 | 12.8.1 | Table | 12.2-1 | |

**Approximate Fundamental Period, $T_a$:** 12.8.2.1

| | | | | | |
|---|---|---|---|---|---|
| $C_t$ = | 0.03 | | *Dependent on structure* | Table | 12.8-2 |
| x = | 0.75 | | | Table | 12.8-2 |
| $h_n$ = | 30 | ft | *Height of heighest level of structure* | | |
| $T_a = C_t h_n^x$ = | 0.385 | s | 12.8.2.1 | Eq | 12.8-7 |

**Seismic Response Coefficient:** 12.8.1.1

| | | | | |
|---|---|---|---|---|
| $C_{Scalc} = S_{DS}/(R/I)$ = | 0.033 | | Eq | 12.8-2 |
| $C_{Smax}$ = if T <= $T_L$ : $S_{D1}/(T*(R/I))$ = | 0.027 | | Eq | 12.8-3 |
| if T > $T_L$ : $S_{D1*}T_L/(T^2*(R/I))$ = | | | Eq | 12.8-4 |
| $C_{Smin}$ = | 0.01 | *Revised Sup. 2* | Eq | 12.8-5/12.8-6 |
| $C_S$ = | 0.033 | | | |

**Seismic Base Shear:** 12.8.1

| | | | | | | |
|---|---|---|---|---|---|---|
| Seismic Weight | W = | 1268 | kip | 12.7.2 | | Table Below |
| Seismic Base Shear | V = | 42.3 | kip | 12.8.1 | Eq | 12.8-1 |

**Vertical Distribution of Seismic Forces:** 12.8.3

Lateral force per level $F_x = C_{vx}V$ Eq 12.8-11

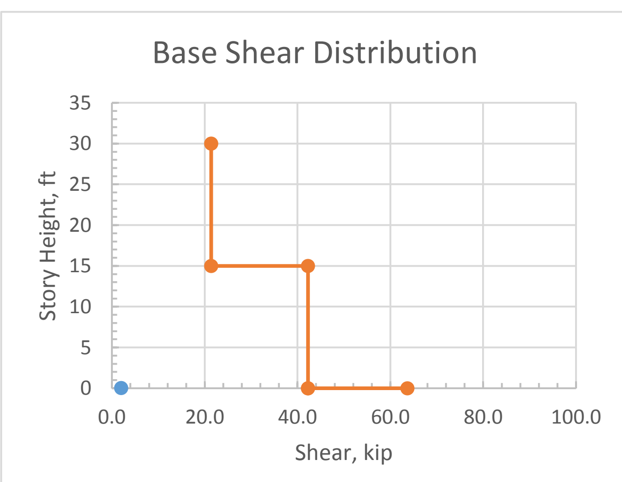
$C_{vx} = (w_x h_x^k)/(\Sigma\ w_i h_i^k)$ *Vertical Distribution Factor*

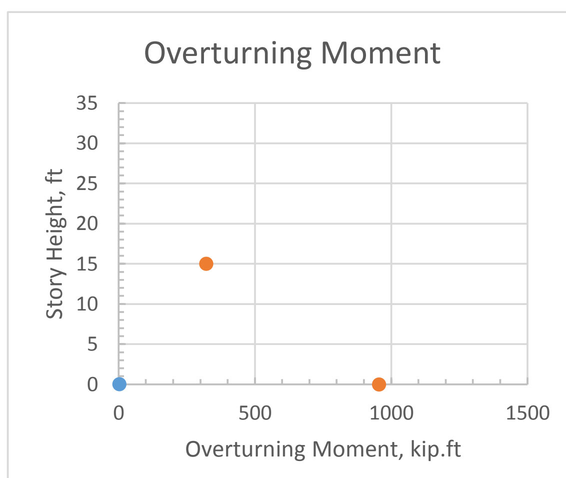
k = 1.00 0.94 12.8.3

**Horizontal Distribution of Seismic Forces:** 12.8.4

$V_x = \Sigma\ F_i$ = Eq 12.8-13

| Floor | Height (ft) | Weight (kip) | $w_x h_x^k$ | $C_{vx}$ | $F_x$ (kip) | $V_x$ (kip) | Overturning Moment (kip.ft) | Total Height (ft) |
|---|---|---|---|---|---|---|---|---|
| **Roof** | 15 | 430 | 12894 | 0.506 | 21.4 | 21.4 | | 30 |
| | | | | | | 21.4 | | 15 |
| **2nd** | 15 | 838 | 12576 | 0.494 | 20.9 | 42.3 | 321 | 15 |
| | | | | | | 42.3 | | 0 |
| **Podium** | 0 | 0 | 0 | 0 | 0 | 42.3 | 955 | 0 |
| **SUM** | 30 | 1268 | 25470 | 1 | 42.3 | 63.7 | | 0 |



## 1. MOMENT FRAME INFORMATION

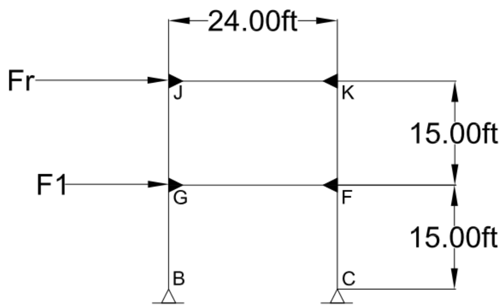



**Figure 1** - Moment Frame

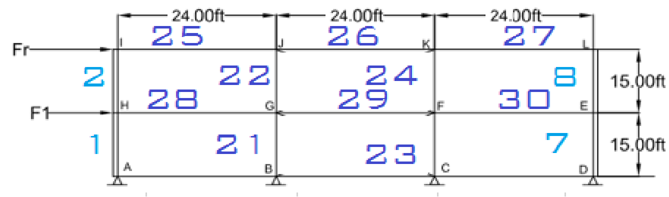



**Figure 2** - Member Reference

| | | |
|---|---|---|
| Story Height, H = | 15 | ft |
| Bay Width, W = | 24 | ft |
| $F_{r} =$ | 9.78 | kip |
| $F_{1} =$ | 19.57 | kip |

## 2.1 MEMBER FORCES DISTRIBUTION

**Compute Shear Forces**

$\sum F_x=0$

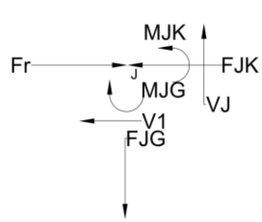
| | | |
|---|---|---|
| $F_{r} =$ | 9.78 | kips |
| # of shear forces | 2 | |
| $V_{1} =$ | 4.89 | kips |

**Compute moments at top of columns**

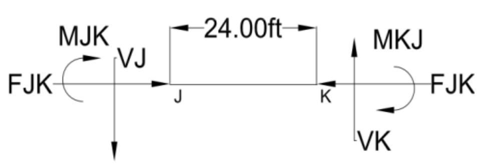
| | | |
|---|---|---|
| $M_{JG} =$ | 36.69 | kip*ft |
| $M_{KF} =$ | 36.69 | kip*ft |

## METHOD OF JOINTS:

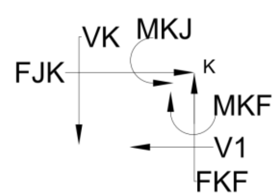
Joint J




Member J-K




Joint K



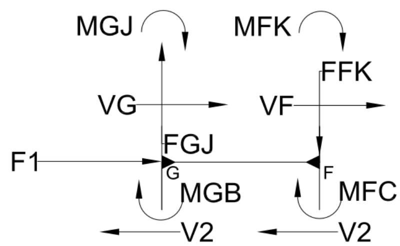

**Joint J:**

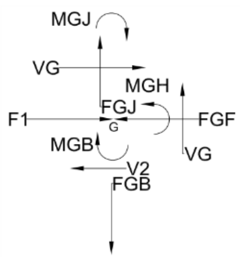
| | | | | |
|---|---|---|---|---|
| | $M_{JK} =$ | 36.69 | kip*ft | |
| $\sum Fx=0$ | $F_{JK} =$ | 4.89 | kips | C |
| $\sum M/W$ | $V_{J} =$ | 3.06 | kips | |
| $\sum Fy=0$ | $F_{JG} =$ | 3.06 | kips | T |

**Joint K:**

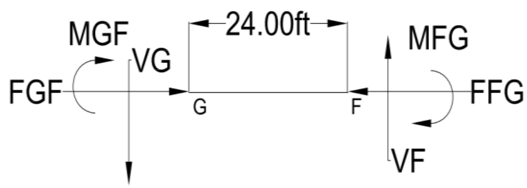
| | | | |
|---|---|---|---|
| $M_{KJ} =$ | 36.69 | kip*ft | |
| $F_{JK} =$ | 4.89 | kips | C |
| $V_{K} =$ | 3.06 | kips | |
| $F_{KF} =$ | 3.06 | kips | C |
| $M_{KF} =$ | 36.69 | kips | |

**Section 2**

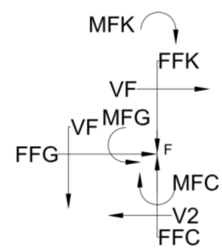



**Joint G**




**Member G-F**




Joint F

**Joint G:**

| | | | | |
|---|---|---|---|---|
| (F1+FR)/2 | $V_{2} =$ | 14.68 | kips | |
| FJG | $F_{GJ} =$ | 3.06 | kips | T |
| V2*H/2 | $M_{GB} =$ | 110.1 | kip*ft | |
| (MJG) | $M_{GJ} =$ | 36.69 | kip*ft | |
| (FJG) | $F_{GJ} =$ | 3.06 | kips | |
| $\sum$Fx=0 | $F_{GF} =$ | 4.89 | kips | C |
| $\sum$M | $M_{GF} =$ | 146.76 | kip*ft | |
| $\sum$M/W | $V_{G} =$ | 12.23 | kips | |
| $\sum$Fy=0 | $F_{GB} =$ | 15.29 | kips | |

**Joint F**

| | | | | |
|---|---|---|---|---|
| (F1+FR)/2 | $V_{2} =$ | 14.68 | kips | |
| FKF | $F_{FK} =$ | 3.06 | kips | T |
| V2*H/2 | $M_{FC} =$ | 110.1 | kip*ft | |
| MKF | $M_{FK} =$ | 36.69 | kip*ft | |
| FKF | $F_{FK} =$ | 3.06 | kips | |
| FGF | $F_{FG} =$ | 4.89 | kips | C |
| $\sum$M | $M_{FG} =$ | 146.76 | kip*ft | |
| $\sum$M/W | $V_{G} =$ | 12.23 | kips | |
| $\sum$Fy=0 | $F_{FC} =$ | 15.29 | kips | |

## 3. RESULTS

| | Member (#) | Frame (type) | Floor (Units) | Function | Force (kip) | T/C | Moment (kip.ft) |
|---|---|---|---|---|---|---|---|
| GF | 29 | Moment | First | Beam | 4.89 | C | 146.76 |
| JK | 26 | Moment | Roof | Beam | 4.89 | C | 36.69 |
| BG | 21 | Moment | First | Column | 15.29 | T | 110.07 |
| CF | 23 | Moment | First | Column | 15.29 | C | 110.07 |
| JG | 22 | Moment | Roof | Column | 4.89 | T | 36.69 |
| KF | 24 | Moment | Roof | Column | 4.89 | C | 36.69 |

## 1. MOMENT FRAME INFORMATION

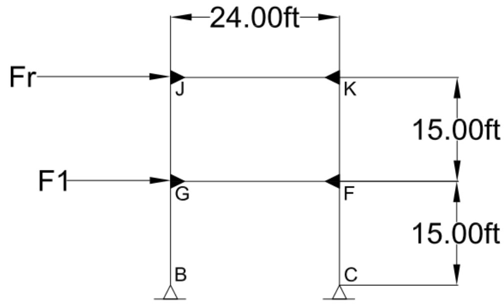



**Figure 1** - Moment Frame

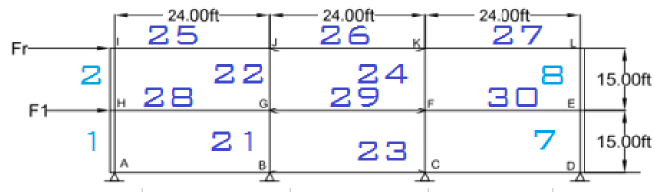



**Figure 2** - Member Reference

| | | |
|---|---|---|
| Story Height, H = | 15 | ft |
| Bay Width, W = | 24 | ft |
| $F_{r\,=}$ | 3.23 | kip |
| $F_{1\,=}$ | 12.62 | kip |

### 2.1 MEMBER FORCES DISTRIBUTION

**Compute Shear Forces**

$\sum F_x=0$

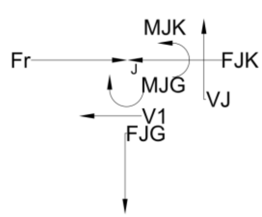
| | | |
|---|---|---|
| $F_{r\,=}$ | 3.23 | kips |
| # of shear forces | 2 | |
| $V_{1\,=}$ | 1.62 | kips |

**Compute moments at top of columns**

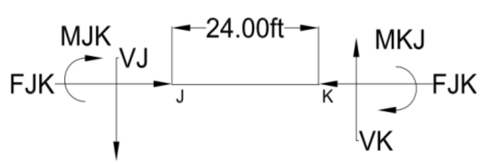
| | | |
|---|---|---|
| $M_{JG\,=}$ | 12.11 | kip*ft |
| $M_{KF\,=}$ | 12.11 | kip*ft |

### METHOD OF JOINTS:

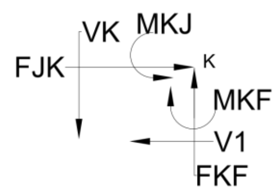
Joint J




Member J-K




Joint K



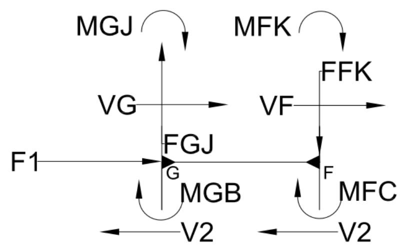

**Joint J:**

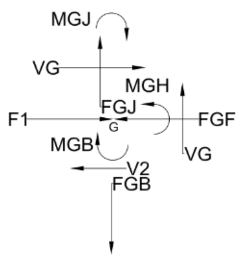
| | | | | |
|---|---|---|---|---|
| | $M_{JK\,=}$ | 12.11 | kip*ft | |
| $\sum$Fx=0 | $F_{JK\,=}$ | 1.62 | kips | C |
| $\sum$M/W | $V_{J\,=}$ | 1.01 | kips | |
| $\sum$Fy=0 | $F_{JG\,=}$ | 1.01 | kips | T |

**Joint K:**

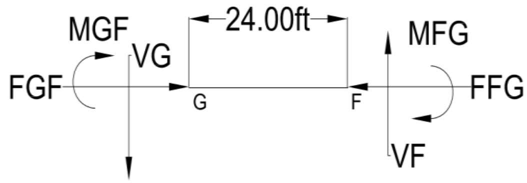
| | | | |
|---|---|---|---|
| $M_{KJ\,=}$ | 12.11 | kip*ft | |
| $F_{JK\,=}$ | 1.62 | kips | C |
| $V_{K\,=}$ | 1.01 | kips | |
| $F_{KF\,=}$ | 1.01 | kips | C |
| $M_{KF\,=}$ | 12.11 | kips | |

**Section 2**

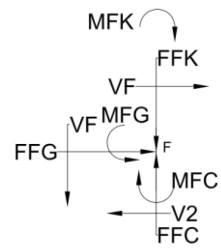



**Joint G**




**Member G-F**




Joint F

**Joint G:**

| | | | | |
|---|---|---|---|---|
| (F1+FR)/2 | $V_{2=}$ | 7.9 | kips | |
| FJG | $F_{GJ=}$ | 1.01 | kips | T |
| V2*H/2 | $M_{GB=}$ | 59.44 | kip*ft | |
| (MJG) | $M_{GJ=}$ | 12.11 | kip*ft | |
| (FJG) | $F_{GJ=}$ | 1.01 | kips | |
| ∑Fx=0 | $F_{GF=}$ | 4.70 | kips | T |
| ∑M | $M_{GF=}$ | 71.6 | kip*ft | |
| ∑M/W | $V_{G=}$ | 6.0 | kips | |
| ∑Fy=0 | $F_{GB=}$ | 7.0 | kips | |

**Joint F**

| | | | | |
|---|---|---|---|---|
| (F1+FR)/2 | $V_{2=}$ | 7.9 | kips | |
| FKF | $F_{FK=}$ | 1.0 | kips | C |
| V2*H/2 | $M_{FC=}$ | 59.4 | kip*ft | |
| MKF | $M_{FK=}$ | 12.1 | kip*ft | |
| FKF | $F_{FK=}$ | 1.01 | kips | |
| FGF | $F_{FG=}$ | 4.70 | kips | T |
| ∑M | $M_{FG=}$ | 71.6 | kip*ft | |
| ∑M/W | $V_{G=}$ | 6.0 | kips | |
| ∑Fy=0 | $F_{FC=}$ | 7.0 | kips | |

### 3. RESULTS

| | Member<br>*(#)* | Frame<br>*(type)* | Floor<br>*(Units)* | Function | Force<br>*(kip)* | T/C | Moment<br>*(kip.ft)* |
|---|---|---|---|---|---|---|---|
| GF | 29 | Moment | First | Beam | 4.70 | T | 71.55 |
| JK | 26 | Moment | Roof | Beam | 1.62 | C | 12.11 |
| BG | 21 | Moment | First | Column | 6.97 | T | 59.44 |
| CF | 23 | Moment | First | Column | 6.97 | C | 59.44 |
| JG | 22 | Moment | Roof | Column | 4.70 | T | 12.11 |
| KF | 24 | Moment | Roof | Column | 4.70 | C | 12.11 |

## 1. MOMENT FRAME INFORMATION

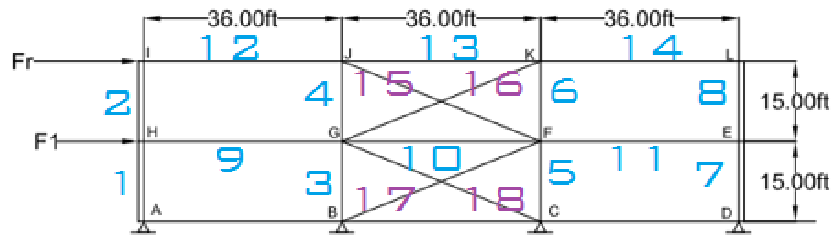



**Figure 1** - Braced-Frame Member Reference

| | | | |
|---|---|---|---|
| Diagonal, D | 39 | ft | |
| Story Height, H | 15 | ft | |
| Bay Width, W | 36 | ft | |
| $F_r$ | 9.78 | kip | 6.44 |
| $F_1$ | 19.57 | kip | 0.00 |

## 2.1 MEMBER FORCES DISTRIBUTION

**Find Reactions**

| | | | |
|---|---|---|---|
| Cy= | | kip | up |
| By= | 0.00 | kip | down |
| Cx= | -14.68 | kip | west |
| Bx= | -14.68 | kip | west |

**Joint B**

| | | | | |
|---|---|---|---|---|
| (Fr+F1)/2 | $V_1$ | 14.68 | kip | |
| | $F_{BFy}$ | -6.12 | kip | |
| Brace Force | $F_{BF}$ | 15.90 | kip | T |
| Vertical Force | $F_{BG}$ | -6.1152 | kip | T |

**Joint C**

| | | | |
|---|---|---|---|
| $V_1$ | 14.68 | kip | |
| $F_{GCy}$ | -6.12 | kip | C |
| $F_{GC}$ | 15.90 | kip | C |
| $F_{FC}$ | -6.12 | kip | C |

**To solve system:**

| | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|
| Moment Equation | 15 | $F_{GF}$ | 27.69 | $F_{GK}$ | -36 | $F_{GJ}$ | = | -72.06 | 513.69 |
| Forces in X | -1 | $F_{GF}$ | -0.923 | $F_{GK}$ | 0 | $F_{GJ}$ | = | -4.8 | -4.89 |
| Forces in Y | 0 | $F_{GF}$ | -0.385 | $F_{GK}$ | 1 | $F_{GJ}$ | = | 4 | -12 |

**Inverse Matrix**

| | | |
|---|---|---|
| 61.533 | 922 | 2215.2 |
| -66.666 | -1000 | -2400.0 |
| -25.666 | -385 | -923.0 |

**Solution**

| | |
|---|---|
| $F_{GF}$ | 1.13 |
| $F_{GK}$ | 3.95 |
| $F_{GJ}$ | 5.49 |

**Joint G**

| | | | | |
|---|---|---|---|---|
| Brace Force | $F_{GK}$ | 3.95 | kip | C |
| Horizontal Force | $F_{GF}$ | 1.13 | kip | C |
| Vertical Force | $F_{GJ}$ | 5.49 | kip | T |

**Joint F**

| | | | |
|---|---|---|---|
| $F_{FJ}$ | 14.67 | kip | C |
| $F_{GF}$ | 1.13 | kip | C |
| $F_{FK}$ | 1.52 | kip | T |

**Joint J**

**Joint K**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Brace Force | $F_{JF}$ | 14.7 | kip | C | $F_{GK}$ | 4.0 | kip | C |
| Horizontal Force | $\mathbf{F_{JK}}$ | 3.6 | kip | T | $F_{KJ}$ | 3.6 | kip | T |
| Vertical Force | $F_{JG}$ | 5.49 | kip | T | $F_{KF}$ | 1.5 | kip | T |

## 3. RESULTS

| | Member (#) | Frame (type) | Floor (Units) | Function | Force (kip) | T/C | Moment (kip.ft) |
|---|---|---|---|---|---|---|---|
| GF | 10 | Braced | First | Beam | 14.67 | C | |
| JK | 13 | Braced | Roof | Beam | 3.65 | T | |
| JF | 15 | Braced | Roof | Brace | 14.67 | C | |
| GK | 16 | Braced | Roof | Brace | 3.95 | C | |
| BF | 17 | Braced | First | Brace | 15.90 | T | |
| GC | 18 | Braced | First | Brace | 15.90 | C | |
| BG | 3 | Braced | First | Column | -6.12 | T | |
| JG | 4 | Braced | Roof | Column | 5.49 | T | |
| CF | 5 | Braced | First | Column | -6.12 | C | |
| KF | 6 | Braced | Roof | Column | 1.52 | T | |

## 1. MOMENT FRAME INFORMATION

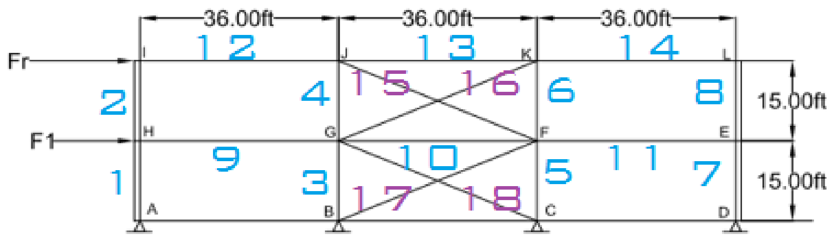



**Figure 1** - Braced-Frame Member Reference

| | | |
|---|---|---|
| Diagonal, D | 39 | ft |
| Story Height, H | 15 | ft |
| Bay Width, W | 36 | ft |
| $F_r$ | 4.31 | kip |
| $F_1$ | 16.82 | kip |

## 2.1 MEMBER FORCES DISTRIBUTION

**Find Reactions**

| | | | |
|---|---|---|---|
| Cy= | | kip | up |
| By= | 0 | kip | down |
| Cx= | -10.565 | kip | west |
| Bx= | -10.565 | kip | west |

**Joint B**

| | | | | |
|---|---|---|---|---|
| (Fr+F1)/2 | $V_1$ | 10.57 | kip | |
| | $F_{BFy}$ | -4.40 | kip | |
| Brace Force | $F_{BF}$ | 11.45 | kip | T |
| Vertical Force | $F_{BG}$ | -4.40 | kip | T |

**Joint C**

| | | | |
|---|---|---|---|
| $V_1$ | 10.57 | kip | |
| $F_{GCy}$ | -4.40 | kip | C |
| $F_{GC}$ | 11.45 | kip | C |
| $F_{FC}$ | -4.40 | kip | C |

**To solve system:**

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Moment Equation | 15 | $F_{GF}$ | 27.69 | $F_{GK}$ | -36 | $F_{GJ}$ | = | 29.1 | ##### |
| Forces in X | -1 | $F_{GF}$ | -0.923 | $F_{GK}$ | 0 | $F_{GJ}$ | = | -6.25 | -6.26 |
| Forces in Y | 0 | $F_{GF}$ | -0.385 | $F_{GK}$ | 1 | $F_{GJ}$ | = | 1.796 | -8.80 |

**Inverse Matrix**

| | | |
|---|---|---|
| 61.533 | 922 | 2215.2 |
| -66.666 | -1000 | -2400.0 |
| -25.666 | -385 | -923.0 |

**Solution**

| | |
|---|---|
| $F_{GF}$ | 6.61 |
| $F_{GK}$ | -0.38 |
| $F_{GJ}$ | 1.66 |

**Joint G**

| | | | | |
|---|---|---|---|---|
| Brace Force | $F_{GK}$ | -0.38 | kip | C |
| Horizontal Force | $F_{GF}$ | 6.61 | kip | C |
| Vertical Force | $F_{GJ}$ | 1.66 | kip | T |

**Joint F**

| | | | |
|---|---|---|---|
| $F_{FJ}$ | 4.29 | kip | C |
| $F_{GF}$ | 6.61 | kip | C |
| $F_{FK}$ | -0.15 | kip | T |

**Joint J**

**Joint K**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Brace Force | $F_{JF}$ | 4.3 | kip | C | $F_{GK}$ | -0.4 | kip | C |
| Horizontal Force | $\mathbf{F_{JK}}$ | -0.4 | kip | T | $F_{KJ}$ | -0.4 | kip | T |
| Vertical Force | $F_{JG}$ | 1.66 | kip | T | $F_{KF}$ | -0.1 | kip | T |

**3. RESULTS**

| | Member<br>*(#)* | Frame<br>*(type)* | Floor<br>*(Units)* | Function | Force<br>*(kip)* | T/C | Moment<br>*(kip.ft)* |
|---|---|---|---|---|---|---|---|
| GF | 10 | Braced | First | Beam | 4.29 | C | |
| JK | 13 | Braced | Roof | Beam | -0.35 | T | |
| JF | 15 | Braced | Roof | Brace | 4.29 | C | |
| GK | 16 | Braced | Roof | Brace | -0.38 | C | |
| BF | 17 | Braced | First | Brace | 11.45 | T | |
| GC | 18 | Braced | First | Brace | 11.45 | C | |
| BG | 3 | Braced | First | Column | -4.40 | T | |
| JG | 4 | Braced | Roof | Column | 1.66 | T | |
| CF | 5 | Braced | First | Column | -4.40 | C | |
| KF | 6 | Braced | Roof | Column | -0.15 | T | |

## GIVEN:

| | | | | | | First | Roof |
|---|---|---|---|---|---|---|---|
| **Number of Floors:** | 2 | **Number of Bays/Rov** | 3 | **Number of Rows:** | | 2 | 3 |
| **Beam Length** | 36 | **Girder Length** | 24 | **Interior Girder** | | 48 | |
| **Short Column Height** | 15 | **Long Column Height** | 30 | | | | |

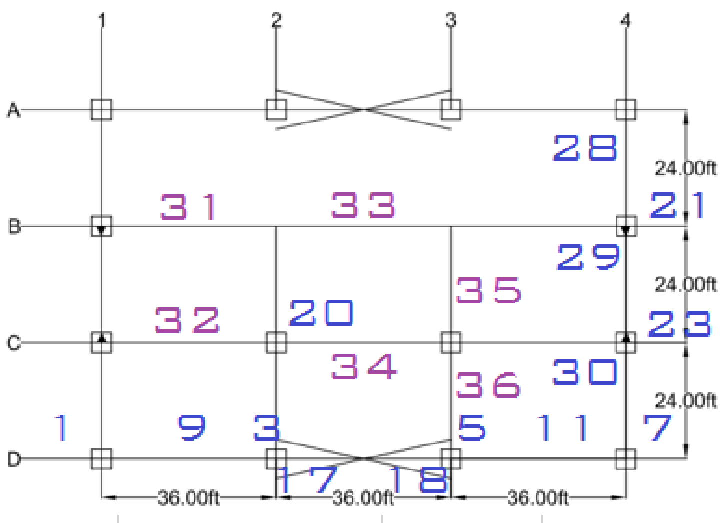



**Figure 1 -** First Floor Plan

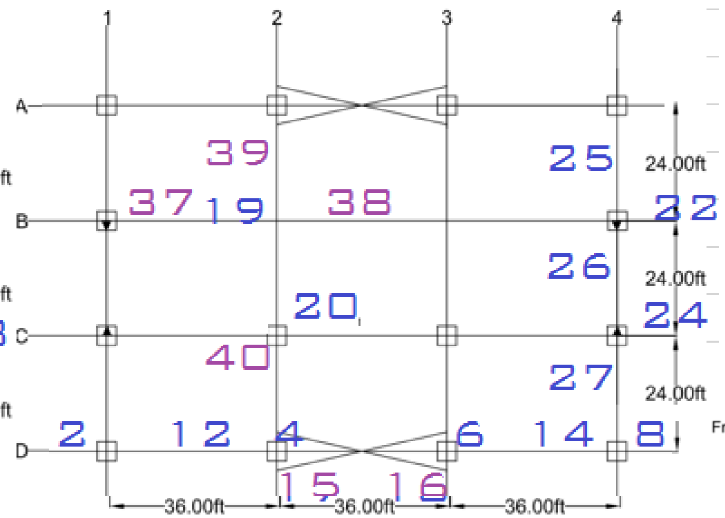



**Figure 2 -** Roof Floor Plan

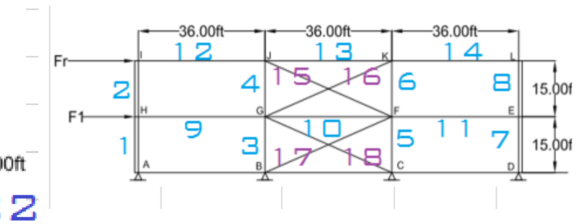



**Figure 3 -** Braced-Elevation

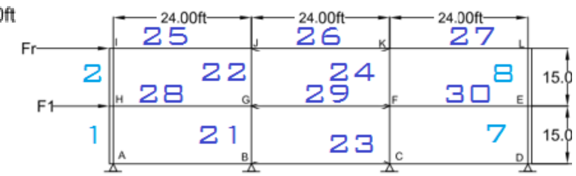
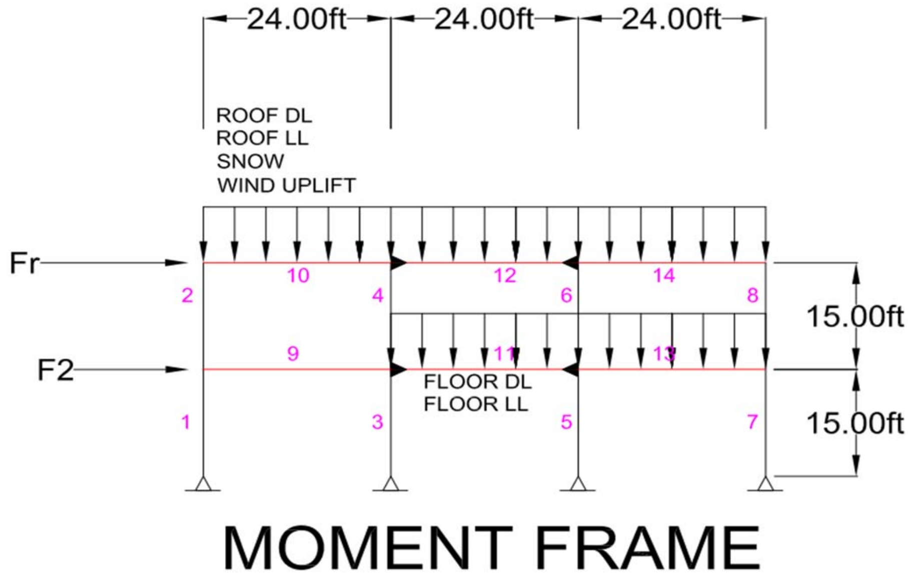



**Figure 4 -** Moment Frame - Elevation

## 1. MEMBERS FOR ANALYSIS:

| Member Ref. | Frame | Floor | Member | Section | New Section | Length | Unit Weight | Spacing or a | Beams/Bay | Qty of Members | Amount of Steel | New Amount | Difference |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| (#) | (type) | (Units) | | (Shape) | | (ft) | (plf) | (ft) | (Units) | (Units) | (kips) | (kips) | (%) |
| 1 | Braced | First | Column | W8X40 | W12X50 | 15 | 40 | 12 | | 2 | 1.20 | 1.50 | 0.25 |
| 2 | Braced | Roof | Column | W8X40 | W12X50 | 15 | 40 | 1.7 | | 2 | 1.20 | 1.50 | 0.25 |
| **3** | Braced | First | Column | W12X40 | W12X50 | 15 | 40 | 5 | | 2 | 1.20 | 1.50 | 0.25 |
| **4** | Braced | Roof | Column | W12X40 | W12X40 | 15 | 40 | | | 2 | 1.20 | 1.20 | 0 |
| 5 | Braced | First | Column | W12X40 | W12X50 | 15 | 40 | | | 2 | 1.20 | 1.50 | 0.25 |
| 6 | Braced | Roof | Column | W12X40 | W12X50 | 15 | 40 | | | 2 | 1.20 | 1.50 | 0.25 |
| 7 | Braced | First | Column | W8X40 | W12X50 | 15 | 40 | | | 2 | 1.20 | 1.50 | 0.25 |
| 8 | Braced | Roof | Column | W8X40 | W12X50 | 15 | 40 | | | 2 | 1.20 | 1.50 | 0.25 |
| 1m | Moment | First | Column | W8X40 | W10X68 | 15 | 40 | | | 2 | 1.20 | 2.04 | 0.7 |
| 2m | Moment | Roof | Column | W8X40 | W10X68 | 15 | 40 | | | 2 | 1.20 | 2.04 | 0.7 |
| 7m | Moment | First | Column | W8X40 | W10X68 | 15 | 40 | | | 2 | 1.20 | 2.04 | 0.7 |
| 8m | Moment | Roof | Column | W8X40 | W10X68 | 15 | 40 | | | 2 | 1.20 | 2.04 | 0.7 |
| 9 | Braced | First | Beam | W14X30 | W14X30 | 36 | 30 | | | 2 | 2.16 | 2.16 | 0 |
| 10 | Braced | First | nterior Bean | W14X30 | W14X30 | 36 | 30 | | | 2 | 2.16 | 2.16 | 0 |
| 11 | Braced | First | Beam | W14X30 | W14X30 | 36 | 30 | | | 2 | 2.16 | 2.16 | 0 |
| 12 | Braced | Roof | Beam | W10X19 | W10X19 | 36 | 19 | | | 2 | 1.37 | 1.37 | 0 |
| 13 | Braced | Roof | nterior Bean | W10X19 | W10X19 | 36 | 19 | | | 2 | 1.37 | 1.37 | 0 |
| 14 | Braced | Roof | Beam | W10X19 | W10X19 | 36 | 19 | | | 2 | 1.37 | 1.37 | 0 |
| 15 | Braced | Roof | Braces | WT9X48.5 | W10X33 | 39 | 48.5 | | | 2 | 3.78 | 2.57 | -0.32 |
| 16 | Braced | Roof | Braces | WT9X48.5 | W10X33 | 39 | 48.5 | | | 2 | 3.78 | 2.57 | -0.32 |
| 17 | Braced | First | Braces | WT9X48.5 | W10X33 | 39 | 48.5 | | | 2 | 3.78 | 2.57 | -0.32 |
| 18 | Braced | First | Braces | WT9X48.5 | W10X33 | 39 | 48.5 | | | 2 | 3.78 | 2.57 | -0.32 |
| 19 | Interior | Roof | Roof Columr | W8X40 | W8X40 | 15 | 40 | | | 2 | 1.20 | 1.20 | 0.00 |
| 20 | Interior | First | ıterior Colum | W8X40 | W8X40 | 30 | 40 | | | 2 | 2.40 | 2.40 | 0.00 |
| 21 | Moment | First | Column | W10X39 | W10X68 | 15 | 39 | | | 2 | 1.17 | 2.04 | 0.74 |
| 22 | Moment | Roof | Column | W10X39 | W10X68 | 15 | 39 | | | 2 | 1.17 | 2.04 | 0.74 |

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| **23** | Moment | First | Column | W10X39 | W10X68 | 15 | 39 | | | 2 | 1.17 | 2.04 | 0.74 |
| 24 | Moment | Roof | Column | W10X39 | W10X68 | 15 | 39 | 5 | | 2 | 1.17 | 2.04 | 0.74 |
| 25 | Moment | Roof | Beam | W14X30 | W14X30 | 24 | 30 | 8 | | 2 | 1.44 | 1.44 | 0.00 |
| **26** | Moment | Roof | nterior Bean | W8X40 | W10X39 | 24 | 40 | | | 2 | 1.92 | 1.87 | -0.02 |
| 27 | Moment | Roof | Beam | W14X30 | W14X30 | 24 | 30 | | | 2 | 1.44 | 1.44 | 0.00 |
| 28 | Moment | First | Beam | W8X40 | W8X40 | 24 | 40 | | | 2 | 1.92 | 1.92 | 0.00 |
| **29** | Moment | First | nterior Bean | W8X40 | W12X53 | 24 | 40 | | | 2 | 1.92 | 2.54 | 0.33 |
| 30 | Moment | First | Beam | W8X40 | W8X40 | 24 | 40 | | | 2 | 1.92 | 1.92 | 0.00 |
| 31 | Interior | First | Beam | W21X44 | W21X44 | 36 | 44 | | | 0 | 0.00 | 0.00 | |
| 32 | Interior | First | Beam | W21X44 | W21X44 | 36 | 44 | | | 0 | 0.00 | 0.00 | |
| 33 | Interior | First | nterior Bean | W21X44 | W21X44 | 36 | 44 | | | 0 | 0.00 | 0.00 | |
| 34 | Interior | First | nterior Bean | W21X44 | W21X44 | 36 | 44 | 12 | 6.00 | 18 | 28.51 | 28.51 | 0.00 |
| 35 | Interior | First | Girder | W30X108 | W30X108 | 24 | 108 | 8 | | 2 | 5.18 | 5.18 | 0.00 |
| 36 | Interior | First | Girder | W30X108 | W30X108 | 24 | 108 | 8 | | 2 | 5.18 | 5.18 | 0.00 |
| 37 | Interior | Roof | Beam | W14X22 | W14X22 | 36 | 22 | | | 0 | 0.00 | 0.00 | |
| 38 | Interior | Roof | nterior Bean | W14X22 | W14X22 | 36 | 22 | 12 | 8.00 | 24 | 19.01 | 19.01 | 0.00 |
| 39 | Interior | Roof | Girder | W27X84 | W27X84 | 48 | 84 | | | 2 | 8.06 | 8.06 | 0.00 |
| 40 | Interior | First | Girder | W27X84 | W27X84 | 24 | 84 | | | 2 | 4.03 | 4.03 | 0 |
| TOTAL | | | | | | | | | | 118 | 128.94 | 133.62 | 6.55 |




Sizing (From 30%)

| | |
|---|---|
| Members 10 & 14 | W18x40 |
| Member 12 | W12x19 |
| Members 9 & 13 | W24x68 |
| Member 11 | W18x35 |
| Exterior Columns | W8x40 |
| Interior Columns | W8x40 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft | Plan Area | 7776 | $ft^2$ |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft | | | |

Vertical Loading

Roof

| | | |
|---|---|---|
| Roof Live Load, $W_{RLL}$ | 0.9 | klf |
| Snow, $W_s$ | 0.57 | klf |
| Wind Uplift, $W_{WU}$ | -0.42 | klf |
| Roof Dead Load=self weight of beam | | |

Level 2

| | | |
|---|---|---|
| Floor Dead Load, $W_{DL}$ | 1.67 | klf |
| Floor Live Load, $W_{FIL}$ | 1.8 | klf |

| | | |
|---|---|---|
| Girder Reactions on Interior Columns | 0 | kips |
| Girder Reactions on Exterior Columns | 0 | kips |

Lateral Loading

| | | |
|---|---|---|
| Seismic Force, Roof Level | 3.23 | kips |
| Seismic Force, Level 2 | 12.62 | kips |
| Wind Force, Roof Level | 9.61 | kips |
| Wind Force, Level 2 | 19.22 | kips |

Determine Notional Loads

Roof

| | | |
|---|---|---|
| Roof Live Load | 0.9 | klf |
| Snow Load | 0.57 | klf |
| Wind Uplift | -0.42 | klf |
| Dead Load (Beam Self Weight) | 0.04 | klf |
| $W_{u\text{-}roof}$ | 1.49 | klf |

Level 2

| | | | |
|---|---|---|---|
| Floor System Load | 1.67 | klf | |
| Beam Self weight | 0.068 | klf | |
| Dead Load | 1.74 | klf | |
| Live Load | 1.80 | klf | |
| $W_{u\text{-}2}$ | 4.99 | klf | |
| $Y_R$ | 107.14 | kips | |
| $Y_2$ | 358.99 | kips | |
| $\alpha$ | 1 | | |
| $N_R$ | 0.214 | kips | Eq C2-2 |
| $N_2$ | 0.718 | kips | Eq C2-1 |
| $F_y$ | 50 | ksi | |

***Run Computer model in SAP2000***

**Design of Columns (Data from Members 3 & 5)**

First order analysis forces

| | top | | bottom | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 57.15 | kips | 57.15 | kips | SAP | $\Delta_h/L$ | 0.003 | drift limit | |
| Ultimate Axial Load, $P_{ult}$ | 29.35 | kips | 29.35 | kips | SAP | $\Delta_h/L$ | 0.00267 | Actual drift | |
| Ultimate Moment, $M_{untx}$ | 31.08 | kip*ft | 0 | kip*ft | SAP | H | 30.215 | kips | SAP |
| Ultimate Moment, $M_{ultx}$ | 158.84 | kip*ft | 0 | kip*ft | SAP | | | | |
| Unbraced Length, $L_{bx}$ | 15 | ft | | | | | | | |
| | | | Effective Length Factor, $K_y$ | | | | 1 | | |

$P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story\text{-}r}$ | 622.08 | kips |
| $P_{story\text{-}2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |
| TRIAL SECTION | W10x68 | |

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 85.3 | $in^3$ | $S_x$ | 26.4 | $in^3$ |
| Moment of Inertia, $I_x$ | 394 | $in^4$ | $r_{ts}$ | 2.59 | in |
| $r_x$ | 4.44 | in | $h_o$ | 9.63 | in |
| $r_y$ | 2.59 | in | J | 3.56 | $in^4$ |
| $A_g$ | 19.9 | in2 | | | |
| $b_f$ | 10.1 | in | | | |
| $t_f$ | 0.77 | in | | | |
| h | 7.5 | in | | | |
| $t_w$ | 0.47 | in | | | |

Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 9.15 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 40.6 | ft | Table 3-2 |
| $\phi BF$ | 3.85 | kips | Table 3-2 |
| $\phi M_{px}$ | 320 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | NONSLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 15 | ft | | |
| KL/r | 69.50 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 59.26 | ksi | | |
| $F_{cr}$ | 35.12 | ksi | E3-2 | |
| $F_{cr}$ | 51.97 | ksi | E3-3 | |
| $F_{cr}$ | 35.12 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 629.06 | kips | Table 4-1 | |

Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 158.84 | kip*ft | $M_{ult}$ | |
| $M_A$ | 23.54 | kip*ft | SAP | |
| $M_B$ | 54.31 | kip*ft | SAP | |
| $M_C$ | 81.47 | kip*ft | SAP | |
| $C_b$ | 2.14 | | | Eq F1-1 |
| | | | | |
| Zone 1, $M_c$ | 320 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 320.00 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 317.14 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 697.72 | kip*ft | | Eq F2-3 |
| $M_c$ | 320.00 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 86.50 | kips | $P_{nt}$+$P_{lt}$ |
| $P_y$ | 995.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.09 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2784.45 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *Eq A-8-4* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| | | | |
| $M_r$ | 261.96 | kip*ft | *Eq A-8-1* |
| $M_c$ | 320.00 | kip*ft | *Referenced Above* |
| $P_r$ | 99.8 | kips | *Eq A-8-2* |
| $P_c$ | 629.06 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.16 | | |
| $P_r/P_c$>0.2 | 0.89 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.90 | | Eq H1-1b |
| Design Check | 0.90 | **SECTION OK** | |

## Design of Level 2 Beam (Member 11)

First order analysis forces

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 11.62 | kips | SAP | $\Delta_h/L$ | 0.003 | drift limit | | |
| Ultimate Axial Load, $P_{ult}$ | 5.69 | kips | SAP | $\Delta_h/L$ | 0.00267 | Actual drift | | |
| Ultimate Moment, $M_{untx}$ | 74.27 | kip*ft | SAP | H | 30.215 | kips | SAP | |
| Ultimate Moment, $M_{ultx}$ | 112.88 | kip*ft | SAP | | | | | |
| Unbraced Length, $L_{bx}$ | 24 | ft | | | | | | |
| | | | Effective Length Factor, $K_y$ | | 1 | | | |

$P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story-r}$ | 622.08 | kips |
| $P_{story-2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |

TRIAL SECTION W12x53

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 77.9 | $in^3$ | $S_x$ | 70.6 | $in^3$ |
| Moment of Inertia, $I_x$ | 425 | $in^4$ | $r_{ts}$ | 2.79 | in |
| $r_x$ | 5.23 | in | $h_o$ | 11.5 | in |
| $r_y$ | 2.48 | in | J | 1.58 | $in^4$ |
| $A_g$ | 15.6 | in2 | | | |
| $b_f$ | 10 | in | | | |
| $t_f$ | 0.575 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.345 | in | | | |

Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 8.76 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 28.2 | ft | Table 3-2 |
| $\phi$BF | 5.5 | kips | Table 3-2 |
| $\phi M_{px}$ | 292 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | SLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 24 | ft | | |
| KL/r | 116.13 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 21.22 | ksi | | |
| $F_{cr}$ | 18.65 | ksi | E3-2 | |
| $F_{cr}$ | 18.61 | ksi | E3-3 | |
| $F_{cr}$ | 18.61 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 261.33 | kips | | |

Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $C_b$ | 1.14 | | Table 3-1 | |
| Zone 1, $M_c$ | 292 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 249.06 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 49.54 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 291.47 | kip*ft | | Eq F2-3 |
| $M_c$ | 249.06 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 17.31 | kips | $P_{nt}+P_{lt}$ |
| $P_y$ | 780.00 | kips | $A_g*F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.02 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 1173.25 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.99 | | *Eq A-8-4* |
| $B_1$ | 1.01 | | *Eq A-8-3* |

Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| $M_r$ | 239.01 | kip*ft | *Eq A-8-1* |
| $M_c$ | 249.06 | kip*ft | *Referenced Above* |
| $P_r$ | 19.9 | kips | *Eq A-8-2* |
| $P_c$ | 261.33 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.08 | | |
| $P_r/P_c>0.2$ | 0.93 | | Eq H1-1a |
| $P_r/P_c<0.2$ | 1.00 | | Eq H1-1b |
| Design Check | 0.998 | **SECTION OK** | |

## Design of Roof Level Beam (Member 12)

First order analysis forces

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 7.29 | kips | SAP | $\Delta_h/L$ | 0.003 | drift limit | |
| Ultimate Axial Load, $P_{ult}$ | 2.35 | kips | SAP | $\Delta_h/L$ | 0.00267 | Actual drift | |
| Ultimate Moment, $M_{untx}$ | 14.41 | kip*ft | SAP | H | 30.215 | kips | SAP |
| Ultimate Moment, $M_{ultx}$ | 43.08 | kip*ft | SAP | | | | |
| Unbraced Length, $L_{bx}$ | 24 | ft | | | | | |
| | | | Effective Length Factor, $K_y$ | | 1 | | |

$P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story-r}$ | 622.08 | kips |
| $P_{story-2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |
| TRIAL SECTION | W10x39 | |

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 46.8 | $in^3$ | $S_x$ | 42.1 | $in^3$ |
| Moment of Inertia, $I_x$ | 209 | $in^4$ | $r_{ts}$ | 2.24 | in |
| $r_x$ | 4.27 | in | $h_o$ | 9.39 | in |
| $r_y$ | 1.98 | in | J | 0.976 | $in^4$ |
| $A_g$ | 11.5 | in2 | | | |
| $b_f$ | 7.99 | in | | | |
| $t_f$ | 0.53 | in | | | |
| h | 7.5 | in | | | |
| $t_w$ | 0.315 | in | | | |

Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.99 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 24.2 | ft | Table 3-2 |
| $\phi$BF | 3.78 | kips | Table 3-2 |
| $\phi M_{px}$ | 176 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | SLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 24 | ft | | |
| KL/r | 145.45 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 13.53 | ksi | | |
| $F_{cr}$ | 10.64 | ksi | E3-2 | |
| $F_{cr}$ | 11.86 | ksi | E3-3 | |
| $F_{cr}$ | 11.86 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 122.80 | kips | | |

## Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $C_b$ | 1.14 | | Table 3-1 | |
| Zone 1, $M_c$ | 176 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 136.34 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 40.37 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 141.64 | kip*ft | | Eq F2-3 |
| $M_c$ | 136.34 | kip*ft | | |

## Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 9.64 | kips | $P_{nt}+P_{lt}$ |
| $P_y$ | 575.00 | kips | $A_g*F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.02 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

## Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 576.96 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.99 | | *Eq A-8-4* |
| $B_1$ | 1.01 | | *Eq A-8-3* |

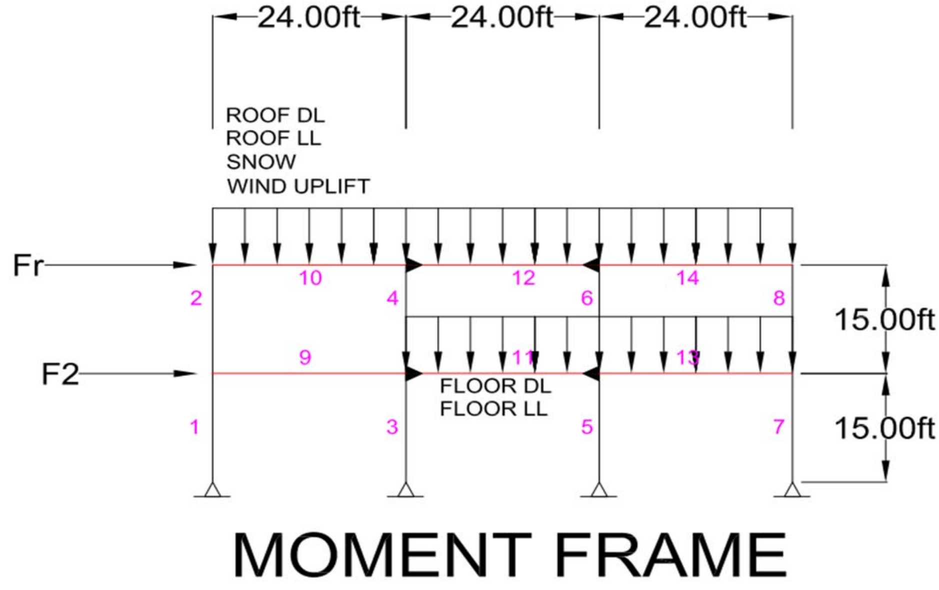
## Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| $M_r$ | 77.18 | kip*ft | *Eq A-8-1* |
| $M_c$ | 136.34 | kip*ft | *Referenced Above* |
| $P_r$ | 10.7 | kips | *Eq A-8-2* |
| $P_c$ | 122.80 | kips | *Referenced Above* |

## Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.09 | | |
| $P_r/P_c>0.2$ | 0.59 | | Eq H1-1a |
| $P_r/P_c<0.2$ | 0.61 | | Eq H1-1b |
| Design Check | 0.61 | **SECTION OK** | |

## Moment Frame Design




Sizing (From 30%)

| | |
|---|---|
| Members 10 & 14 | W18x40 |
| Member 12 | W12x19 |
| Members 9 & 13 | W24x68 |
| Member 11 | W18x35 |
| Exterior Columns | W8x40 |
| Interior Columns | W8x40 |

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft | Plan Area | 7776 | $ft^2$ |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft | | | |

### Vertical Loading

Roof

| | | |
|---|---|---|
| Roof Live Load, $W_{RLL}$ | 0.9 | klf |
| Snow, $W_s$ | 0.57 | klf |
| Wind Uplift, $W_{WU}$ | -0.42 | klf |
| Roof Dead Load=self weight of beam | | |

Level 2

| | | |
|---|---|---|
| Floor Dead Load, $W_{DL}$ | 1.67 | klf |
| Floor Live Load, $W_{FlL}$ | 1.8 | klf |
| | | |
| Girder Reactions on Interior Columns | 0 | kips |
| Girder Reactions on Exterior Columns | 0 | kips |

### Lateral Loading

| | | |
|---|---|---|
| Seismic Force, Roof Level | 3.23 | kips |
| Seismic Force, Level 2 | 12.62 | kips |
| | | |
| Wind Force, Roof Level | 9.61 | kips |
| Wind Force, Level 2 | 19.22 | kips |

Determine Notional Loads

Roof

| | | |
|---|---|---|
| Roof Live Load | 0.9 | klf |
| Snow Load | 0.57 | klf |
| Wind Uplift | -0.42 | klf |
| Dead Load (Beam Self Weight) | 0.04 | klf |
| $W_{u\text{-}roof}$ | 1.49 | klf |

Level 2

| | | |
|---|---|---|
| Floor System Load | 1.67 | klf |
| Beam Self weight | 0.068 | klf |
| Dead Load | 1.74 | klf |
| Live Load | 1.80 | klf |
| $W_{u\text{-}2}$ | 4.99 | klf |

| | | | |
|---|---|---|---|
| $Y_R$ | 107.14 | kips | |
| $Y_2$ | 358.99 | kips | |
| $\alpha$ | 1 | | |
| $N_R$ | 0.214 | kips | Eq C2-2 |
| $N_2$ | 0.718 | kips | Eq C2-1 |
| $F_y$ | 50 | ksi | |

***Run Computer model in SAP2000***

**Design of Columns (Data from Members 3 & 5)**

First order analysis forces

| | top | | bottom | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 57.15 | kips | 57.15 | kips | SAP | $\Delta_h/L$ | 0.003 | | |
| Ultimate Axial Load, $P_{ult}$ | 29.35 | kips | 29.35 | kips | SAP | H | 30.215 | kips | SAP |
| Ultimate Moment, $M_{untx}$ | 31.08 | kip*ft | 0 | kip*ft | SAP | | | | |
| Ultimate Moment, $M_{ultx}$ | 158.84 | kip*ft | 0 | kip*ft | SAP | | | | |
| Unbraced Length, $L_{bx}$ | 15 | ft | | | | | | | |

Effective Length Factor, $K_y$ 1

$P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story\text{-}r}$ | 622.08 | kips |
| $P_{story\text{-}2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |

TRIAL SECTION W10x68

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 85.3 | $in^3$ | $S_x$ | 26.4 | $in^3$ |
| Moment of Inertia, $I_x$ | 394 | $in^4$ | $r_{ts}$ | 2.59 | in |
| $r_x$ | 4.44 | in | $h_o$ | 9.63 | in |
| $r_y$ | 2.59 | in | J | 3.56 | $in^4$ |
| $A_g$ | 19.9 | in2 | | | |
| $b_f$ | 10.1 | in | | | |
| $t_f$ | 0.77 | in | | | |
| h | 7.5 | in | | | |
| $t_w$ | 0.47 | in | | | |

## Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 9.15 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 40.6 | ft | Table 3-2 |
| $\phi$BF | 3.85 | kips | Table 3-2 |
| $\phi M_{px}$ | 320 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | NONSLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

## Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 15 | ft | | |
| KL/r | 69.50 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 59.26 | ksi | | |
| $F_{cr}$ | 35.12 | ksi | E3-2 | |
| $F_{cr}$ | 51.97 | ksi | E3-3 | |
| $F_{cr}$ | 35.12 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 629.06 | kips | Table 4-1 | |

## Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 158.84 | kip*ft | $M_{ult}$ | |
| $M_A$ | 23.54 | kip*ft | SAP | |
| $M_B$ | 54.31 | kip*ft | SAP | |
| $M_C$ | 81.47 | kip*ft | SAP | |
| $C_b$ | 2.14 | | | Eq F1-1 |
| | | | | |
| Zone 1, $M_c$ | 320 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 320.00 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 317.14 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 697.72 | kip*ft | | Eq F2-3 |
| $M_c$ | 320.00 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 86.50 | kips | $P_{nt}$+$P_{lt}$ |
| $P_y$ | 995.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.09 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2784.45 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *Eq A-8-4* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| | | | |
| $M_r$ | 261.96 | kip*ft | *Eq A-8-1* |
| $M_c$ | 320.00 | kip*ft | *Referenced Above* |
| $P_r$ | 99.8 | kips | *Eq A-8-2* |
| $P_c$ | 629.06 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.16 | | |
| $P_r/P_c$>0.2 | 0.89 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.90 | | Eq H1-1b |
| Design Check | 0.90 | **SECTION OK** | |

## Design of Level 2 Beam (Member 11)

### First order analysis forces

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 11.62 | kips | SAP | $\Delta_h/L$ | 0.003 | | |
| Ultimate Axial Load, $P_{ult}$ | 5.69 | kips | SAP | H | 30.215 | kips | SAP |
| Ultimate Moment, $M_{untx}$ | 74.27 | kip*ft | SAP | | | | |
| Ultimate Moment, $M_{ultx}$ | 112.88 | kip*ft | SAP | | | | |
| Unbraced Length, $L_{bx}$ | 24 | ft | | | | | |
| | | | Effective Length Factor, $K_y$ | | 1 | | |

### $P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story-r}$ | 622.08 | kips |
| $P_{story-2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |

TRIAL SECTION W12x53

### Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 77.9 | $in^3$ | $S_x$ | 70.6 | $in^3$ |
| Moment of Inertia, $I_x$ | 425 | $in^4$ | $r_{ts}$ | 2.79 | in |
| $r_x$ | 5.23 | in | $h_o$ | 11.5 | in |
| $r_y$ | 2.48 | in | J | 1.58 | $in^4$ |
| $A_g$ | 15.6 | in2 | | | |
| $b_f$ | 10 | in | | | |
| $t_f$ | 0.575 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.345 | in | | | |

### Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 8.76 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 28.2 | ft | Table 3-2 |
| $\phi$BF | 5.5 | kips | Table 3-2 |
| $\phi M_{px}$ | 292 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | SLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

### Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 24 | ft | | |
| KL/r | 116.13 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 21.22 | ksi | | |
| $F_{cr}$ | 18.65 | ksi | E3-2 | |
| $F_{cr}$ | 18.61 | ksi | E3-3 | |
| $F_{cr}$ | 18.61 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 261.33 | kips | | |

Flexure Capacity

| | | | |
|---|---|---|---|
| Zone? | ZONE 2 | | |
| $C_b$ | 1.14 | | Table 3-1 |
| | | | |
| Zone 1, $M_c$ | 292 | kip*ft | Eq F2-1 |
| Zone 2, $M_c$ | 249.06 | kip*ft | Eq F2-2 |
| Zone 3, $F_{cr}$ | 49.54 | ksi | Eq F2-4 |
| Zone 3, $M_c$ | 291.47 | kip*ft | Eq F2-3 |
| $M_c$ | 249.06 | kip*ft | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 17.31 | kips | $P_{nt}+P_{lt}$ |
| $P_y$ | 780.00 | kips | $A_g*F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.02 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 1173.25 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.99 | | *Eq A-8-4* |
| $B_1$ | 1.01 | | *Eq A-8-3* |

Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| | | | |
| $M_r$ | 239.01 | kip*ft | *Eq A-8-1* |
| $M_c$ | 249.06 | kip*ft | *Referenced Above* |
| $P_r$ | 19.9 | kips | *Eq A-8-2* |
| $P_c$ | 261.33 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.08 | | |
| $P_r/P_c>0.2$ | 0.93 | | Eq H1-1a |
| $P_r/P_c<0.2$ | 1.00 | | Eq H1-1b |
| Design Check | 0.998 | **SECTION OK** | |

## Design of Roof Level Beam (Member 12)

### First order analysis forces

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 7.29 | kips | SAP | $\Delta_h/L$ | 0.003 | | |
| Ultimate Axial Load, $P_{ult}$ | 2.35 | kips | SAP | H | 30.215 | kips | SAP |
| Ultimate Moment, $M_{untx}$ | 14.41 | kip*ft | SAP | | | | |
| Ultimate Moment, $M_{ultx}$ | 43.08 | kip*ft | SAP | | | | |
| Unbraced Length, $L_{bx}$ | 24 | ft | | | | | |
| | | | Effective Length Factor, $K_y$ | | 1 | | |

### $P_{story}$ & $P_{mf}$

| | | |
|---|---|---|
| Roof Level Uniform Pressure | 0.08 | ksf |
| Level 2 Uniform Pressure | 0.27 | ksf |
| $P_{story-r}$ | 622.08 | kips |
| $P_{story-2}$ | 2111.96 | kips |
| $P_{story}$ | 2734.04 | kips |
| Moment Frame Column Tributary Area | 432.00 | $ft^2$ |
| $P_{mf}$ | 607.56 | kips |

TRIAL SECTION W10x39

### Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 46.8 | $in^3$ | $S_x$ | 42.1 | $in^3$ |
| Moment of Inertia, $I_x$ | 209 | $in^4$ | $r_{ts}$ | 2.24 | in |
| $r_x$ | 4.27 | in | $h_o$ | 9.39 | in |
| $r_y$ | 1.98 | in | J | 0.976 | $in^4$ |
| $A_g$ | 11.5 | in2 | | | |
| $b_f$ | 7.99 | in | | | |
| $t_f$ | 0.53 | in | | | |
| h | 7.5 | in | | | |
| $t_w$ | 0.315 | in | | | |

### Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.99 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 24.2 | ft | Table 3-2 |
| $\phi$BF | 3.78 | kips | Table 3-2 |
| $\phi M_{px}$ | 176 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | SLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

### Axial Capacity

| | | | | |
|---|---|---|---|---|
| K | 1 | | K=1 for columns in DAM | |
| Kl | 24 | ft | | |
| KL/r | 145.45 | | 4.71*sqrt(E/Fy) | 113.4318 |
| $F_e$ | 13.53 | ksi | | |
| $F_{cr}$ | 10.64 | ksi | E3-2 | |
| $F_{cr}$ | 11.86 | ksi | E3-3 | |
| $F_{cr}$ | 11.86 | ksi | | |
| $\phi$ | 0.90 | | | |
| $P_c$ | 122.80 | kips | | |

Flexure Capacity

| | | | |
|---|---|---|---|
| Zone? | ZONE 2 | | |
| $C_b$ | 1.14 | | Table 3-1 |
| Zone 1, $M_c$ | 176 | kip*ft | Eq F2-1 |
| Zone 2, $M_c$ | 136.34 | kip*ft | Eq F2-2 |
| Zone 3, $F_{cr}$ | 40.37 | ksi | Eq F2-4 |
| Zone 3, $M_c$ | 141.64 | kip*ft | Eq F2-3 |
| $M_c$ | 136.34 | kip*ft | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 9.64 | kips | $P_{nt}$+$P_{lt}$ |
| $P_y$ | 575.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.02 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 576.96 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.99 | | *Eq A-8-4* |
| $B_1$ | 1.01 | | *Eq A-8-3* |

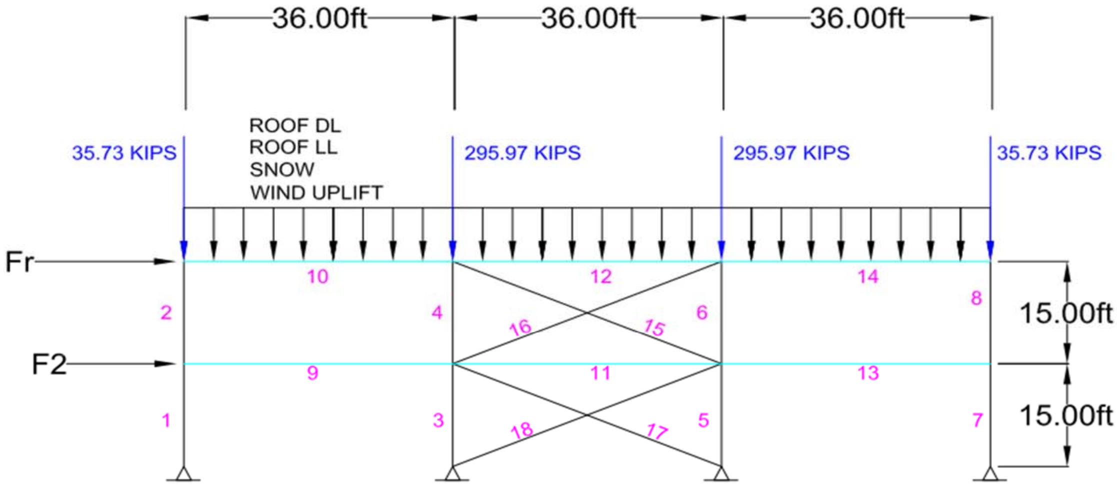
Determine B2

| | | | |
|---|---|---|---|
| $R_M$ | 0.97 | | *Eq A-8-8* |
| $P_{estory}$ | 8762.35 | kips | *Eq A-8-7* |
| $B_2$ | 1.45 | | *Eq A-8-6* |
| $M_r$ | 77.18 | kip*ft | *Eq A-8-1* |
| $M_c$ | 136.34 | kip*ft | *Referenced Above* |
| $P_r$ | 10.7 | kips | *Eq A-8-2* |
| $P_c$ | 122.80 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.09 | | |
| $P_r/P_c$>0.2 | 0.59 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.61 | | Eq H1-1b |
| Design Check | 0.61 | **SECTION OK** | |

## North Braced Frame Design




NORTH BRACED FRAME
(COLUMN LINE A)

| | | |
|---|---|---|
| $F_y$ | 50 | ksi |

Sizing (From 30%)

| | |
|---|---|
| Level 2 Beams/Girders | W14x132 |
| Roof Level Beams/Girders | W14x90 |
| Columns | W12x40 |
| Bracing | W10x33 |

Vertical Loading

Roof

| | | |
|---|---|---|
| Roof Live Load, $W_{RLL}$ | 0.6 | klf |
| Snow, $W_s$ | 0.38 | klf |
| Wind Uplift, $W_{WU}$ | -0.28 | klf |
| Roof Dead Load=self weight of beam | | |

Level 2

| | | |
|---|---|---|
| Floor Dead Load, $W_{DL}$ | 1.12 | klf |
| Floor Live Load, $W_{FIL}$ | 1.2 | klf |
| | | |
| Girder Reactions on Interior Columns | 295.97 | kips |
| Girder Reactions on Exterior Columns | 35.73 | kips |

Lateral Loading

| | | |
|---|---|---|
| Seismic Force, Roof Level | 4.31 | kips |
| Seismic Force, Level 2 | 16.82 | kips |
| | | |
| Wind Force, Roof Level | 6.27 | kips |
| Wind Force, Level 2 | 12.54 | kips |

Determine Notional Loads

Roof

| | | |
|---|---|---|
| Roof Live Load | 0.6 | klf |
| Snow Load | 0.38 | klf |
| Wind Uplift | -0.28 | klf |
| Dead Load (Beam Self Weight) | 0.09 | klf |
| $W_{u\text{-}roof}$ | 1.07 | klf |
| Girder Reactions | 331.70 | kips |

Level 2

| | | | |
|---|---|---|---|
| Floor System Load | 1.12 | klf | |
| Beam Self weight | 0.132 | klf | |
| Dead Load | 1.25 | klf | |
| Live Load | 1.20 | klf | |
| $W_{u\text{-}2}$ | 3.42 | klf | |
| $Y_R$ | 447.26 | kips | |
| $Y_2$ | 369.36 | kips | |
| $\alpha$ | 1 | | |
| $N_R$ | 0.895 | kips | Eq C2-2 |
| $N_2$ | 0.739 | kips | Eq C2-1 |

Eccentricity

| | | |
|---|---|---|
| Column Web Thickness, $t_w$ | 0.295 | in |
| Connection Distance (Assumed) | 2.5 | in |
| e | 2.6475 | **in** |
| Say 3" eccentricity | | |

***Run Computer model in SAP2000***

## Design of Bottom Columns (Members 3 & 5)

| First Order Analysis Forces | Top | | Bottom | |
|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 283.85 | kips | 283.85 | kips |
| Ultimate Moment, $M_{unt}$ | 14.2 | kip*ft | 0 | kip*ft |

| | | | | | |
|---|---|---|---|---|---|
| Unbraced Length, $L_b$ | 15 | ft | | | |
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft |

Section Used *W12x50*

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 71.9 | $in^3$ | $S_x$ | 64.2 | $in^3$ |
| Moment of Inertia, $I_x$ | 391 | $in^4$ | $r_{ts}$ | 2.25 | in |
| $r_x$ | 5.18 | in | $h_o$ | 11.6 | in |
| $A_g$ | 14.6 | in2 | J | 1.71 | $in^4$ |
| $b_f$ | 8.08 | in | $c_w$ | 1880 | $in^6$ |
| $t_f$ | 0.64 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.37 | in | | | |

## Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.92 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 23.8 | ft | Table 3-2 |
| $\phi BF$ | 5.98 | kips | Table 3-2 |
| $\phi M_{px}$ | 270 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | NONSLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

## Axial Capacity

| | | | |
|---|---|---|---|
| K | 1 | | K=1 for columns in DAM |
| Kl | 15 | ft | |
| $P_c$ | 355 | kips | Table 4-1 |

## Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 14.20 | kip*ft | $M_{ult}$ | |
| $M_A$ | 4.50 | kip*ft | SAP | |
| $M_B$ | 10.40 | kip*ft | SAP | |
| $M_C$ | 15.59 | kip*ft | SAP | |
| $C_b$ | 1.29 | | | Eq F1-1 |
| Zone 1, $M_c$ | 270 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 270.00 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 84.66 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 452.92 | kip*ft | | Eq F2-3 |
| $M_c$ | 270.00 | kip*ft | | |

## Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 283.85 | kips | $P_{unt}$ |
| $P_y$ | 730.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.39 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

## Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2763.25 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *0.6-0.4*(M1/M2)* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

## Summary

| | | | |
|---|---|---|---|
| $M_r$ | 14.20 | kip*ft | *Eq A-8-1* |
| $M_c$ | 270.00 | kip*ft | *Referenced Above* |
| $P_r$ | 283.9 | kips | *Eq A-8-2* |
| $P_c$ | 355 | kips | *Referenced Above* |

## Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.80 | | |
| $P_r/P_c$>0.2 | 0.85 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.45 | | Eq H1-1b |
| Design Check | 0.85 | **SECTION OK** | |

## Design of Top Columns (Members 4 & 6)

| First Order Analysis Forces | Top | | Bottom | |
|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 284.8 | kips | 284.8 | kips |
| Ultimate Moment, $M_{unt}$ | 15.09 | kip*ft | 0 | kip*ft |

| | | | | | |
|---|---|---|---|---|---|
| Unbraced Length, $L_b$ | 15 | ft | | | |
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft |

Section Used *W12x50*

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 71.9 | $in^3$ | $S_x$ | 64.2 | $in^3$ |
| Moment of Inertia, $I_x$ | 391 | $in^4$ | $r_{ts}$ | 2.25 | in |
| $r_x$ | 5.18 | in | $h_o$ | 11.6 | in |
| $A_g$ | 14.6 | in2 | J | 1.71 | $in^4$ |
| $b_f$ | 8.08 | in | $c_w$ | 1880 | $in^6$ |
| $t_f$ | 0.64 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.37 | in | | | |

Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.92 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 23.8 | ft | Table 3-2 |
| $\phi$BF | 5.98 | kips | Table 3-2 |
| $\phi M_{px}$ | 270 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | NONSLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

Axial Capacity

| | | | |
|---|---|---|---|
| K | 1 | | K=1 for columns in DAM |
| Kl | 15 | ft | |
| $P_c$ | 355 | kips | Table 4-1 |

Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 15.09 | kip*ft | $M_{ult}$ | |
| $M_A$ | 5.42 | kip*ft | SAP | |
| $M_B$ | 10.84 | kip*ft | SAP | |
| $M_C$ | 16.26 | kip*ft | SAP | |
| $C_b$ | 1.29 | | | Eq F1-1 |
| | | | | |
| Zone 1, $M_c$ | 270 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 270.00 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 84.57 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 452.47 | kip*ft | | Eq F2-3 |
| $M_c$ | 270.00 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 284.80 | kips | $P_{unt}$ |
| $P_y$ | 730.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.39 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2763.25 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *0.6-0.4*(M1/M2)* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

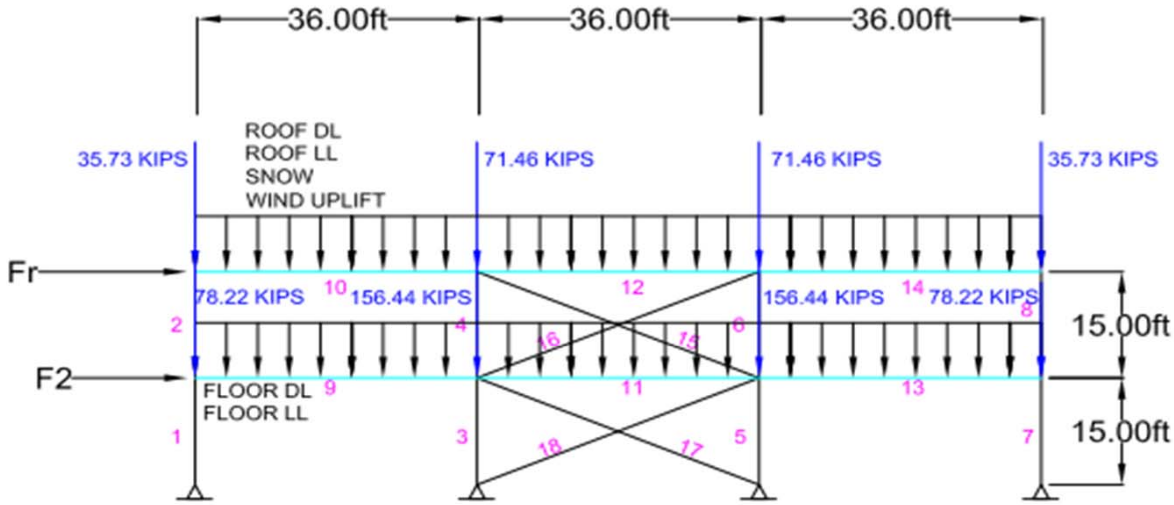
Summary

| | | | |
|---|---|---|---|
| $M_r$ | 15.09 | kip*ft | *Eq A-8-1* |
| $M_c$ | 270.00 | kip*ft | *Referenced Above* |
| $P_r$ | 284.8 | kips | *Eq A-8-2* |
| $P_c$ | 355 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.80 | | |
| $P_r/P_c$>0.2 | 0.85 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.46 | | Eq H1-1b |
| Design Check | 0.85 | **SECTION OK** | |

## South Braced Frame Design




SOUTH BRACED FRAME
(COLUMN LINE D)

| | | |
|---|---|---|
| $F_y$ | 50 | ksi |

Sizing (From 30%)

| | |
|---|---|
| Level 2 Beams/Girders | W14x132 |
| Roof Level Beams/Girders | W14x90 |
| Columns | W12x40 |
| Bracing | W10x33 |

Vertical Loading

| | | |
|---|---|---|
| Roof | | |
| Roof Live Load, $W_{RLL}$ | 0.6 | klf |
| Snow, $W_s$ | 0.38 | klf |
| Wind Uplift, $W_{WU}$ | -0.28 | klf |
| Roof Dead Load=self weight of beam | | |
| Girder Reactions on Interior Columns | 71.46 | kips |
| Girder Reactions on Exterior Columns | 35.73 | kips |
| Level 2 | | |
| Floor Dead Load, $W_{DL}$ | 1.12 | klf |
| Floor Live Load, $W_{FIL}$ | 1.2 | klf |
| Girder Reactions on Interior Columns | 156.44 | kips |
| Girder Reactions on Exterior Columns | 78.22 | kips |

Lateral Loading

| | | |
|---|---|---|
| Seismic Force, Roof Level | 4.31 | kips |
| Seismic Force, Level 2 | 16.82 | kips |
| Wind Force, Roof Level | 6.27 | kips |
| Wind Force, Level 2 | 12.54 | kips |

Determine Notional Loads

Roof

| | | |
|---|---|---|
| Roof Live Load | 0.6 | klf |
| Snow Load | 0.38 | klf |
| Wind Uplift | -0.28 | klf |
| Dead Load (Beam Self Weight) | 0.09 | klf |
| $W_{u\text{-}roof}$ | 1.07 | klf |
| Girder Reactions | 107.19 | kips |

Level 2

| | | | |
|---|---|---|---|
| Floor System Load | 1.12 | klf | |
| Beam Self weight | 0.132 | klf | |
| Dead Load | 1.25 | klf | |
| Live Load | 1.20 | klf | |
| $W_{u\text{-}2}$ | 3.42 | klf | |
| Girder Reactions | 234.66 | kips | |
| $Y_R$ | 222.75 | kips | |
| $Y_2$ | 369.36 | kips | |
| $\alpha$ | 1 | | |
| $N_R$ | 0.446 | kips | Eq C2-2 |
| $N_2$ | 0.739 | kips | Eq C2-1 |

Eccentricity

| | | |
|---|---|---|
| Column Web Thickness, $t_w$ | 0.295 | in |
| Connection Distance (Assumed) | 2.5 | in |
| e | 2.6475 | **in** |
| Say 3" eccentricity | | |

***Run Computer model in SAP2000***

## Design of Bottom Columns (Members 3 & 5)

| First Order Analysis Forces | Top | | Bottom | |
|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 295.93 | kips | 295.93 | kips |
| Ultimate Moment, $M_{unt}$ | 13.37 | kip*ft | 0 | kip*ft |

| | | | | | |
|---|---|---|---|---|---|
| Unbraced Length, $L_b$ | 15 | ft | | | |
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft |

Section Used *W12x50*

Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 71.9 | $in^3$ | $S_x$ | 64.2 | $in^3$ |
| Moment of Inertia, $I_x$ | 391 | $in^4$ | $r_{ts}$ | 2.25 | in |
| $r_x$ | 5.18 | in | $h_o$ | 11.6 | in |
| $A_g$ | 14.6 | in2 | J | 1.71 | $in^4$ |
| $b_f$ | 8.08 | in | $c_w$ | 1880 | $in^6$ |
| $t_f$ | 0.64 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.37 | in | | | |

Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.92 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 23.8 | ft | Table 3-2 |
| $\phi$BF | 5.98 | kips | Table 3-2 |
| $\phi M_{px}$ | 270 | kip*ft | Table 3-2 |

| Slenderness Characteristics | Flexure $\lambda_p$ | Flexure $\lambda_r$ | Compression $\lambda_r$ |
|---|---|---|---|
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | NONSLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

Axial Capacity

| | | | |
|---|---|---|---|
| K | 1 | | K=1 for columns in DAM |
| Kl | 15 | ft | |
| $P_c$ | 355 | kips | Table 4-1 |

Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 13.37 | kip*ft | $M_{ult}$ | |
| $M_A$ | 4.79 | kip*ft | SAP | |
| $M_B$ | 9.58 | kip*ft | SAP | |
| $M_C$ | 14.37 | kip*ft | SAP | |
| $C_b$ | 1.29 | | | Eq F1-1 |
| Zone 1, $M_c$ | 270 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 270.00 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 84.73 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 453.32 | kip*ft | | Eq F2-3 |
| $M_c$ | 270.00 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 295.93 | kips | $P_{unt}$ |
| $P_y$ | 730.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.41 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2763.25 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *0.6-0.4*(M1/M2)* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

Summary

| | | | |
|---|---|---|---|
| $M_r$ | 13.37 | kip*ft | *Eq A-8-1* |
| $M_c$ | 270.00 | kip*ft | *Referenced Above* |
| $P_r$ | 295.9 | kips | *Eq A-8-2* |
| $P_c$ | 355 | kips | *Referenced Above* |

Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.83 | | |
| $P_r/P_c$>0.2 | 0.88 | | Eq H1-1a |
| $P_r/P_c$<0.2 | 0.47 | | Eq H1-1b |
| Design Check | 0.88 | **SECTION OK** | |

## Design of Top Columns (Members 4 & 6)

| First Order Analysis Forces | Top | | Bottom | |
|---|---|---|---|---|
| Ultimate Axial Load, $P_{unt}$ | 103.15 | kips | 103.15 | kips |
| Ultimate Moment, $M_{unt}$ | 12.57 | kip*ft | 0 | kip*ft |

| | | | | | |
|---|---|---|---|---|---|
| Unbraced Length, $L_b$ | 15 | ft | | | |
| Braced Frame Bay Width, $W_b$ | 36 | ft | Plan Length, L | 108 | ft |
| Moment Frame Bay Width, $W_m$ | 24 | ft | Plan Width, W | 72 | ft |

Section Used *W12x40*

### Section Properties (Table 1-1)

| | | | | | |
|---|---|---|---|---|---|
| Plastic Section Modulus strong axis, $Z_x$ | 57 | $in^3$ | $S_x$ | 51.5 | $in^3$ |
| Moment of Inertia, $I_x$ | 307 | $in^4$ | $r_{ts}$ | 2.21 | in |
| $r_x$ | 5.13 | in | $h_o$ | 11.4 | in |
| $A_g$ | 11.7 | in2 | J | 0.906 | $in^4$ |
| $b_f$ | 8.01 | in | $c_w$ | 1440 | $in^6$ |
| $t_f$ | 0.515 | in | | | |
| h | 9.25 | in | | | |
| $t_w$ | 0.295 | in | | | |

### Beam Properties

| | | | |
|---|---|---|---|
| Full plastic yielding unbraced length, $L_p$ | 6.85 | ft | Table 3-2 |
| Elastic/Inelastic LTB unbraced length, $L_r$ | 21.1 | ft | Table 3-2 |
| $\phi$BF | 5.54 | kips | Table 3-2 |
| $\phi M_{px}$ | 214 | kip*ft | Table 3-2 |

| | Flexure | | Compression |
|---|---|---|---|
| Slenderness Characteristics | $\lambda_p$ | $\lambda_r$ | $\lambda_r$ |
| Flanges | 9.15 | 24.08 | 13.49 |
| | NONCOMPACT | | SLENDER |
| Web | 90.55 | 137.27 | 35.88 |
| | COMPACT | | NONSLENDER |

### Axial Capacity

| | | | |
|---|---|---|---|
| K | 1 | | K=1 for columns in DAM |
| Kl | 15 | ft | |
| $P_c$ | 281 | kips | Table 4-1 |

Flexure Capacity

| | | | | |
|---|---|---|---|---|
| Zone? | ZONE 2 | | | |
| $M_{max}$ | 12.57 | kip*ft | $M_{ult}$ | |
| $M_A$ | 14.37 | kip*ft | SAP | |
| $M_B$ | 9.58 | kip*ft | SAP | |
| $M_C$ | 4.79 | kip*ft | SAP | |
| $C_b$ | 1.24 | | | Eq F1-1 |
| Zone 1, $M_c$ | 214 | kip*ft | | Eq F2-1 |
| Zone 2, $M_c$ | 219.14 | kip*ft | | Eq F2-2 |
| Zone 3, $F_{cr}$ | 71.46 | ksi | | Eq F2-4 |
| Zone 3, $M_c$ | 306.68 | kip*ft | | Eq F2-3 |
| $M_c$ | 219.14 | kip*ft | | |

Determine $\tau_b$

| | | | |
|---|---|---|---|
| $P_r$ | 103.15 | kips | $P_{unt}$ |
| $P_y$ | 585.00 | kips | $A_g$*$F_y$ |
| $\alpha$ | 1 | | |
| $\alpha P_r/P_y$ | 0.18 | | |
| $\tau_b$ | 1.00 | | Eq C2-2a |

Determine B1

| | | | |
|---|---|---|---|
| $P_{e1}$ | 2169.61 | kips | *Eq A-8-5* |
| Modification Coefficient, $C_{mx}$ | 0.60 | | *0.6-0.4*(M1/M2)* |
| $B_1$ | 1.00 | | *Eq A-8-3* |

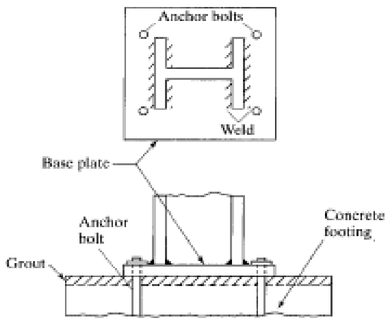
Summary

| | | | |
|---|---|---|---|
| $M_r$ | 12.57 | kip*ft | *Eq A-8-1* |
| $M_c$ | 219.14 | kip*ft | *Referenced Above* |
| $P_r$ | 103.2 | kips | *Eq A-8-2* |
| $P_c$ | 281 | kips | *Referenced Above* |

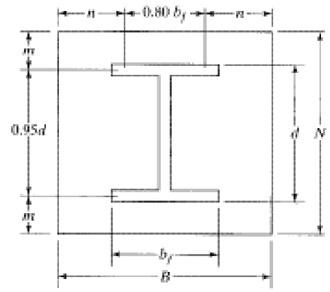
Combined Forces Ineraction Equation

| | | | |
|---|---|---|---|
| $P_r/P_c$ | 0.37 | | |
| $P_r/P_c>0.2$ | 0.42 | | Eq H1-1a |
| $P_r/P_c<0.2$ | 0.24 | | Eq H1-1b |
| Design Check | 0.42 | **SECTION OK** | |

## CONNECTIONS




(a)




| | |
|---|---|
| Member Ref: | 23 |
| Frame: | Moment |
| Floor: | First |
| Member: | Column |
| Ref. 2: | 1-M |

## ASSUMPTIONS:

| | | | |
|---|---|---|---|
| **Footing dimensions** | B= | 9 | ft |
| | L= | 9 | ft |
| | A= | 81 | $ft^2$ |

**Loads*

## 1. MATERIAL PROPERTIES:

| | | | |
|---|---|---|---|
| Modulus of Elasticity: | E = | 29000 | ksi |
| Shear Modulus: | G = | 11200 | ksi |
| Yield Strength: | $F_{y}$ = | 36 | ksi |
| Concrete Compressive | f'c = | 3 | ksi |

## 2. LOADS:

| | | | |
|---|---|---|---|
| Dead Load | DL= | 200 | kip |
| Live Load | LL= | 300 | kip |
| Factors | $\phi_t$= | 0.9 | |
| | $\phi_r$= | 0.75 | |
| Compression | $\phi_c$= | 0.65 | |

LRFD

**1) Demand:**

Load | Pu = 295 kip

**1. PREVIOUS MEMBER GEOMETRIC INFORMATION:** **Demand**

| | | | |
|---|---|---|---|
| **Previous Selection:** | | **W10X68** | |
| Plastic Modulus | Z = | 85.3 | $in^3$ |
| Flange Width | bf = | 10.1 | in |
| Depth: | d = | 10.4 | in |

**2. NEW MEMBER GEOMETRIC INFORMATION:**

| | | | | |
|---|---|---|---|---|
| **Base Plate:** | | | | |
| Width | Bw = | 12 | in | 15 *Project Information* |
| Length | Nl= | 12 | in | Least volume |
| Area | A = | 144 | $in^2$ | |
| Thickness | t = | 1.00 | in | |

| | | | |
|---|---|---|---|
| **Minimal Area Check:** | Amin = | 89.0 | |
| | A>Amin ? | OK | |
| **Ratio Check** | | 31.18 | |
| | V(A2/A1)= | OK | |
| **Given Column Used:** | m= | 1.06 | in |
| | n= | 1.96 | in |
| | n'= | 2.56 | in |
| | l = | 2.56 | in |
| Optimization | m~n | | |
| | D = | 0.90 | |
| | N = | 10.33 | |
| | B = | 8.61 | |
| | Acheck= | 88.99 | $in^2$ |
| Check | bf.d = | 105.04 | $in^2$ |
| | Acheck>bf.df ? | YES | |

**Concrete Bearing Strength Check**

| | | |
|---|---|---|
| $\phi_{c.}$ P= | 477.36 | k |
| $\phi_{c.}$ P > Pu ? | YES | |

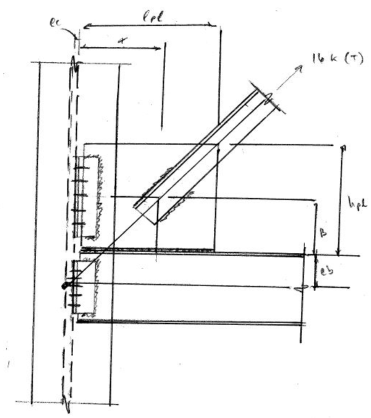
| | | | | | | |
|---|---|---|---|---|---|---|
| **USE** | | **1** | **18.00** | **18** | in | **ANSWER** |
| | w/ | 2 in clearance for 3/4" diameter bolts | | | | |

Prepared by: Matthew Laskey  Date: 11/29/2014
Checked by: Ana Gouveia

This calculation is representative of all second floor bracing conncections. This is due to the symmetrical values
and the loading characteristics. The connection is also over designed, so any irregularities will be ok by inspection
The tensile forces acting on the connection from the brace below. Will also reduce the forces on the connection

Connection Information:

| | |
|---|---|
| Grid Line: | 2 |
| Column Line: | A |
| Elevation: | 2nd Floor |
| | |
| Story ht | 15 ft |
| Run | 36 ft |
| Angle from vert | 67.38 deg |
| | |
| P(t/c) | 16 k |
| E | 29000 ksi |




| Column | | | Beam | | | Brace | | | |
|---|---|---|---|---|---|---|---|---|---|
| Section | W12x50 | | Section | W14x30 | | section | L6x4x.5 | | |
| dbm | 12.2 | in | dbm | 13.8 | in | wbr | | 6 | in |
| Fy | 50 | ksi | Fy | 50 | ksi | Fy | | 46 | ksi |
| tf | 0.64 | in | k | 0.785 | in | Weld1 (br to gpl) | | 3 | /16 in |
| tw | 0.37 | in | tw | 0.27 | in | lw=lamin | | 6 | in |

| Gusset plate | | |
|---|---|---|
| hpl | 18 | in |
| lpl | 12 | in |
| tpl | 0.375 | in |
| fy | 36 | ksi |
| Weld2 (gpl to bm) | 6 | /16 in |
| L1 (whit sec) | 12.40 | in |

Note: 1) highest edge of brace set at 2" above top of beam
2) left most edge of brace set at 1" from edge of connection angle

| | | | Angle | | | | Fy | Bolts | |
|---|---|---|---|---|---|---|---|---|---|
| | | | size | | | Length | Fu | | |
| Connection angles | Desc | n rows | osl / gage | leg2 / weld | Thick | in | ksi | diam | spec |
| Gusset pl to column | dbl angle | 5 | 4 | 3 | 0.5 | 15 | 36 | 0.75 | A325 |
| | | | 2.81 | 3 | | | 58 | | |
| Beam to column | dbl angle | 5 | 4 | 3 | 0.5 | 15 | 36 | 0.75 | A325 |
| | | | 2.81 | 3 | | | 58 | | |

Phi 0.75

**1. Brace to gusset connection**

lamin load 1.077586 in > 6 ? USE 6 in

Use lamin= 6 in

**2. calc min plate size**

Check lplmin (min length to fit brace)

lplmin=(leg2-.5"setback)+1"clear+(wbrcos(theta))+(lamin)sin(theta)

| | | | | | |
|---|---|---|---|---|---|
| lplmin= | 11.35 | in | > | 12 | USE lpl stated |
| Use lplmin | 12.00 | = | | 12 | in |

Check hplmin (min height to fit brace)
hplmin= 2" clear + (wbrsin(theta))+(laminCos(theta))

| | | | | |
|---|---|---|---|---|
| hplmin= | 9.85 | in | > | 18 |
| Use hplmir | 18.00 | in | | |

**3. Whitmore section check** 0.449219
Lwmin= 12.9282 in

calc tension yielding on whit sec (assumes all Lw in Gpl)

| | | | | | | |
|---|---|---|---|---|---|---|
| Ra | 130.8981 | k | > | 16 | ? | Plate OK |

calc compr buckling on whit sec of Gpl (assume all Lw in Gpl)

| | | |
|---|---|---|
| K | 0.5 | |
| L1 | 12.40 | |
| r | 0.108253 | |
| KL/r | 57.27279 | check if Kl/r<>25 eq for Ra buclking |
| Lambda c | 0.642319 | |
| Fcr | 30.29056 | |
| Ra buckling | 110.1382 | k > 16 ? Plate OK |

Geometry and force parameters

| | | |
|---|---|---|
| tan(theta)= | 2.399984 | |
| eb=dbm/2 | 6.9 | in |
| ec=dcol/2 | 0.185 | in |
| Beta= (((ng-1)+3")/2)+ | 9 | in |

alphaideal=ebtan(theta)-ec+betatan(theta)

| | | |
|---|---|---|
| alphaideal | 37.97 | in |

aphaactual= lp/2+.5"setback

| | | |
|---|---|---|
| alphaactual | 6.5 | in |
| Difference | -31.47 | in |

**4. Forces**
r=sqrt((alpahi+ec)^2+(beta+eb)^2)
r= 41.33977 in

At gsset to col

| | | |
|---|---|---|
| Vnc=(beta/r)Pn | 3.483329 | k |
| Hnc=(ec/r)Pn | 0.071602 | k |

At gusset to beam

| | | |
|---|---|---|
| Vnb= (eb/r)Pn | 2.670552 | k |
| Hnb= (alpha ideal/r)Pn | 14.69761 | k |

At beam to col

| | | |
|---|---|---|
| H= Hnc+Hnb | 14.76922 | k |
| V=Vnb+Vnc | 6.153881 | k |

**5. Gusset to column**
Check bolts:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| tensile force per bolt | rnt=Hc/2n | 0.00716 | k/bolt | < | 19.9 | allowable | Bolts OK |
| Shear force per bolt | rnv=Vnc/2 | 0.348333 | k/bolt | < | 7.38 | (slip critical) | |
| | | | | Bolts OK | | | |
| fv=rnv/Abolt | | 0.788464 | ksi | | | | |

Check bearing strength at bolt holes

| | | | | |
|---|---|---|---|---|
| rbrg=phi*2.4*dbolt*tl*Fu | 29.3625 k/bolt | > | 0.348332885 | Bearing strength OK |

**6. Check Angle**

| | | | |
|---|---|---|---|
| Prying action | b=g-tl/2 | 2.56 | in |
| | a=osl-g | 1.19 | in |
| | b'=b-d/2 | 2.185 | in |
| | a'=a+d/2 | 1.565 | in |
| | p=b'/a' | 1.396166 | |
| | d'=d+1/16 | 0.8125 | in |
| | p=L/n | 3 | in |
| | small delta=1-(d'/p) | 0.729167 | in |

Beta = (1/p)*(B/T)-1)  3.581236  if Beta>1 set alpa'=  1

treq=sqrt((4.44*rnt*b')/(p*Fu*(1+deltaalpha')))

treq=  0.022273 in  <  0.5 Angle thickness OK

**7. Check Weld of Angle to Gusset Plate**

fillet welds on 3 sides of both angles

Pnc=sqrt(Hnc^2+Vnc^2)

Pnc=  3.484065

Theta= arctan(Hnc/Vnc)

theta=  34.13 deg

Length of Dbl Angle=  15 in

kl= leg2-1/2" setback =  2.5 in

k=kl/l=  0.167

From Table 8-8 w/ theta=30 Find x by interpolation.

x=  0.022

xl=  0.33 in

al= leg2-xl  2.67 in

a=al/l=  0.178

| | | | | | |
|---|---|---|---|---|---|
| Find C by interpolation: | | | 0.1 | 0.167 | 0.2 |
| C= | 2.69 | 0.15 | 2.43 | 2.7784 | 2.95 |
| | | 0.178 | | 2.69 | |
| ϕ= | 0.75 | 0.2 | 2.29 | 2.625 | 2.79 |

ϕRn=  181.575 k

ϕRn>  Pnc=  3.484065 Weld OK

**8. Check strength of angles**

Check shear yielding due to Vnc

ϕ=  1

Ag 1 angle=  7.5 in2

ϕRn=  ϕ.6Fy2Ag=  324 k  > Vnc?=  3.483328849 OK  (J4-3)

Check Shear Rupture

ϕ=  0.75

Anv=  5.46875

ϕRn=  ϕ.6Fu2Anv=  285.4688 k  > Vnc?=  3.483328849 OK  (J4-4)

Check Block Shear Rupture

ϕRn=  ϕ[.6FuAnv + UbsFuAnt < .6FyAgv+UbsFuAnt]*2

ϕ=  0.75

Ubs=  1 Tension stress uniform

| | | | | | |
|---|---|---|---|---|---|
| | .6FuAnv + UbsFuAnt= | | 507.5 | | |
| | .6FyAgv+UbsFuAnt= | | 479.1875 | | |
| ϕRn= | 718.7813 k | >Vnc?= | 3.483329 OK | | (J4-5) |

Check Column Flange
tflange= 0.64 > treq= 0.022273 OK

**9. Gusset to Beam Design**

| | | |
|---|---|---|
| Hnb= | 14.69761 k | [From Above] |
| Vnb= | 2.670552 k | [From Above] |
| Mb=Hbeb= | 101.4135 kin | |

Sx=(hpl-1)^2/3 96.33333 in^2 Weld treated as a line.

Check gusset stress
fv=Hb/lpltpl= 3.266137 < ϕ.6Fy?= 21.6 OK

fa= 0.773649 ksi <ϕFy= 36 OK

User note on pg 16.1-126: When required stress is less than 30% of available stress, combinec
effects of stress need not be considered.

Weld Load
fr= 0.629346 k/in
For ductility multiply by 1.4 = 0.881085 k/in

Weld Capacity AISC Part 8

| | | | |
|---|---|---|---|
| D= | 0.375 in | | |
| ϕ= | 0.75 | | |
| θ= | 71.10 | | |
| △u= | 0.024192 | < .17D= | 0.6375 |
| △i= | 0.024192 | | |
| △m= | 0.019848 | | |
| p= | 1.218859 | | |
| f(p)= | 0.993585 | | |
| Weld Cap= | 12.11488 k/in | > 1.4fr?= | 0.881 OK |

Check beam web yielding @ gusset plate
fb= 1.286635 ksi <ϕFybm?= 37.5 OK

**10. Beam to Column**

| | | |
|---|---|---|
| Hnc= | 14.76922 | [From Above] |
| Vnc= | 6.153881 | [From Above] |
| n= | 5 | |

Check Bolts

| | | | |
|---|---|---|---|
| Abolt= | 0.441786 in2 | | |
| Tensile force per bolt | | | |
| rnt= | 1.476922 k/bolt | < | 19.9 OK |
| Shear force per bolt | | | |
| rnv= | 0.615388 k/bolt | < | 7.38 OK |
| fv= | 1.392954 ksi | | |

Tension-shear interaction

| | | | |
|---|---|---|---|
| ft= | 114.3534 < | | 90 Use 90 |
| ϕrn= | 29.82059 k | >rnt? | OK |

Check Angle

Prying
Because using same sized angle, all values same as section 6.
treq= 0.022273 in

check weld of angles to column
fillet welds on 3 sides of both angles
Pnc= 16 k
θ= 30.00

Length of Dbl Angle= 15 in
kl= leg2-1/2" setback = 2.5 in
k=kl/l= 0.167
From Table 8-8 w/ theta=30 Find x by interpolation.
x= 0.022
xl= 0.33 in
al= leg2-xl 14.67 in
a=al/l= 0.978

| | | | | | | |
|---|---|---|---|---|---|---|
| Find C by interpolation: | | | | 0.1 | 0.167 | 0.2 |
| C= | 2.69 | | 0.15 | 2.43 | 2.7784 | 2.95 |
| C1= | 1 | | 0.178 | | 2.69 | |
| ϕ= | 0.75 | | 0.2 | 2.29 | 2.625 | 2.79 |

ϕRn= 181.575 k
ϕRn> Pnc= 16 Weld OK

Check Strength of angles
Check shear yielding due to Vnc
ϕ= 1
Ag 1 angle= 7.5 in2
ϕRn= ϕ.6Fy2Ag= 324 k > Vnc?= 6.153880967 OK (J4-3)

Check Shear Rupture
ϕ= 0.75
Anv= 5.46875
ϕRn= ϕ.6Fu2Anv= 285.4688 k > Vnc?= 6.153880967 OK (J4-4)

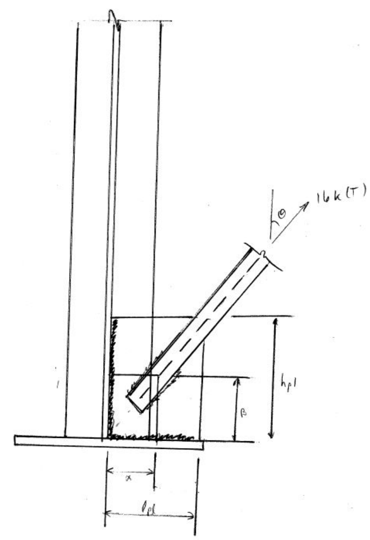
Check Block Shear Rupture
ϕRn= ϕ[.6FuAnv + UbsFuAnt < .6FyAgv+UbsFuAnt]*2
ϕ= 0.75
Ubs= 1 Tension stress uniform

.6FuAnv + UbsFuAnt= 507.5
.6FyAgv+UbsFuAnt= 479.1875
ϕRn= 718.7813 k >Vnc?= 6.153881 OK (J4-5)

Check Column Flange
tflange= 0.64 > treq= 0.022273 OK

Prepared by: Matthew Laskey Date: 11/29/2014
Checked by: Ana Gouveia

This calculation is representative of all first floor bracing conncections. This is due to the symmetrical values and the loading characteristics. The connection is also over designed, so any irregularities will be ok by inspection.




Connection Information:

| | |
|---|---|
| Grid Line: | 2 |
| Column Line: | A |
| Elevation: | 1st Floor |

| | | |
|---|---|---|
| Story ht | 15 | ft |
| Run | 36 | ft |
| Angle from vert | 67.38 | deg |

| | | |
|---|---|---|
| P(t/c) | 16 | k |
| E | 29000 | ksi |

| Column | | |
|---|---|---|
| Section | W12x50 | |
| dbm | 12.2 | in |
| Fy | 50 | ksi |
| tw | 0.37 | in |
| tbplate | 0.75 | in |
| wbplate | 16 | in |

| Brace | | |
|---|---|---|
| section | L6x4x.5 | |
| wbr | 6 | in |
| Fy | 46 | ksi |
| Weld1 (br to gpl) | 3 | /16 in |
| lw=lamin | 8 | in |

| Gusset plate | | |
|---|---|---|
| hpl | 18 | in |
| lpl | 12 | in |
| tpl | 0.375 | in |
| fy | 36 | ksi |
| Weld2 (gpl to bm) | 6 | /16 in |
| L1 (whit sec) | 12.40 | in |

Note: 1) highest edge of brace set at 2" above top of beam
2) left most edge of brace set at 1" from edge of connection angle

Phi 0.75

**1. Brace to gusset connection**

| | | | | |
|---|---|---|---|---|
| lamin load | 1.077586 | in > | 6 ? | **USE 6 in** |
| Use lamin= | **6** | in | | |
| U= | 0.75 | | | |
| Lamin= | 8 | in | | |

**2. calc min plate size**

Check lplmin (min length to fit brace)

| | | | | | |
|---|---|---|---|---|---|
| lplmin=(leg2-.5"setback)+1"clear+(wbrcos(theta))+(lamin)sin(theta) | | | | | |
| lplmin= | 11.69 | in | > | 12 | USE lpl stated |
| Use lplmin | 12.00 | = | | 12 in | |

| | | | | |
|---|---|---|---|---|
| Check hplmin (min height to fit brace) | | | | |
| hplmin= 2" clear + (wbrsin(theta))+(laminCos(theta)) | | | | |
| hplmin= | 10.62 | in | > | 18 |
| Use hplmir | 18.00 | in | | |

**3. Whitmore section check** 0.449219

Lwmin= 15.2376 in

calc tension yielding on whit sec (assumes all Lw in Gpl)

| | | | | | |
|---|---|---|---|---|---|
| Ra | 154.2807 | k | > | 16 ? | Plate OK |

calc compr buckling on whit sec of Gpl (assume all Lw in Gpl)

| | | | | | |
|---|---|---|---|---|---|
| K | 0.5 | | | | |
| L1 | 12.40 | | | | |
| r | 0.108253 | | | | |
| KL/r | 57.27279 | | | | |
| Lambda c | 0.642319 | | | | |
| Fcr | 30.29056 | | | | |
| Ra buckling | 129.8125 | k | > | 16 ? | Plate OK |

Geometry and force parameters

| | | |
|---|---|---|
| tan(theta)= | 2.40 | |
| eb=dbm/2 | 0.75 | in |
| ec=dcol/2 | 0.375 | in |
| Beta= | 1.242722 | in |
| alphaactua | 4.41 | in |
| Beta actl= | 9 | |
| alphaactua | 4.41 | in |
| Difference | 7.76 | in |

**4. Forces**

r=sqrt((alpahi+ec)^2+(beta+eb)^2)

r= 10.85978 in

At gsset to col

| | | |
|---|---|---|
| Vnc=(beta/r)Pn | 13.25994 | k |
| Hnc=(ec/r)Pn | 0.552497 | k |

At gusset to base plate

| | | |
|---|---|---|
| Vnb= (eb/r)Pn | 1.104995 | k |
| Hnb= (alpha ideal/r)Pn | 6.493686 | k |

**5. Gusset to column Design**

| | | | |
|---|---|---|---|
| Hnc= | 0.552497 | k | [From Above] |
| Vnc= | 13.25994 | k | [From Above] |

Sx=(hpl-1)^2/3 96.33333 in^2 Weld treated as a line.

Check weld strength of vertical weld

| | | | | | | |
|---|---|---|---|---|---|---|
| fx=fa= | 0.06074 | k/in | | | | |
| fy=fv= | 0.389998 | k/in | | | | |
| | | | | | | |
| fr= | 0.3947 | k/in | | | | |
| For ductility, multiply by 1.4= | | | 0.55258 | k/in | | |
| | | | | | | |
| Weld Capacity | | AISC Part 8 | | | | |
| D= | 0.375 | in | | | | |
| ϕ= | 0.75 | | | | | |
| θ= | 9.86 | | | | | |
| △u= | 0.067608 | < .17D= | 0.6375 | | | |
| △i= | 0.067608 | | | | | |
| △m= | 0.035516 | | | | | |
| p= | 1.903577 | | | | | |
| f(p)= | 0.733282 | | | | | |
| Weld Cap= | | 8.94 | k/in | > 1.4fr?= | 0.553 | OK |

**6. Gusset to Base Plate**

| | | | |
|---|---|---|---|
| Hnb= | 6.493686 | k | [From Above] |
| Vnb= | 1.104995 | k | [From Above] |

Check weld strength of horizontal weld

| | | | | | | |
|---|---|---|---|---|---|---|
| lwh= | 6.815 | in | | | | |
| fx=fv= | 0.48 | k/in | | | | |
| fy=fa= | 0.08 | k/in | | | | |
| | | | | | | |
| fr= | 0.48 | k/in | | | | |
| for ductility, multiply by 1.4= | | | 0.677 | k/in | | |
| | | | | | | |
| Weld Capacity | | AISC Part 8 | | | | |
| D= | 0.375 | in | | | | |
| ϕ= | 0.75 | | | | | |
| θ= | 9.66 | | | | | |
| △u= | 0.068 | < .17D= | 0.6375 | | | |
| △i= | 0.068 | | | | | |
| △m= | 0.036 | | | | | |
| p= | 1.909 | | | | | |
| f(p)= | 0.727998 | | | | | |
| Weld Cap= | | 8.88 | k/in | > 1.4fr?= | 0.677 | OK |

**7. Gusset plate stresses**

a. Normal load

normal load = larger of fxtop or fybott

| | | |
|---|---|---|
| fxtop= | 0.06 | |
| fybott= | 0.08 | |
| Normal load= | 0.08 | k/in |

Plate stress= normalload* 2welds* 1in long/ tpl*1in long

| | | | | | |
|---|---|---|---|---|---|
| plate stress= | 0.432378 | ksi | < ϕFy?= | 32.4 | OK |

b. Shear Load

Shear load larger of fytop or fxbott

| | | |
|---|---|---|
| fytop | 0.389998 | |
| fxbott | 0.48 | |
| shear load | 0.48 | k/in |

Plate stress= shearload*2welds*1in long / tpl*1in long

| | | | | | |
|---|---|---|---|---|---|
| Plate stress= | 2.540939 | ksi | < ϕ.6Fy?= | 19.44 | OK |

# Moment Frame Connection Design-Level 2

**Given:**

| | | | | | |
|---|---|---|---|---|---|
| **Girder** | W12x53 | | **Column** | W10x68 | |
| depth, d | 12.1 | in | depth, d | 10.4 | in |
| $t_f$ | 0.575 | in | $t_f$ | 0.77 | in |
| $b_f$ | 10 | in | $b_f$ | 10.1 | in |
| $t_w$ | 0.345 | in | $t_w$ | 0.47 | in |
| | | | k | 1.27 | in |

**Flange Plates**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| t | 3/4 | in | $F_y$ | 36 | ksi | $F_u$ | 58 | ksi |
| w | 10 | in | <0.15*bf (column)? | | CHECK FLANGE LOCAL BENDING | | | J10.1 |
| unbraced length, $L_p$ | 3 | in | | | | | | |
| # of rows | 2 | | Gage | 7 | in | Spacing | 3 | in |
| A325N, bolt dia. | 7/8 | in | Hole | std | | # bolts | 10 | per flange |
| | | | | | | Conn. Length | 12 | in |
| | | | | | | Edge Distance | 2 | in |

**Flange-Column Weld**

| | | |
|---|---|---|
| Electrode Strength, $F_u$ | 70 | ksi |
| Flange CJP Weld: | | |
| Length | 10 | in |
| Size | 1/2 | in |

**Double Angle Connection**

| | | | | | | |
|---|---|---|---|---|---|---|
| t | 1/4 | in | weld size | 1/4 | in | |
| # bolts/row, n | 4 | | Weld Lengt | 11.5 | in | (L) |
| A325N, bolt dia. | 3/4 | in | Hole | std | | |

**Beam End Forces (LRFD)**

| | | |
|---|---|---|
| Shear, V | 78.22 | kips |
| Moment, M | 239.01 | kip*ft |

**Determine:**

Connection adequacy and need for column stiffeners

## Solution

**Determine Flange and Web Shear Forces**

| | | | |
|---|---|---|---|
| Flange Force, T=C | 237.03 | kips | M/d |
| Web Shear Force | 78.22 | kips | Given |

**Tensile Strength of Flange Plate**

a. Tensile Yield, $P_n$

| | | | |
|---|---|---|---|
| Gross Area, $A_g$ | 7 1/2 | $in^2$ | |
| Nom. Tensile Yield, $P_n$ | 270 | kips | Eq D2-1 |
| $\Phi$ | 0.9 | | |
| $\Phi P_n$ | 243 | kips | |
| | OK | | |

b. Tensile Rupture, $P_n$

| | | |
|---|---|---|
| Effective Area, $A_e$ | 6.00 | $in^2$ |
| Nom. Tensile Rupt., $P_n$ | 348.00 | kips |
| $\Phi$ | 0.75 | |
| $\Phi P_n$ | 261.00 | kips |
| | OK | |

**Flange Plate Block Shear**

| | | |
|---|---|---|
| $A_{nt}$ | 3.00 | $in^2$ |
| $A_{nv}$ | 28.50 | $in^2$ |
| $A_{gv}$ | 42.00 | $in^2$ |
| x=y for WT6x26.5 | 1.02 | in |
| U | 1.00 | |

| | | |
|---|---|---|
| $0.6*F_u*A_{nv}+U*F_u*A_{nt}$ | $0.6*F_y*A_{gv}+U*F_u*A_{nt}$ | Eq J4-5 |
| 1165.8 kips | 1081 kips | |

| | | |
|---|---|---|
| $R_n$ | 1081.20 | kips |
| $\Phi$ | 0.75 | |
| $\Phi R_n$ | 810.90 | kips |
| | OK | |

**Flange Plate Bolt Bearing**

Find Minimum clear distance, $L_c$

| | | |
|---|---|---|
| Interior $L_c$ | 2.00 | in |
| Edge $L_c$ | 1.50 | in |
| Min $L_c$ | 1.50 | in |

| | Tearout | | Deformation | | |
|---|---|---|---|---|---|
| | 78.30 | kips/bolt | 91.35 | kips/bolt | Eq J3-6A |

| | | |
|---|---|---|
| Nominal Strength, $R_n$ | 78.30 | kips/bolt |
| Nominal Plate Bearing Strength, $P_n$ | 783 | kips |
| $\Phi$ | 0.75 | |
| $\Phi P_n$ | 587.25 | kips |
| | OK | |

**Bolt Shear**

| | | | |
|---|---|---|---|
| Bolt Area, $A_b$ | 0.60 | $in^2$ | |
| A325N, $F_{nv}$ | 54 | ksi | Table J3.2 |
| $R_n$ | 32.47 | kips/bolt | |
| Bolt Shear Strength | 324.71 | kips | |
| $\Phi$ | 0.75 | | |
| $\Phi P_n$ | 243.53 | | |
| | OK | | |

**Flange Plate Compressive Strength**

| | | | |
|---|---|---|---|
| K | 0.65 | | Table C-A-7.1 |
| $r_y$ | 0.22 | in | t/sqrt(12) |
| $KL_p/r$ | 9.01 | | Eq J-4-6 Applies |
| Effective Plate width, $l_w$ | 10 | in | |
| Effective gross area, $A_{ge}$ | 7.5 | $in^2$ | |
| $P_n$ | 270 | kips | Nominal Compressive Strength, Eq J4-6 |
| $F_{cr}$ | 32.3 | ksi | Table 4-22 |
| $\Phi$ | 0.9 | | |
| $\Phi P_n$ | 243 | | |
| | OK | | |

**Evaluate Double-angle connection**

| | | | | |
|---|---|---|---|---|
| Strength of welds | 79.9 | kips | | Table 10-2 (Case II) |
| | OK | | | |
| Bolt/Angle Strength | 101 | kips | | Table 10-1 |
| | OK | | | |
| Bolt bearing on web | 351 | k/in | 121.10 kips | Table 10-1 |
| | OK | | | |

**Check Column with concentrated forces**

Flange local bending

| | | | |
|---|---|---|---|
| $R_n$ | 133.40 | kips | J10-1 |
| $\Phi$ | 0.9 | | |
| $\Phi R_n$ | 120.06 | kips | |
| | OK | | |

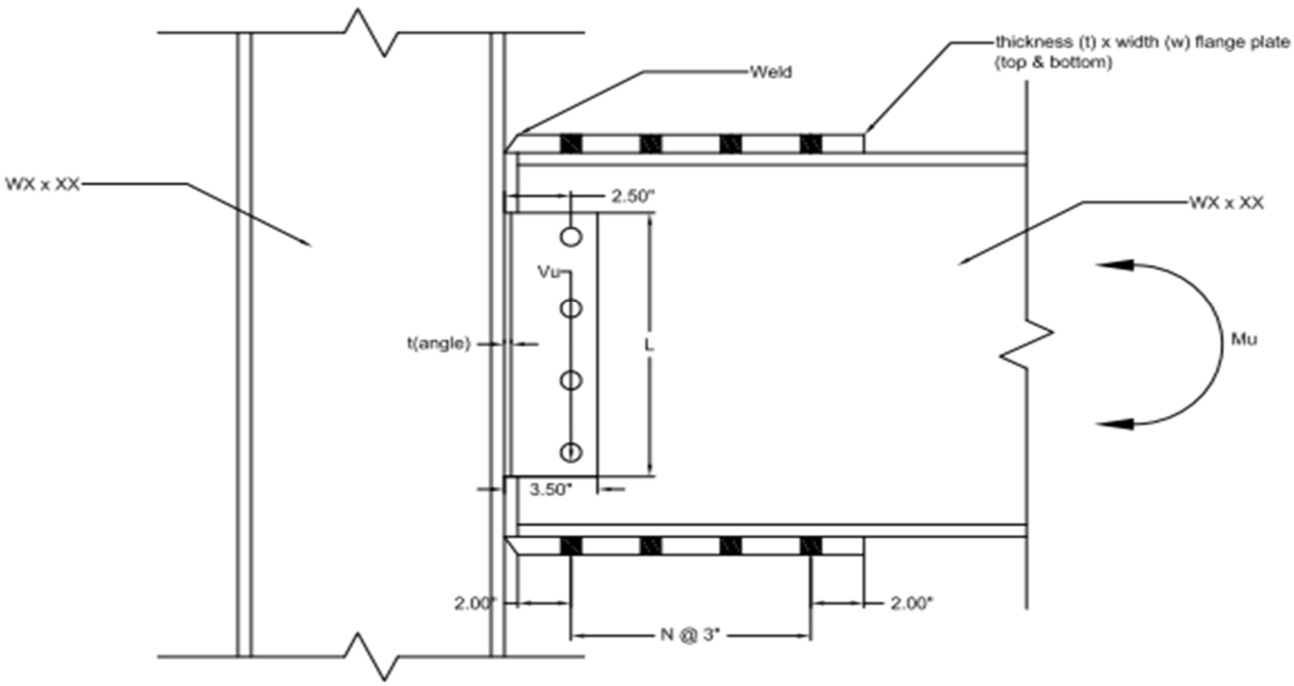
Web Local Yielding

| | | | |
|---|---|---|---|
| $R_n$ | 104.48 | kips | J10-3 |
| $\Phi$ | 1 | | |
| $\Phi R_n$ | 104.48 | kips | |
| | OK | | |

Web Crippling Parameters

| | | | |
|---|---|---|---|
| $R_n$ | 326.50 | | J10-4 |
| $\Phi$ | 0.75 | | |
| $\Phi R_n$ | 244.87 | | |
| | OK | | |

**Summary**


thickness (t) x width (w) flange plate
(top & bottom)
Weld
WX x XX
2.50"
WX x XX
Vu
t(angle)
L
Mu
3.50"
2.00"
2.00"
N @ 3"

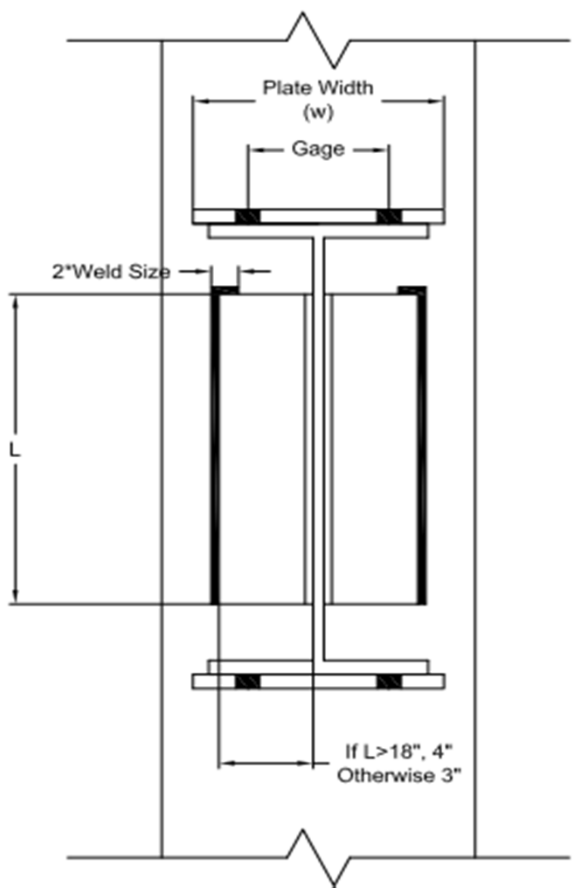


Plate Width
(w)
Gage
2*Weld Size
L
If L>18", 4"
Otherwise 3"

| | | | |
|---|---|---|---|
| Girder: | W12x53 | $M_u$ | 239.01 kip*ft |
| Column: | W10x68 | $V_u$ | 78.22 kips |

## Flange Plate:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Thickness, t | 3/4 | in | $F_y$ | 36 | ksi | $F_u$ | 58 ksi |
| width, w | 10 | in | | | | | |
| # of rows | 2 | | | | | | |
| Hole | std | | | | | | |
| A325N, bolt dia. | 7/8 | in | | | | | |
| # bolts | 10 | per flange | | | | | |
| Gage | 7 | in | | | | | |

## Flange-Column Weld:

| | | |
|---|---|---|
| Electrode Strength, $F_u$ | 70 | ksi |
| Flange CJP Weld: | | |
| Length | 10 | in |
| Size | 1/2 | in |

## Double Angle Connection:

Weld to column, bolt to girder

Angle properties:

| | | |
|---|---|---|
| t | 1/4 | in |
| L | 11.5 | in |
| Leg Connect. to Column | 3 | in |
| Leg connect. To girder | 3.5 | in |
| | | |
| weld size | 1/4 | in |
| Hole | std | |
| # bolts/row, n | 4 | (Bolts spaced at 3" O.C.) |
| A325N, bolt dia. | 3/4 | in |

# Moment Frame Connection Design-Roof Level

**Given:**

| | | | | | |
|---|---|---|---|---|---|
| **Girder** | W10x39 | | **Column** | W10x68 | |
| depth, d | 9.92 | in | depth, d | 10.4 | in |
| $t_f$ | 0.53 | in | $t_f$ | 0.77 | in |
| $b_f$ | 7.99 | in | $b_f$ | 10.1 | in |
| $t_w$ | 0.315 | in | $t_w$ | 0.47 | in |
| | | | k | 1.27 | in |

**Flange Plates**

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| t | 1/2 | in | $F_y$ | 36 | ksi | $F_u$ | 58 | ksi |
| w | 8 | in | <0.15*bf (column)? | CHECK FLANGE LOCAL BENDING | | | J10.1 | |
| unbraced length, $L_p$ | 3 | in | | | | | | |
| # of rows | 2 | | Gage | 6 | in | Spacing | 3 | in |
| A325N, bolt dia. | 7/8 | in | Hole | std | | # bolts | 4 | per flange |
| | | | | | | Conn. Length | 3 | in |
| | | | | | | Edge Distance | 2 | in |

**Flange-Column Weld**

| | | |
|---|---|---|
| Electrode Strength, $F_u$ | 70 | ksi |
| Flange CJP Weld: | | |
| Length | 8 | in |
| Size | 1/2 | in |

**Double Angle Connection**

| | | | | | | |
|---|---|---|---|---|---|---|
| Thickness, t | 1/4 | in | weld size | 1/4 | in | |
| # bolts/row, n | 3 | | Weld Lengt | 8.5 | in | (L) |
| A325N, bolt dia. | 3/4 | in | Hole | std | | |

**Beam End Forces (LRFD)**

| | | |
|---|---|---|
| Shear, V | 35.73 | kips |
| Moment, M | 77.18 | kip*ft |

**Determine:**

Connection adequacy and need for column stiffeners

**Solution**

**Determine Flange and Web Shear Forces**

| | | | |
|---|---|---|---|
| Flange Force, T=C | 93.36 | kips | M/d |
| Web Shear Force | 35.73 | kips | Given |

**Tensile Strength of Flange Plate**

a. Tensile Yield, $P_n$

| | | | |
|---|---|---|---|
| Gross Area, $A_g$ | 4 | $in^2$ | |
| Nom. Tensile Yield, $P_n$ | 144 | kips | Eq D2-1 |
| $\Phi$ | 0.9 | | |
| $\Phi P_n$ | 129.6 | kips | |
| | OK | | |

b. Tensile Rupture, $P_n$

| | | |
|---|---|---|
| Effective Area, $A_e$ | 3.00 | $in^2$ |
| Nom. Tensile Rupt., $P_n$ | 174.00 | kips |
| $\Phi$ | 0.75 | |
| $\Phi P_n$ | 130.50 | kips |
| | OK | |

**Flange Plate Block Shear**

| | | |
|---|---|---|
| $A_{nt}$ | 1.00 | $in^2$ |
| $A_{nv}$ | 7.00 | $in^2$ |
| $A_{gv}$ | 10.00 | $in^2$ |
| x=y for WT6x26.5 | 1.02 | in |
| U | 1.00 | |

| | | | | |
|---|---|---|---|---|
| $0.6*F_u*A_{nv}+U*F_u*A_{nt}$ | | $0.6*F_y*A_{gv}+U*F_u*A_{nt}$ | | Eq J4-5 |
| 301.6 | kips | 274 | kips | |

| | | |
|---|---|---|
| $R_n$ | 274.00 | kips |
| $\Phi$ | 0.75 | |
| $\Phi R_n$ | 205.50 | kips |
| | OK | |

**Flange Plate Bolt Bearing**

Find Minimum clear distance, $L_c$

| | | |
|---|---|---|
| Interior $L_c$ | 2.00 | in |
| Edge $L_c$ | 1.50 | in |
| Min $L_c$ | 1.50 | in |

| Tearout | | Deformation | | |
|---|---|---|---|---|
| 52.20 | kips/bolt | 60.9 | kips/bolt | Eq J3-6A |

| | | |
|---|---|---|
| Nominal Strength, $R_n$ | 52.20 | kips/bolt |
| Nominal Plate Bearing Strength, $P_n$ | 208.8 | kips |
| $\Phi$ | 0.75 | |
| $\Phi P_n$ | 156.6 | kips |
| | OK | |

**Bolt Shear**

| | | | |
|---|---|---|---|
| Bolt Area, $A_b$ | 0.60 | $in^2$ | |
| A325N, $F_{nv}$ | 54 | ksi | Table J3.2 |
| $R_n$ | 32.47 | kips/bolt | |
| Bolt Shear Strength | 129.89 | kips | |
| $\Phi$ | 0.75 | | |
| $\Phi P_n$ | 97.41 | | |
| | OK | | |

**Flange Plate Compressive Strength**

| | | | |
|---|---|---|---|
| K | 0.65 | | Table C-A-7.1 |
| $r_y$ | 0.14 | in | t/sqrt(12) |
| $KL_p/r$ | 13.51 | | Eq J-4-6 Applies |
| Effective Plate width, $l_w$ | 8 | in | |
| Effective gross area, $A_{ge}$ | 4 | $in^2$ | |
| $P_n$ | 144 | kips | Nominal Compressive Strength, Eq J4-6 |
| $F_{cr}$ | 32.3 | ksi | Table 4-22 |
| $\Phi$ | 0.9 | | |
| $\Phi P_n$ | 129.6 | | |
| | OK | | |

**Evaluate Double-angle connection**

| | | | | | |
|---|---|---|---|---|---|
| Strength of welds | 48.1 | kips | | | Table 10-2 (Case II) |
| | OK | | | | |
| Bolt/Angle Strength | 76.4 | kips | | | Table 10-1 |
| | OK | | | | |
| Bolt bearing on web | 263 | k/in | 82.85 | kips | Table 10-1 |
| | OK | | | | |

**Check Column with concentrated forces**

Flange local bending

| | | | |
|---|---|---|---|
| $R_n$ | 133.40 | kips | J10-1 |
| $\Phi$ | 0.9 | | |
| $\Phi R_n$ | 120.06 | kips | |
| | OK | | |

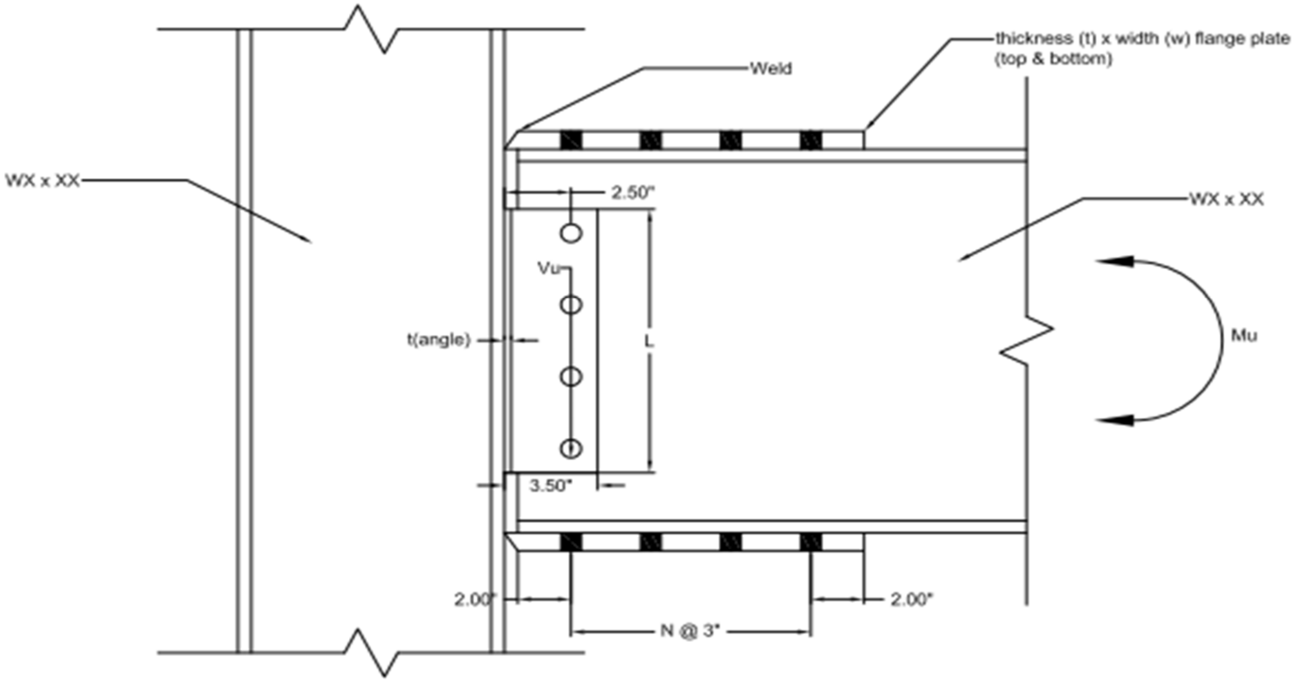
Web Local Yielding

| | | | |
|---|---|---|---|
| $R_n$ | 104.48 | kips | J10-3 |
| $\Phi$ | 1 | | |
| $\Phi R_n$ | 104.48 | kips | |
| | OK | | |

Web Crippling Parameters

| | | | |
|---|---|---|---|
| $R_n$ | 326.50 | | J10-4 |
| $\Phi$ | 0.75 | | |
| $\Phi R_n$ | 244.87 | | |
| | OK | | |

**Summary**


thickness (t) x width (w) flange plate
(top & bottom)
Weld
WX x XX
2.50"
WX x XX
Vu
t(angle)
L
Mu
3.50"
2.00"
2.00"
N @ 3"

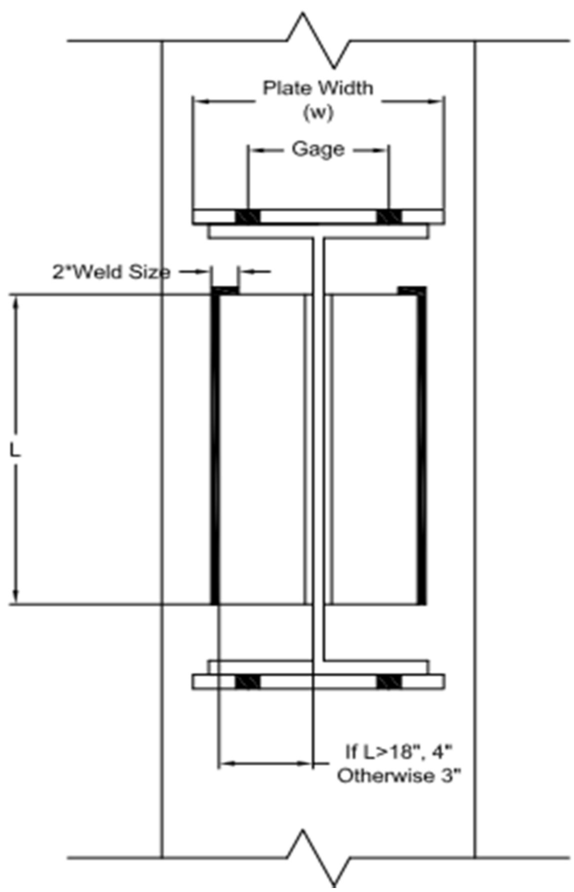


Plate Width
(w)
Gage
2*Weld Size
L
If L>18", 4"
Otherwise 3"

| | | | |
|---|---|---|---|
| Girder: | W10x39 | $M_u$ | 77.18 kip*ft |
| Column: | W10x68 | $V_u$ | 35.73 kips |

## Flange Plate:

| | | | | | |
|---|---|---|---|---|---|
| Thickness, t | 1/2 in | $F_y$ | 36 ksi | $F_u$ | 58 ksi |
| width, w | 8 in | | | | |
| # of rows | 2 | | | | |
| Hole | std | | | | |
| A325N, bolt dia. | 7/8 in | | | | |
| # bolts | 4 per flange | | | | |
| Gage | 6 in | | | | |

## Flange-Column Weld:

Electrode Strength, $F_u$ 70 ksi

Flange CJP Weld:

| | |
|---|---|
| Length | 8 in |
| Size | 1/2 in |

## Double Angle Connection:

Weld to column, bolt to girder

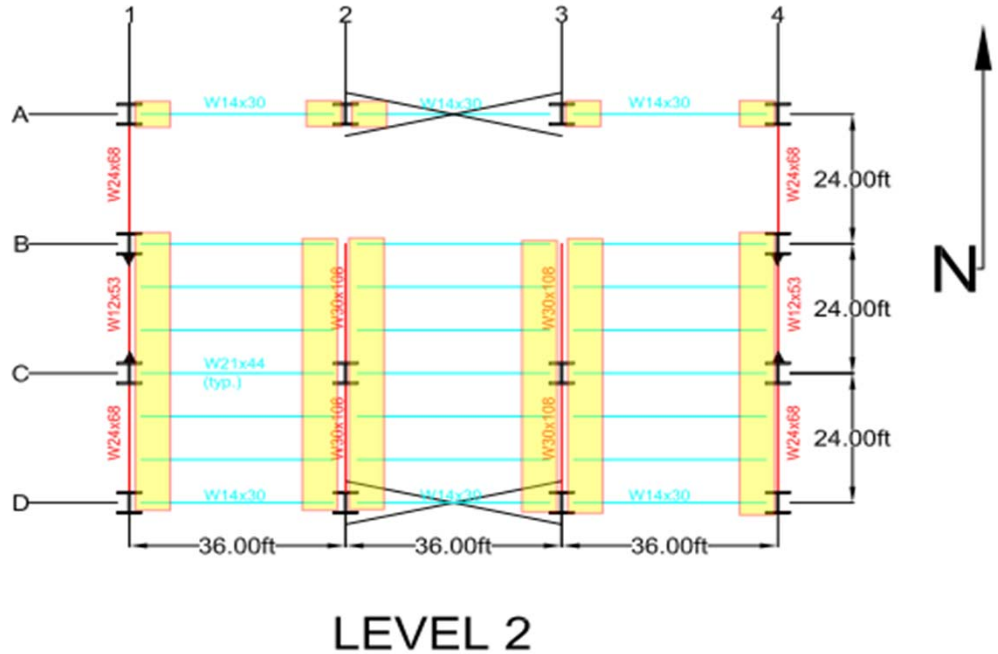
Angle properties:

| | |
|---|---|
| t | 1/4 in |
| L | 8.5 in |
| Leg Connect. to Column | 3 in |
| Leg connect. To girder | 3.5 in |
| weld size | 1/4 in |
| Hole | std |
| # bolts/row, n | 3 (Bolts spaced at 3" O.C.) |
| A325N, bolt dia. | 3/4 in |

# Simple Shear Connection Design - Level 2

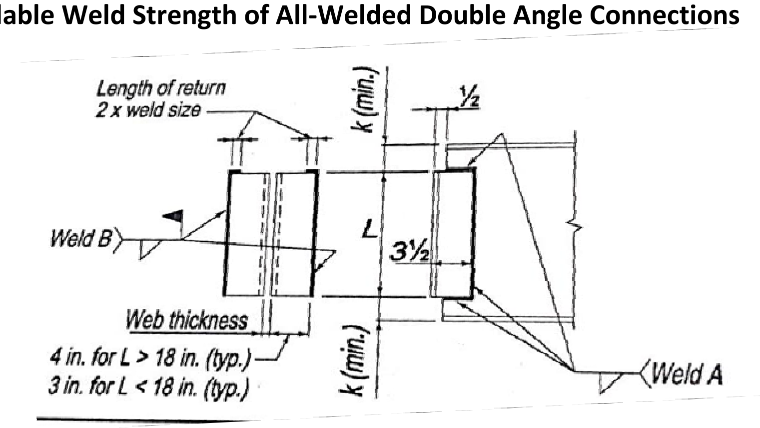
## BEAM CONNECTIONS

Connections highlighted below:




| | | | |
|---|---|---|---|
| $P_u$ | 39.11 | kips | Shear reaction |

**Use all welded double angle connections**
**Table 10-3: Available Weld Strength of All-Welded Double Angle Connections**



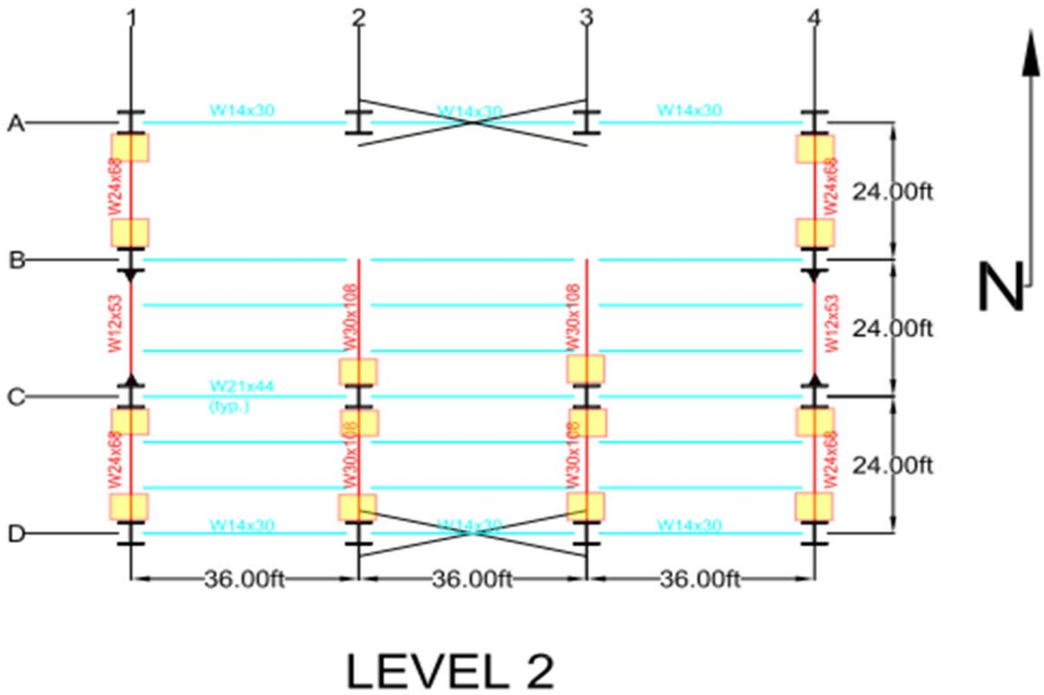
Welds with the following specifications meet requirements:

| | | | |
|---|---|---|---|
| L | 6 | in | |
| Weld B | 5/16 | in | |
| Angle Width on Supporting element | 3 | in | |
| Weld A | 3/16 | in | |
| Angle Thickness | 0.25 | in | Weld size+(1/16) |

*Use (2) 3.5x3x1/4 angles*

## GIRDER CONNECTIONS

Connections highlighted below:

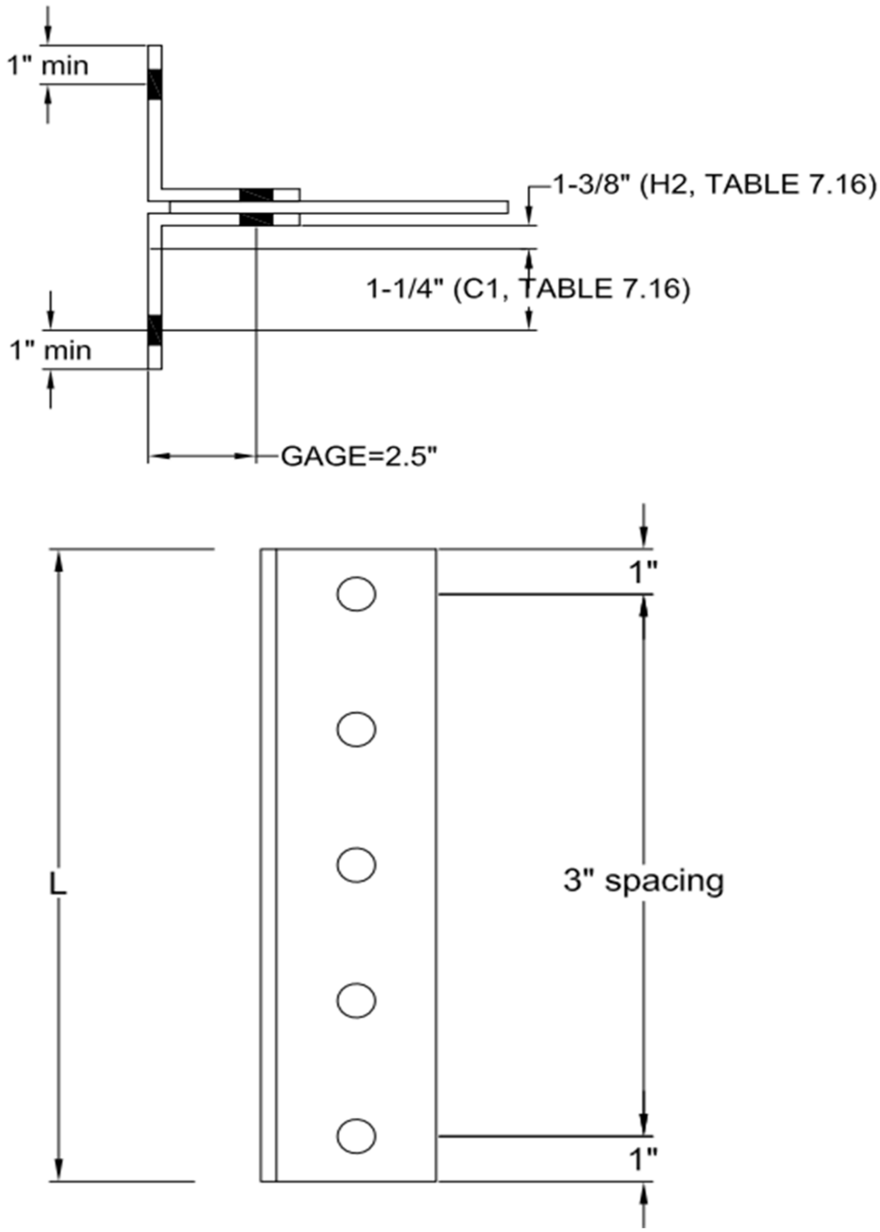



$P_u$ 156.44 kips

**Use all bolted double angle connection**
Table 10-1: All Bolted Double-Angle Connections

Following bolt specifications meet requirements:
*5 rows, 3/4" bolts*
*Group A, threads included, STD hole*
*3/8" angle thickness*

Size angle:




| | | | |
|---|---|---|---|
| Number of spaces, N | 4 | | |
| Girder $t_w$ | 0.545 | in | |
| Angle thickness, t | 3/8 | in | |
| $H_2$ | 1 3/8 | in | Table 7.16 |
| $C_1$ | 1 1/4 | in | Table 7.16 |
| Gage | 2.5 | in | Assumed |
| Min. edge distance | 1 | in | Assumed |

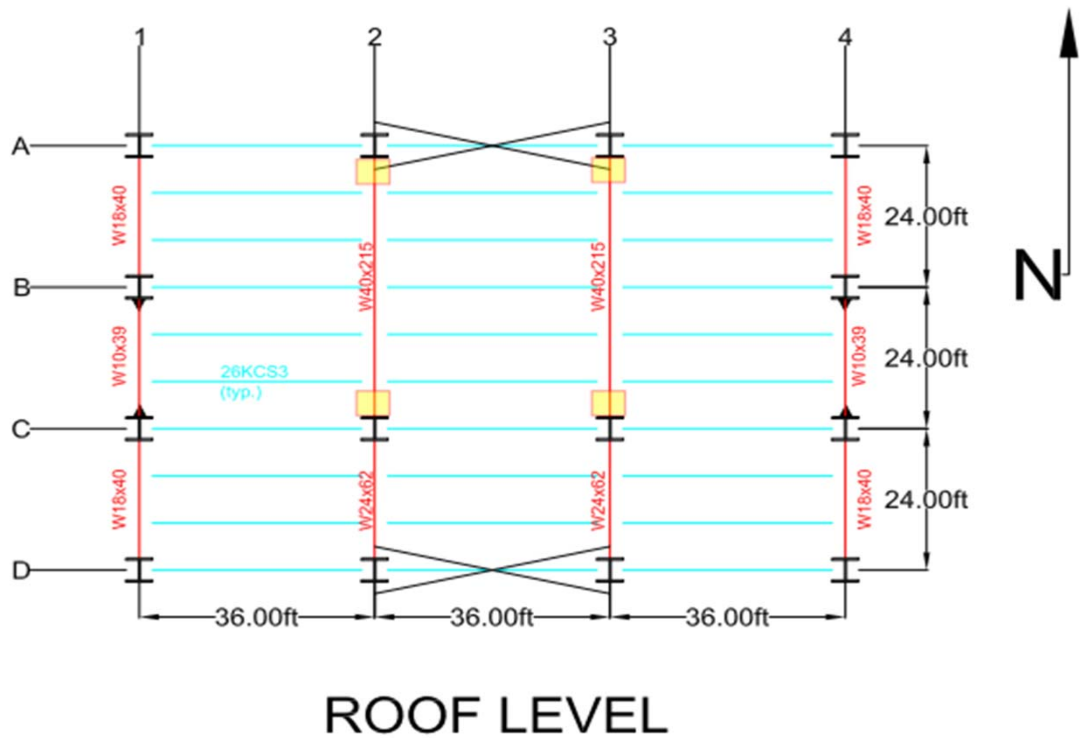
| | | | |
|---|---|---|---|
| Leg Length bolted to girder: | 3.5 | in | (Gage+1") |
| Use | 3.5 | in | |
| Leg length bolted to column: | 4.00 | in | ($t+H_2+C_1+1"$) |
| Use | 4 | in | |
| Angle Length | 14 | | |

**Use (2) 4 x 3.5 x 3/8 x 1'-2" angles**

# Simple Shear Connection Design - Roof Level

## GIRDER CONNECTIONS

Connections highlighted below:

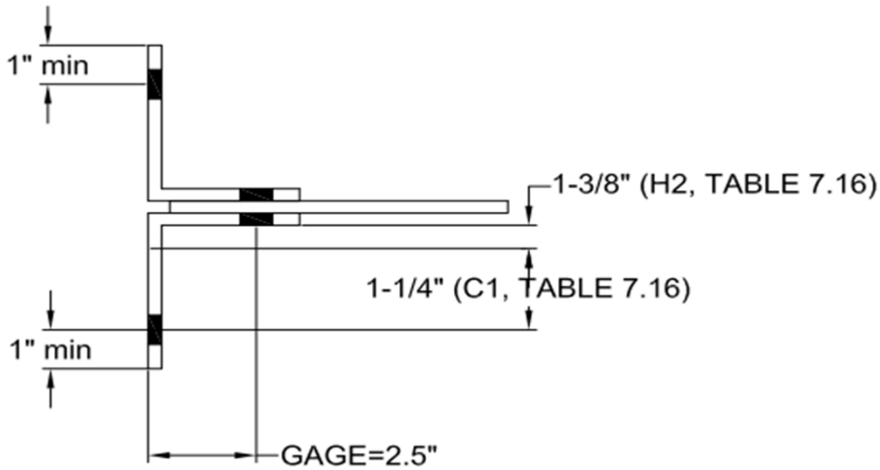


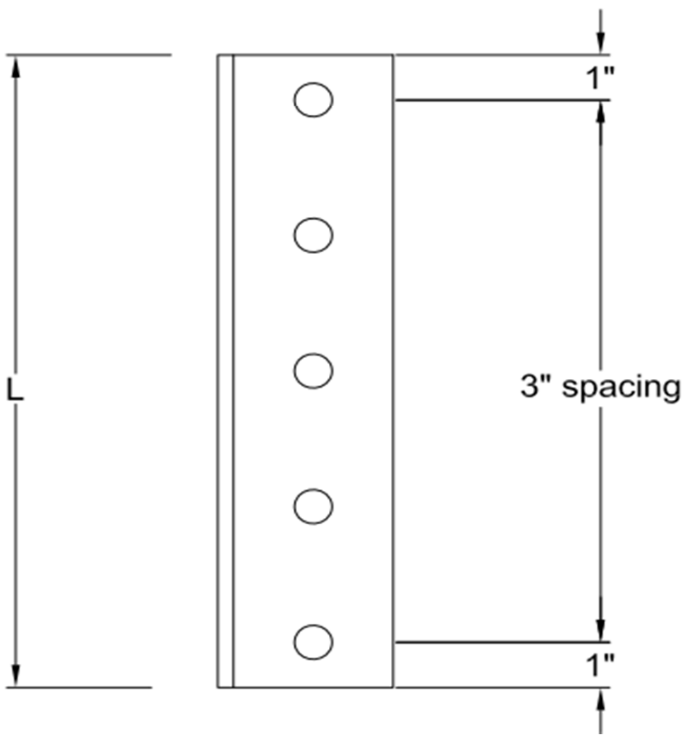

$P_u$ 295.97 kips

**Use all bolted double angle connection**
Table 10-1: All Bolted Double-Angle Connections

Following bolt specifications meet requirements:
*9 rows, 3/4" bolts*
*Group A, threads included, STD hole*
*3/8" angle thickness*

Size angle:



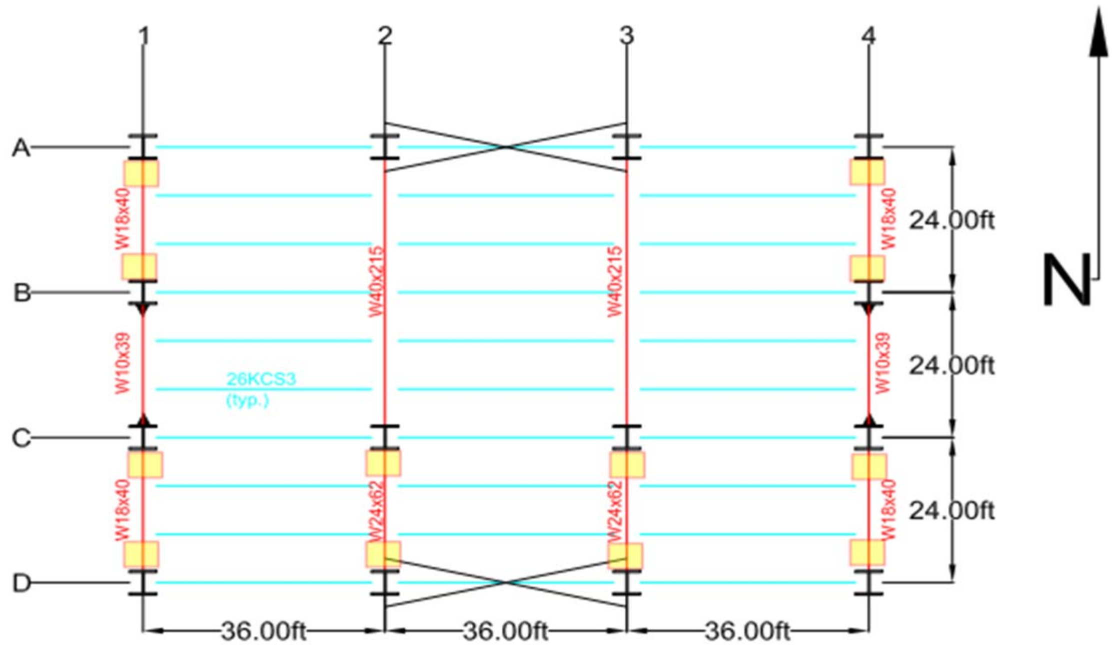




| | | | |
|---|---|---|---|
| Number of spaces, N | 8 | | |
| Girder $t_w$ | 0.65 | in | |
| Angle thickness, t | 3/8 | in | |
| $H_2$ | 1 3/8 | in | Table 7.16 |
| $C_1$ | 1 1/4 | in | Table 7.16 |
| Gage | 2.5 | in | Assumed |
| Min. edge distance | 1 | in | Assumed |

| | | | |
|---|---|---|---|
| Leg Length bolted to girder: | 3.5 | in | (Gage+1") |
| Use | 3.5 | in | |
| Leg length bolted to column: | 4.00 | in | ($t+H_2+C_1+1$") |
| Use | 4 | in | |
| Angle Length | 26 | | |

**Use (2) 4 x 3.5 x 3/8 x 2'-2" angles**

Connections highlighted below:



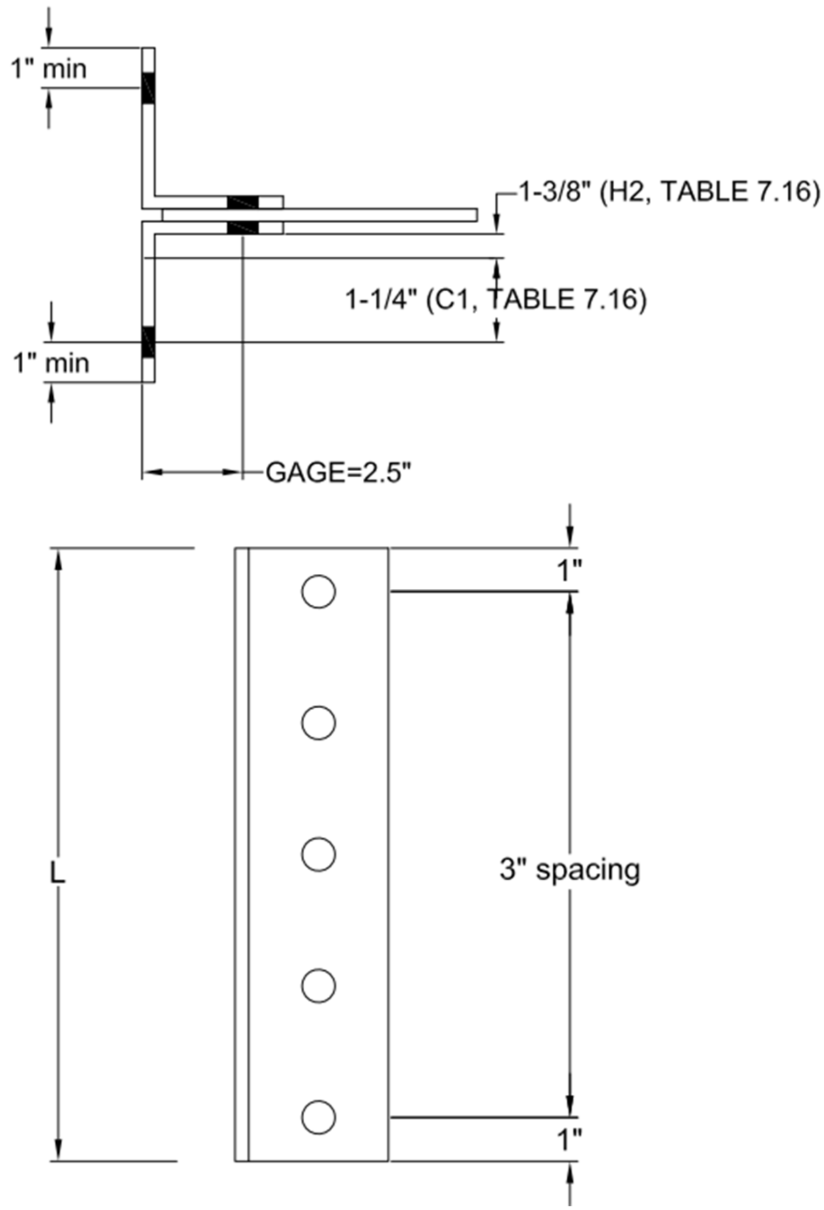

ROOF LEVEL

| | | |
|---|---|---|
| $P_u$ | 71.46 | kips |

**Use all bolted double angle connection**
Table 10-1: All Bolted Double-Angle Connections

Following bolt specifications meet requirements:
*4 rows, 3/4" bolts*
*Group A, threads included, STD hole*
*1/4" angle thickness*

Size angle:




| | | | |
|---|---|---|---|
| Number of spaces, N | 3 | | |
| Girder $t_w$ | 0.43 | in | |
| Angle thickness, t | 1/4 | in | |
| $H_2$ | 1 3/8 | in | Table 7.16 |
| $C_1$ | 1 1/4 | in | Table 7.16 |
| Gage | 2.5 | in | Assumed |
| Min. edge distance | 1 | in | Assumed |

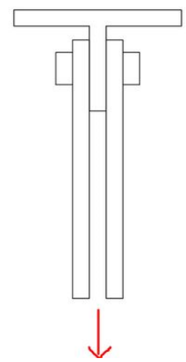
| | | | |
|---|---|---|---|
| Leg Length bolted to girder: | 3.5 | in | (Gage+1") |
| Use | 3.5 | in | |
| Leg length bolted to column: | 3.88 | in | ($t+H_2+C_1+1"$) |
| Use | 4 | in | |
| Angle Length | 11 | | |

**Use (2) 4 x 3.5 x 1/4 x 11" angles**

| | | | |
|---|---|---|---|
| Prepared by: | Matthew Laskey | Date: | 11/29/2014 |
| Checked by: | Ana Gouveia | | |

Hanger Tension MemberDesign
Loading Referenced from "Hanger Connection Design"

The tension member will consist of two narrow steel plates bolted together at the WT member. The only forces acting on this member is tension.



| | | |
|---|---|---|
| Pu= | 117.33 | k |
| Pu= | 58.665 | k/tension member |

| Steel Plates | | | WT12x96 Member | | |
|---|---|---|---|---|---|
| tpl= | 1 | in | d= | 12.7 | in |
| wpl= | 14 | in | k= | 1.96 | in |
| Fy= | 36 | ksi | d-k= | 10.74 | in |
| Fu= | 58 | ksi | | | |

**Plates**

Allowable tensile yielding (AISC 16.D.2)

| | | |
|---|---|---|
| ϕ= | 0.9 | |
| Ag= | 14 | in2 |
| Pn= | 504 | k |
| ϕPn= | 453.6 | k/member |

Allowable Tensile rupture

| | | |
|---|---|---|
| ϕ= | 0.75 | |
| An= | 11.375 | in2 |
| U= | 1 | Table D3.1 |
| Ae= | 11.375 | in2 |
| Pn= | 659.75 | k |
| ϕPn= | 494.8125 | k/member |

Block Shear of Plates

| | | |
|---|---|---|
| ϕ= | 0.75 | |
| Ubs= | 1 | Uniform tension stress |

Rn=.6FuAnv+UbsFuAnt<.6FyAgv+UbsFuAnt

| | | |
|---|---|---|
| .6FuAnv+UbsFuAnt= | 659.75 | k |
| .6FyAgv+UbsFuAnt= | 659.75 | k |

| | | |
|---|---|---|
| ϕRn= | 395.85 | k/member |

Check

| | | | | | |
|---|---|---|---|---|---|
| yield | 453.6 | k/member | > | 58.665 | OK |
| Rupture | 494.8125 | k/member | > | 58.665 | OK |
| Block | 395.85 | k/member | > | 58.665 | OK |

**Bolts**

Using A325N 3/4" diameter bolts

| | | | |
|---|---|---|---|
| n= | 3 | | |
| dia= | 0.75 | in | |
| Ab= | 0.44 | in2 | |
| Fnt= | 90 | ksi | Table J3.2 |
| Fnv= | 54 | ksi | Table J3.3 |

Tensile Strength of Bolts AISC J3.6

| | | |
|---|---|---|
| ϕ= | 0.75 | |
| Rnt= | 39.76 | k |
| ϕRnt= | 29.82 | k/bolt |

Shear Strength of Bolts (double Shear) AISC J3.6

| | | |
|---|---|---|
| ϕ= | 0.75 | |
| Rnv= | 23.86 | k |
| ϕRnv= | 17.89 | k/bolt |

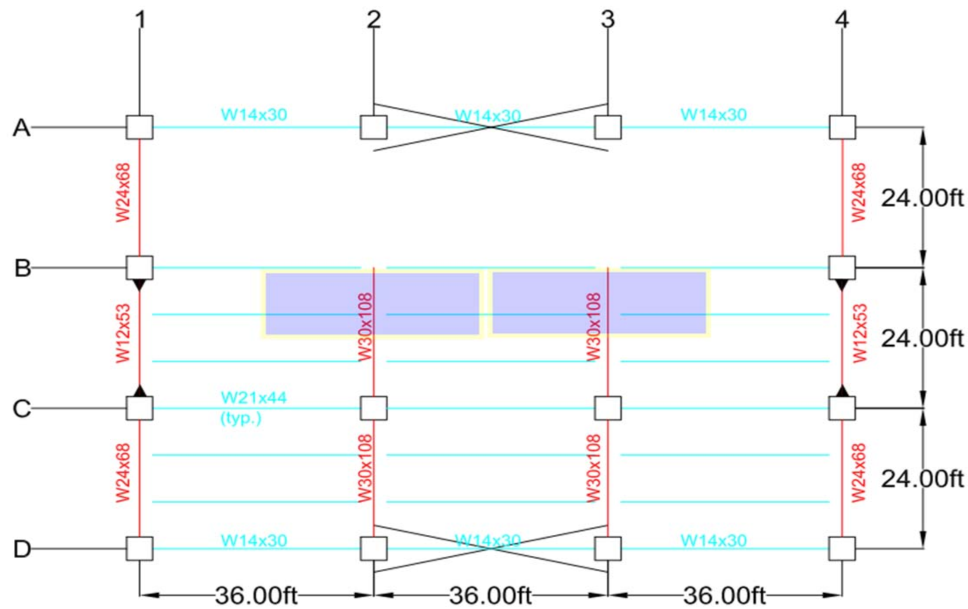
Check shear force per bolt

| | | | | |
|---|---|---|---|---|
| rnv=Pu/2n | 9.78 | < | 17.89 | OK |

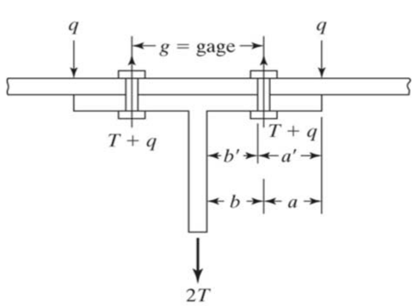
Check Tension force per bolt

| | | | | |
|---|---|---|---|---|
| rnt=Pu/n | 19.56 | < | 29.82 | OK |

# Hanger Connection Design



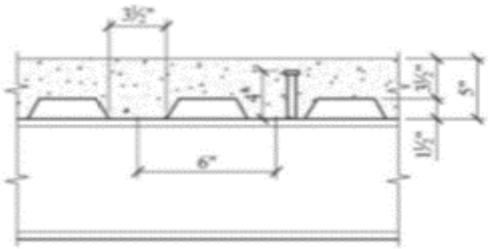

| | | |
|---|---|---|
| Dead Load, DL | 93 | psf |
| Live Load, LL | 100 | psf |
| | | |
| Tributary width | 36 | ft |
| Tributary Length | 12 | ft |
| Tributary Area | 432 | $ft^2$ |
| | | |
| $W_u$ | 271.6 | psf |
| $P_u$ | 117.33 | kips/hanger |



| | | | |
|---|---|---|---|
| Specify WT: | ***WT12x96*** | | |
| Hanger will bear on a W30 girder | | | |
| $\phi$ | 0.9 | | |
| $\Omega$ | 1.67 | | |
| $F_u$ | 58 | ksi | Table 2-4 |
| $b_f$ | 13 | in | Table 1-8 |
| $t_f$ | 1.46 | in | Table 1-8 |
| $t_w$ | 0.81 | in | Table 1-8 |
| bolt diameter, $d_b$ | 3/4 | in | |
| hole diameter, $d_h$ | 7/8 | in | +1/8 |
| Edge distance | 2 | in | |
| Gage | 8.13 | in | $b_f$-(2*Edge distance)-(2*$d_h$/2) |
| a | 2.44 | in | |
| a' | 2.00 | in | |
| b | 3.66 | in | |
| b' | 3.22 | in | |
| S | 8.13 | in | |
| P | 8.13 | in | |
| T | 58.67 | kips | |
| | | | |
| $t_{min}$ | 1.33 | in | |
| $t_f$>$t_{min}$? | YES | | |

**1. Given:**

Unshored Construction:
Steel beams must support all loads prior to .75f'c



**Floor Slab/Deck Details**

**Steel Beam Properties:**

| | | | | | | |
|---|---|---|---|---|---|---|
| Section: | W | **W21X44** | | E | 29000 | ksi |
| | d | **20.7** | in | $F_y$ | 50 | ksi |
| | $b_f$ | **6.5** | in | $r_{ts}$ | **2.81** | in |
| | $t_f$ | **0.45** | in | $h_0$ | **11.6** | in |
| | $t_w$ | **0.35** | in | $I_{xw}$ | **843** | $in^4$ |
| | A | **13** | $in^2$ | $I_{yw}$ | **20.7** | $in^4$ |
| | $S_x$ | **81.6** | $in^3$ | $J_w$ | **0.77** | $in^4$ |
| | Z | **95.4** | $in^3$ | $r_{xw}$ | **8.06** | in |
| | Zy | **10.2** | in | ryw | **1.26** | in |
| | $S_y$ | **6.37** | $in^3$ | | | |

**Beam/Slab Properties:**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Beam Yield Stress, Fy | 50 | ksi | | | | | |
| Beam Spacing | 9 | ft | | | | | |
| Beam Span Length | 36 | ft | Bay Width | 24 | ft | | |
| Deck height, hr | 1.5 | in | rib spacing | 6 | in | OK | |
| Overall Slab thickness, Ycon | 5 | in | rib width | 3.5 | in | | |
| Deck orientation | Perpendicular | | | | | | |
| Thickness of Concrete, tc | 4.25 | in | tc = Yc - hr/2 | | | | *I3.2c* |
| effective slab width | 108 | in | beff = S1/2 + S2/2 | | | | *13.1a* |

**Concrete Properties:**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Compressive Strength, f'c | 4 | ksi | | | | | |
| Unit Weight of Concrete | 145 | pcf | | | | | |
| Elastic Modulus, Ec | 3492 | ksi | | | | | *Eq. I8-1* |

**Load Transfer Device**

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Max Headed stud diameter | 3/4 | in | Stud Used | 3/4 | in | OK | *I8.2d* |
| Min Length | 3 | in | length | 4 | in | OK | |
| ASTM A108, Tensile Strength, Fu | 65.00 | ksi | Stud position | weak | | | |
| Minimum longitudinal spacing | 4.5 | in | Rp | 0.6 | | | *I8.2d* |
| Maximum longitudinal spacing | 36 | in | Rg | 1 | | | *I8.2d* |
| Minimum transverse spacing | 3 | in | clearance | 1.5 | in | | *I8.2d* |

**Loads**

| | | | | | |
|---|---|---|---|---|---|
| Superimposed Floor Live Load | 150 | psf | LRFD, $\phi$ | 0.9 | |
| Superimposed Dead Load | 171.5 | psf | | | |
| Construction Live Load (staging) | 20.00 | psf | | | |
| Metal Deck | 2 | psf | | | |
| Beam Self Wt | 44 | psf | from beam used above | | |
| Ceilling and utilities | 10 | psf | | | |
| Partitions | 20 | psf | | | 8100 |

**Method/Solution**

**1. Design Loads on Beam** — **Service Loads (plf)**

| **1,1 Construction Phase** | D | L | TOTAL | *Assumptions:* |
|---|---|---|---|---|
| Live Load | | 180 | | Value given * beam spacing (twidth) |
| Dead Loads: | | | | |
| Metal Deck | 18 | | | |
| Beam Self Weight | 44 | | | |
| Wet Concrete | 462 | | | Calculated value based on beam spacing |
| Total with concrete | 524 | 180 | 704 | Concrete Thickness |
| Total without concrete | 62 | 180 | 242 | |

| Total Uniform Construction Design Load, wc | **Factored Loads (plf)** | **TOTAL** |
|---|---|---|

| | | | |
|---|---|---|---|
| LRFD, with concrete, wc-c | 629 | 288 | 917 |
| LRFD, without concrete, wc-wc | 74 | 288 | 362 |

| **1,2 Superimpose Loads (post Construction)** | D | L | TOTAL |
|---|---|---|---|
| Superimpose Floor Live Load | | 1543.5 | |
| Superimpose Dead Loads: | 288 | | |
| Total with concrete | 288 | 1543.5 | 1832 |

| Total Uniform Construction Design Load, wc | **Factored Loads (plf)** | | |
|---|---|---|---|
| LRFD, with concrete, wc-c | 346 | 2470 | 2815 |

**2, Determine Required Beam Strength**

**2.1. Construction Phase**

| | | | |
|---|---|---|---|
| Live Moment, Mrcll | 46.656 | k-ft | total live load |
| Total Moment, Mrc | 148.6 | k-ft | with concrete |
| Total Shear, Vrc | 16.5 | kips | with concrete |
| Total Moment, Mr+rc | 58.7 | k-ft | Prior to fastening deck (no conc) |

**2,2 Superimposed (post-construction)**

| | | |
|---|---|---|
| Moment, Mrs | 456.06 | k-ft |
| Shear, Vrs | 50.67 | kips |

**3. Select Feasible shapes assuming 100% composite action and superimposed loads**

| | | |
|---|---|---|
| 3,1 PNA is located in slab, estimate a= ,4 x tc | 1.7 | in |
| 3,2 Calculate Y2 = Ycon - a/2 | 4.15 | in |
| 3,3 Select Feasible shapes | | |

| | | |
|---|---|---|
| **Required Superimposed Moment** | 456.06 | k-ft |

**4,0, Check Construction Phase Required Strength/Capacity** **Cb** 1

**4.1 Construction w/ DL + LL (before bracing top flange with unfastened deck)**

*Check Elastic Lateral Torsional Capacity and temporary bracing requirements*

Required Moment

| Mr-nc | 58.7 | k-ft |
|---|---|---|

| Feasible Shapes | Lb | Sx | rts (in) | ho | J | Lb/rts | Eq. F2-4 Fcr (ksi) | Mn (k-ft) | $\Phi_b M_n$ (k-ft) | Check Mr-nc/ΦbMn | Is Temp. Bracing Req'd? |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **W18X35** | 36 | 57.6 | 1.42 | 15.5 | 0.506 | 304.2 | 6.98 | 33.5 | 30.1 | 1.95 | Yes |
| **W18X35** | 30 | 57.6 | 1.51 | 17.3 | 0.506 | 238.4 | 9.08 | 43.6 | 39.2 | 1.50 | No |

**4.2 Construction w/ DL + LL (with fastened deck) + Assume Fully Braced Top Flange**

Required Moment

| Mrc | 148.6 | k-ft |
|---|---|---|

Construction Dead Load Deflection

| | | |
|---|---|---|
| Superimposed Load, wu | 0.704 | kips |
| Md= | 114.08 | k-ft |

| Feasible Shapes | $\Phi_b M_{px}$ (k-ft) | Ix | $\Phi_b M_n$ (k-in) | Mrc/ΦbMn | $\Phi_b V_n$ (k) | Vrc/ΦbVn | Check Δcdl (in) | Camber? (D>,75 in) | |
|---|---|---|---|---|---|---|---|---|---|
| W18X35 | 125 | 510 | 361.7 | 0.41 | 94.5 | 0.17 | 1.80 | YES | Table 3-2 |
| W18X35 | 166 | 510 | 470.7 | 0.32 | 106 | 0.16 | 1.80 | YES | Table 3-2 |

**5. Check Adequacy of Composite Sections**

**5.1 Flexure & Shear**

| Feasible Shapes | As | d | a | Y2 | Mn (k-ft) | $\Phi_b M_n$ (k-in) | Mrs/ΦbMn | < 1,0 ? | $\Phi_b V_n$ (k) | Vrs/ΦbVn | < 1,0 ? |
|---|---|---|---|---|---|---|---|---|---|---|---|
| **W18X35** | 10.3 | 17.7 | 1.40 | 4.30 | 515 | 463.5 | 0.98 | OK | 94.5 | 0.54 | OK |
| **W18X35** | 10.3 | 17.7 | 1.40 | 4.30 | 515 | 463.5 | 0.98 | OK | 106 | 0.48 | OK |

**5.2 Deflection/Serviceability** C1= 161 [Fig. 3-2 AISC] (l/360) = 1.2 in (l/240) = 1.8 in

| Feasible Shapes | Y2 | Y1 | Table 3-20 $I_{LB}$ | Camber (in) | D (l/360) = Camber - Δcdl (in) | Δsdl (in) | Δsll (in) | Check Δsll (in) < (l/360) | Δtl (in) | Check Δtl (in) < (l/240) |
|---|---|---|---|---|---|---|---|---|---|---|
| W18X35 | 4.30 | 0 | 1055 | 1 | -0.20 | 0.36 | 1.20 | OK | 1.500 | OK |
| W18X35 | 4.30 | 0 | 875 | 0 | 0.00 | 0.43 | 2.30 | NG | 2.730 | NG |

**6.0 Design Steel Anchor Studs** Area of Steel Anchor, Asa **Asa =** 0.44 $in^2$ ea.

**6.1 Strength of Steel Headed Anchors**

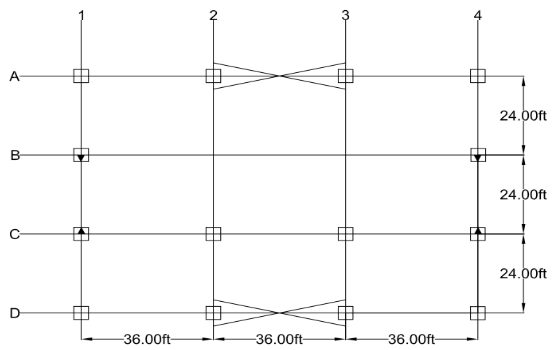
| | | | | |
|---|---|---|---|---|
| **Qn1** | 26.1 | | *Eq. I8-1* |
| **Qn2** | 17.2 | | *Eq. I8-1* |
| **Qn** | 17.2 | Control | |

| | | **W18X35** | **W18X35** |
|---|---|---|---|
| Total Required Shear Transfer, SQn= Fy.As | | 515 | 515 |
| Number Steel Headed Anchors Required, Ns | | 29.9 | 29.9 |
| | USE: | 30 | 30 |
| Distance between max and zero moment = L/2 | | 216 | 216 |
| Required Stud Spacing | | 7.2 | 7.2 |
| | CHECK: | OK | **OK** |
| | TOTAL ANCHORS REQUIRED: | 60 | **60** |

| **RESULTS** | **W18X35** | |
|---|---|---|
| **CAMBER** | 1 | in |
| **# STUDS** | 60 | studs |
| **STUD Φ** | 0.75 | in |
| **STUD LENGTH** | 4 | in |
| **SPACING** | EQUAL | |
| **TEMPORARY BRACING** | | Yes |
| **Δsll (in)** | 1.20 | in |

## Sizing-Roof Level

**Given:**
Roof Level Plan below

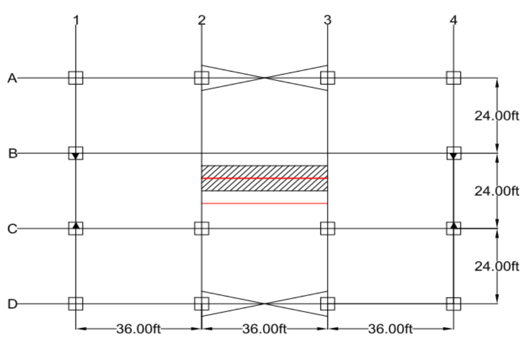


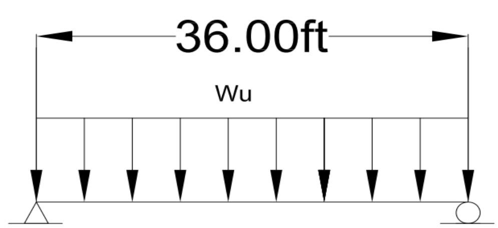

*Use open web steel joists w/ steel girders
*Assume four beams per bay spaced @ 8' O.C.
*Beams run parallel to braced frame

**Determine:**
Preliminary sizes for roof level members

## BEAMS

Typical beam highlighted in RED below:




| | | |
|---|---|---|
| Beam Length | 36 | ft |
| Beam Spacing | 8 | ft |
| Beam Tributary Area | 288 | $ft^2$ |

Roof Dead Load

| | | |
|---|---|---|
| Roof Dead Load | 46.50 | psf |
| **$W_{DLroof}$** | **372** | **plf** |

Roof Live Load

| | | |
|---|---|---|
| Roof Live Load | 50.00 | psf |
| **$W_{LLroof}$** | **400** | **plf** |

Snow Load

| | | |
|---|---|---|
| Snow Load | 31.50 | psf |
| **$W_{snow}$** | **252** | **plf** |

Wind Load

| | | |
|---|---|---|
| Wind Uplift Pressure | -23.49 | psf |
| **$W_{wind}$** | **-187.94** | **plf** |

LRFD Load Combinations

| | | |
|---|---|---|
| Load Combo 2 | 646.4 | plf |

| | | |
|---|---|---|
| Load Combo 3 | 992.432 | plf |
| Load Combo 4 | 458.464 | plf |
| Load Combo 5 | 496.8 | plf |
| Load Combo 6 | 146.864 | plf |
| **$W_u$** | **992.432** | **plf** |
| **$W_u$** | **0.99** | **klf** |

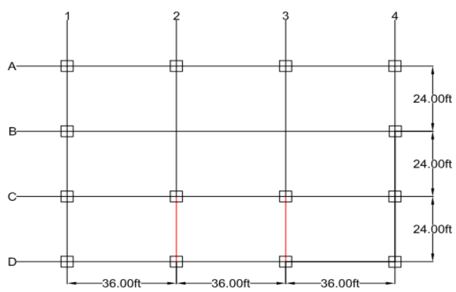
| | | |
|---|---|---|
| $M_u$ | 160.77 | kip*ft |
| $V_u$ | 17.86 | kips |

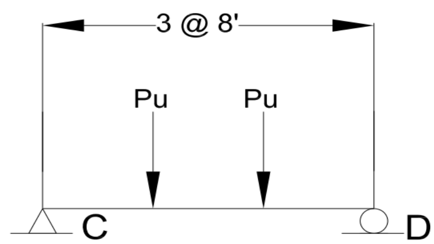
Select Beam

| | | | |
|---|---|---|---|
| Try | **W16x26** | | |
| $\phi M_n$ | 166 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |
| $\phi V_n$ | 106 | kips | Table 3-2 |
| $\phi V_n > V_u$? | YES | | |

## GIRDERS

Design Girders Highlighted in RED below:




Girder "CD" along Column line 2
Girder "CD" along Column line 3

| | | |
|---|---|---|
| Girder Length | 24 | ft |
| Beam Tributary Area | 288 | $ft^2$ |

Loading from beams:

Roof Dead Load

| | | |
|---|---|---|
| Roof Dead Load | 46.50 | psf |
| **$P_{DLroof}$** | **26.784** | **kips** |

Roof Live Load

| | | |
|---|---|---|
| Roof Live Load | 50.00 | psf |
| **$P_{LLroof}$** | **28.8** | **kips** |

Snow Load

| | | |
|---|---|---|
| Snow Load | 31.50 | psf |
| **$P_{SNOW}$** | **18.144** | **kips** |

Wind

| | | |
|---|---|---|
| Wind Uplift Pressure | -23.49 | psf |
| **$W_{wind}$** | **-13.53** | **kips** |

LRFD Load Combinations

| | | |
|---|---|---|
| Load Combo 2 | 46.54 | kips |
| Load Combo 3 | 71.46 | kips |
| Load Combo 4 | 33.01 | kips |
| Load Combo 5 | 35.77 | kips |
| Load Combo 6 | 10.57 | kips |
| **$P_u$** | **71.46** | kips |

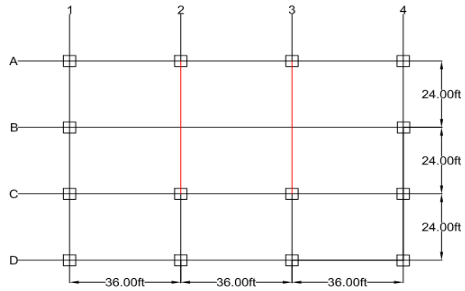
| | | |
|---|---|---|
| $V_u$ | 71.46 | kips |
| $M_u$ | 571.64 | kip*ft |

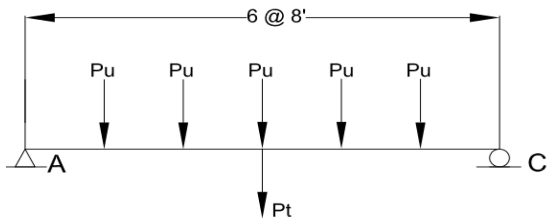
Select Girder

| | | | |
|---|---|---|---|
| Try | **W24x62** | | |
| $\phi M_n$ | 574 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |

| | | | |
|---|---|---|---|
| $\phi V_n$ | 306 | kips | Table 3-2 |
| $\phi V_n > V_u$? | YES | | |

Design Girders Highlighted in RED below :




Girder "AC" along Column line 2
Girder "AC" along Column line 3

| | | |
|---|---|---|
| Girder Length | 48 | ft |
| Beam Spacing | 8 | ft |
| Beam Tributary Area | 288 | $ft^2$ |

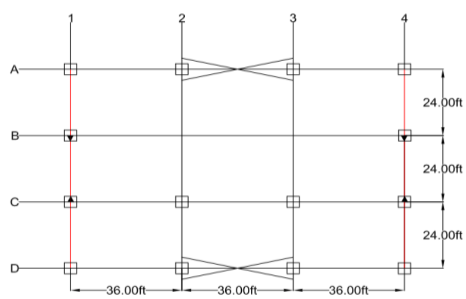
Loading from beams:

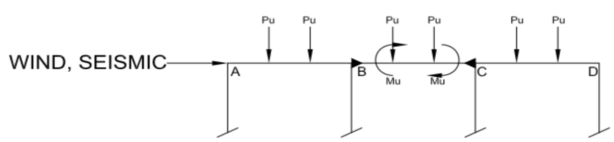
| | | | |
|---|---|---|---|
| $\mathbf{P_u}$ | **71.46** | kips | |
| $\mathbf{P_t}$ | **234.66** | kips | Determined from "Sizing-Level 1" |
| $V_u$ | 295.97 | kips | |
| $M_u$ | 3551.63 | kip*ft | |

Select Girder

| | | | |
|---|---|---|---|
| Try | **W40x215** | | |
| $\phi M_n$ | 3620 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |
| $\phi V_n$ | 761 | kips | Table 3-2 |
| $\phi V_n > V_u$? | YES | | |

Design roof level members of moment frame highlighted in RED below:




| | | |
|---|---|---|
| Member Length | 24 | ft |

**Members "AB" & "CD"**

Flexure:

| | | | |
|---|---|---|---|
| $P_u$ | 35.73 | kips | (1/2*Pu from Girder "CD" along CL 2 & 3) |
| $V_u$ | 35.73 | kips | |
| $M_u$ | 285.82 | kip*ft | |
| Try | **W18x40** | | |
| $\phi M_n$ | 294 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |
| $\phi V_n$ | 146 | kips | Table 3-2 |
| $\phi V_n > V_u$? | YES | | |

Axial:

| | | |
|---|---|---|
| $P_{seismic}$ | 3.23 | kips |
| $P_{wind}$ | 9.61 | kips |

LRFD Load Combinations

| | | |
|---|---|---|
| Load Combination 4 | 9.61 | kips |
| Load Combination 5 | 3.23 | kips |

| | | | |
|---|---|---|---|
| Load Combination 6 | 9.61 | kips | |
| Load Combination 7 | 3.23 | kips | |
| $P_u$ | 9.61 | kips | |
| | | | |
| KL | 24 | | |
| $r_y$ | 1.27 | $in^3$ | Table 1-1 |
| KL/r | 18.90 | | |
| A | 11.8 | $in^2$ | Table 1-1 |
| $\phi F_{cr}$ | 43.8 | ksi | Table 4-22 |
| $\phi P_n$ | 516.84 | kips | |
| $\phi P_n > P_{u?}$ | YES | | |

**Member "BC"**

Flexure:

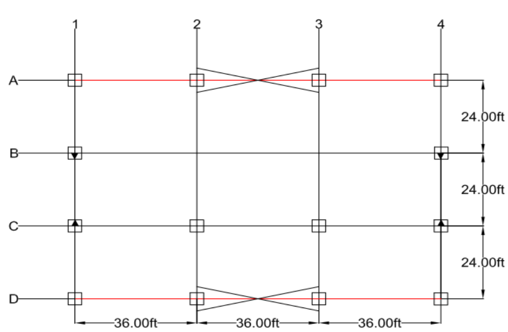
| | | | |
|---|---|---|---|
| Roof Dead Load | 46.50 | psf | |
| $P_{DLroof}$ | 6.70 | kips | Loading from beam |
| $W_{DLroof}$ | 0.56 | klf | |
| **$M_{DLroof}$** | **26.78** | **kip*ft** | Negative moment at ends |
| | | | |
| Roof Live Load | 50.00 | psf | |
| $P_{LLroof}$ | 7.20 | kips | Loading from beam |
| $W_{LLroof}$ | 0.60 | klf | |
| **$M_{LLroof}$** | **28.80** | **kip*ft** | Negative moment at ends |
| | | | |
| Snow Load | 31.50 | psf | |
| $P_{SNOW}$ | 4.54 | kips | Loading from beam |
| $W_{SNOW}$ | 0.38 | klf | |
| **$M_{SNOW}$** | **18.14** | **kip*ft** | Negative moment at ends |
| | | | |
| Wind Uplift Pressure | -23.49 | psf | |
| $P_{WINDU}$ | -3.38 | kips | Loading from beam |
| $W_{WINDU}$ | -0.28 | klf | |
| Vertical $M_{WINDU}$ | -13.53 | kip*ft | Positive moment at ends |
| Lateral $M_{WINDL}$ | 36.04 | kip*ft | Determined in frame analysis |
| **$M_{WIND}$** | **22.50** | **kip*ft** | Resulting Wind Moment |
| | | | |
| **$M_{seismic}$** | **12.13** | **kip*ft** | Determined in frame analysis |

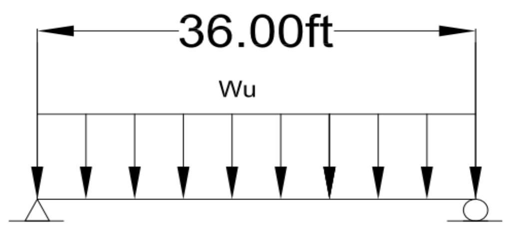
| LRFD Load Combinations | | |
|---|---|---|
| Load Combo 2 | 46.54 | kip*ft |
| Load Combo 3 | 89.47 | kip*ft |
| Load Combo 4 | 69.05 | kip*ft |
| Load Combo 5 | 47.90 | kip*ft |
| Load Combo 6 | 46.61 | kip*ft |
| Load Combo 7 | 36.23 | kip*ft |
| **$M_u$** | **89.47** | **kips** |

| | | | |
|---|---|---|---|
| Try | **W12x19** | | |
| $\phi M_n$ | 92.6 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |

By observation, Member BC is sufficient for axial loads

Design roof level members of braced frame highlighted below

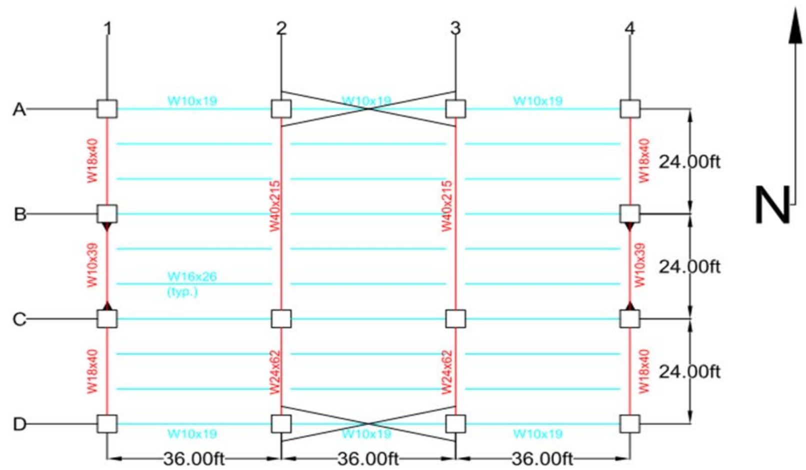



| | | | |
|---|---|---|---|
| $W_u$ | 0.50 | klf | (1/2*Wu-beam) |

| | | | |
|---|---|---|---|
| Member Length | 36 | ft | |
| | | | |
| $M_u$ | 80.39 | kip*ft | |
| $V_u$ | 8.93 | kips | |
| | | | |
| Try | **W10x19** | | |
| $\phi M_n$ | 81 | kip*ft | Table 3-2 |
| $\phi M_n > M_u$? | YES | | |
| $\phi V_n$ | 76.5 | kips | Table 3-2 |
| $\phi V_n > V_u$? | YES | | |

## Roof Deck Specification  (Vulcraft)




| | | |
|---|---|---|
| Beam Spacing, S | 8 | ft |
| Beam Length, L | 36 | ft |

*1) Determine suitable metal roof deck to accommodate construction and permenant loads*

| Construction Loads | | | |
|---|---|---|---|
| Construction Live Load | 20 | psf | (Assumed) |
| Metal Deck Self Weight | 1.6 | psf | (Assumed) |
| $W_a$ | 16.6 | psf | ASD LC3 |

| Superimposed Loads | | | |
|---|---|---|---|
| Roof Live Load | 30 | psf | |
| Dead Roof | 20 | psf | |
| Snow | 31.5 | psf | |
| $W_a$ | 43.625 | psf | ASD LC3 |

Vulcraft Steel Roof and Floor Deck:

**3 Span, B18 meets capacity (83/71 psf)**

*2) Determine fastening details for roof diaphragm to accommodate the design lateral loads in each direction*

By inspection, wind force control over seismic

| | | | |
|---|---|---|---|
| Wind Force, $F_{roof}$ | 6.27 | kips | (per frame) |
| Linear Load, $W_{roof}$ | 87.08 | plf | |
| Linear Load (Allowable) | 52.25 | plf | $0.6*W_{roof}$ |

Vulcraft B18 table:

| | |
|---|---|
| **36/3 fastener layout** | |
| **Support Fasteners:** | **5/8" puddle welds** |
| **Side Lap Fasteners** | **(2) #10 tek screws** |
| Shear Strength | 278 plf |

Shear Strength>Linear Load YES

3) Determine maximum diaphragm deflection in each orthogonal direction

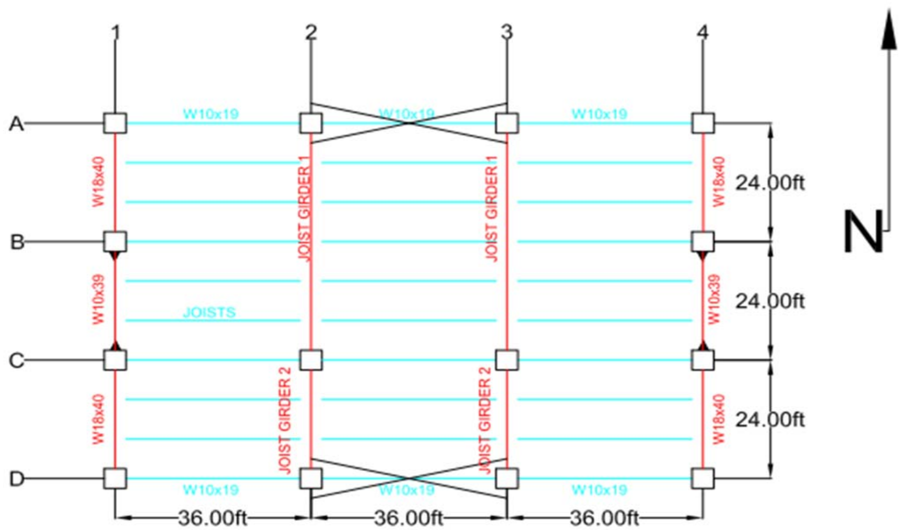
Vertical Diaphragm Deflection

| | | |
|---|---|---|
| w | 43.625 | psf |
| L | 8 | ft |
| E | 2950000 | psi |
| I | 0.289 | $in^4/ft$ |
| $\Delta$ | 0.001446 | in |

Lateral Diaphragm Deflection

| | | |
|---|---|---|
| w | 0.09 | klf |
| $D_B$ | 1084 | |
| $K_1$ | 0.573 | |
| $K_2$ | 1398 | |
| Span | 8 | ft |
| G' | 24.03 | k/in |
| L | 108 | ft |
| B | 72 | ft |
| $\Delta_{CL}$ | 0.073 | in |

## Roof Joist Specification (Vulcraft)




| Superimposed Loads | | |
|---|---|---|
| Roof Live Load | 30 | psf |
| Dead Roof | 20 | psf |
| Snow | 31.5 | psf |

*1) Design open web joist and girder system for the roof*

| Joists | | | |
|---|---|---|---|
| Joist Spacing, S | 8 | ft | |
| Joist Length, L | 36 | ft | |
| $P_U$ | 74.4 | psf | (LRFD LC 3) |
| $W_U$ | 595.20 | plf | |
| $W_U$ | 0.0041 | pli | |
| Moment, $M_u$ | 96.42 | kip*in | |
| Shear, $V_u$ | 10,713.60 | lbs | |

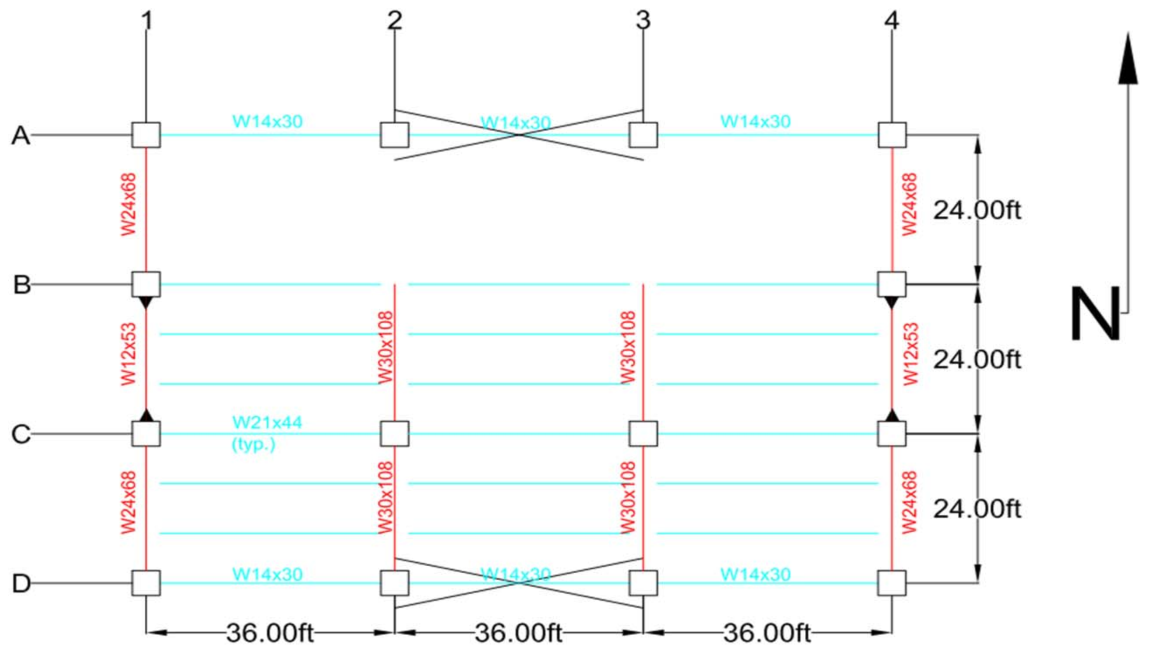
Use KCS joist:

**26KCS3 meets capacity**

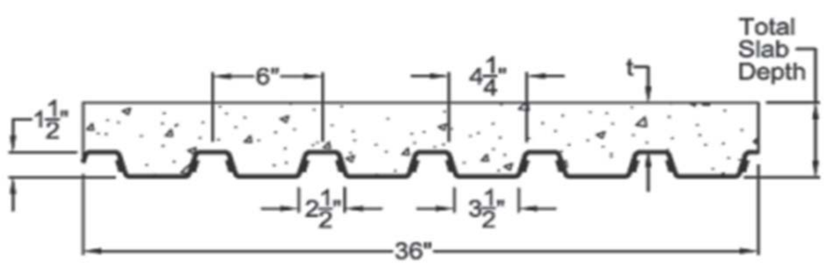
| | | |
|---|---|---|
| $M_c$ | 1662 | kip*in |
| $>M_u$? | YES | |
| $V_c$ | 12600 | lbs |
| $>V_u$? | YES | |

# Floor System Vibration Calculation

Interior Beam/Slab System:




1.5 VLR




Find:

1) Fundamental natural frequency of the interior beam, girder, and combined panels modes of the second floor.

2) Adequacy of the floor system for vibration due to walking excitation.

**Beam/Slab Properties**

| | | | |
|---|---|---|---|
| Beam Yield Stress, $F_y$ | 50 | ksi | |
| Elastic Modulus, $E_s$ | 29000 | ksi | |
| Beam Spacing | 8 | ft | |
| Beam Span Length | 36 | ft | |
| Deck Height, $h_r$ | 1.5 | in | |
| Overall Slab Thickness, $Y_{con}$ | 5.5 | in | |
| Deck Orientation on Beam | Perp. | | |
| Concrete Thickness, $t_c$ | 4 | in | |
| Rib Spacing | 6 | in | |
| Rib Width | 3.5 | in | |

**Concrete Properties**

| | | | |
|---|---|---|---|
| Conc. Comp. Strength, $f'_c$ | 4 | ksi | |
| Unit weight of conc., $w_c$ | 145 | pcf | |
| Elastic Modulus, $E_c$ | 3492 | ksi | (wc^1.5*sqrt(f'c)) |
| Dynamic Elastic Modulus | 4714 | ksi | (1.35*Ec) |
| Modular ratio, n | 6.15 | | (Es/1.35Ec) |

**Loads**

| | | | |
|---|---|---|---|
| Floor Live Load | 100 | psf | |
| Dead Load (No Conc.) | 18 | psf | |
| Concrete Dead Load | 33.23 | psf | $(Y_{con}/2)/12*w_c$ |
| Deck Dead Load | 2.5 | psf | Vulcraft Manual |
| $W_{total}$ | 153.73 | psf | Total uniform load |

**Dynamic Parameters**

| | | | |
|---|---|---|---|
| Excitation force, $P_o$ | 65 | lbs | Table 4.1 |
| Damping Ratio, $\beta$ | 0.05 | lbs | Table 4.1 |

Solution:

*Determine $I_{transformed}$ of Beams and Girders*

| | Girder | | Beam | | | |
|---|---|---|---|---|---|---|
| | W30x108 | | W21x44 | | | |
| Span, L | 24 | ft | 36 | ft | | |
| Tributary Width | 36 | ft | 8 | ft | | |
| Self Weight | 108 | plf | 44 | plf | | |
| $I_x$ | 4470 | $in^4$ | 843 | $in^4$ | | |
| A | 31.7 | $in^2$ | 13 | $in^2$ | | |
| d | 29.8 | in | 20.7 | in | | |
| effective slab depth, $d_e$ | 4.75 | in | 4 | in | | |
| Effective w of conc., $b_e$ | 115.2 | in | 96 | in | | |
| Transformed Conc., $A_{TC}$ | 88.95 | $in^2$ | 62.42 | $in^2$ | $b_e*d_e/n$ | |
| dist. To centroid, y' | 2.38 | in | 2 | in | $d_e/2$ | (from top of slab) |
| Area of Steel, $A_s$ | 31.7 | $in^2$ | 13 | $in^2$ | | |
| dist to centroid, y' | 20.4 | in | 15.85 | in | $Y_{con}+d/2$ | (from top of slab) |
| Dist to Trans, N.A., y'' | 7.11 | in | 4.39 | in | | (from top of slab) |
| $I_x$ of transformed conc., $I_{TC}$ | 1028.85 | $in^4$ | 512 | $in^4$ | | |
| $A_{TC}(y'-y'')^2$ | 1995.04 | $in^4$ | 355.73 | $in^4$ | | |
| $A_S(y'-y'')^2$ | 5598.30 | $in^4$ | 1708.15 | $in^4$ | | |
| $I_x$ | 4470 | $in^4$ | 843 | $in^4$ | | |
| **$I_x$ (transformed)** | 13092.20 | $in^4$ | 3418.88 | $in^4$ | | |

*Determine effective panel width*

| | Girder | | Beam | | |
|---|---|---|---|---|---|
| Coefficient, C | 1.8 | | 2 | | |
| $D_s$ | 17.42 | $in^4/ft$ | 10.40 | $in^4/ft$ | 16 |
| $D_{j/g}$ | 363.67 | $in^4/ft$ | 427.36 | $in^4/ft$ | |
| Effective Panel width, B | **44.98** | **ft** | **28.44** | **ft** | 4.3a or b |

*Determine max deflection, Δ*

| | **Girder** | | **Beam** | | **Combined** | |
|---|---|---|---|---|---|---|
| Uniform load, w | 5642.25 | plf | 1273.83 | plf | | |
| $L_g/B_j$ | 0.84 | <1? | YES | | | |
| max deflection, Δ (ft) | 0.094 | ft | 0.410 | ft | 0.50 | ft |
| max deflection, Δ (in) | 1.12 | in | 4.92 | in | 6.04 | in |

*Determine Fundamental Frequency of each panel, $f_n$*

| | **Girder** | | **Beam** | | **Combined** | |
|---|---|---|---|---|---|---|
| $f_n$ | 3.34 | Hz | 1.59 | Hz | 4.93 | Hz |

*Determine peak acceleration ratio, a/g*

| | **Girder** | | **Beam** | | **Combined** | |
|---|---|---|---|---|---|---|
| Total uniform floor load | 153.73 | psf | 153.73 | psf | | |
| Girder/beam self weight | 108 | plf | 44 | plf | | |
| Self Weight uniform load | 0.125 | psf | 0.153 | psf | | |
| w | 153.85 | psf | 153.88 | psf | | |
| $W_j$ & $W_g$ | 166083.02 | lbs | 157551.89 | lbs | | |
| Comb. Equiv panel weight | 30888.9841 | lbs | 128249.5719 | lbs | 159138.6 | lbs |
| peak acceleration ratio, a/g | 0.24% | | 0.47% | | 0.15% | |
| | | | | | | |
| Peak accel<accel limit | 0.50% | OK | 0.50% | OK | 0.50% | OK |

# Appendix A: Calculations

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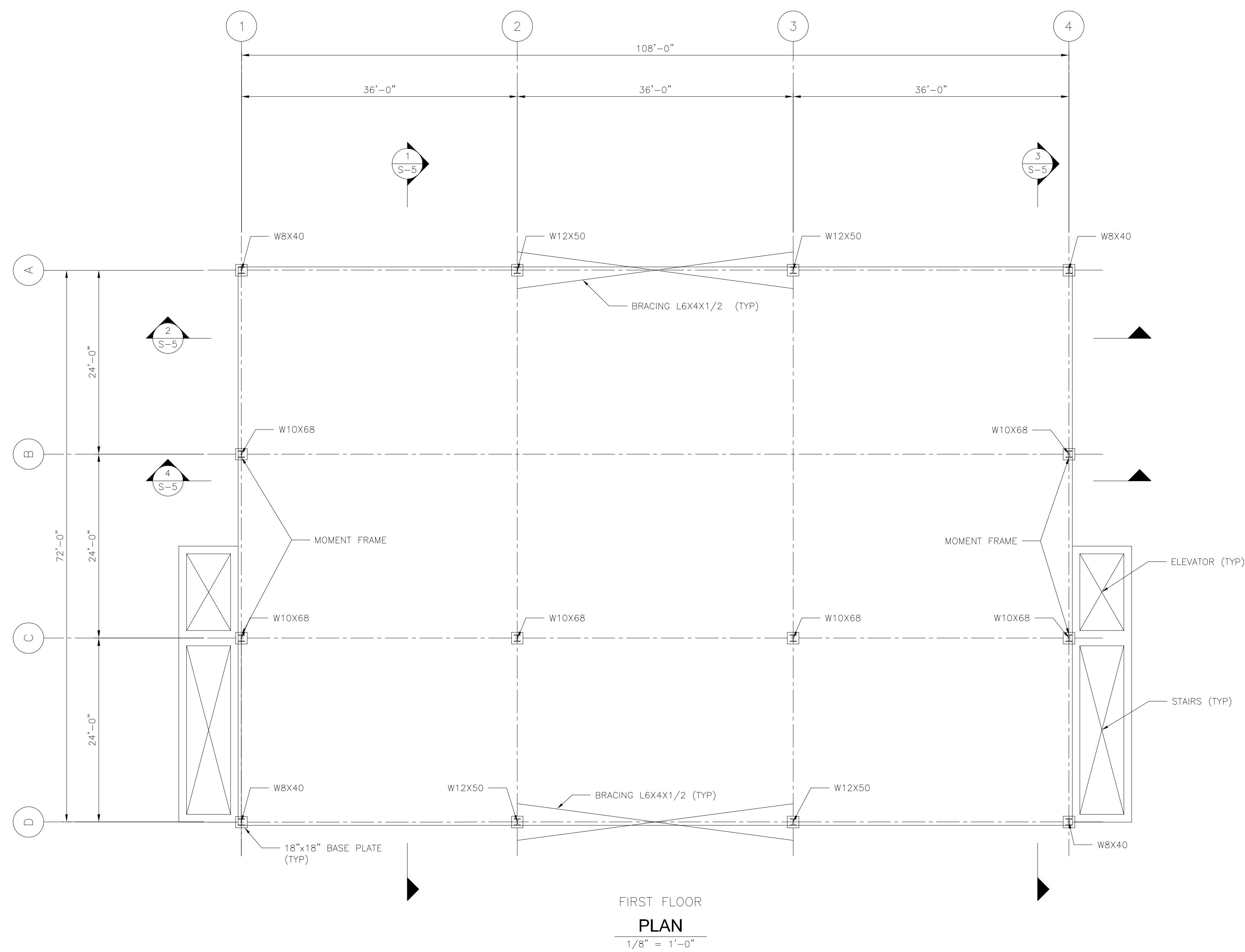



FIRST FLOOR

PLAN

1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: DEC 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

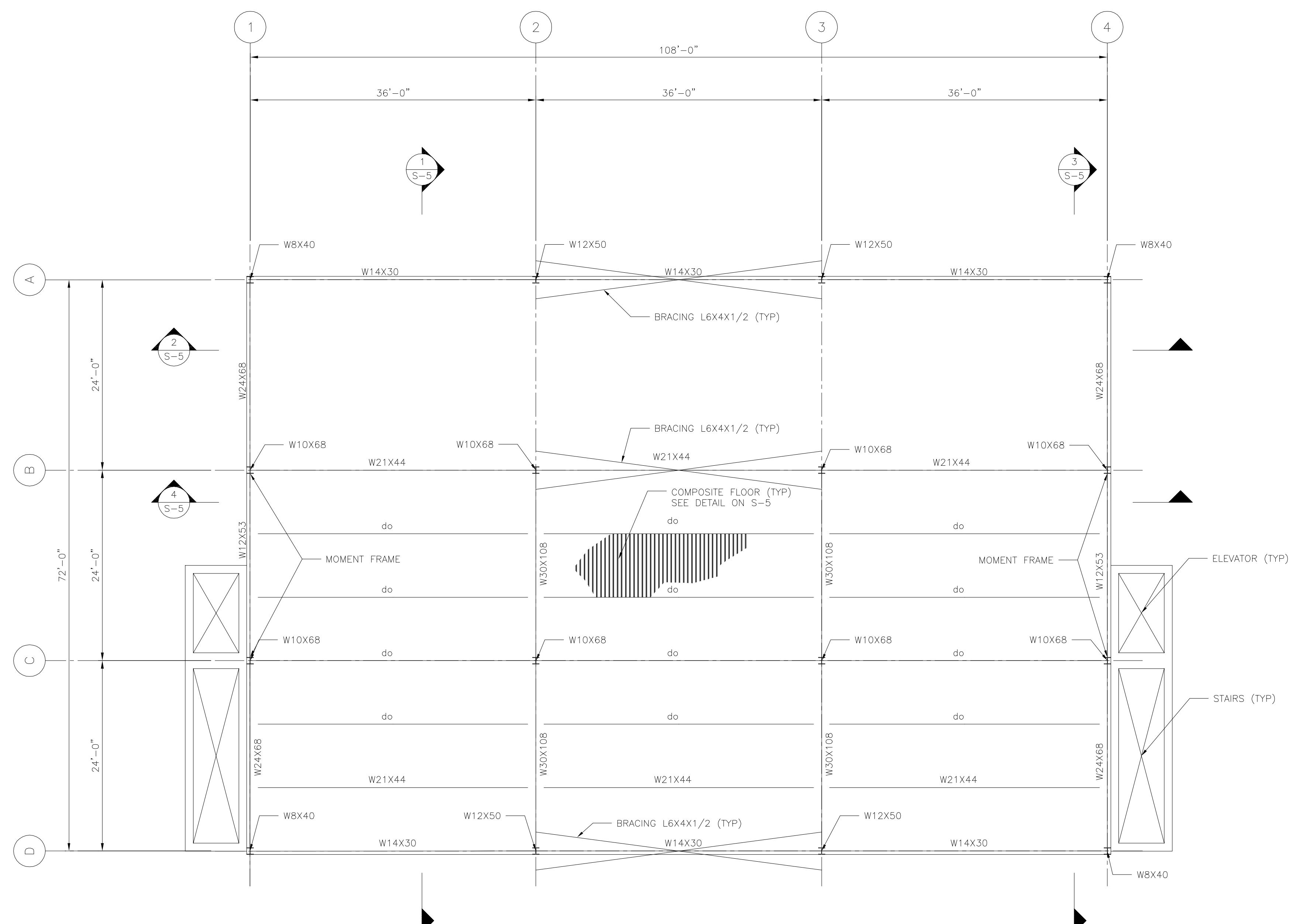
FIRST FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S001PD
SHEET NO.
S-1

100% SUBMITTAL - FOR CONSTRUCTION

NOTE:

1. TOS EL +15.0
2. DECK VULCRAFT 1.5VLR19




SECOND FLOOR
PLAN
1/8" = 1'-0"

| | | | | |
|---|---|---|---|---|
| REV. NO. | DATE | DRWN | CHKD | REMARKS |

| | |
|---|---|
| DESIGNED BY: | Z.ZAVALIANOS |
| DRAWN BY: | M.LASKEY |
| SHEET CHK'D BY: | A.GOUVEIA |
| CROSS CHK'D BY: | Z.ZAVALIANOS |
| APPROVED BY: | M.LASKEY |
| DATE: | DEC 2014 |

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

SECOND FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S002PD
SHEET NO.
S-2

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XREFs: [CDMS_2436] Images: []
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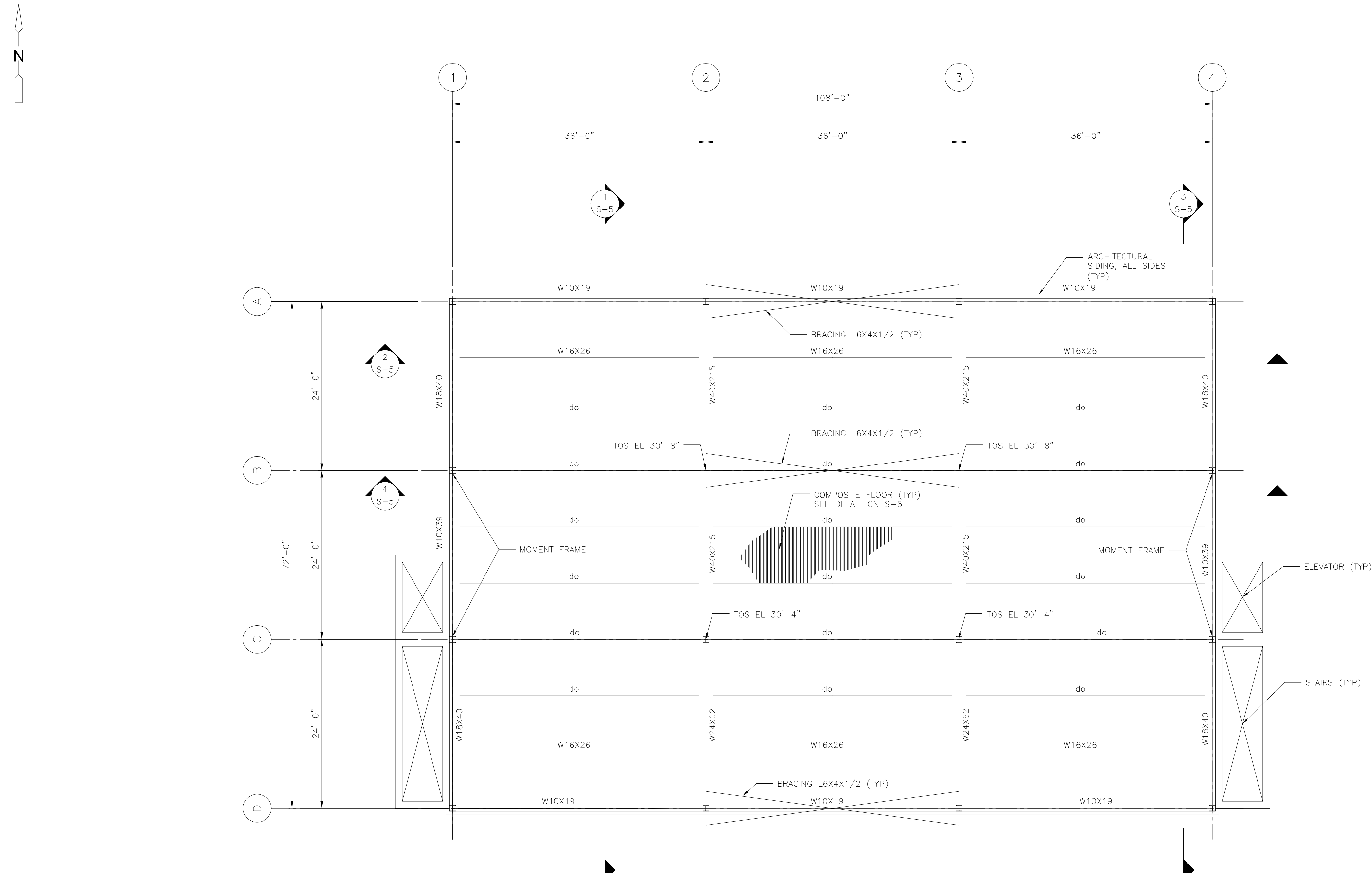
NOTE:

1. TOS EL +30.0 UON.
2. DECK VULCRAFT B18 3-SPAN.




ROOF
PLAN
1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

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DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: DEC 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

ROOF STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S003PD
SHEET NO.
S-3

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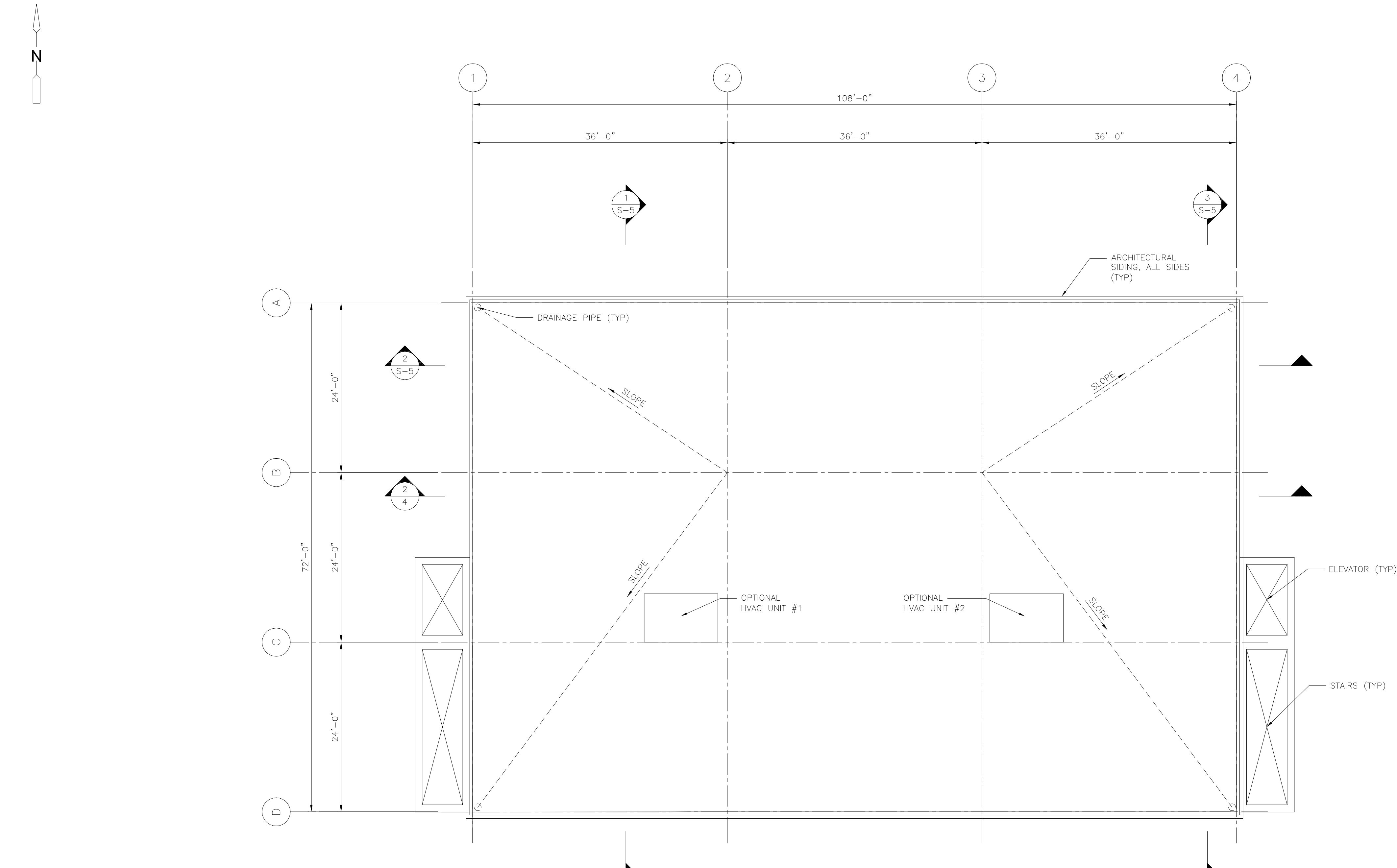


ROOF
PLAN
1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

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CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: DEC 2014

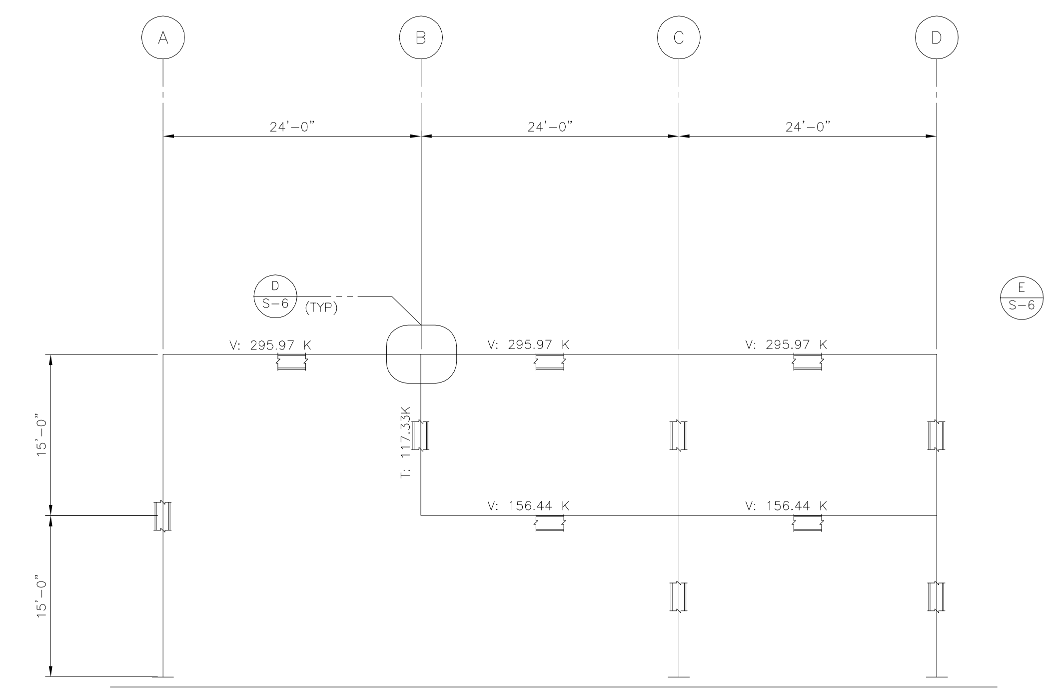
14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

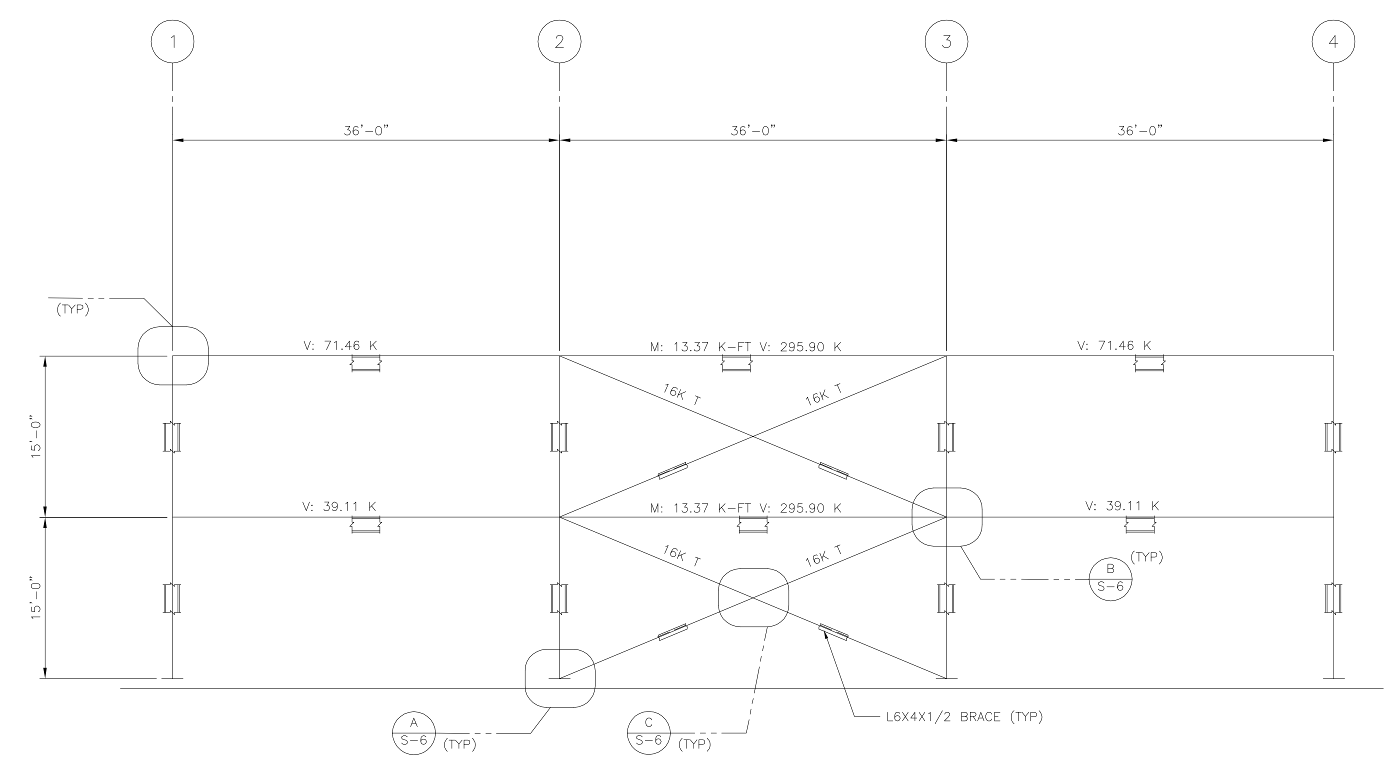
ROOF PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S004PD
SHEET NO.
S-4

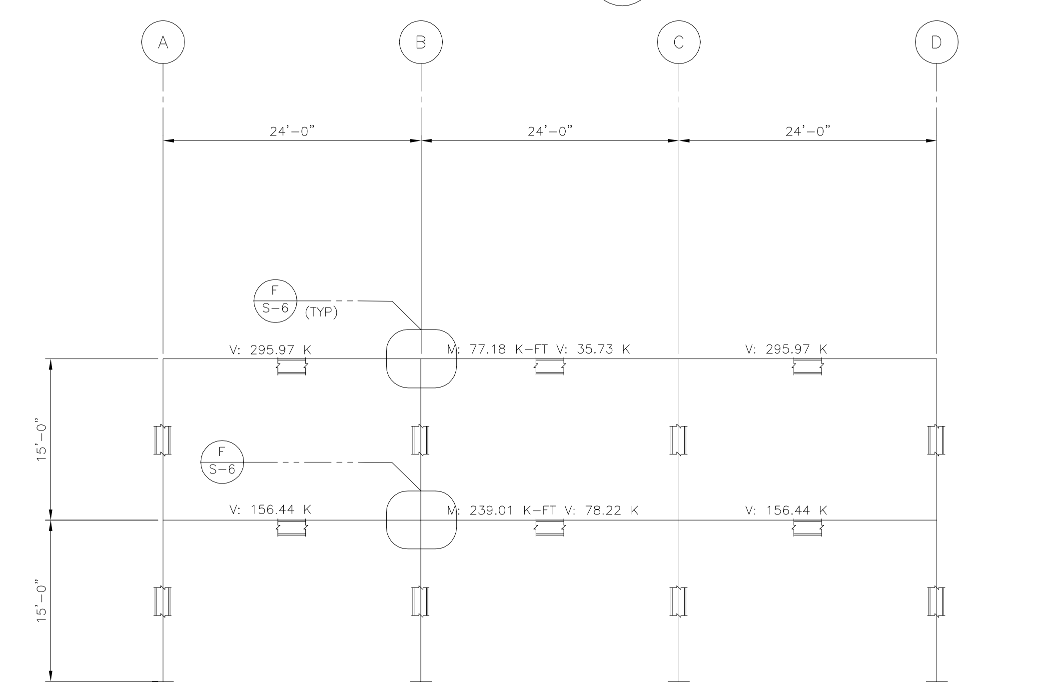
100% SUBMITTAL - FOR CONSTRUCTION



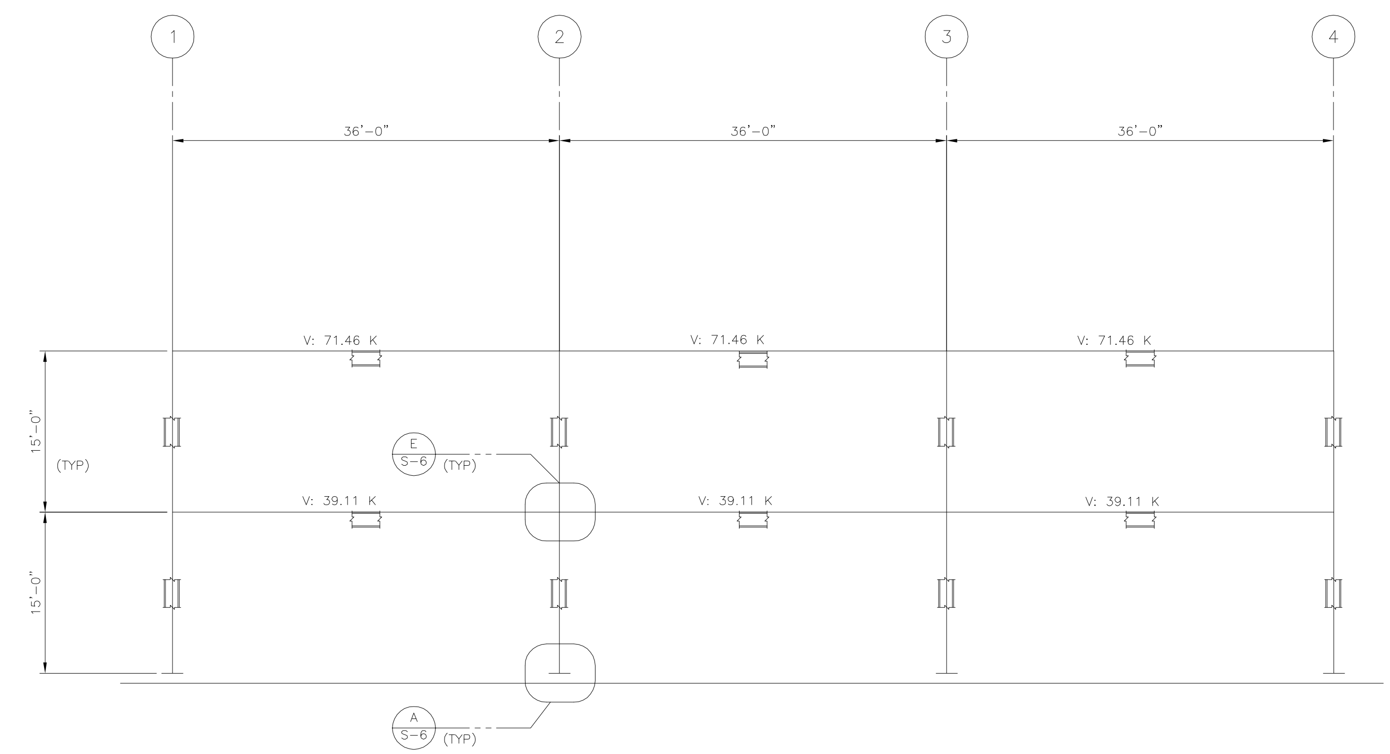

ELEVATION ALONG GRID 2
SECTION 1
1/8" = 1'-0"




ELEVATION ALONG GRID A
SECTION 2
1/8" = 1'-0"




ELEVATION ALONG GRID 4
SECTION 3
1/8" = 1'-0"




ELEVATION ALONG GRID B
SECTION 4
1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| | | | | |

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SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: DEC 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

STEEL ELEVATIONS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S005PD
SHEET NO.
S-5

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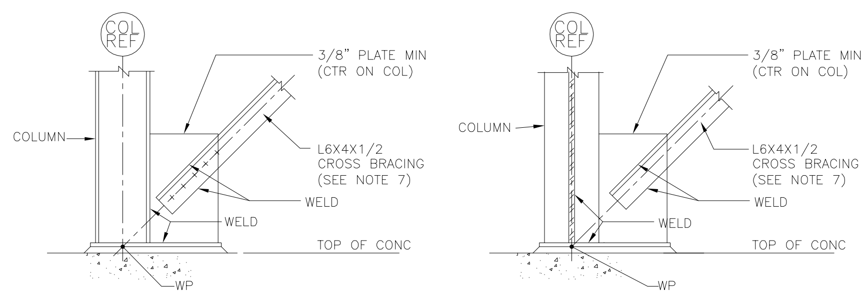
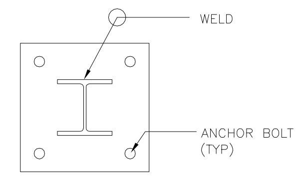


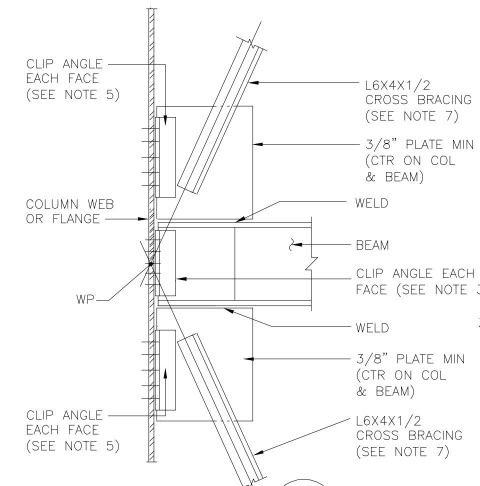

@ COLUMN FLANGE

@ COLUMN WEB

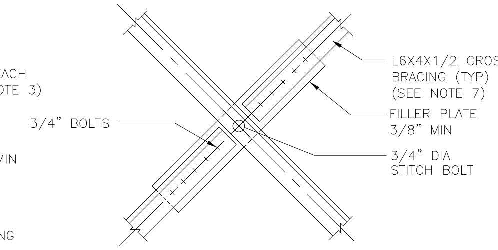
**DETAIL A**
NTS




TYPICAL BASE PLATE CONNECTION




**DETAIL B**
NTS




**DETAIL C**
NTS

STRUCTURAL STEEL NOTES:

1. STRUCTURAL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH THE THIRTEENTH EDITION OF THE AISC MANUAL OF STEEL CONSTRUCTION, PROVISIONS FOR ALLOWABLE STRENGTH DESIGN.

2. MATERIAL FOR STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING UON:
STRUCTURAL STEEL WIDE FLANGE SHAPES ASTM A992
ALL OTHER STRUCTURAL STEEL SHAPES, PLATES, AND BARS ASTM A36

3. ALL BEAM CONNECTIONS SHALL BE DESIGNED FOR ONE HALF OF THE MAXIMUM TOTAL UNIFORM LOAD TABULATED IN TABLE 3-6, PART 3 OF THE AISC MANUAL, OR FOR DESIGN REACTION SHOWN, PLUS AXIAL LOAD (#K T/C) WHERE INDICATED ON THE DRAWINGS, UON. ALL CLIP ANGLES SHALL BE ⅜" MINIMUM THICKNESS.

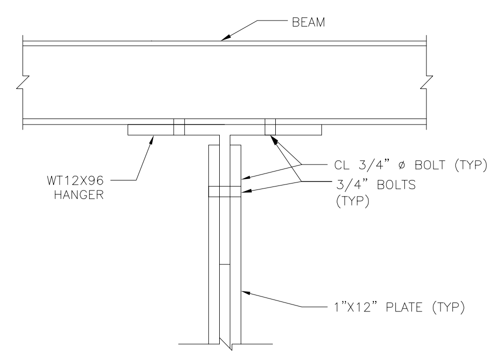
4. ALL BOLTED CONNECTIONS SHALL BE SLIP-CRITICAL UON.

5. FIELD CONNECTIONS SHALL BE BY ASTM A-325 HIGH STRENGTH BOLTS, UON. BOLTS SHALL BE 3/4" DIAMETER IN 13/16" DIAMETER HOLES, UON.

6. WELDING SHALL BE PERFORMED BY CERTIFIED WELDERS IN ACCORDANCE WITH AWS D1.1"STRUCTURAL WELDING CODE-STEEL". WELDING FILLER METAL SHALL BE AWS A5.1 E70XX ELECTRODES. SURFACES TO BE WELDED SHALL BE WIRE BRUSHED CLEAN BEFORE WELDING. THE AREA TO BE FIELD WELDED SHALL BE CLEANED FREE OF PAINT, DIRT, RUST, GREASE, AND OTHER FOREIGN MATTER PRIOR TO WELDING.

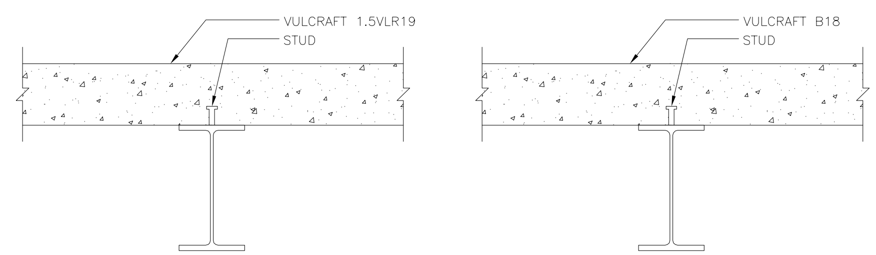
7. BRACING CONNECTIONS SHALL BE DESIGNED FOR AXIAL LOADS SHOWN ON DRAWINGS. WHEN BRACING IS CONNECTED TO ONE MEMBER ONLY, THE CONNECTION SHALL DEVELOP THE FULL LOAD. BOLTED CONNECTIONS SHALL HAVE A MINIMUM OF 2 BOLTS, UON.

8. UON - UNLESS OTHERWISE NOTED




CONNECTION TO ROOF ELEVATION
SIMILAR FOR CONNECTION TO SECOND FLOOR

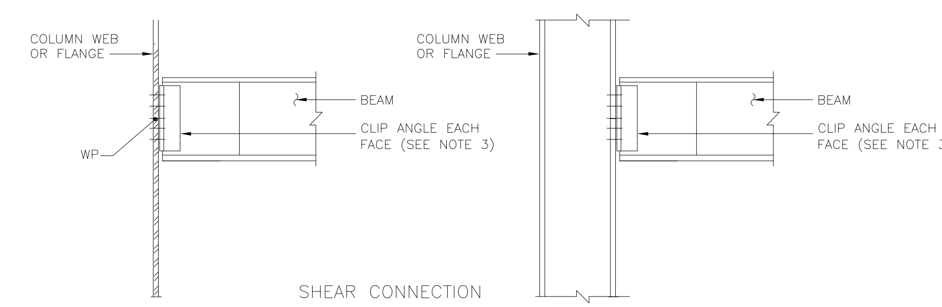
**DETAIL C**
NTS




2ND FLOOR

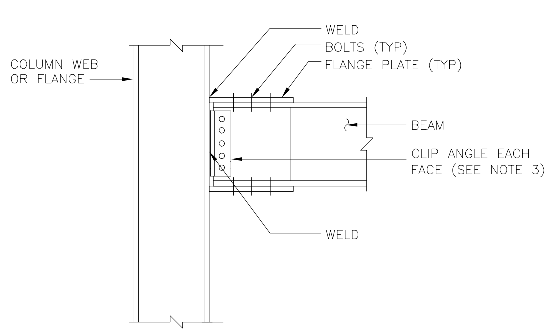
ROOF

**DETAIL D**
NTS



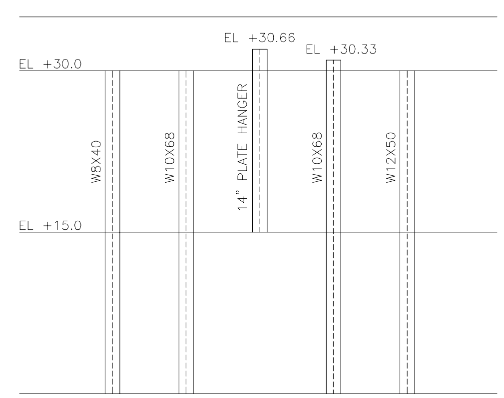

SHEAR CONNECTION

**DETAIL E**
NTS




MOMENT CONNECTION

**DETAIL F**
NTS




COLUMN SCHEDULE

**BEAM SCHEDULE**

| FLOOR | BEAM SHAPE | SPAN (FT) | COMMENTS |
|---|---|---|---|
| 2ND | W24X68 | 24 | |
| 2ND | W30X108 | 24 | |
| 2ND | W12X53 | 24 | |
| 2ND | W14X30 | 30 | |
| 2ND | W21X44 | 30 | |
| 3RD | W24X62 | 24 | |
| 3RD | W18X40 | 24 | |
| 3RD | W10X39 | 24 | |
| 3RD | W40X215 | 24 | |
| 3RD | W10X19 | 30 | |
| 3RD | W16X26 | 30 | |

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

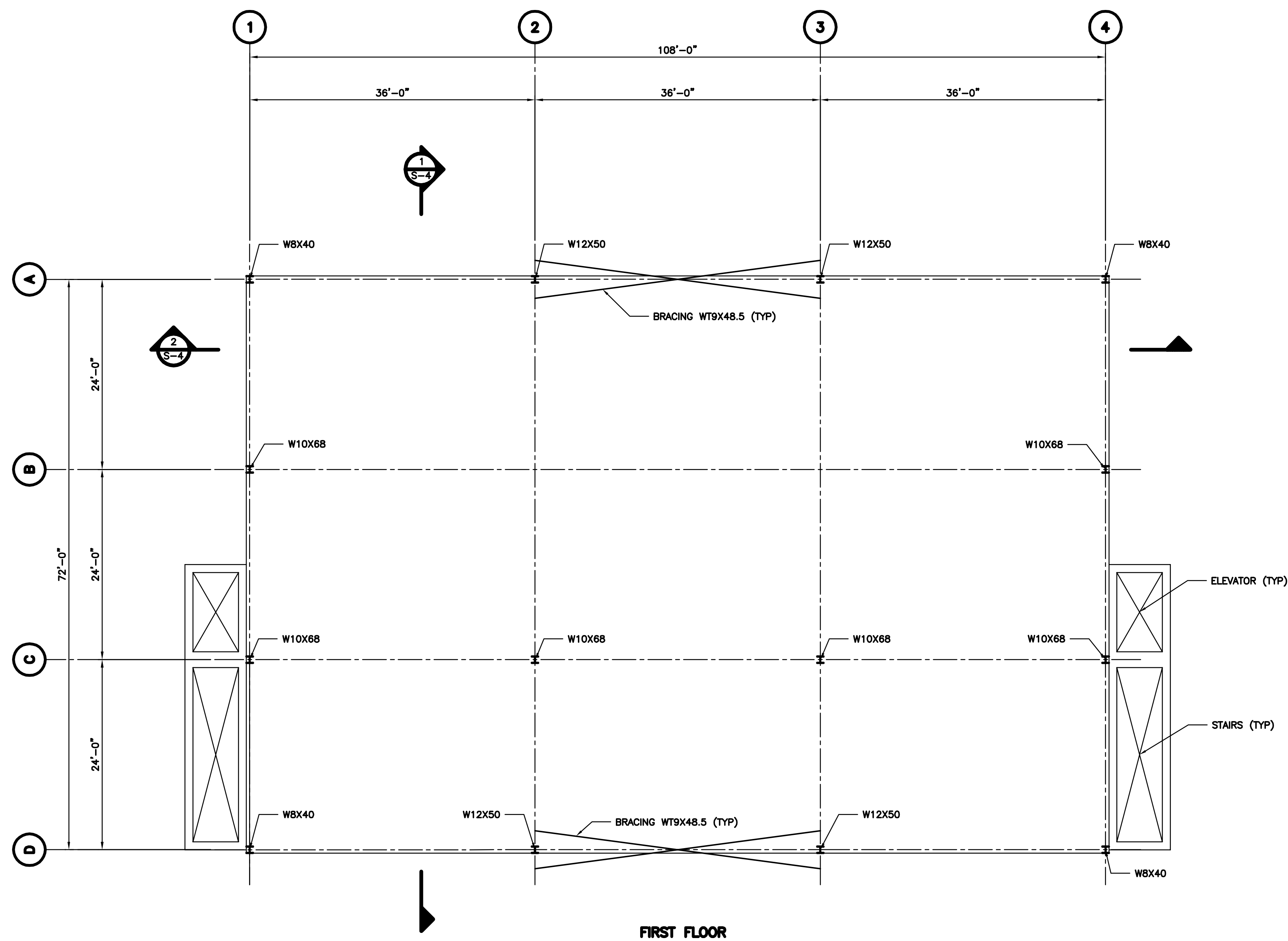
DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: DEC 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

STANDARD STEEL DETAILS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S006PD
SHEET NO.
S-6

100% SUBMITTAL - FOR CONSTRUCTION




**FIRST FLOOR**
PLAN
1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
FIRST FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S001PD
SHEET NO.
S-1

30% SUBMITTAL - NOT FOR CONSTRUCTION

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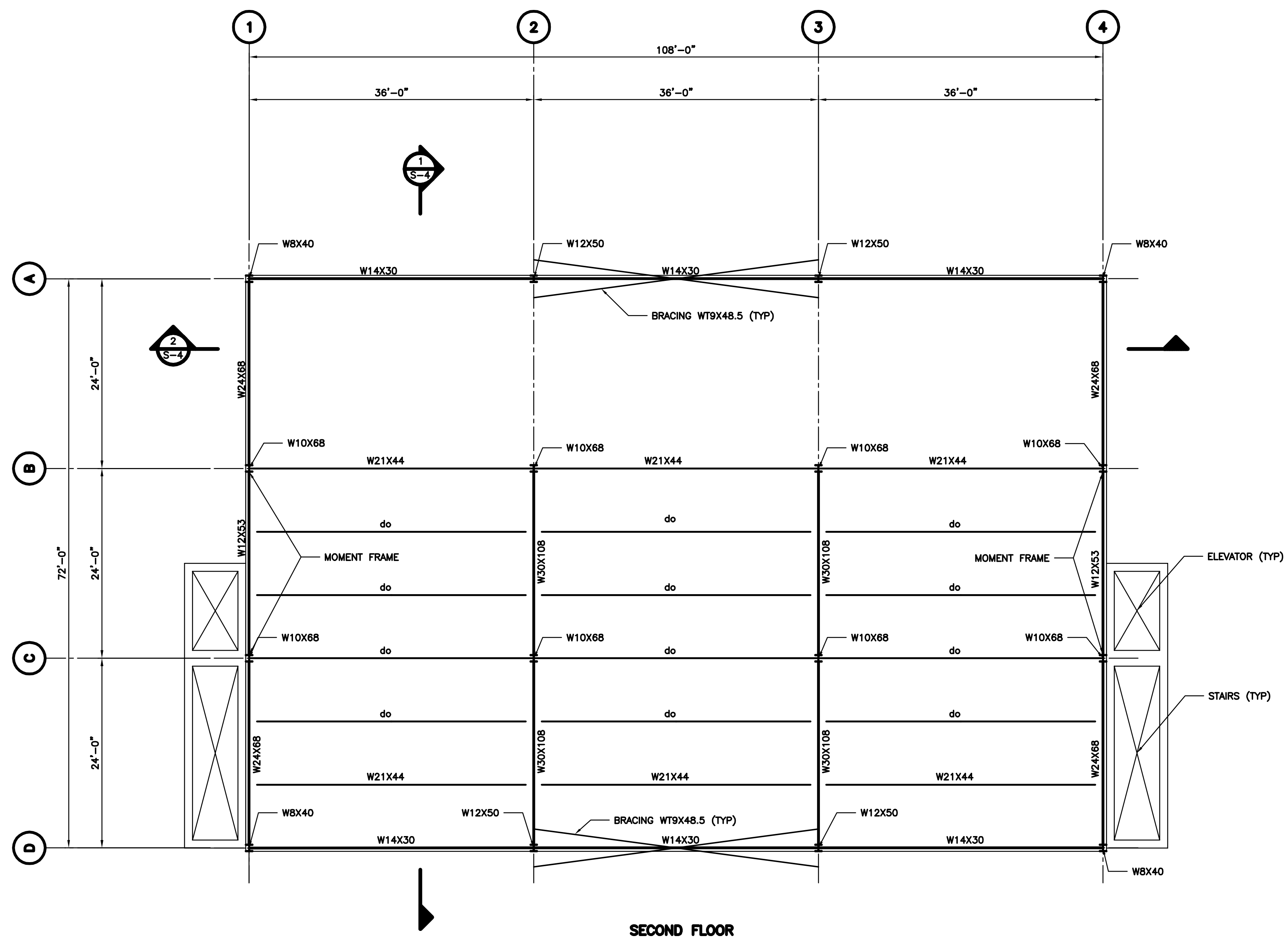


**SECOND FLOOR**

PLAN

1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
SECOND FLOOR STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S002PD
SHEET NO.
S-2

30% SUBMITTAL - NOT FOR CONSTRUCTION

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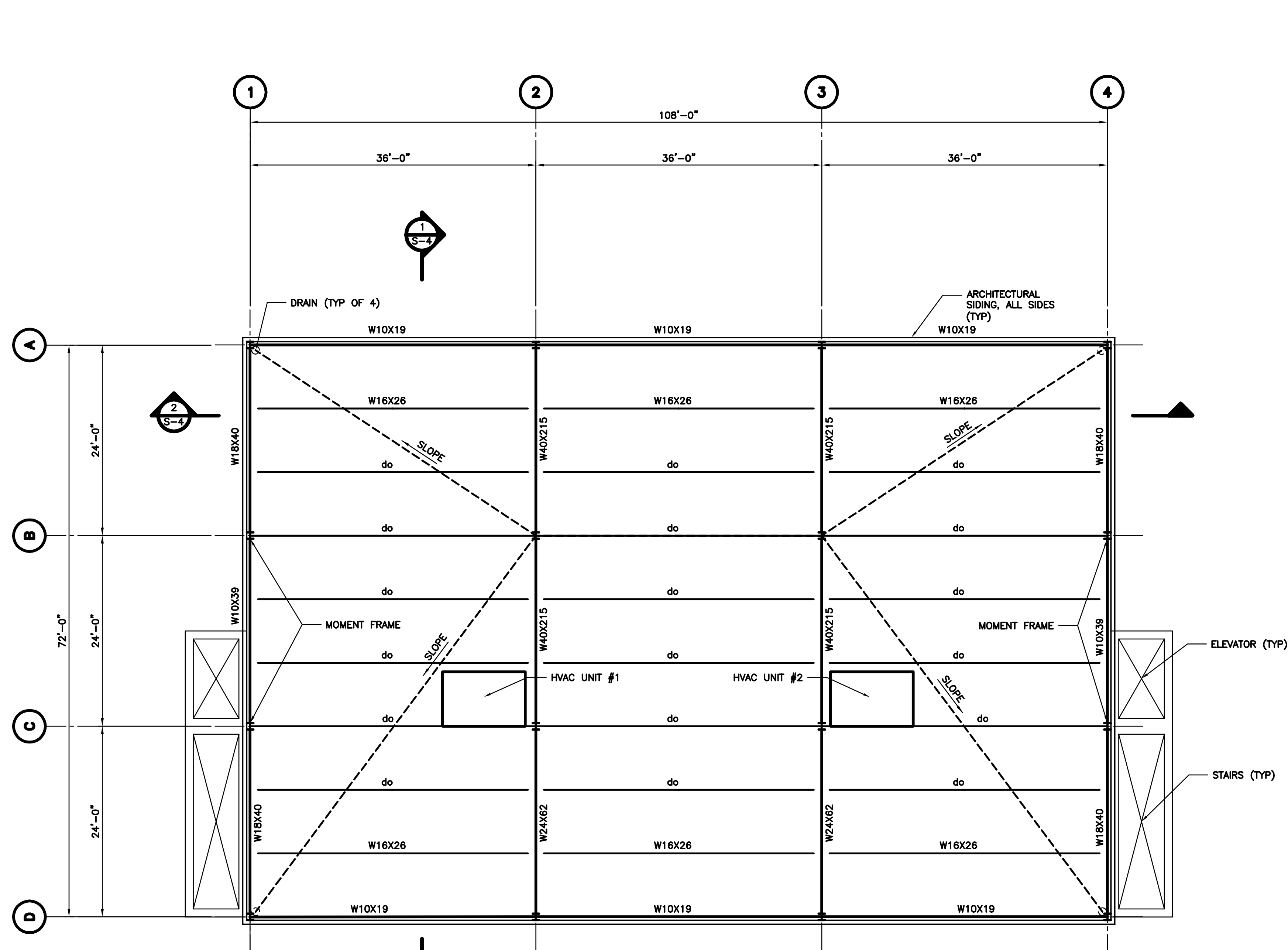


**ROOF**
PLAN
1/8" = 1'-0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| | | | | |

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
ROOF STEEL PLANS

PROJECT NO. XXXXX-XXXXX
FILE NAME: S003PD
SHEET NO.
S-3

30% SUBMITTAL - NOT FOR CONSTRUCTION

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Last saved by: LASKEYMA Time: 9/26/2014 4:31:50 PM
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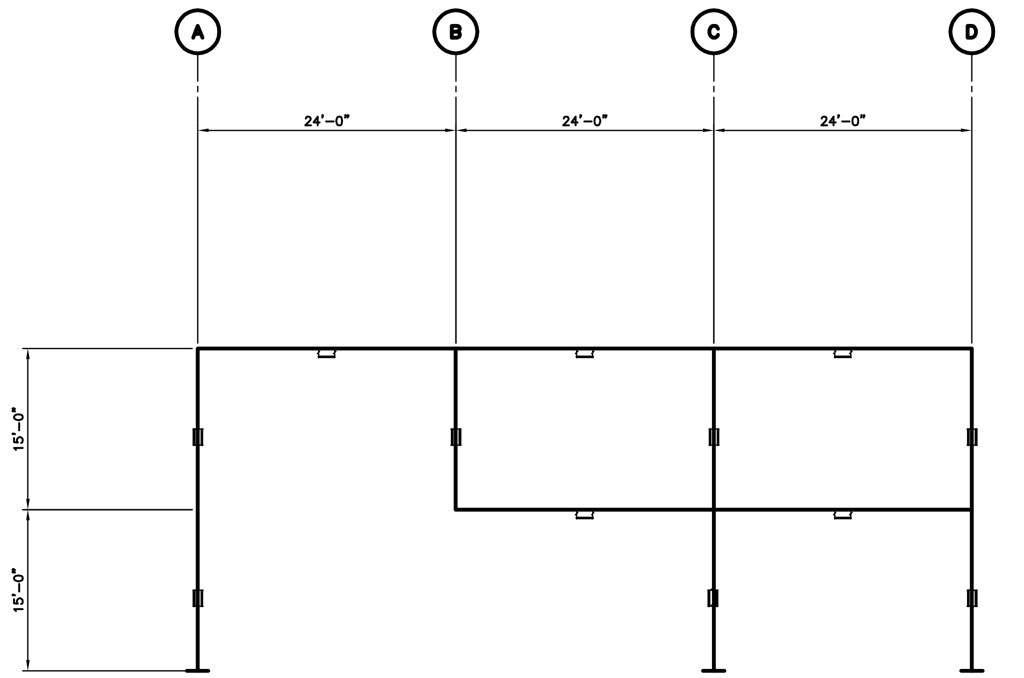
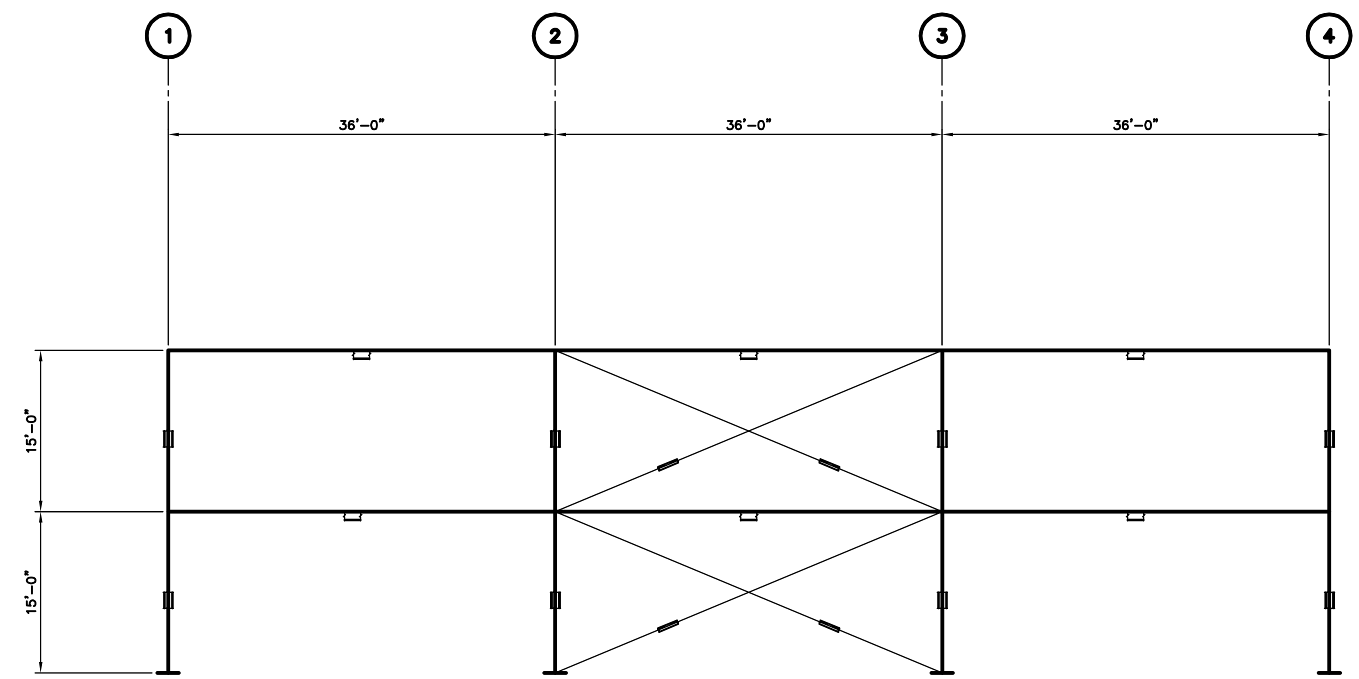



**ELEVATION ALONG GRID 2**

SECTION 1/– 
1/8" = 1'–0"




**ELEVATION ALONG GRID A**

SECTION 2/– 
1/8" = 1'–0"

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|
| | | | | |

DESIGNED BY: Z.ZAVALIANOS
DRAWN BY: M.LASKEY
SHEET CHK'D BY: A.GOUVEIA
CROSS CHK'D BY: Z.ZAVALIANOS
APPROVED BY: M.LASKEY
DATE: NOV 2014

14.551 STEEL DESIGN
STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
STEEL ELEVATIONS

PROJECT NO. XXXXX–XXXXX
FILE NAME: S004PD
SHEET NO.
S-4

30% SUBMITTAL - NOT FOR CONSTRUCTION

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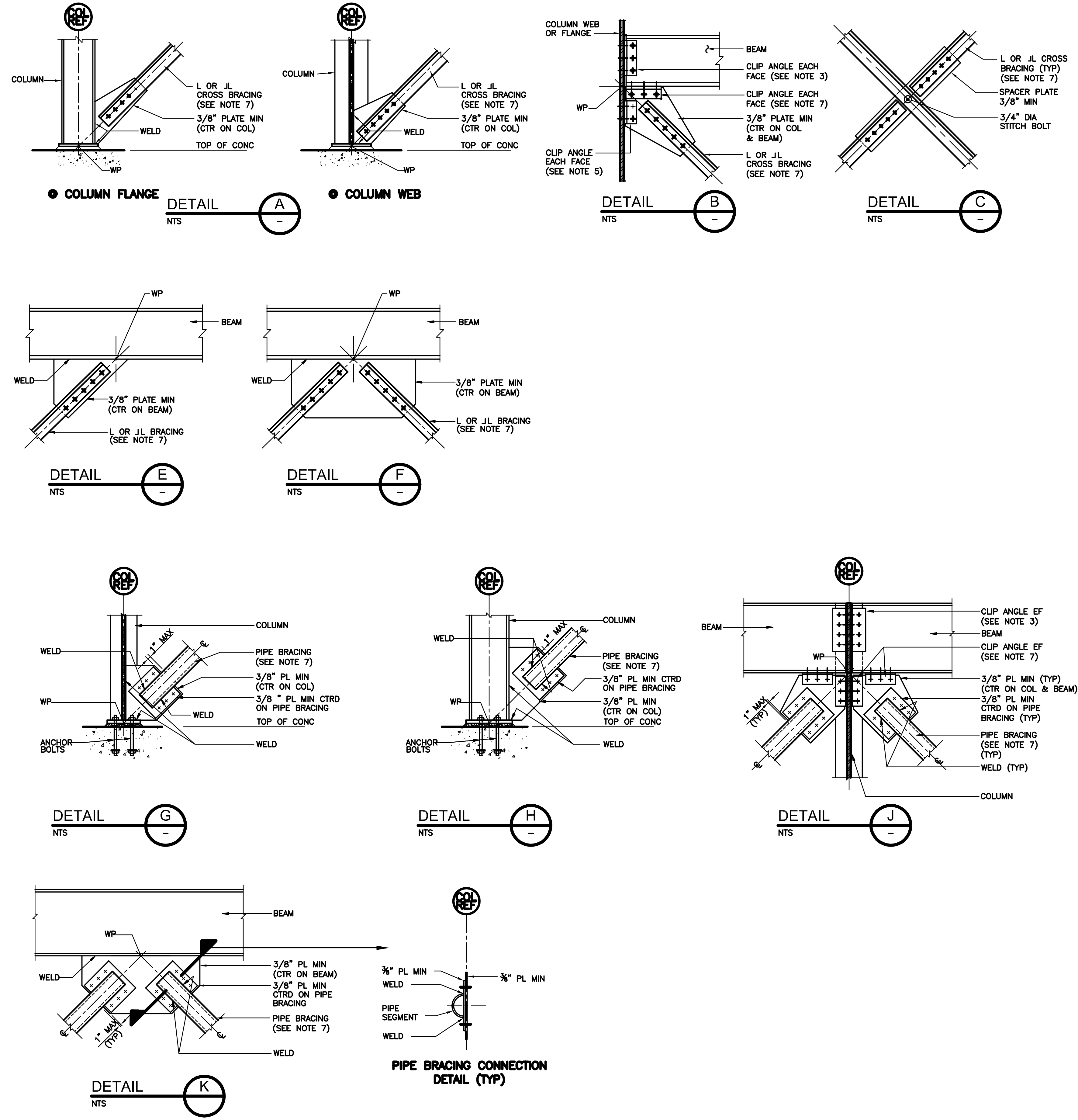



## STRUCTURAL STEEL NOTES:

1. STRUCTURAL STEEL SHALL BE FABRICATED IN ACCORDANCE WITH THE THIRTEENTH EDITION OF THE AISC MANUAL OF STEEL CONSTRUCTION, PROVISIONS FOR ALLOWABLE STRENGTH DESIGN.

2. MATERIAL FOR STRUCTURAL STEEL SHALL CONFORM TO THE FOLLOWING UON:
   - STRUCTURAL STEEL WIDE FLANGE SHAPES ASTM A992
   - STRUCTURAL STEEL HSS TUBE ASTM A500, GRADE B
   - ALL OTHER STRUCTURAL STEEL SHAPES, PLATES, AND BARS ASTM A36

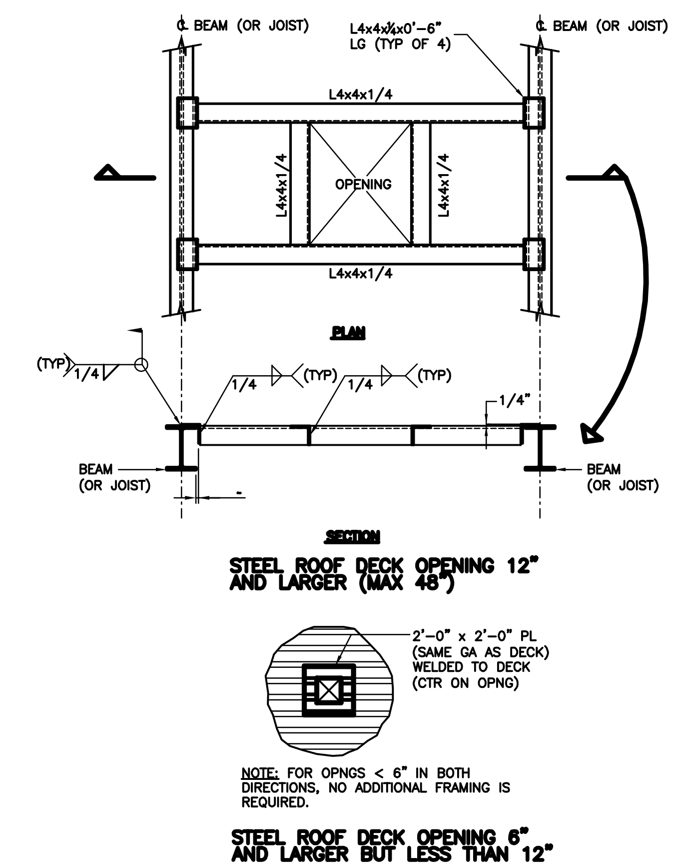
3. ALL BEAM CONNECTIONS SHALL BE DESIGNED FOR ONE HALF OF THE MAXIMUM TOTAL UNIFORM LOAD TABULATED IN TABLE 3-6, PART 3 OF THE AISC MANUAL, OR FOR DESIGN REACTION SHOWN, PLUS AXIAL LOAD (#K T/C) WHERE INDICATED ON THE DRAWINGS, UON. ALL CLIP ANGLES SHALL BE ⅜" MINIMUM THICKNESS.

4. ALL BOLTED CONNECTIONS SHALL BE SLIP-CRITICAL UON.

5. FIELD CONNECTIONS SHALL BE BY ASTM A-325 HIGH STRENGTH BOLTS, UON. BOLTS SHALL BE 3/4" DIAMETER IN 13/16" DIAMETER HOLES, UON. WHERE CONNECTIONS ARE TO EXISTING MEMBERS, HOLES SHALL BE FIELD DRILLED.

6. WELDING SHALL BE PERFORMED BY CERTIFIED WELDERS IN ACCORDANCE WITH AWS D1.1"STRUCTURAL WELDING CODE-STEEL". WELDING FILLER METAL SHALL BE AWS A5.1 E70XX ELECTRODES. SURFACES TO BE WELDED SHALL BE WIRE BRUSHED CLEAN BEFORE WELDING. THE AREA TO BE FIELD WELDED SHALL BE CLEANED FREE OF PAINT, DIRT, RUST, GREASE, AND OTHER FOREIGN MATTER PRIOR TO WELDING.

7. BRACING CONNECTIONS SHALL BE DESIGNED FOR AXIAL LOADS SHOWN ON DRAWINGS. FOR VERTICAL BRACING, THE HORIZONTAL COMPONENT OF THE LOAD SHALL BE DEVELOPED IN THE CONNECTION TO THE BEAM OR COLUMN BASE PLATE AND THE VERTICAL COMPONENT IN CONNECTION TO THE COLUMN. WHEN BRACING IS CONNECTED TO ONE MEMBER ONLY, THE CONNECTION SHALL DEVELOP THE FULL LOAD. BOLTED CONNECTIONS SHALL HAVE A MINIMUM OF 2 BOLTS, UON.




ROOF OPENING FRAMING
NTS

NOTE: SEE ROOF FRAMING PLAN FOR OPENING SIZES & LOCATIONS.

| REV. NO. | DATE | DRWN | CHKD | REMARKS |
|---|---|---|---|---|

| | |
|---|---|
| DESIGNED BY: | Z.ZAVALIANOS |
| DRAWN BY: | M.LASKEY |
| SHEET CHK'D BY: | A.GOUVEIA |
| CROSS CHK'D BY: | Z.ZAVALIANOS |
| APPROVED BY: | M.LASKEY |
| DATE: | NOV 2014 |

14.551 STEEL DESIGN

STEEL DESIGN CLASS PROJECT

PRELIMINARY DESIGN
STANDARD STEEL DETAILS

| | |
|---|---|
| PROJECT NO. | XXXXX-XXXXX |
| FILE NAME: | S005PD |
| SHEET NO. | S-5 |

30% SUBMITTAL - NOT FOR CONSTRUCTION

# APPENDIX C: CONSTRUCTION ESTIMATE

## Appendix C: Construction Estimate

Originator: ZZ
Check: ML

# COST ESTIMATE

Level 2-Beams/Girders

| | | | | | |
|---|---|---|---|---|---|
| Beams: | | | Girders: | | |
| Beam Length | 36 | ft | Girder Length | 24 | ft |
| | | | | | |
| Interior Beams | W21x44 | | Interior Girders | W30x108 | |
| Weight | 44 | plf | Weight | 108 | plf |
| Weight/beam | 1584 | lbs/beam | Weight/girder | 2592 | lbs/girder |
| # of Beams | 18 | | # of girder | 4 | |
| Total (Interior) | 28512 | lbs | Total (Interior) | 10368 | lbs |
| | | | | | |
| Extrerior Beams | W14x30 | | Exterior Girders | | |
| Weight | 30 | plf | Girder 1 | W24x68 | |
| Weight/beam | 1080 | lbs/beam | Weight | 68 | plf |
| # of Beams | 6 | | Weight/girder | 1632 | lbs/girder |
| Total (Exterior) | 6480 | lbs | # of girders | 4 | |
| | | | Total (G1) | 6528 | lbs |
| | | | | | |
| | | | Girder 2 | W12x53 | |
| | | | Weight | 53 | plf |
| | | | Weight/girder | 1272 | lbs/girder |
| | | | # of girders | 2 | |
| | | | Total (G2) | 2544 | lbs |
| | | | | | |
| Total Level 2 Beam | 34992 | lbs | Total Level 2 Girder | 9072 | lbs |
| | | | | | |
| Total (Level 2) | 44064 | lbs | | | |

Originator: ZZ
Check: ML

## Roof Level-Joists/Girders

**Joists:**

| | | |
|---|---|---|
| Joist Length | 36 | ft |
| Joist | 26KCS3 | |
| Linear Weight | 12.5 | plf |
| Weight/joist | 450 | lbs/beam |
| # of Joists | 30 | |
| Total (Joists) | 13500 | lbs |

**Girders:**

Interior Girders

| | | |
|---|---|---|
| Girder 1 | W40x215 | |
| Girder Length | 48 | ft |
| Weight | 215 | plf |
| Weight/girder | 10320 | lbs/girder |
| # of girders | 2 | |
| Total (G1) | 20640 | lbs |
| Girder 2 | W24x62 | |
| Girder Length | 24 | ft |
| Weight | 62 | plf |
| Weight/girder | 1488 | lbs/girder |
| # of girders | 2 | |
| Total (G2) | 2976 | lbs |

Exterior Girders

| | | |
|---|---|---|
| Girder 1 | W18x40 | |
| Girder Length | 24 | ft |
| Weight | 40 | plf |
| Weight/girder | 960 | lbs/girder |
| # of girders | 4 | |
| Total (G1) | 3840 | lbs |
| Girder 2 | W10x39 | |
| Girder Length | 24 | ft |
| Weight | 39 | plf |
| Weight/girder | 936 | lbs/girder |
| # of girders | 2 | |
| Total (G2) | 1872 | lbs |

| | | | | | |
|---|---|---|---|---|---|
| Total (Joists) | 13500 | lbs | Total (Girders) | 29328 | lbs |

| | | |
|---|---|---|
| Total (Roof Level) | 42828 | lbs |

Originator: ZZ
Check: ML

## Columns

| | | |
|---|---|---|
| Interior/Corner Columns | W8x40 | |
| Column Height | 30 | ft |
| Linear Weight | 40 | plf |
| Weight/column | 1200 | lbs/column |
| # of columns | 6 | |
| Total (int/ext) | 7200 | lbs |
| | | |
| Moment Frame Columns | W10x68 | |
| Column Height | 30 | ft |
| Linear Weight | 68 | plf |
| Weight/column | 2040 | lbs/column |
| # of columns | 4 | |
| Total (int/ext) | 8160 | lbs |
| | | |
| Braced Frame Columns | W12x50 | |
| Column Height | 30 | ft |
| Linear Weight | 50 | plf |
| Weight/column | 1500 | lbs/column |
| # of columns | 4 | |
| Total (int/ext) | 6000 | lbs |
| | | |
| Total (Columns) | 21360 | lbs |

## Bracing

| | | |
|---|---|---|
| Bracing | L6x4x0.5 | |
| Bracing Length | 39 | ft |
| Linear Weight | 16.2 | plf |
| Weight/brace | 631.8 | lbs/brace |
| # of braces | 6 | |
| Total (Bracing) | 3790.8 | lbs |

## Hangers

Plate

| | | |
|---|---|---|
| thickness | 1 | in |
| width | 14 | in |
| Height | 15 | ft |
| # plates | 2 | |
| Volume | 2.92 | $ft^3$ |
| Steel Unit Weight | 490 | pcf |
| Total (Hanger) | 2858.333 | lbs |

Originator: ZZ
Check: ML

*Material (Steel) Cost*

| | | |
|---|---|---|
| Total Main Structural Steel | 57.45 | tons |
| Add 20% for connections/miscellaneous details | 68.94 | tons |
| Steel Rate | 2500 | $/ton |
| Total Material Cost | $ 172,351.70 | |

*Labor/Equipment Cost*

| | | |
|---|---|---|
| Crew Unit Cost | 1100 | $/ton |
| | $ 75,834.75 | |
| 20% for equipment | $ 15,166.95 | |
| Total Labor/equipment cost | $ 91,001.70 | |

| | |
|---|---|
| Sub Total | $ 187,518.65 |
| O&P (25%) | $ 46,879.66 |

| | |
|---|---|
| Total Project Cost | $ 234,398.31 |