Prince Mohammad Bin Fahd University

College of Engineering

Department of Civil Engineering

Senior Design Project (ASSE III)

Fall 2015

# *Comparative Design & Analysis of a Mosque Using Steel and Reinforced Concrete*


Presented by:

| | |
|---|---|
| Hussain Ahmed Alabuo | 200900408 |
| Mohammed Essa Alabuo | 200900403 |
| Abdullan Nami Alnami | 200901322 |
| Mufarreh Mohammed Alshammary | 201101147 |

Supervised by:

Dr. Andi Asiz

Eng. Danish Ahmed

# ACKNWOLDGEMENT

We take this opportunity to express our profound gratitude and deep regards to our supervisors Dr. Andi Asiz and Eng. Danish Ahmed for their exemplary guidance, monitoring and constant encouragement throughout this project. The blessing, help and guidance given by them shall carry us a long way in the journey of life.

We also take this opportunity to express a deep sense of gratitude to the Dean of College of Engineering Dr. Jamal Nayfeh for his cordial support, valuable information and guidance, which helped us in completing our engineering degree.

We would like to thank all faculty members of civil engineering department: Dr. Omar Ouda, Dr. Alaa Salman, and Dr. Abas for the valuable information provided by them in their respective fields. We are grateful for their cooperation during our engineering study.

We would like to thank Almulhim Consulting Engineering represented by Eng. Mohammed Jamal for assisting in the preliminary design of mosque by AutoCAD program which was used as the main drawing for this project.

Lastly, we thank our families, friends and all citizens of Eastern Province for their constant encouragement and help.

# ABSTRACT

The Kingdom of Saudi Arabia has been applying the Shari`ah of Islam, acting in accordance with the Holy Qur'an and Sunnah, and making judgments based on Allah's Shari`ah. It also strives to propagate and call people to Islam with wisdom and fair preaching.

This is the essential cornerstone of its system, duties, and obligations, to which it pays due attention. The Basic Law of Saudi Arabia provides for deriving power from the Holy Qur'an and Sunnah, which rule over this Law and all other laws of the Kingdom.

It also stipulates that the Kingdom should protect the Islamic creed, apply the Shari`ah, enjoin good, forbid evil, and call people to Allah.

The Sectors of Islamic Affairs, Da`wah, Guidance, and Endowments have been among the outstanding pillars and essential parts of the Kingdom. Since the foundations of the kingdom were laid by King `Abdul `Aziz Al Saud and his sons after him, the Kingdom has established, sponsored, supported, and developed large organizations for this purpose, flourishing in the era of the Custodian of the Two Holy Mosques, his Crown Prince, and his Second Deputy Prime Minister.

Subsequently wherever the Muslim's have gone, they built Mosques for their needs in that community and worship Allah. Moreover mosque indeed is a great example of traditional and long lasting buildings which civil engineers do best making a strong, stable, and sustainable buildings.

Through our civil engineering study at Prince Mohammed bin Fahd University, as well as taking the opportunity to apply what we have studied at the university level, we decided to choose the design of the mosque. We make Comparative Design & Analysis of a Mosque Using Steel and Reinforced Concrete Elements design and analysis ( figure 1), including calculation of the foundation.

Our project covered by several programs such as Etabs2015, Sap2000, AutoCAD, SkiCiv, LimCon, and steel connections.

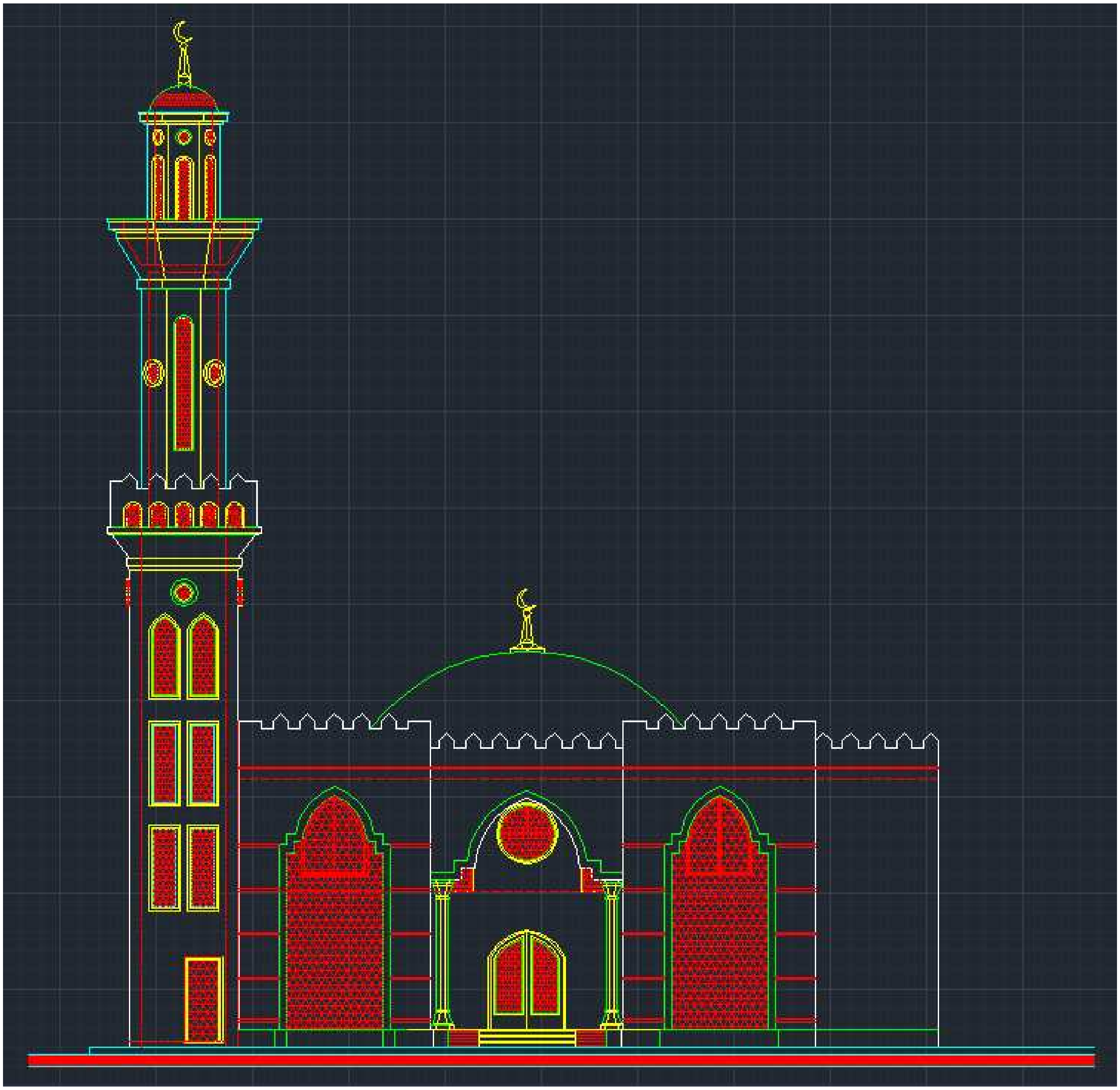


**Figure 1 Mosque overview**

# Table of Contents

# List of figures

# List of Tables

# Chapter 1: Introduction

## 1.1 General

There are strict and detailed requirements in Sunni fiqh for a place of worship to be considered a mosque, with places that do not meet these requirements regarded as musallas. There are stringent restrictions on the uses of the area formally demarcated as the mosque (which is often a small portion of the larger complex), and, in the Islamic Sharia law, after an area is formally designated as a mosque, it remains so until the Last Day.

Many mosques have elaborate domes, minarets, and prayer halls, in varying styles of architecture. Mosques originated on the Arabian Peninsula, but are now found in all inhabited continents. The mosque serves as a place where Muslims can come together for salat (meaning "prayer") as well as a center for information, education, and dispute settlement. The imam leads the congregation in prayer.

The mosque played a major part in the spread of education in the Muslim World, and the association of the mosque with education remained one of its main characteristics throughout history, and, the school became an indispensable appendage to the mosque. From the earliest days of Islam, the mosque was the center of the Muslim community, a place for prayer, meditation, religious instruction, political discussion, and a school. And anywhere Islam took hold, mosques were established, and basic religious and educational instruction began. Once established, mosques developed into well-known places of learning, often with hundreds, even thousands, of students, and frequently contained important libraries In Iraq, pharmacology, engineering, astronomy and other subjects were taught in the mosques of Baghdad, and students came from Syria, Persia and India to learn these sciences. While at the Qarawiyin Mosque, there were courses on grammar, rhetoric, logic, mathematics, and astronomy, and possibly history, geography and chemistry.

## 1.2 The main objectives for our project are:

1- To apply all theories that have been learned "in all structural courses" to this Project.
2- To learn how to design a building from A to Z and passing through all design stages.
3- To be familiar with other elements that is used in real life and wasn't learned in the courses.

**Codes in practice:**

1- American Concrete Institute (ACI 318-08) code.
2- Saudi Building Code. SBC.
3- International Building Code UBC-1997.
4- Minimum Design Loads for Buildings and Other Structures ASCE_7-10

**The materials used, in this project include steel, concrete and foundation with the following properties:**

1- Fy = 420 MPa steel.

2- f'c = 30MPa concrete (Slab, beam).

3- F'c = 40MPa concrete.(column and shear wall)

The building will be planted on a soil with 450 KPa bearing capacity.

## 1.3 Scope of the report:

The present report is composed of five chapters. A detailed description of the project and a historical background is presented in chapter 1. The concrete design is introduced in chapter 2 including floors, beams, dome, and columns. Following that, the steel design with connections are given in chapter 3. The foundation design discussed in chapter 4.

# Chapter 2: Concrete

## 2.1 Loads Calculations:

### 2.1.1 Types of loads:

Structural loads are forces applied to a component of a structure or to the structure as a unit.

In structural design, assumed loads are specified international and local design codes for types of structures, geographic locations, and usage. In addition to the load magnitude, its frequency of occurrence, distribution, and nature (static or dynamic) are important factors in design. Loads cause stresses, deformation in structures. Assessment of their effects is carried out by the methods of structural analysis. Excess load or overloading may cause structural failure, and hence such possibility should be either considered in the design or strictly controlled.

In the Euro codes, the term actions have a similar meaning to loads, but encompass applied deformations as well as forces.

The following lists the common loading types primarily for civil infrastructure and land machinery. Structures for aerospace (e.g. aircraft, satellites, rockets, space stations, etc...) and marine environments (e.g. boats, submarines, etc.) have their own particular design loads and consideration includes dead loads but also includes forces set up by irreversible changes in a structure's constraints - for example, loads due to settlement, the secondary effects of pre-stress or due to shrinkage and creep in concrete.

#### A. Dead Load

The dead load is the weight of the structure acting with gravity on the foundations below. Snow load is the weight of the dead load and the imposed load but also the weight of the snow on top which could cause damage to the roof.

#### B. Live Loads

Live loads, or imposed loads, are temporary, of short duration, or moving. Examples include snow, wind, earthquake, traffic, movements, water pressures in tanks, and occupancy loads. For certain specialized structures, vibro-acoustic loads may be considered.

#### C. Environmental Loads

- Temperature changes leading to thermal expansion cause thermal loads
- loads caused by humidity or moisture induced expansion
- Water waves
- Shrinkage

### D. Static Loads

These are loads that build up gradually over time, or with negligible dynamic effects. Since structural analysis for static loads is much simpler than for dynamic loads, design codes usually specify statically-equivalent loads for dynamic loads caused by wind, traffic or earthquake

### E. Dynamic Loads

These are loads that display significant dynamic effects. Examples include impact loads, waves, wind gusts and strong earthquakes. Because of the complexity of analysis, dynamic loads are normally treated using statically equivalent loads for routine design of common structures. Dynamic loads are also caused by a force other than gravity.

### F. Load Combinations

A load combination results when more than one load type acts on the structure. Design codes usually specify a variety of load combinations together with weighting factors for each load type in order to ensure the safety of the structure under different probable loading scenarios.

- **The Load Combination Equations as per ACI 318 M- 08:**

1. $1.4(D + F)$
2. $1.2(D + F + T) + 1.6(L + H) + 0.5(L_r \text{ or } S \text{ or } R)$
3. $1.2D + 1.6(L_r \text{ or } S \text{ or } R) + ((0.5 \text{ or } 1.0)^{*}L \text{ or } 0.8W)$
4. $1.2D + 1.6W + (0.5 \text{ or } 1.0)^{*}L + 0.5(L_r \text{ or } S \text{ or } R)$
5. $1.2D + 1.0E + (0.5 \text{ or } 1.0)^{*}L + 0.2S$
6. $0.9D + 1.6W + 1.6H$
7. $0.9D + 1.0E + 1.6H$

### 2.1.2 Load parameters and super imposed dead load:

**A. Load parameters according to ACI & SBC code:**

- Ceramic tiles thickness = 0.03 m.
- Ceramic tiles density = 23.5 kN / $m^3$.
- Density of RC = 24.525 kN / $m^3$.
- Slabs thickness = 0.25 m.

**B. Super imposed dead load:**

- Ceramic tiles = 0.03 * 23.5 = 0.705 kN/$m^2$.
- Concrete = 24.525 * 0.25 = 6.13125 kN/$m^2$.

### 2.1.3 Loads on slabs:

Since our design have roof and one floor (Mezzanine floor) so we will have two different LL (live load).

**a) $1^{st}$ Floor (Mezzanine floor)**

Equivalent L.L (SBC 301 table 4-1) = 5 kN/$m^2$.

**b) Roof**

Equivalent L.L (SBC 301 table 4-1) = 2.5 kN/$m^2$

## 2.2: Design of slabs

### 2.2.1 Types of Slab:

**Introduction:**

Reinforced concrete slab is used to provide flat, useful surface. It might be supported either by beam or directly by column. Slabs might be classified to one way or two way slab depending on the deflection shape of slab as shown on the figures 2.1 and 2.2.

The selection of the system of slab used in the structure is depending on the architectural plan and the construction requirement and the economy.

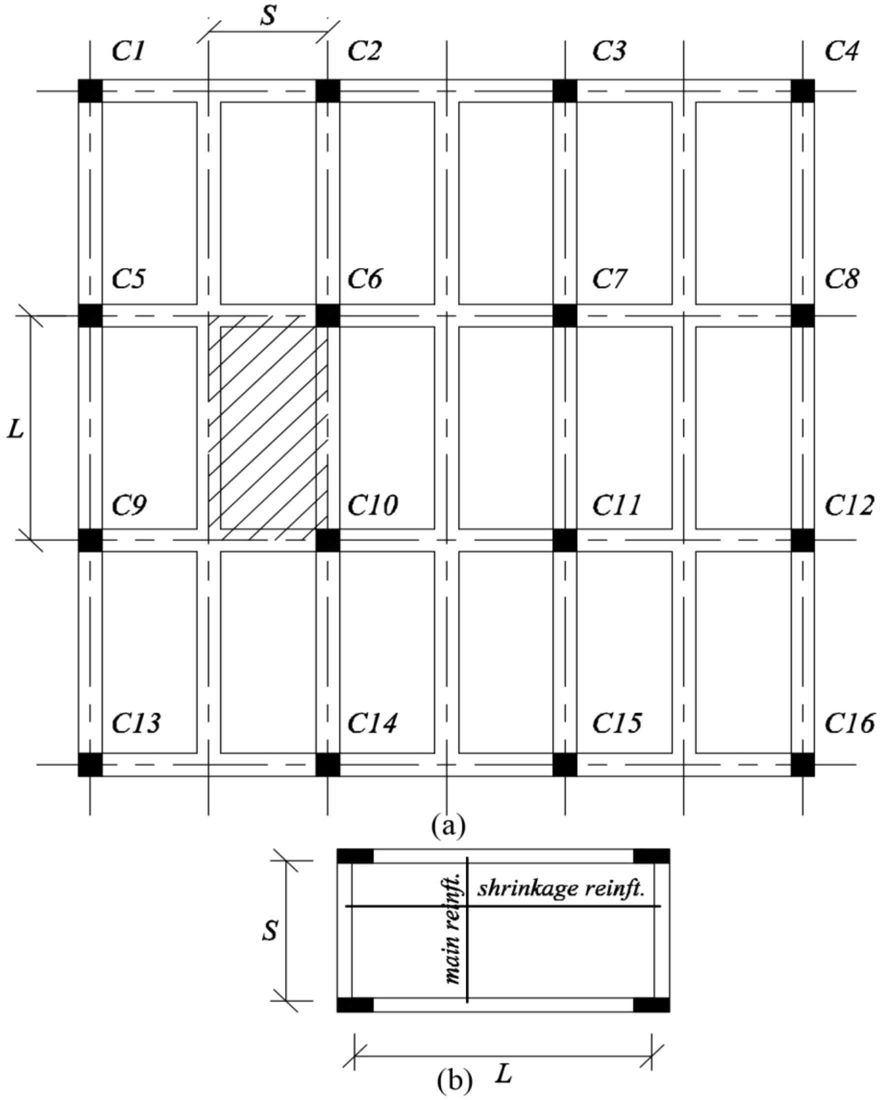



**Figure 2 One way slab; (a) classification; (b) reinforcement**

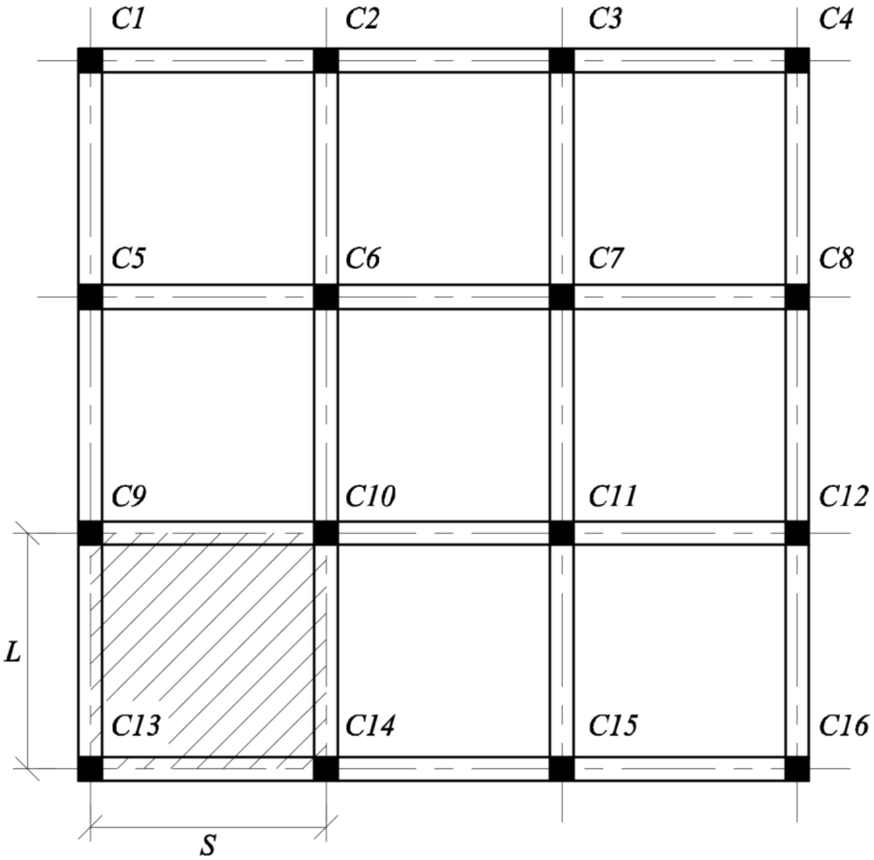
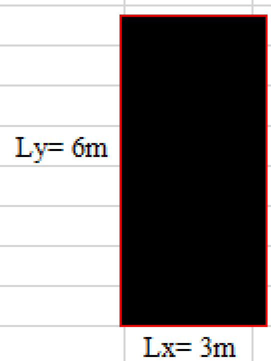



**Figure 3 Two way slab**

**To know which type is our slab we need to calculate it:**

| |
|---|
| **Design of Slab** |
| **Type of Slab** |
| $Ly / Lx = 6/3 = 2 \geq 2$ |
| It's a One way Slab |



### 2.2.2 Steps to Design one way slab:

Step 1: Determine the minimum required thickness of slab from table 9.5(c) ACI 318

Step 2: Determine the ultimate loads that effect on the slab.

Step 3: Determine the total factored static moment.

Step 4: Determine the required area of steel with the required spacing.

### 2.2.3 Slab Minimum Thickness:

**Table 1 Minimum thickness of one-way solid slabs**

| Element | Simply supported | One end continuous | Both ends continuous | Cantilever |
|---|---|---|---|---|
| One-way solid slabs | *l*/20 | *l*/24 | *l*/28 | *l*/10 |

Where ***l*** is the span length in the direction of bending.

| | | | | |
|---|---|---|---|---|
| Minimum thickness of one-way slabs. | | | | |
| Both ends Continous. | | | | |
| One-way solid slab = L/28 | | | | |
| where L is the span length in the direction of binding. | | | | |
| | L | | Total | |
| L/28 = | 236.592 | ÷ 28 = | 8.449714 | in |
| 8.449714 in = 0.21572 m ≈ 0.25 m thikness of slabs | | | | |

### 2.2.4 Materials properties:

**Table 2 Properties**

| | | |
|---|---|---|
| concrete | fc´ | 30 MPa |
| steel | Fy | 420 MPa |

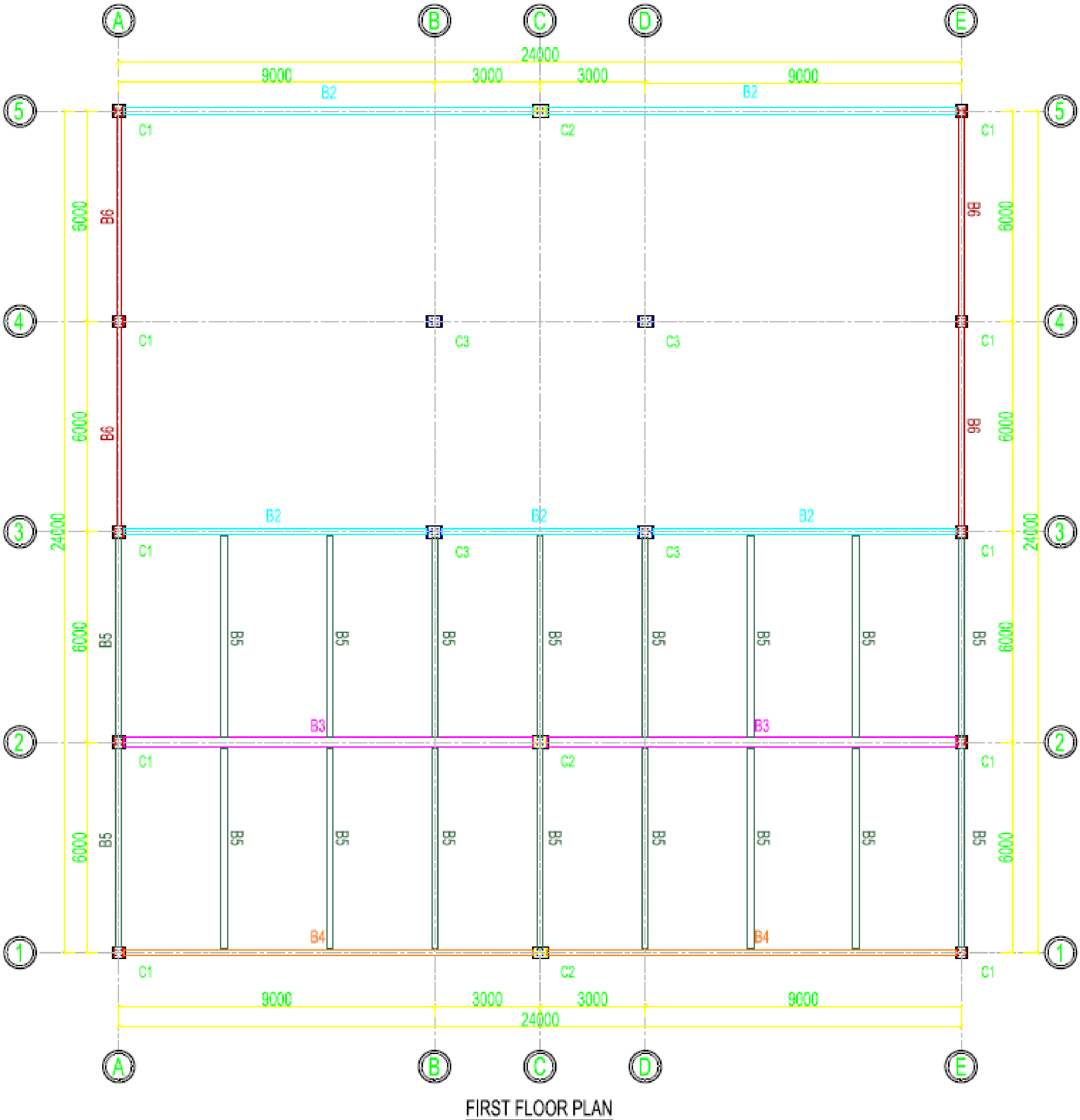
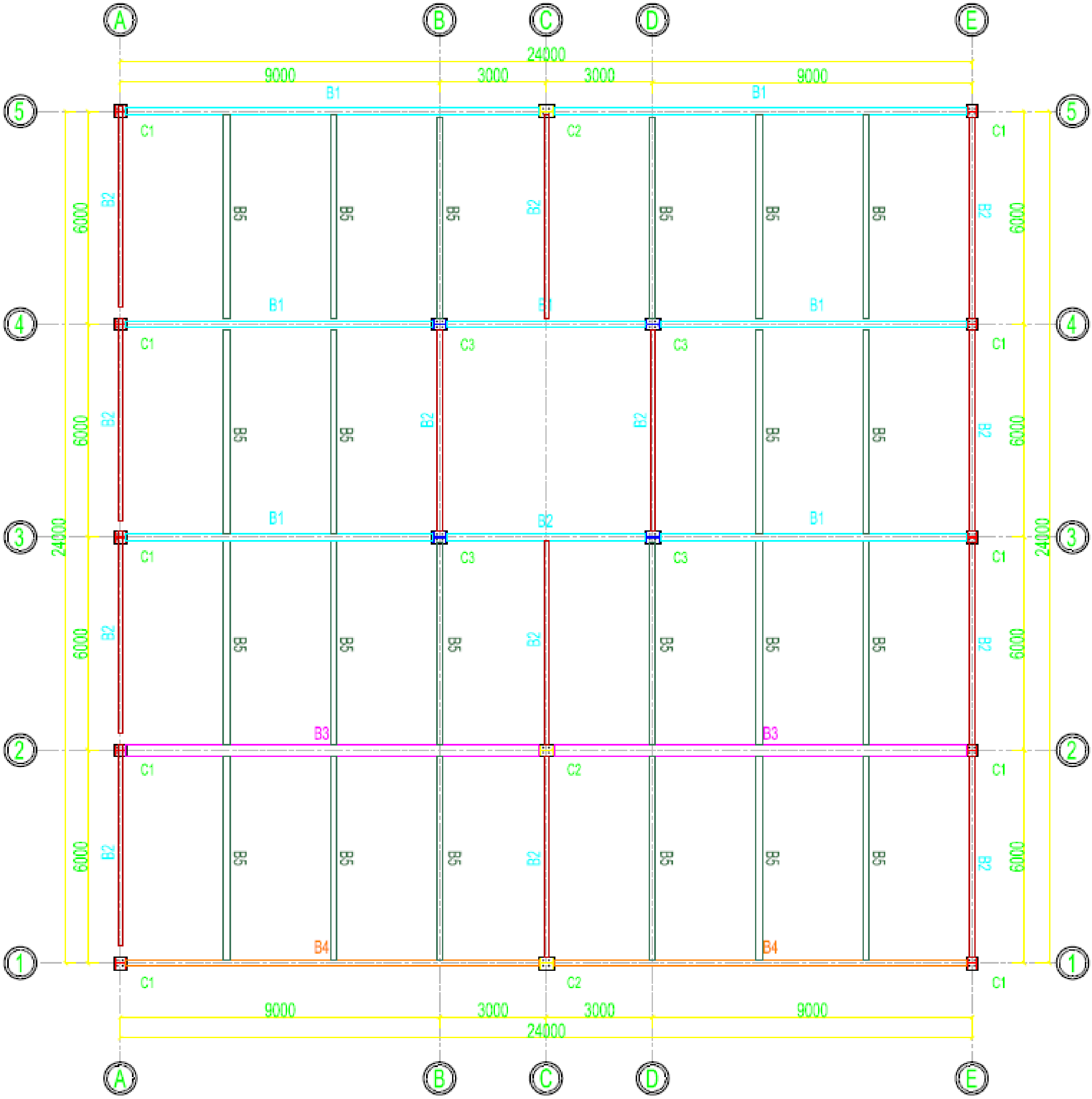



**Figure 4 1st Floor (Mezzanine floor)**




SECOND FLOOR PLAN

**Figure 5 Roof Floor**

### 2.2.5 Load Calculation:

#### a) $1^{st}$ Floor (Mezzanine floor) loads:

| | | | | | |
|---|---|---|---|---|---|
| **First Floor (Mezanine)** | | | | | |
| **Loads** | | | | | |
| Dead Load: | | | | | |
| | | | | | |
| Ceramic Tiles: | | | | | |
| D=kN/m$^3$ | Th=m | Total | | | |
| 23.5 × | 0.03 | = 0.705 kN/m$^2$ | | | |
| | | | | | |
| Concrete: | | | | | |
| D=kN/m$^3$ | Th=m | Total | | | |
| 24.525 × | 0.25 | = 6.13125 kN/m$^2$ | | | |
| | | | | | |
| Total Dead Load: | | | | | |
| | Ceramic | Concrete | Total | | |
| | 0.705 × | 6.13125 | = 6.83625 kN/m$^2$ | | |
| | | | | | |
| Live Load: | | | | | |
| Live Load = | | 5 kN/m$^2$ | | | |
| Ultimate Load: | | | | | |
| WU = 1.2 x DL + 1.6 x LL | | | | Total | |
| 1.2 × | 6.83625 | + 1.6 | × 5 | = 16.2035 kN/m$^2$ | |

Saudi Building Code, 301 Structural – Loading and Forces
CHAPTER 4: LIVE LOADS

### b) Roof Floor loads:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| Design of Slab | | | | | | | |
| Loads | | | | | | | |
| Roof Floor | | | | | | | |
| Dead Load: | | | | | | | |
| Concrete: | | | | | | | |
| D=kN/m³ | | Th =m | | Total | | | |
| 24.525 | × | 0.25 | = | 6.13125 kN/m² | | | |
| | | | | | | | |
| Live Load: | | | | | | | |
| Live Load = | | | | 2.5 kN/m² | | | |
| Ultimate Load: | | | | | | | |
| WU = 1.2 x DL + 1.6 x LL | | | | | | Total | |
| 1.2 | × | 6.13125 | + | 1.6 | × | 2.5 | = 11.3575 kN/m² |

Saudi Building Code, 301 Structural – Loading and Forces
CHAPTER 4: LIVE LOADS

### 2.2.6 Flexural Design

The total factored static moment $M_u$ is given by:

$$M_u = \frac{W_u l_2 l_n^2}{8}$$

Where $W_u$ = factored load per unit area, $L_2$ length of span, measured center to center of supports in the direction perpendicular to the direction moments are being determined , and $L_n$ =is the average of adjacent clear span length.

### a) 1st Floor (Mezzanine floor) design:

| Design of Slab |
|---|
| One way Slab |
| Floor Slab |

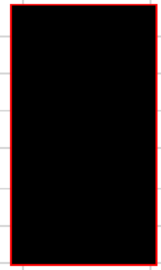
Yield point of the Steel

Fy = 60000 psi = 60 ksi.

More than two spans.

$$Mu = \frac{Wu * L2 * Ln^2}{8}$$

$L_n$ = 6m



$L_2$ = 3m

| | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Wu = | 16.2035 | $kN/m^2$ | → | = | 0.338491 | $kip/ft^2$ | | |
| | | | | | Total | | | |
| Mu = | $\frac{16.204 \times 3 \times 36}{8}$ | | → | = | 218.7473 | kN-m | → | 397.5777 kip-in |

**Area of Steel**

| | | | | | | |
|---|---|---|---|---|---|---|
| La = 0.9 x H | | | H | | Total | |
| | La = | 0.9 | × | 10 | = | 9 in |

$$A_s = \frac{Mu}{0.9 * fy * la}$$

| | | | | |
|---|---|---|---|---|
| $A_s$ = | $\frac{397.5777}{0.9 \times 60 \times 9}$ | → | = | Total 0.818061 $in^2$ |

| | | | | | |
|---|---|---|---|---|---|
| $A_s$ min = | 200/Fy | | | Total | |
| | 200 | ÷ | 60000 | = | 0.0033 $in^2$ |

$A_s$ max = 0.75 x $A_{sb}$

$A_{sb}$ = $N_{cb}$/Fy

$N_{cb}$ = 0.85*f'c*$a_b$*b

$$\frac{Cb}{0.003} = \frac{La - Cb}{0.00207}$$

from the formula above we get Cb= 5.4 in

a= β*Cb → β= 0.85 for f'c ≤ 4000 psi

| | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| a= | 0.85 | × | 5.4 | = | 4.59 | in | | | | | | |
| b = | 1 ft | | → | | 12 in | ( assuming ) | | | | | | |
| $N_{cb}$= | 0.85 | × | 4000 | × | 4.59 | × | 12 | = | 187272 | $Ib/in^2$ | → | 187.272 KIb |
| $A_{sb}$= | 187.272 | × | 60 | = | 3.1212 | $in^2$ | | | | | | |
| $A_s$ max = | 0.75 | × | 3.1212 | = | 2.3409 | $in^2$ | | | | | | |

we will use As as the area of steel bars = 0.818061 $in^2$

**Design of steel bars in Floor slab**

we will use As as the area of steel bars = 0.818061 $in^2$

Use the table to Find the Number of Bars and Diamater of bars

Table 2.3 Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | $\boxed{0.88}$ | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

**Take 2#6 Ø** → **2 No. 6 bars**

**b) Roof design:**

Design of Slab

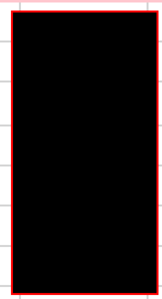
One way Slab

Roof Slab

Yield point of the Steel

Fy = 60000 psi = 60 ksi.

$L_n$= 6m



More than two spans.

$$Mu = \frac{Wu * L2 * Ln^2}{8}$$

Where Ln is the average of adjacent clear span length.

$L_2$= 3m

Wu = 11.3575 $kN/m^2$ → = 0.237258 $kip/ft^2$

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | | | | | | Total | | | |
| Mu = | $\frac{11.358 \times 3 \times 36}{8}$ | | | → | = | 153.3263 | kN-m | → | 278.6737 kip-in |

## Area of Steel

| $La = 0.9 \times H$ | | | H | | Total | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | La = | 0.9 | × | 10 | = | 9 in | | | |

$$A_s = \frac{Mu}{0.9 * fy * la}$$

$$A_s = \frac{278.6737}{0.9 \times 60 \times 9} \rightarrow = \text{Total } 0.573403 \text{ in}^2$$

$A_s$ min = 200/Fy

200 ÷ 60000 = Total 0.0033 $in^2$

$A_s$ max = $0.75 \times A_{sb}$

$A_{sb} = N_{cb}/Fy$

$N_{cb} = 0.85 * f'c * a_b * b$

$$\frac{Cb}{0.003} = \frac{La - Cb}{0.00207}$$

from the formula above we get Cb= 5.4 in

a= β*Cb → β= 0.85 for f'c ≤ 4000 psi

| | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| a= | 0.85 | × | 5.4 | = | 4.59 | in | | | |
| b = | 1 ft | → | 12 in | ( assuming ) | | | | | |
| $N_{cb}$= | 0.85 | × | 4000 | × | 4.59 | × | 12 | = 187272 Ib/$in^2$ → | 187.272 KIb |
| $A_{sb}$= | 187.272 | × | 60 | = | 3.1212 | $in^2$ | | | |
| $A_s$ max = | 0.75 | × | 3.1212 | = | 2.3409 | $in^2$ | | | |

we will use As as the area of steel bars = 0.573403 $in^2$

## Design of steel bars in Roof slab

we will use As as the area of steel bars = 0.573403 $in^2$

Use the table to Find the Number of Bars and Diamater of bars

Table2.4 Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

**Take 1#7 Ø** → **1 No. 7 bars**

| Find the actual La | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | | | | | Prefixes | |
| | Actual d = h − Cover − stirrup − $\frac{d_b}{2}$ | | | | | h = | 10 in |
| | | | | | | $d_b$ = | 1 |
| | La = 0.9 x H | | | Total | | | |
| | La= | 0.9 × | 10 in = | 9 | in | | |
| | | | | | 1 | | |
| | Actual d = | 10 − | 1.50 − | 0 − | —— = | 8 in | |
| | | | | | 2 | | |

Table 2.5 Minimum Required Beam Width, *b* (in.)

| Number of bars | Bar number | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | # 3 and #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
| 2 | 6.0 | 6.0 | 6.5 | 6.5 | 7.0 | 7.5 | 8.0 | 8.0 |
| 3 | 7.5 | 8.0 | 8.0 | 8.5 | 9.0 | 9.5 | 10.5 | 11.0 |
| 4 | 9.0 | 9.5 | 10.0 | 10.5 | 11.0 | 12.0 | 13.0 | 14.0 |
| 5 | 10.5 | 11.0 | 11.5 | 12.5 | 13.0 | 14.0 | 15.5 | 16.5 |
| 6 | 12.0 | 12.5 | 13.5 | 14.0 | 15.0 | 16.5 | 18.0 | 19.5 |
| 7 | 13.5 | 14.5 | 15.0 | 16.0 | 17.0 | 18.5 | 20.5 | 22.5 |
| 8 | 15.0 | 16.0 | 17.0 | 18.0 | 19.0 | 21.0 | 23.0 | 25.0 |
| 9 | 16.5 | 17.5 | 18.5 | 20.0 | 21.0 | 23.0 | 25.5 | 28.0 |
| 10 | 18.0 | 19.0 | 20.5 | 21.5 | 23.0 | 25.5 | 28.0 | 31.0 |

| Find the actual b | | |
|---|---|---|
| from the As we will find the actual b after we assume it 12 in | | |
| Take 3#8 Ø | → | from table 5. the minimum width b = 9 in |

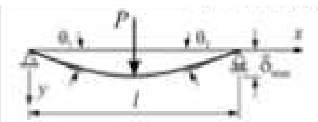
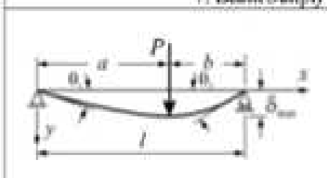
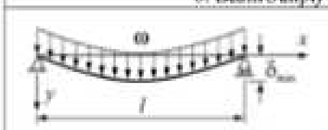
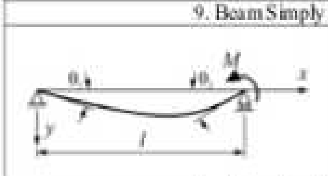
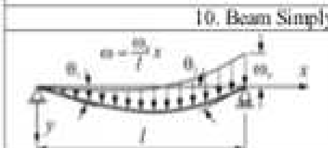
## 2.3 Design of beams & sub beams:

- **Introduction:**

Beams are the structural elements that carry the transferred loads from the slabs; they are treated as the slab's supports.

- **Beams types according to the reinforcement are:**
  1. Simply supported - a beam supported on the ends which are free to rotate and have no moment resistance.
  2. Fixed - a beam supported on both ends and restrained from rotation.
  3. Over hanging - a simple beam extending beyond its support on one end.
  4. Double overhanging - a simple beam with both ends extending beyond its supports on both ends.
  5. Continuous - a beam extending over more than two supports.
  6. Cantilever - a projecting beam fixed only at one end.
  7. Trussed - a beam strengthened by adding a cable or rod to form a truss.

**Table 3 Design of rectangular beam & sub-beam section for moment (simply supported beam)**

| | | | |
|---|---|---|---|
|  | $\theta_1 = \theta_2 = \frac{Pl^2}{16EI}$ | $y = \frac{Px}{12EI}\left(\frac{3l^2}{4} - x^2\right)$ for $0 < x < \frac{l}{2}$ | $\delta_{max} = \frac{Pl^3}{48EI}$ |
| 7. Beam Simply Supported at Ends – Concentrated load $P$ at any point | | | |
|  | $\theta_1 = \frac{Pb(l^2 - b^2)}{6lEI}$ <br> $\theta_2 = \frac{Pab(2l - b)}{6lEI}$ | $y = \frac{Pbx}{6lEI}\left(l^2 - x^2 - b^2\right)$ for $0 < x < a$ <br> $y = \frac{Pb}{6lEI}\left[\frac{l}{b}(x-a)^3 + \left(l^2 - b^2\right)x - x^3\right]$ <br> for $a < x < l$ | $\delta_{max} = \frac{Pb\left(l^2 - b^2\right)^{3/2}}{9\sqrt{3}lEI}$ at $x = \sqrt{\left(l^2 - b^2\right)/3}$ <br> $\delta = \frac{Pb}{48EI}\left(3l^2 - 4b^2\right)$ at the center, if $a > b$ |
| 8. Beam Simply Supported at Ends – Uniformly distributed load $\omega$ (N/m) | | | |
|  | $\theta_1 = \theta_2 = \frac{\omega l^3}{24EI}$ | $y = \frac{\omega x}{24EI}\left(l^3 - 2lx^2 + x^3\right)$ | $\delta_{max} = \frac{5\omega l^4}{384EI}$ |
| 9. Beam Simply Supported at Ends – Couple moment $M$ at the right end | | | |
|  | $\theta_1 = \frac{Ml}{6EI}$ <br> $\theta_2 = \frac{Ml}{3EI}$ | $y = \frac{Mlx}{6EI}\left(1 - \frac{x^2}{l^2}\right)$ | $\delta_{max} = \frac{Ml^2}{9\sqrt{3}\,EI}$ at $x = \frac{l}{\sqrt{3}}$ <br> $\delta = \frac{Ml^2}{16EI}$ at the center |
| 10. Beam Simply Supported at Ends – Uniformly varying load: Maximum intensity $\omega_o$ (N/m) | | | |
|  | $\theta_1 = \frac{7\omega_o l^3}{360EI}$ <br> $\theta_2 = \frac{\omega_o l^3}{45EI}$ | $y = \frac{\omega_o x}{360lEI}\left(7l^4 - 10l^2x^2 + 3x^4\right)$ | $\delta_{max} = 0.00652\frac{\omega_o l^4}{EI}$ at $x = 0.519l$ <br> $\delta = 0.00651\frac{\omega_o l^4}{EI}$ at the center |

### 2.3.1 Steps to Design Rectangular beam & sub beams:

Step 1: Determine the minimum required depth of beam & sub beam from table 4.1 ACI 318 code then compute the width.

Step 2: Determine the ultimate load that effect on the beam & sub beam.

Step 3: Determine the total factored static moment.

Step 4: Determine the required area of steel for flexural design.

Step 5: check the ductility.

Step 6: Determine the required area of stirrups for the shear.

### 2.3.2 Materials properties:

**Table 4 Properties**

| | | |
|---|---|---|
| concrete | fc´ | 30 MPa |
| steel | Fy | 420 MPa |

### 2.3.3 Load calculation for sub beam:

#### a) Load of Sub beam for 1st floor (mezzanine floor):

| Sub Beam of floor | | | | | | | |
|---|---|---|---|---|---|---|---|
| Loading | | | | | | | |
| Loading on Floor = | | | Wu × Spacing = | | | Total | |
| | | 16.2035 | kN/m2 | × | 3 m | = 48.6105 | kN/m |

#### b) Load of Sub beam for Roof:

| Sub Beam of Roof | | | | | | |
|---|---|---|---|---|---|---|
| Loading | | | | | | |
| Loading on Roof = | | | Wu × Spacing = | | Total | |
| | | 11.3575 | kN/m2 | 3 m | 34.0725 | kN/m |

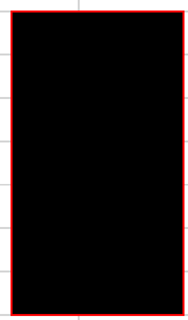
### 2.3.4 Design of sub beams:

a) **Design of Sub beam for 1st floor (mezzanine floor):**

**TABLE 2.8** Minimum Thickness of Nonprestressed Beams or One-Way Slabs Unless Deflections Are Computed[1,2]

| | Minimum Thickness, *h* | | | |
|---|---|---|---|---|
| | Simply supported | One end continuous | Both ends continuous | Cantilever |
| Member | Members not supporting or attached to partitions or other construction likely to be damaged by large deflections | | | |
| Solid one-way slabs | $\ell/20$ | $\ell/24$ | $\ell/28$ | $\ell/10$ |
| Beams or ribbed one-way slabs | $\ell/16$ | $\ell/18.5$ | $\ell/21$ | $\ell/8$ |

| Beam Weight | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| | | | | | | | | |
| Length = | | 6 m | | | | | | |
| | | | | | | | | |
| 1- Find Depth of Beam (h) = | | | L/16 = | 7.38 in | ≈ 8 in | = 203 mm | | |
| | | | | | | | | |
| 2- Find Widt of Beam (b) = | | | h/2 = | 3.69 in | ≈ 4 in | = 102 mm | | |
| | | | | | | | | |
| 3- Self weight of Beam (Bw) = | | | b × h × Density = | | 8 in | 4 in | 150 lb/ft^3 | |
| | | | | → | 0.667 ft × | 0.333 ft × | 150 lb/ft^3 = | 33.33 lb/ft |
| convert | 33.33 lb/ft | → ≈ | 0.04518 kN/m | | | | | |
| $W_{ub}$ = | 0.04518 × | 1.2 = | 0.054216 kN/m | | | | | |
| | | | | | | | | |
| 4- reaction force acting on beam: | | | | | | | | |
| | $\frac{W*L}{2}+\frac{P}{2}$ | | | | h = | 203 mm | | |
| | | | | | | | | |
| R1 = | 24.467898 kN | | | | | | | |
| R2 = | 24.467898 kN | | | | | | | |
| | | | | | | b = | 102 mm | |
| 5- moment: | | | | | | | | |
| $\frac{WL^2}{8}+\frac{PL}{4}$ | | → | 0.054216 × | 36 m | + 6 m | = 73.159722 | kN/m | |
| | | | | 8 | 4 | | | |
| | | | | | | | | |
| moment formula from steel book from moments table 3-23 | | | | | | | | |



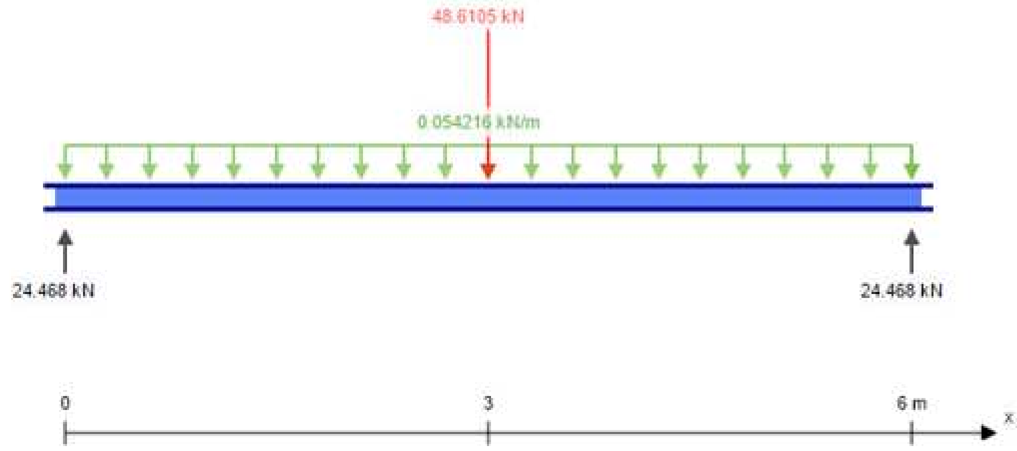



**Figure 6 Force acting on sub-beam & reaction force**

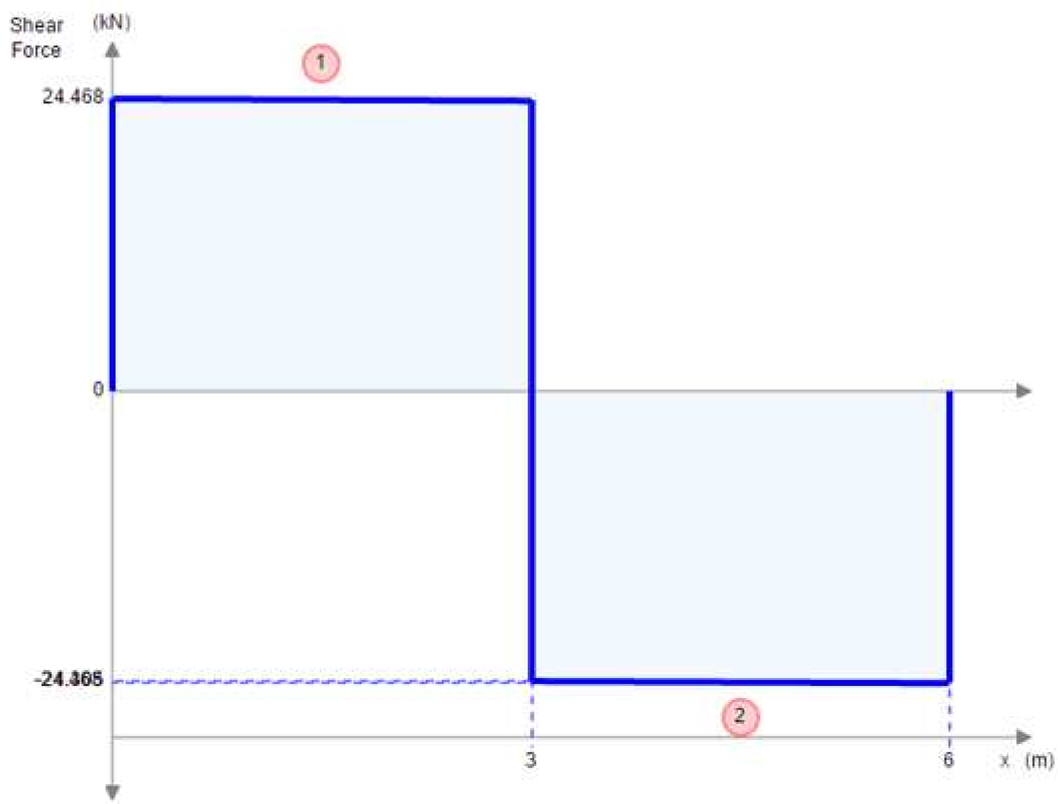



**Figure 7 Shear force**

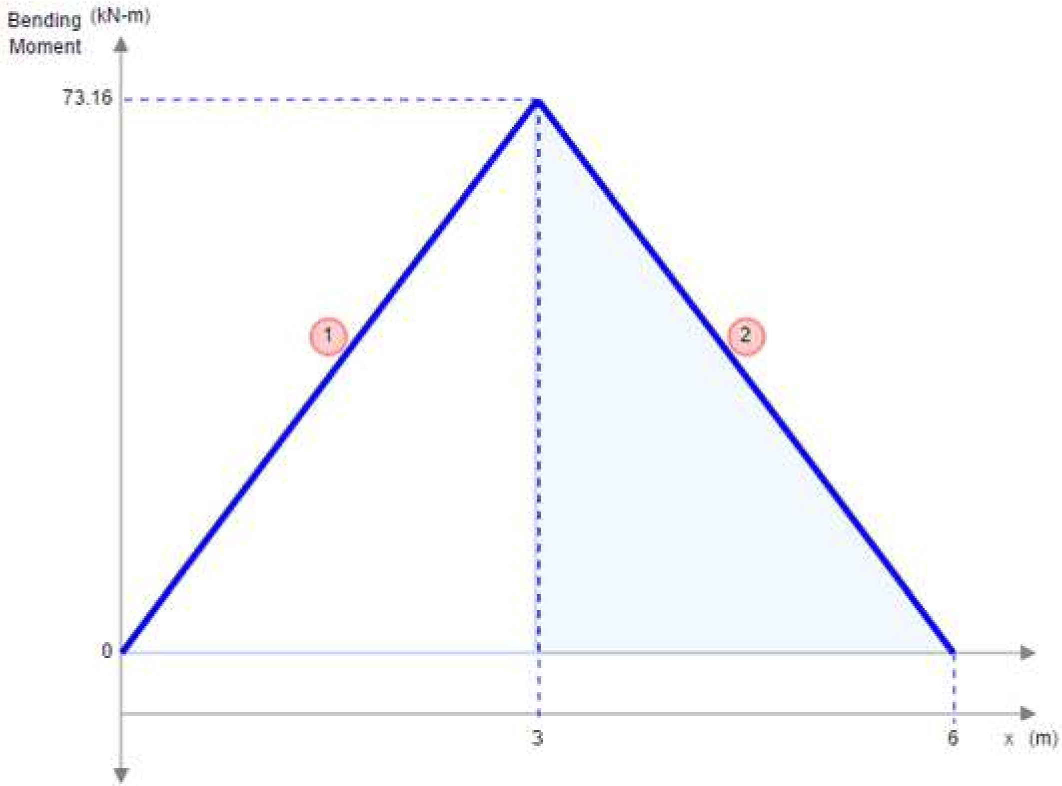



**Figure 8 Bending moment**

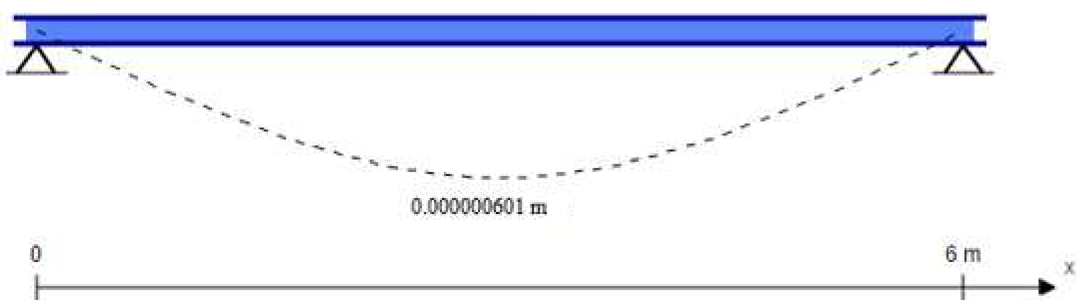



**Figure 9 Deflection**

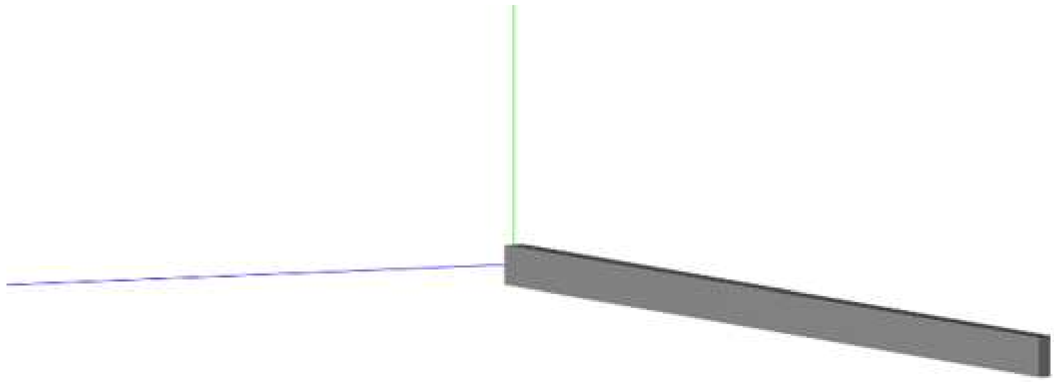


**Figure 10 (3D) renderer and colored results**

| Area of Steel | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **La = 0.9 x H** | | H | | Total | | | | |
| **La=** | 0.9 × | 8 | = 7.2 | in | | | | |
| | | | | | | | | |
| | $A_s = \frac{Mu}{0.9*fy*la}$ | | | Mu = | 73.159722 | kN/m → ≈ | 132.9693239 | kip-in |
| | | | | | | | | |
| $A_s$= | | 132.9693239 | | → | Total | | | |
| | 0.9 × | 60 × | 7.2 | | 0.34199929 $in^2$ | | | |
| | | | | | | | | |
| $A_s$ min = | 200/Fy | | | Total | | | | |
| | 200 ÷ | 60000 | = 0.0033 | $in^2$ | | | | |
| | | | | | | | | |
| $A_s$ max = | 0.75 x $A_{sb}$ | | | | | | | |
| $A_{sb}$= $N_{cb}$/Fy | | | | | | | | |
| $N_{cb}$= 0.85*f'c*$a_b$*b | | | | | | | | |
| $\frac{\textbf{Cb}}{\textbf{0.003}} = \frac{\textbf{La} - \textbf{Cb}}{\textbf{0.00207}}$ | | | | | | | | |
| from the formula above we get Cb= 5.4 in | | | | | | | | |
| a= β*Cb | → β= 0.85 for f'c ≤ 4000 psi | | | | | | | |
| a= | 0.85 × | 5.4 = | 4.59 | in | | | | |
| b = | 1 ft | → | 12 in | ( assuming ) | | | | |
| $N_{cb}$= | 0.85 × | 4000 × | 4.59 × | 12 = | 187272 | lb/$in^2$ → | 187.272 | KIb |
| $A_{sb}$= | 187.272 × | 60 = | 3.1212 | $in^2$ | | | | |
| $A_s$ max = | 0.75 × | 3.1212 = | 2.3409 | $in^2$ | | | | |
| | | | | | | | | |
| we will use As as the area of steel bars = | | | 0.34199929 | $in^2$ | | | | |

Use the table to Find the Number of Bars and Diamater of bars

Table2.9 Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

Take 2#4 Ø → 2 No. 4 bars

## Shear

$$V_v = R - W_{ub} * (h')$$
$$h' = 2 * b$$

$V_v$ = 24.468 − 0.054 * ( 6 m − 5.8 m) = 24.457 kN

$$V_v = \varnothing V_c + V_s$$
$$V_c = 2\ \varnothing \sqrt{f'c}\ b_w\ d$$

$V_c = 2 * 0.85 * \sqrt{4351.131} * 4 * 8 = 3588.3928$ lb

3588.3928 lb → 1628.127414 kg → 15955.64866 N → 15.9556 kN

Vs → $\dfrac{24.457 \quad 15.9556}{0.850}$ = 10.00165428 kN

from Vs calculate stirrups :

$$S = \frac{Av * fy * d}{Vs}$$

$$S = \frac{0.220 \quad 4000 \quad 7.20}{10.02} = 632.335 \text{ in}$$

according to ACI the spacing of shear reinforcment should not be longer of the smaller of the following :

| | | | | | | |
|---|---|---|---|---|---|---|
| ❶ | d/2 | 8 in | × | 2 | = 4 in → = | 102mm |
| ❷ | $\dfrac{A_v f y}{50 b w}$ | $\dfrac{0.220 \times 60}{50 \times 8 \text{ in}}$ | | | = 0.033 in | |

## Find the actual La

| Prefixes | | |
|---|---|---|
| h | = | 8 in |
| cover | = | 1.50 in |
| $d_b$ | = | 1 |

$$\text{Actual d} = \text{h} - \text{Cover} - \text{stirrup} - \frac{d_b}{2}$$

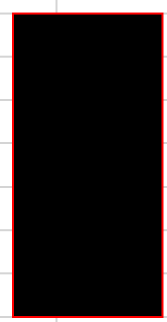
| La = 0.9 x H | | H | Total | |
|---|---|---|---|---|
| La= | 0.9 | × 8 in | = 7.2 | in |

$$\text{Actual d} = 8 - 1.50 - 0.033 - \frac{1}{2} = 6 \text{ in}$$

| Deflection for Sub-beam Floor | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| $\Delta_{max} = \frac{5*\omega*L^4}{384*E*I_x} + \frac{P*L^3}{48*E*I_x}$ | | 5 / 384 | × 0.000054216 / × 25,000 | × 1.296E+15 / × 71050704.3 | + | 0.034073 / 48 | × 2.16E+11 / × 25,000 | × 71050704.3 | |
| | | $\Delta_{max} =$ | 0.000515066 | + | 8.63206E-05 | | | | |
| | | $\Delta_{max} =$ | 0.000601387 | mm | $\rightarrow$ = | 6.01387E-07 | m | | |
| $\Delta_{actual} = \frac{L}{240}$ | | 6 / 240 | | | | Prefixes | | | |
| | | | | | | E= | 25,000 | kN/m | |
| | | | | | | $I_{x\ sub\text{-}beam}$= | 71050704.3 | $mm^4$ | |
| $\Delta_{actual} =$ | 0.025 | mm | $\rightarrow$ = | 0.000025 | m | $I_{x\ girder}$= | 14047810614 | $mm^4$ | |
| $\Delta_{actual} > \Delta_{max}$ | 0.000025 | > | 6.01387E-07 | m | | | | | |

## b) Design of Sub beam for Roof:

| Beam Weight | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Length = | 6 m | | | | | | | |
| 1- Find Depth of Beam (h) = | | L/16 = | 7.38 in | ≈ | 8 in | = 203 mm | | |
| 2- Find Widt of Beam (b) = | | h/2 = | 3.69 in | ≈ | 4 in | = 102 mm | | |
| 3- Self weight of Beam (Bw) = | | b × h × Density = | | 8 in | 4 in | 150 lb/ft^3 | | |
| | | | $\rightarrow$ | 0.667 ft | × 0.333 ft | × 150 lb/ft^3 | = | 33.33 lb/ft |
| convert | 33.33 lb/ft | $\rightarrow$ ≈ | 0.04518 kN/m | | | | | |
| $W_{ub}$ = | 0.04518 | × 1.2 = | 0.054216 kN/m | | | | | |
| 4- reaction force acting on beam: | | | | | | | | |
| | $\frac{W*L}{2} + \frac{P}{2}$ | | | | | h = 203 mm | | |
| R1 = | 17.198898 kN | | | | | | | |
| R2 = | 17.198898 kN | | | | | | | |
| | | | | | | | b = 102 mm | |
| 5- moment: | | | | | | | | |
| $\frac{WL^2}{8} + \frac{PL}{4}$ | | $\rightarrow$ | $\frac{0.054216 \times 36\ m}{8}$ | + | $\frac{6\ m}{4}$ | = 51.352722 kN/m | | |
| moment formula from steel book from moments table 3-23 | | | | | | | | |



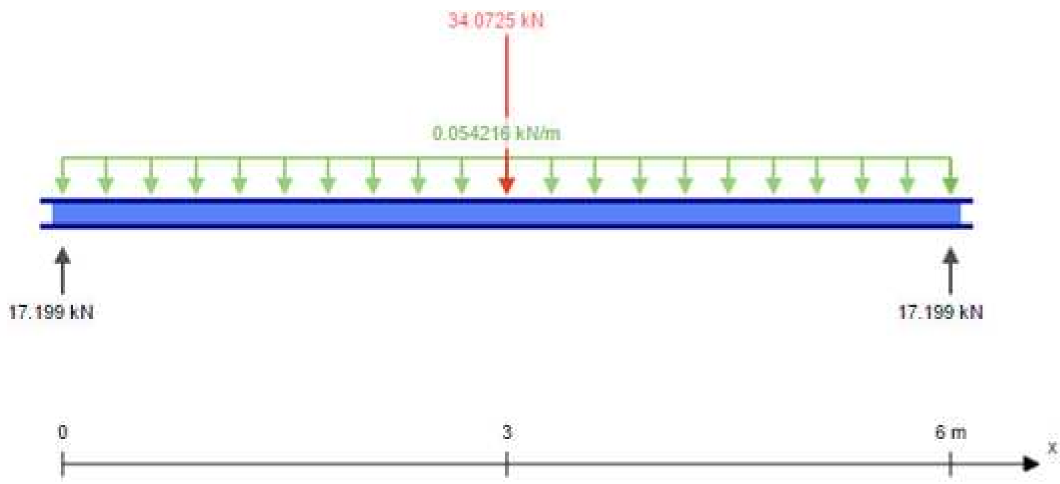



**Figure 11 Force acting on sub-beam & reaction force.**

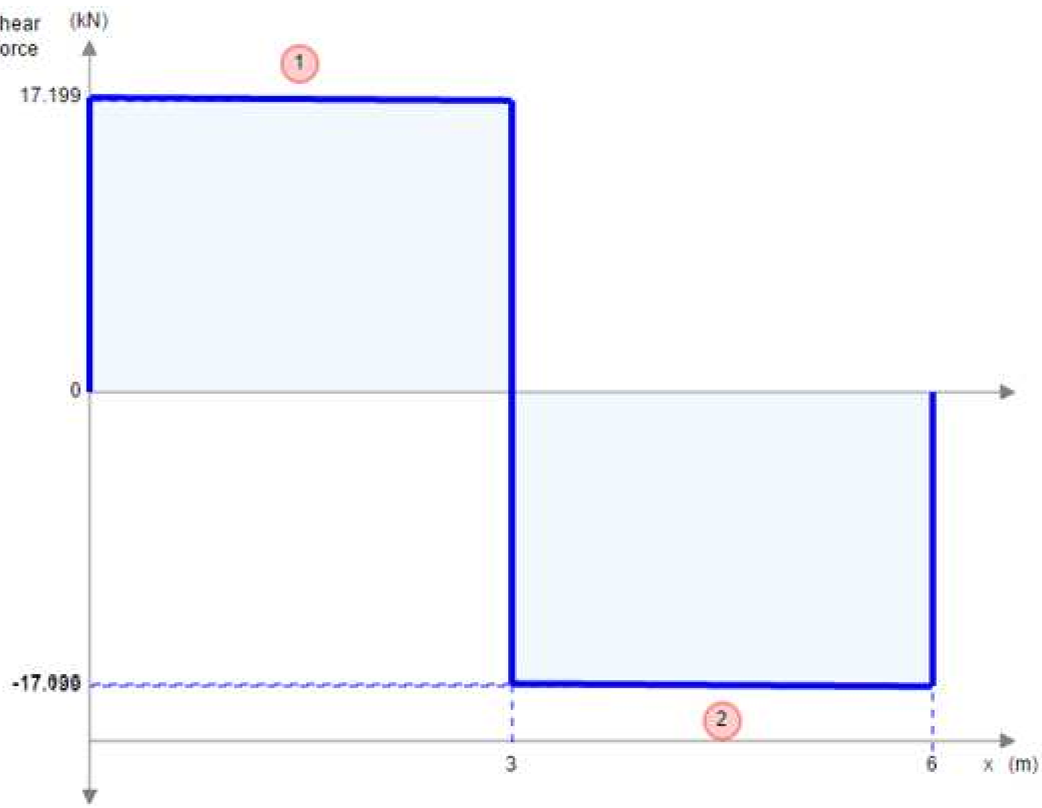



**Figure 12 Shear force**

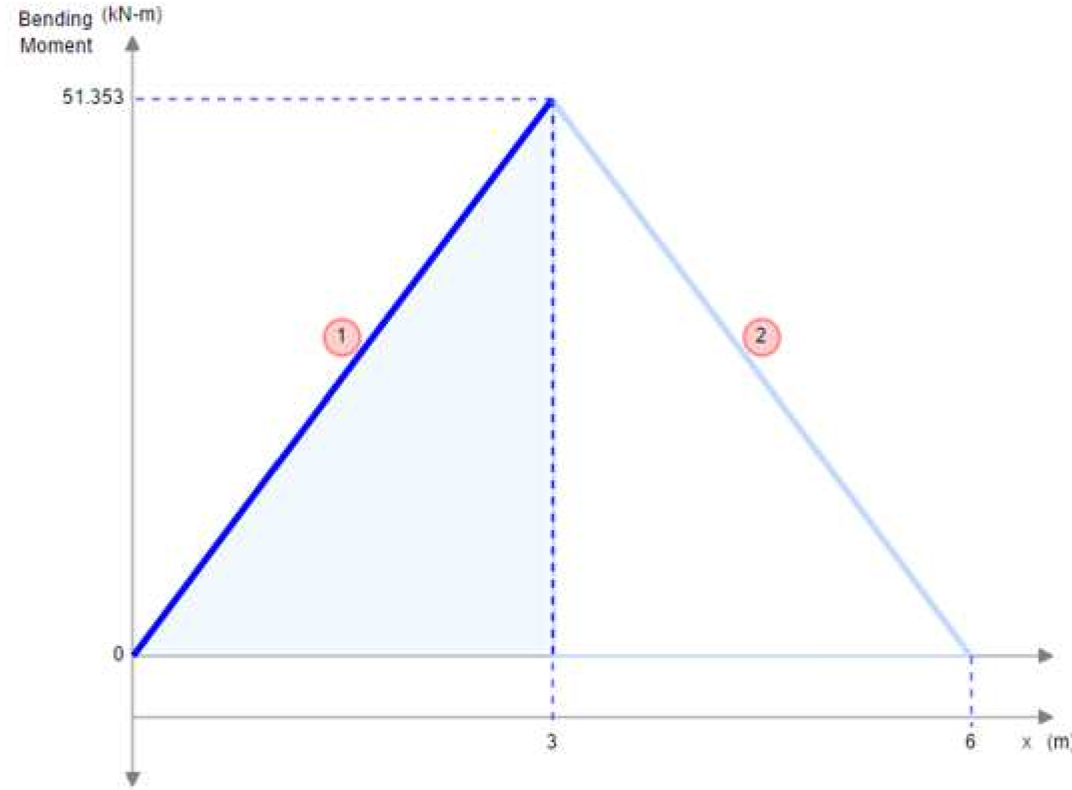



**Figure 13 Bending moment**

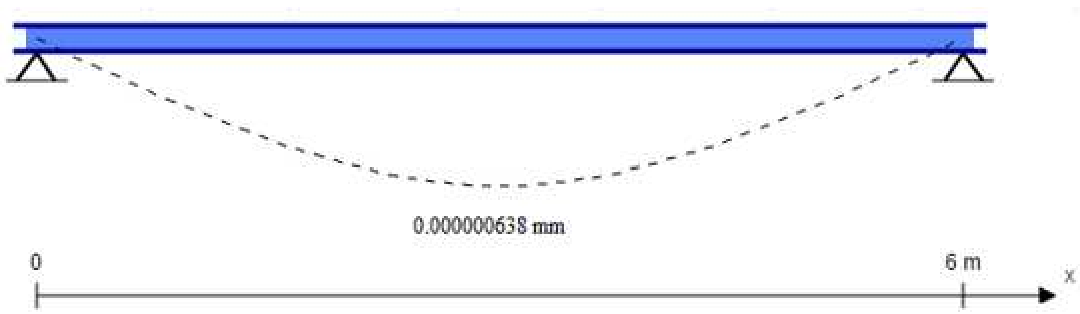



**Figure 14 Deflection**

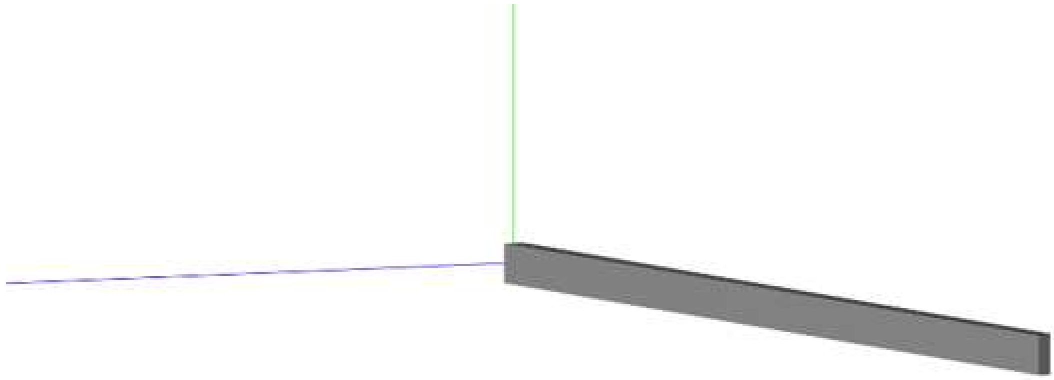


**Figure 15 (3D) renderer and colored results**

| **Area of Steel** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **La = 0.9 x H** | | H | Total | | | | | |
| **La=** | 0.9 × | 8 | = 7.2 | in | | | | |
| | | | | | | | | |
| $A_s = \frac{Mu}{0.9 * fy * la}$ | | | | Mu = | 51.352722 | kN/m → ≈ | 93.33464558 | kip-in |
| | | | | | | | | |
| $A_s$= | 93.33464558 | | | → | Total | | | |
| | 0.9 × | 60 × | 7.2 | | 0.240058245 | $in^2$ | | |
| | | | | | | | | |
| $A_s$ min = | 200/Fy | | Total | | | | | |
| | 200 | ÷ 60000 | = 0.0033 | $in^2$ | | | | |
| | | | | | | | | |
| $A_s$ max = | 0.75 x $A_{sb}$ | | | | | | | |
| $A_{sb}$= $N_{cb}$/Fy | | | | | | | | |
| $N_{cb}$= 0.85*f'c*$a_b$*b | | | | | | | | |
| $\frac{Cb}{0.003} = \frac{La - Cb}{0.00207}$ | | | | | | | | |
| from the formula above we get Cb= 5.4 in | | | | | | | | |
| a= β*Cb | → β= 0.85 for f'c ≤ 4000 psi | | | | | | | |
| a= | 0.85 × | 5.4 = | 4.59 | in | | | | |
| b = | 1 ft | → | 12 in | ( assuming ) | | | | |
| $N_{cb}$= | 0.85 × | 4000 × | 4.59 × | 12 = | 187272 | Ib/$in^2$ → | 187.272 | KIb |
| $A_{sb}$= | 187.272 × | 60 = | 3.1212 | $in^2$ | | | | |
| $A_s$ max = | 0.75 × | 3.1212 = | 2.3409 | $in^2$ | | | | |
| | | | | | | | | |
| we will use As as the area of steel bars = 0.240058245 $in^2$ | | | | | | | | |

**Use the table to Find the Number of Bars and Diamater of bars**

Table2.10Areas of Multiple of Reinforcing Bars (in²)

| Number of bars | #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
|---|---|---|---|---|---|---|---|---|---|
| | Bar number | | | | | | | | |
| 1 | 0.11 | 0.20 | $\boxed{0.31}$ | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

Take1#5 Ø → 1 No. 5 bars

**Shear**

$$Vv = R - Wub * (h')$$

$$h' = 2 * b$$

$V_v$ = 17.199 − 0.054 *( 6 m − 5.8 m) = 17.188 kN

$$Vv = \text{Ø } Vc + Vs$$

$$Vc = 2 \text{ Ø} \sqrt{f'c}\, bw\, d$$

$V_c$ = 2 * 0.85 * $\sqrt{4351.131 *}$ 4 * 8= 3588.3928 psi

3588.3928 lb → 1628.127414 kg → 15955.64866 N → 15.9556 kN

Vs → $\dfrac{17.188 \quad 15.9556}{0.850}$ = 1.449889578 kN

from Vs calculate stirrups :

$$Vs = \frac{Av * fy * d}{S}$$

S = $\dfrac{0.220 \quad 5.4 \quad 7.20}{1.449889578}$ = 5.899 in

according to ACI the spacing of shear reinforcment should not be longer of the smaller of the following :

| | | | | |
|---|---|---|---|---|
| ❶ | d/2 | 8 in × 2 | = 4 in → = | 102mm |
| ❷ | $\dfrac{A_v f y}{50 bw}$ | $\dfrac{0.220 \times 60}{50 \times 8 \text{ in}}$ | = 0.033 in | |

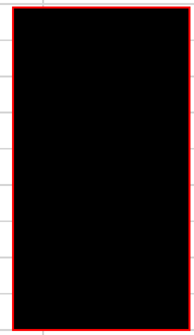
| Deflection for Sub-beam Roof | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| $\Delta_{max} = \frac{5*\omega*L^4}{384*E*I_x} + \frac{P*L^3}{48*E*I_x}$ | | | 5 × | 0.000054216 × | 1.296E+15 | + | 0.0486105 × | 2.16E+11 | |
| | | | 384 × | 25,000 × | 71050704.3 | | 48 × | 25,000 × | 71050704.3 |
| | | | $\Delta_{max} =$ | 0.000515066 | + | 0.00012315 | | | |
| | | | $\Delta_{max} =$ | 0.000638216 | mm | → | 6.38216E-07 | m | |
| $\Delta_{actual} = \frac{L}{240}$ | | | 6 | | | | Prefixes | | |
| | | | 240 | | | | E= | 25,000 | kN/m |
| | | | | | | | $I_{x\ sub\text{--}beam}$= | 71050704.3 | $mm^4$ |
| $\Delta_{actual} =$ | | 0.025 | mm | → = | 0.000025 m | | $I_{x\ girder}$= | 14047810614 | $mm^4$ |
| $\Delta_{actual} > \Delta_{max}$ | | 0.025 | > | 6.38216E-07 m | | | | | |

### 2.3.5 Calculations of loads for beam:

| critical beam | | | | | | | |
|---|---|---|---|---|---|---|---|
| Loading | | | | | | | |
| Loading on Floor = | | | Wu × Spacing = | | | Total | |
| | | | 16.2035 | $kN/m^2$ × | 6 m = | 97.221 | kN/m |

### 2.3.6 Design of the beam:

| Beam Weight | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Length = | | L | 12 m | 472.32 in | | | | |
| 1- Find Depth of Beam (h) = | | | L/16 = | 29.52 in | ≈ 30 in = | 762 mm | | |
| 2- Find Widt of Beam (b) = | | | h/2 = | 14.76 in | ≈ 15 in = | 381 mm | | |
| 3- Self weight of Beam (Bw) = | | | b × h × Density = | | 30 in | 15 in | 150 lb/ft^3 | |
| | | | | → | 2.500 ft × | 1.250 ft × | 150 lb/ft^3 = | 468.75 lb/ft |
| convert | 468.75 lb/ft | → ≈ | 0.6355 | kN/m | | | | |
| $W_{ub}$ = | 0.6355 | 1.2 | 0.7626 | kN/m | | | | |
| point load from reaction acting on sub beam of floor R = | | | | | | 24.467898 | kN | |
| P1= | 24.467898 | × 2 | = 48.935796 | kN | | | | |
| P2= | 24.467898 | × 2 | = 48.935796 | kN | | | | |
| P3= | 24.467898 | × 2 | = 48.935796 | kN | | | | |
| | | | | | | h = | 762 mm | |
| 4- reaction force acting on beam: | | | | | | | | |
| $R_1,R_2 \rightarrow$ | $\frac{3P}{2}+\frac{WL}{2}$ | | | | | | b = | 381 mm |
| R1 → | $\frac{3 \times 48.9358}{2}$ | | $+ \frac{0.7626 \times 12\ m}{2}$ | | → = | 77.979294 | kN | |
| R2 → | $\frac{3 \times 48.9358}{2}$ | | $+ \frac{0.7626 \times 12\ m}{2}$ | | → = | 77.979294 | kN | |
| 5- moment: | | | | | | | | |
| $\frac{WL^2}{8}+\frac{PL}{4}+Pa$ | | → | $\frac{0.7626 \times 144\ m}{8}$ | | $+ \frac{48.9358 \times 12\ m}{4}$ | | + 48.9358 × | 3 |
| | M = | → | 307.34158 | kN/m | | | | |
| Moment formula from steel book from moments table 3-23 | | | | | | | | |



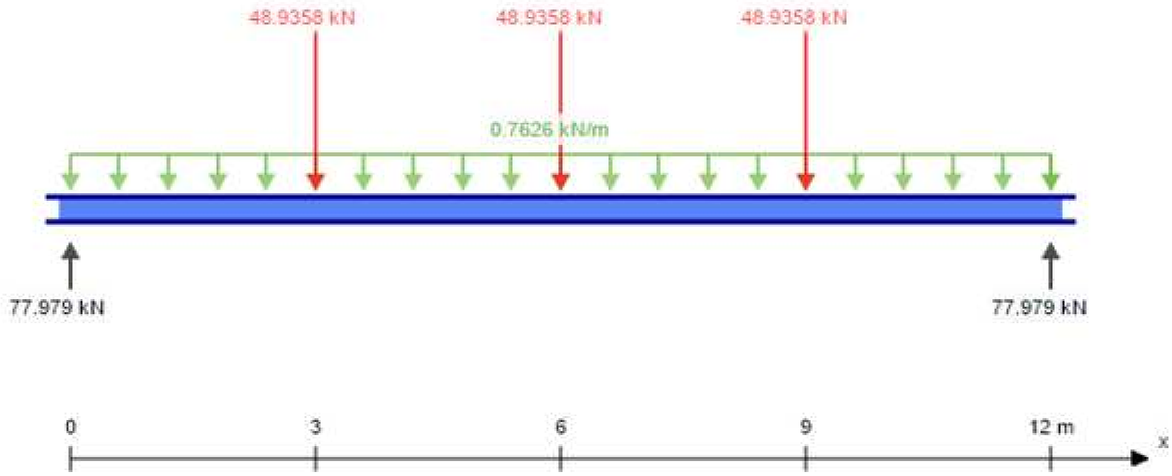



**Figure 16 Force acting on sub-beam & reaction force**

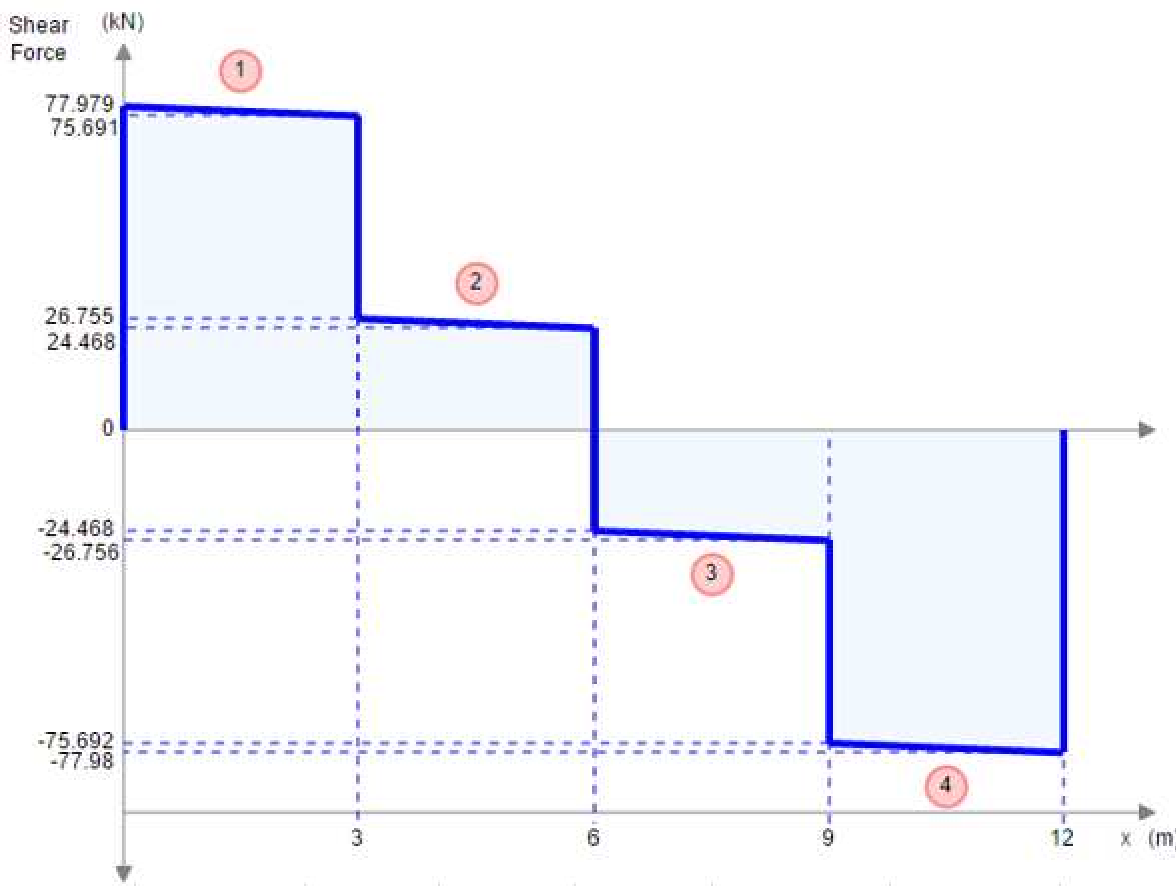



**Figure 17 Shear force**

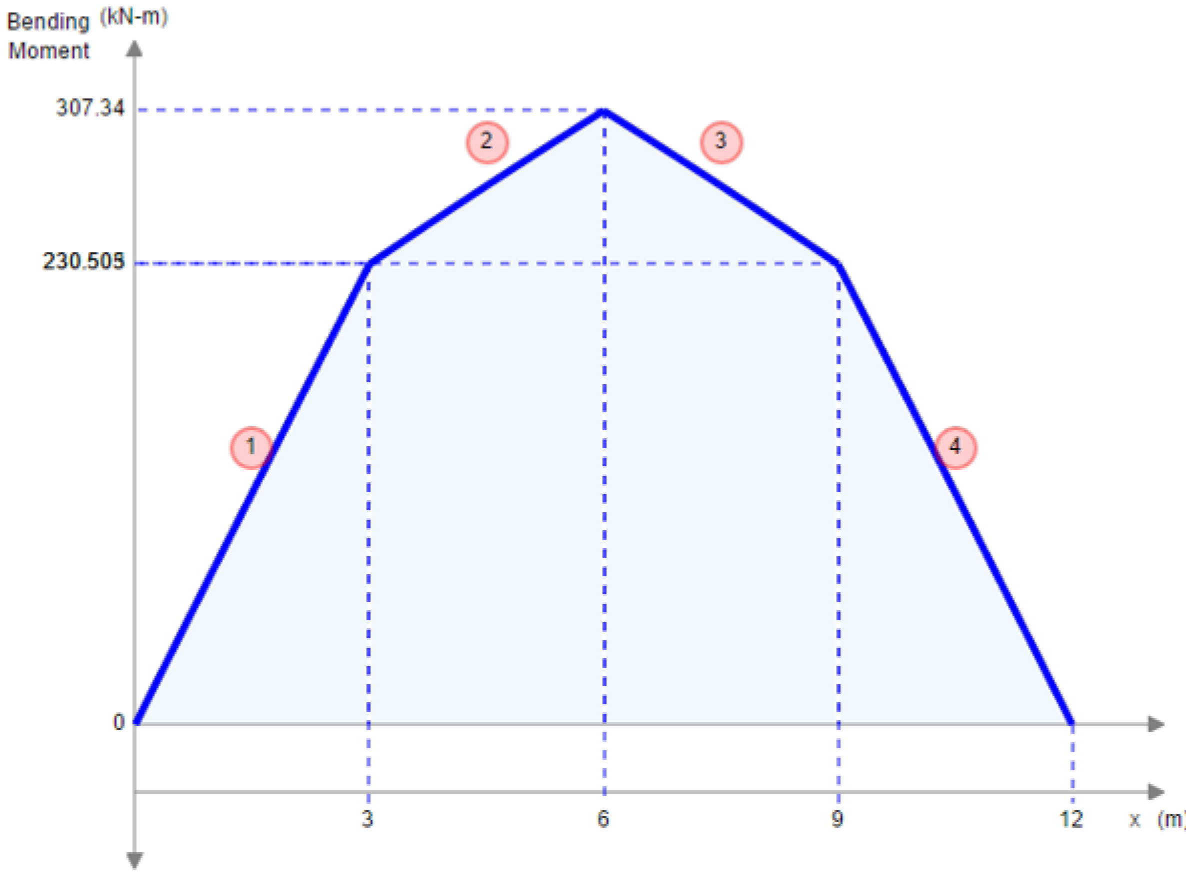



**Figure 18 Bending moment**

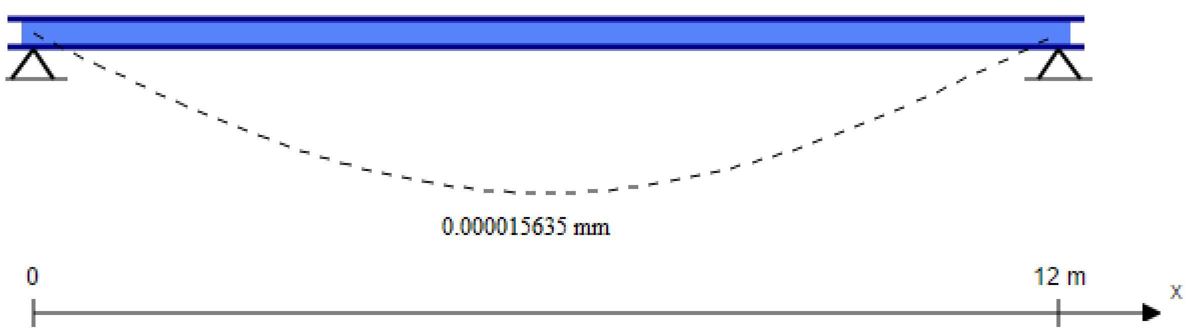



**Figure 19 Deflection**

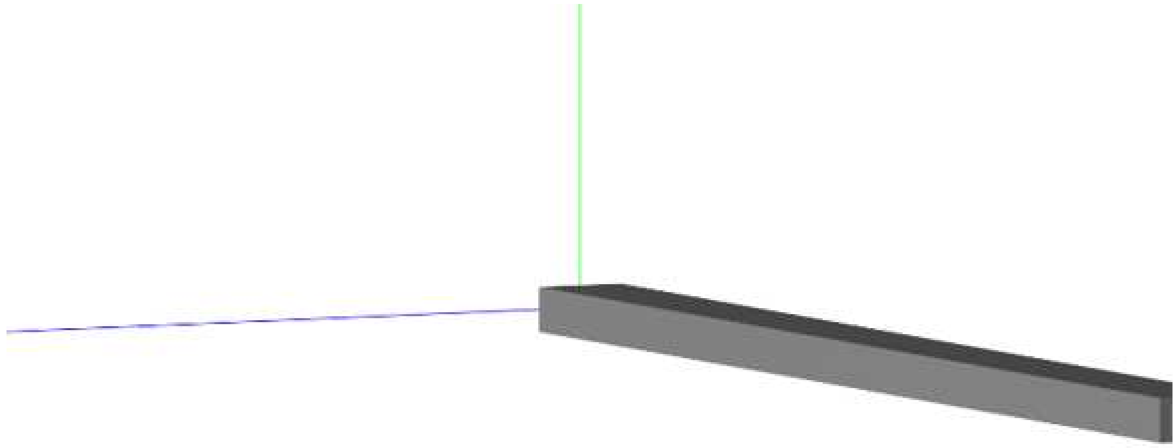


**Figure 20 (3D) renderer and colored results**

| **Area of Steel** | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| **La = 0.9 x H** | | H | Total | | | | | |
| **La=** | 0.9 × | 30 in | = 27 | in | | | | |
| | | | | | | | | |
| $A_s = \frac{Mu}{0.9*fy*la}$ | | | | Mu = | 307.341576 kN/m | → ≈ | 558.5997383 kip-in | |
| | | | | | | | | |
| $A_s=$ | | 558.5997 | | | Total | | | |
| | 0.9 × | 60 | × 27 | → | 0.383127393 $in^2$ | | | |
| | | | | | | | | |
| $A_s$ min = | 200/Fy | | Total | | | | | |
| | 200 | ÷ 60000 | = 0.0033 $in^2$ | | | | | |
| | | | | | | | | |
| $A_s$ max = | 0.75 x $A_{sb}$ | | | | | | | |
| $A_{sb}= N_{cb}/Fy$ | | | | | | | | |
| $N_{cb}= 0.85*f'c*a_b*b$ | | | | | | | | |
| $\frac{Cb}{0.003} = \frac{La - Cb}{0.00207}$ | | | | | | | | |
| from the formula above we get Cb= 5.4 in | | | | | | | | |
| a= β*Cb | → β= 0.85 for f'c ≤ 4000 psi | | | | | | | |
| a= | 0.85 × | 5.4 | = 4.59 | in | | | | |
| b = | 1 ft | → | 12 in | ( assuming ) | | | | |
| $N_{cb}$= | 0.85 × | 4000 | × 4.59 × | 12 | = 187272 Ib/$in^2$ | → | 187.272 | KIb |
| $A_{sb}$= | 187.272 | × 60 | = 3.1212 | $in^2$ | | | | |
| $A_s$ max = | 0.75 × | 3.1212 | = 2.3409 | $in^2$ | | | | |
| | | | | | | | | |
| we will use As as the area of steel bar | | | 0.3831274 | $in^2$ | | | | |

| Use the table to Find the Number of Bars and Diamater of bars |
|---|

Table 2.11 Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | $\boxed{0.40}$ | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

**Take 2#4 Ø** ⟶ **2 No. 4 bars**

| Find the actual La | | | | | | | |
|---|---|---|---|---|---|---|---|
| | | | | | | Prefixes | |
| | Actual d $= h - \text{Cover} - \text{stirrup} - \frac{d_b}{2}$ | | | | | h = | 8 in |
| | | | | | | cover = | 1.50 in |
| | **La = 0.9 x H** | | H | | Total | $d_b$ = | 1 |
| | **La=** | 0.9 | × 30 in | = 27 | in | | |
| | Actual d = | 30 | − 1.50 | − 0.0088 − | $\frac{1}{2}$ = | 28 in | |

## Shear

$$Vv = R - Wub * (h')$$
$$h' = 2 * b$$

$V_v =$ 77.979 − 0.763* ( 6 m − 5.8 m) = 77.827 kN

$$Vv = \varnothing Vc + Vs$$
$$Vc = 2\ \varnothing \sqrt{f'c}\ bw\ d$$

$V_c =$ 2 * 0.85 * $\sqrt{4351.131}$ * 30 * 15 = 50461.7740 lb

50461.7740 lb → 22895.54 kg → 224376.31 N → 224.3763 kN

Vs → $\frac{224.3763 \quad 77.827}{0.850}$ 172.41122 kN

from Vs calculate stirrups :

$$Vs = \frac{Av * fy * d}{S}$$

S = $\frac{0.220 \quad 4000 \quad 27.00}{172.411218}$ = 137.810 in

according to ACI the spacing of shear reinforcment should not be longer of the smaller of the following :

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| ❶ | d/2 | 30 in | × | 2 | = 15 in | → = | 381 mm |
| ❷ | $\frac{A_v f y}{50 bw}$ | $\frac{0.220}{50}$ | × | $\frac{60}{30 \text{ in}}$ | = 0.0088 in | | |

## Deflection for Girder

$$\Delta_{max} = \frac{5*\omega*L^4}{384*E*I_x} + \frac{P*L^3}{48*E*I_x}$$

$\frac{5 \times 0.0007626 \times 2.0736E+16}{384 \times 25000 \times 14047810614}$ + $\frac{48.935796 \times 1.728E+12}{48 \times 25000 \times 14047810614}$

$\Delta_{max} =$ 0.0005863 0.015048796

$\Delta_{max} =$ 0.0156351 mm → 1.56351E-05 m

$$\Delta_{actual} = \frac{L}{240}$$

$\frac{12 \text{ m}}{240}$

$\Delta_{actual} =$ 0.05 mm → = 0.00005 m

$\Delta_{actual} > \Delta_{max}$ 0.05 > 1.564E-05 m

| Prefixes | | |
|---|---|---|
| E= | 25,000 | kN/m |
| $I_{x\ sub-beam}$= | 71050704.3 | $mm^4$ |
| $I_{x\ girder}$= | 14047810614 | $mm^4$ |

## 2.4 Design of columns

Columns are the third important element in structural system which are defined as members that carry load chiefly in compression .usually columns carry bending moment as well about one or both axis of the cross section

### 2.4.1 Classification of columns:

Column are generally classified according to:

a) **The load acting on column:**

1) Axially loaded column: where the load act at the center of the column Section

2) Eccentrically loaded columns: where the load is acting at distance e form the center of column cross section in any direction.

3) Biaxial loaded columns: where the loads are acting at any point on the columns section causing moment about axis of the section

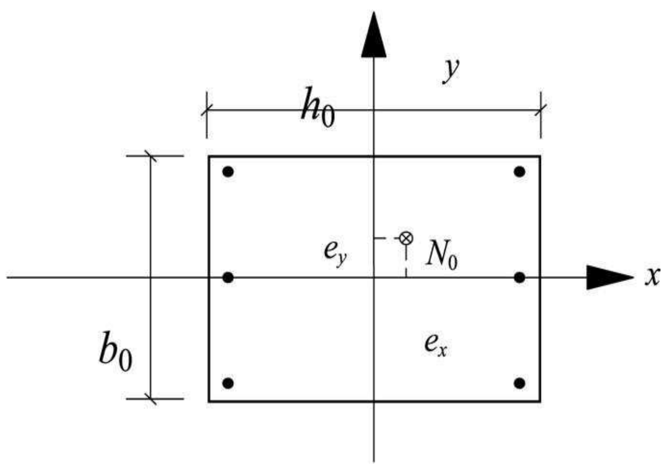



**Figure 21 Cross section for eccentrically loaded column.**

**b) The column length:**

1) Short columns: where the columns failure is due to the crushing of concrete or yielding of the steel bars under the capacity of the columns.
2) Long columns: Where the effect of the buckling and the slenderness ratio must be taken into consideration in the design.

**c) The columns ties:**

1) Members reinforced with longitudinal bars and lateral ties(Rectangular columns)
2) Members reinforced with longitudinal bars and continuous spiral (circular columns)

**d) sample of calculations of rectangular columns:**

- **Steps of design:**
    1) Check the type of the floor in each direction (sway or non-sway)
    2) Determine the load applied and the critical load combination
    3) Check if the column is slender or not.
- **If columns is slender finds its magnified moment, then compare this with the calculated minimum moment:**

    4) Then find out the required area of steel
    5) Calculate the ties or stirrup reinforcement
    6) Draw the detailing of the columns

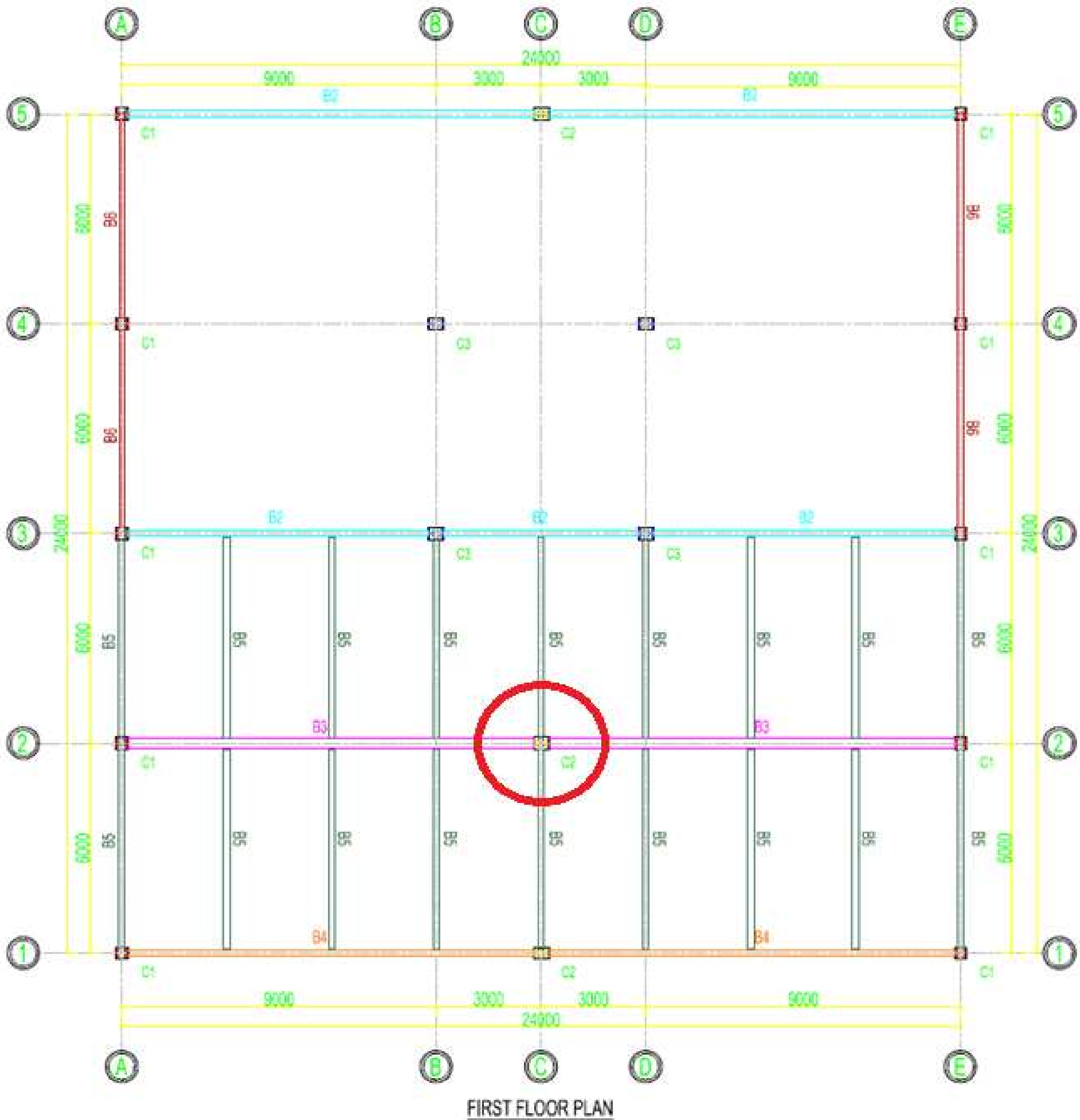



**Figure 22 Column selected for the sample of calculation is 2C Shown in the Figure**

### 2.4.3 Materials properties:

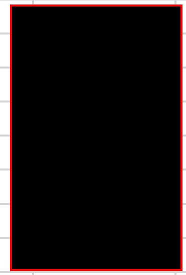
**Table 5 Properties**

| | | |
|---|---|---|
| concrete | fc′ | 30 MPa |
| steel | Fy | 420 MPa |

### 2.4.4 Load Calculation:

| | | | | | | | |
|---|---|---|---|---|---|---|---|
| **General formula for Pu of column** | | | | | | | |
| **Pu = 0.8 ø *o.85 f'c Ag** | | | | | | | |
| **→ for rectangular beam ø = 0.7** | | | | | | | |
| **For Column # 2C in Drawing : ↓** | | | | | | | |
| **1- ROOF :** | | | | | | | |
| loads acting on column are 1/4 of 4 slabs | | | | | | | |
| $W_u$ = | 11.3575 | kN/m² | | | | | |
| for 1/4 of 4 slabs → = | | 18 | m² → | 18 × | 11.3575 | = 204.435 kN | |
| $P_{girder}$ = | 0.7626 × | 9 | = 6.8634 kN | | | | |
| Pu = | 204.435 × | 6.8634 | = 211.2984 kN | | | | |
| **2- FLOOR:** | | | | | | | |
| loads acting on column are 1/4 of 4 slabs | | | | | | | |
| $W_u$ = | 16.2035 | kN/m² | | | | | |
| for 1/4 of 4 slabs → | | 18 | m² → | 18 × | 16.2035 | = 291.663 kN | |
| $P_{girder}$ = | 0.7626 × | 9 | = 6.8634 kN | | | | |
| Pu = | 291.663 + | 6.8634 | = 298.5264 kN | | | | |
| ROOF + FLOOR | | → | 211.2984 + 298.5264 | | = 509.8248 kN | | |
| convert: | 509.8248 | kN →= | 114.6131744 kips | | | | |
| 114.6131744 | = 0.8 | × 0.7 | × 0.85 | × 6 | × $A_g$ | | |
| Ag = | 40.13066331 in² | →= | 25890.69874 mm² | | | | |

| Column Weight | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| Length = | | 12 m | → | 472.44 in | | | | |
| 1- Find Depth of column (h) = | | | L/16 = | 29.53 in | ≈ 30 in | = 762 mm | | |
| 2- Find Widt of column (b) = | | | h/2 = | 14.76 in | ≈ 15 in | = 381 mm | | |
| 3- Self weight of column (Bw) = | | | b × h × Density = | | 30 in | 15 in | 150 lb/ft^3 | |
| | | | | → | 2.500 ft × | 1.250 ft × | 150 lb/ft^3 | 468.75 lb/ft |
| convert | 468.75 lb/ft | → ≈ | 0.04518 kN/m | | | | | |
| Wub = | 0.04518 × | 1.2 = | 0.054216 kN/m | | | | | |
| 4- reaction force acting on column = | | | | | h = | 762 mm | | |
| R = | 281.1951 kN | | | | | | | |
| 5- moment: | | | | | | | | |
| | | | | | | b = | 381 mm | |
| M = | 712.766 kN/m | | | | | | | |



| Area of Steel | | | | | | | |
|---|---|---|---|---|---|---|---|
| $A_g$ = | 40.13066331 $in^2$ | | | | | | |
| to find $A_s$ → | $\rho = \frac{A_s}{bh}$ | → | ρ = | between | 0.01 | & | 0.08 |
| | | | assuming that ρ = | | 0.01 | | |
| 0.01 = | $\frac{A_s}{15\text{ in} \times 30\text{ in}}$ | | As → = | 4.5 | $in^2$ | | |
| As min = | 200/Fy | | Total | | | | |
| | 200 ÷ | 60000 | = | 0.0033 $in^2$ | | | |
| **La = 0.9 x H** | | H | | Total | | | |
| **La=** | 0.9 × | 30 in | = | 27 in | | | |
| we will use As as the area of steel bars = | | | | 4.5 $in^2$ | | | |

**Use the table to Find the Number of Bars and Diamater of bars**

Table2.13 Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|
| | #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | 1.60 | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

**Take 3 # 11 Ø** → **3 No. 11 bars**

**Shear**

$$Vv = R - Wub * (h')$$
$$h' = 2 * b$$

$V_v =$ 281.195 − 0.054 * ( 6 m − 5.8 m ) = 281.184 kN

$$Vv = Ø\,Vc + Vs$$
$$Vc = 2\,Ø\sqrt{f'c}\,bw\,d$$

$V_c = 2 * 0.85 * \sqrt{4351.131} * 30 * 15 =$ 50461.7740 lb

50461.7740 lb → = 22895.54176 kg → = 224376.3093 N → = 224.3763 kN

Vs → $\dfrac{281.1843 \times 224.376}{0.850}$ =66.83287946 kN

from Vs calculate stirrups :

$$Vs = \frac{Av * fy * d}{S}$$

S = $\dfrac{0.220 \times 60 \times 27.00}{66.83287946}$ = 5.333 in

according to ACI the spacing of shear reinforcment should not be longer of the smaller of the following :

❶ d/2   30 in × 2 = 15 in→ = 381 mm

❷ $\dfrac{A_v fy}{50bw}$   $\dfrac{0.220 \times 60}{50 \times 30\text{ in}}$ = 0.0088 in

## 2.5 Design of dome

### 2.5.1 Classification of dome:

A dome is an architectural element that resembles the hollow upper half of a sphere. The precise definition has been a matter of controversy. There are also a wide variety of forms and specialized terms to describe them. A dome can rest upon a rotunda or drum, and can be supported by columns or piers that transition to the dome through squinches or pendentives. A lantern may cover an oculus and may itself have another dome.

Domes have a long architectural lineage that extends back into prehistory and they have been constructed from mud, stone, wood, brick, concrete, metal, glass, and plastic over the centuries. The symbolism associated with domes includes mortuary, celestial, and governmental traditions that have likewise developed over time.

Domes have been found from early Mesopotamia, which may explain the form's spread. They are found in Persian, Hellenistic, Roman, and Chinese architecture in the Ancient world, as well as among a number of contemporary indigenous building traditions. They were popular in Byzantine and medieval Islamic architecture, and there are numerous examples from Western Europe in the middle Ages. The Renaissance style spread from Italy in the early modern period. Advancements in mathematics, materials, and production techniques since that time resulted in new dome types. The domes of the modern world can be found over religious buildings, legislative chambers, sports stadiums, and a variety of functional structures.

**Domes in Islamic architecture:**

When he built the Messenger of Allah bless him and his mosque in Medina, was the roof of the mobile fronds on the palm trunks, the case remained on it among the mosques was not the dome has entered the building of mosques.

The first dome was built in Islam are Dome of the Rock mosque in Jerusalem built by the Umayyad Caliph Abdul Malik bin Marwan year 72 AH. Muslims and was able to be transferred to the customs of the Maghreb and Al-Andalus, and today we find a beautiful example of the city and the valley will (State of the valley) in the east of Algeria domes that make up the foundation element in roofing Hjradtha. And people still call in the Orient and some of Morocco on the room or the Dome of the room

### 2.5.2 Calculations of dome:

| Dome | | | | | |
|---|---|---|---|---|---|
| Given Data of Concrete Dome: | | | | | |
| $H_1$ = | 3.8 | m | | | |
| $H_2$ = | 4 | m | | | |
| $R_1$ = | 2.8 | m | | | |
| $R_2$ = | 3 | m | | | |
| Th = | 0.2 | m | | Prefixes | |
| | | | | ∏ = | 3.14 |
| $V_1 = \pi / 6 * h_1 ( 3 R_1^2 + H_1^2)$ | | | | | |
| $V_1$= | 75.48979 | $m^3$ | | | |
| | | | | | |
| $V_2 = \pi / 6 * h_1 ( 3 R_2^2 + H_2^2)$ | | | | | |
| $V_2$= | 85.51267 | $m^3$ | | | |
| | | | | | |
| Total V = | $V_2 - V_1$ | | | | |
| | 85.51267 − 75.48979 = 10.02288 $m^3$ | | | | |
| | | | | | |
| D=kN/$m^3$ | V=$m^3$ | Total | | | |
| 24.525 × | 10.02288 = | 245.8111 kN | | | |
| | | | | | |
| 1/4 OF DOME | | | | | |
| 245.811132 ÷ | 4 = | 61.45278 kN | | | |

- **Sap2000 design:**

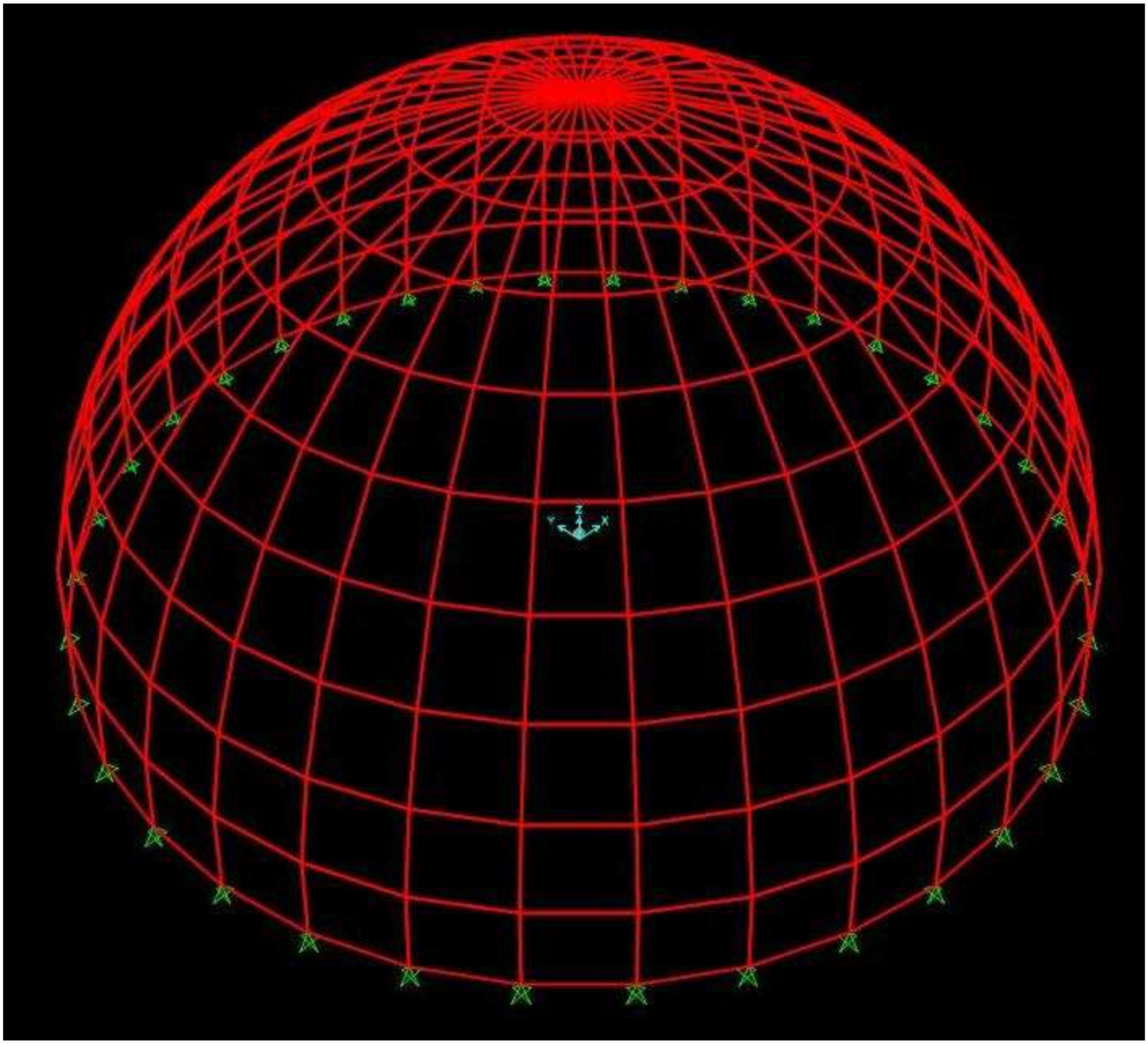


**Figure 23 (3D) Dome structural design.**

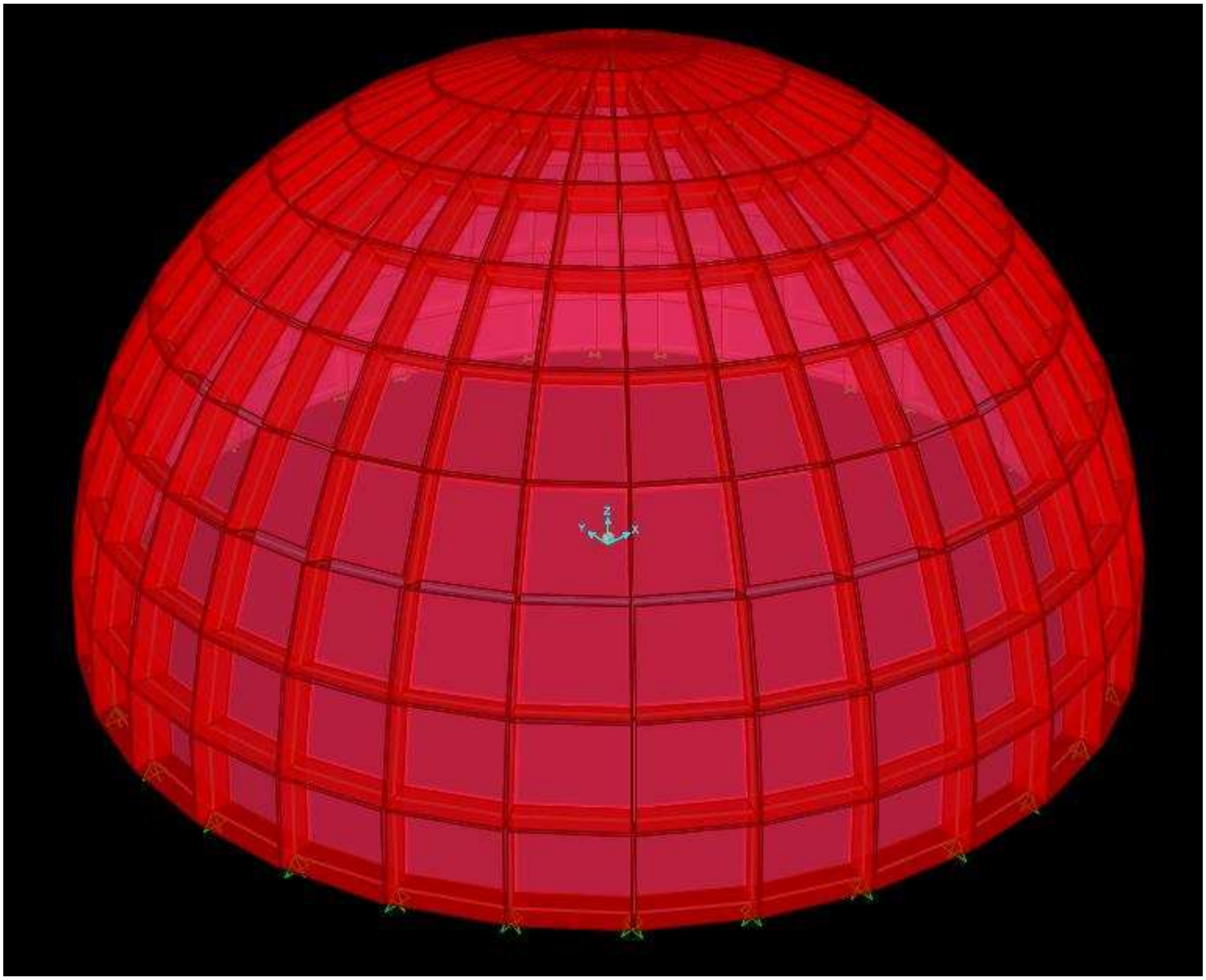


**Figure 24 (3D) dome overview design.**

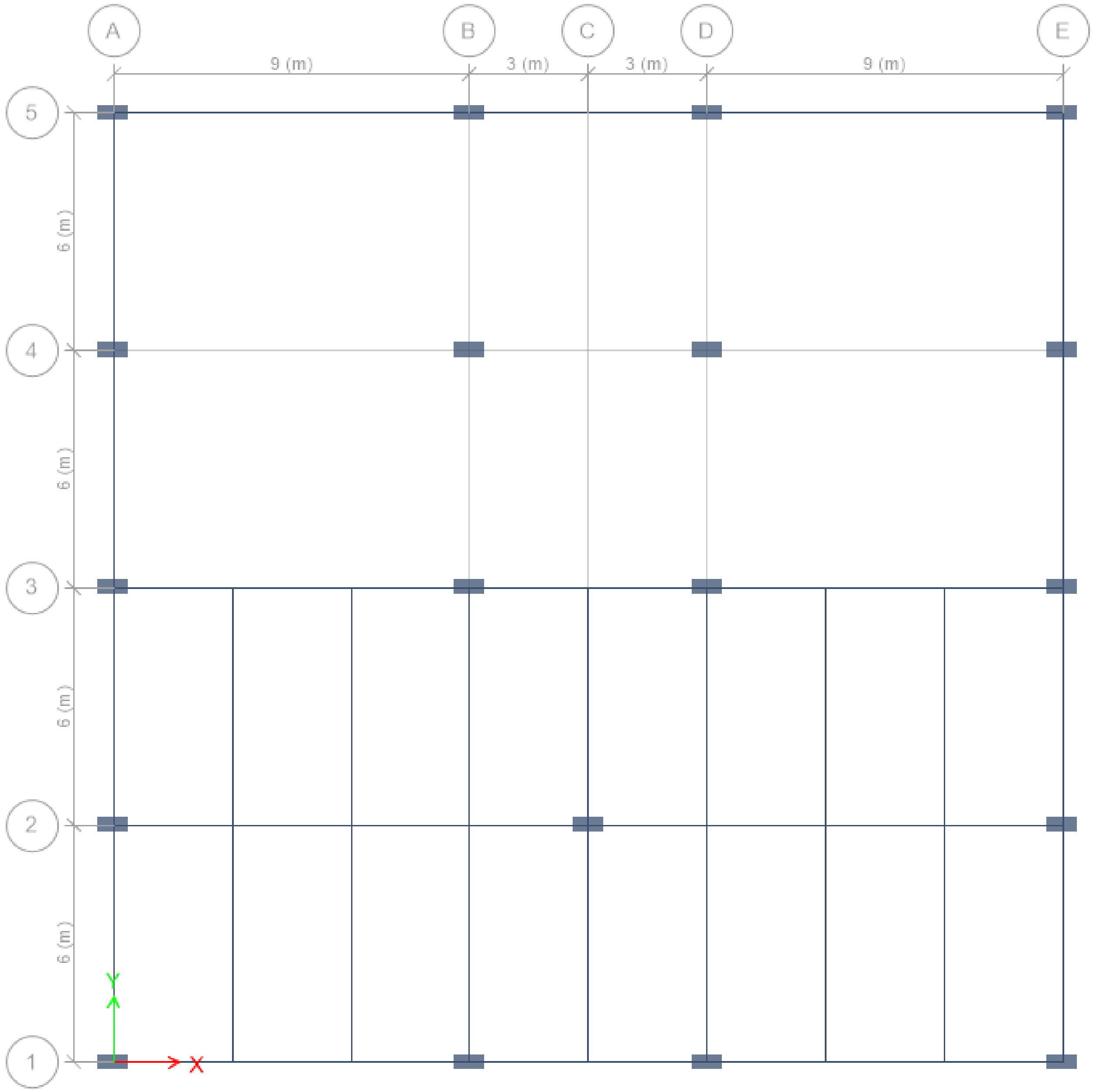



**Figure 25 Top view for 1st floor (mezzanine)**

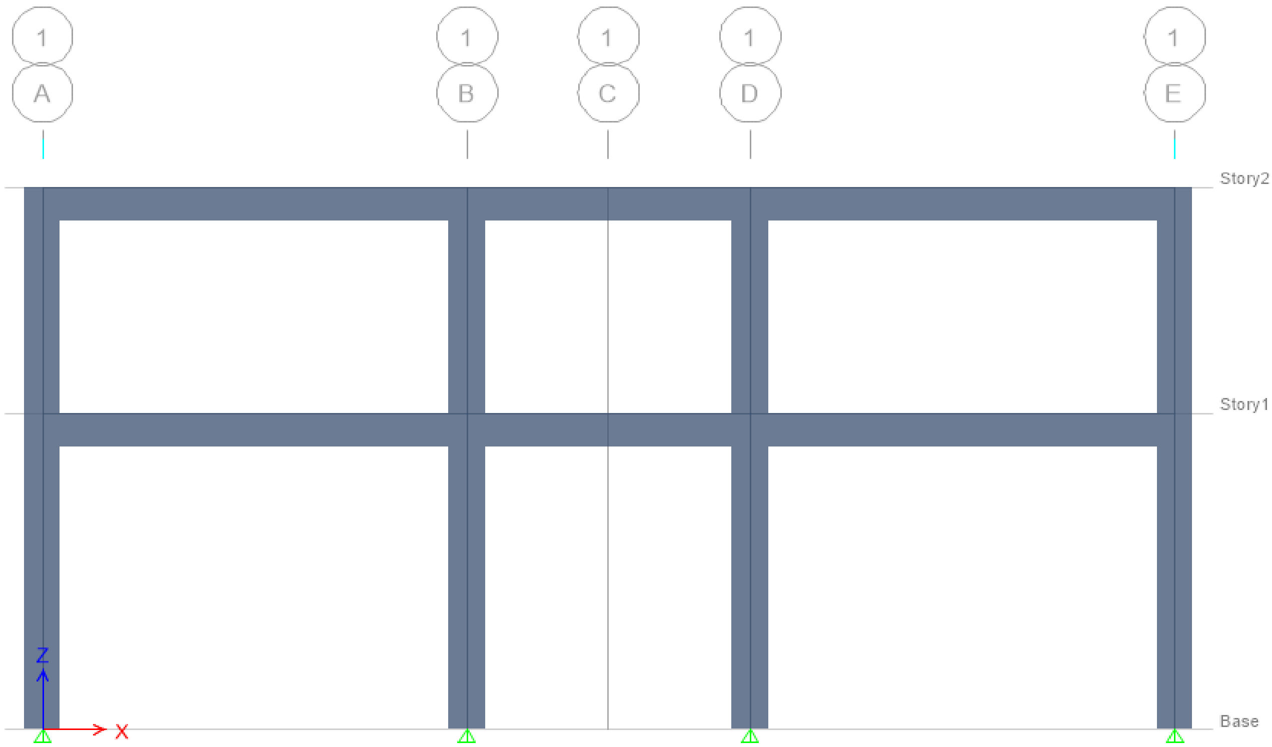



**Figure 26 Side view for 1st floor (mezzanine)**

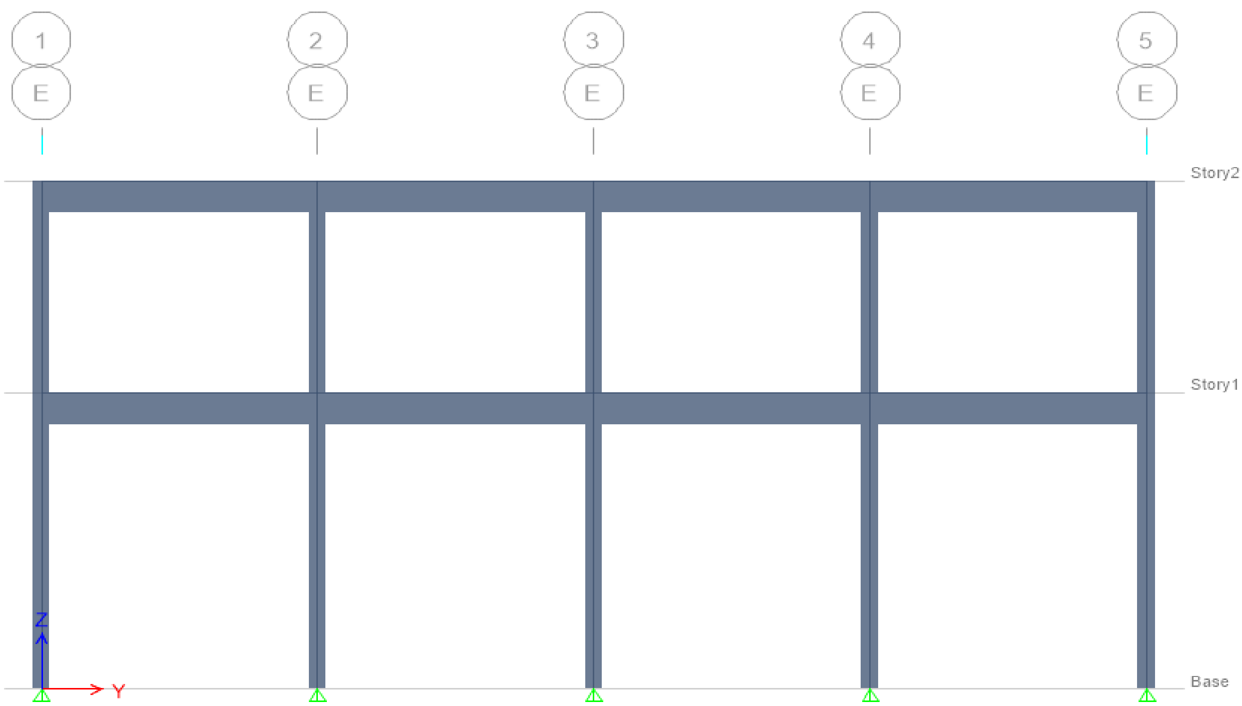



**Figure 27 Side view**

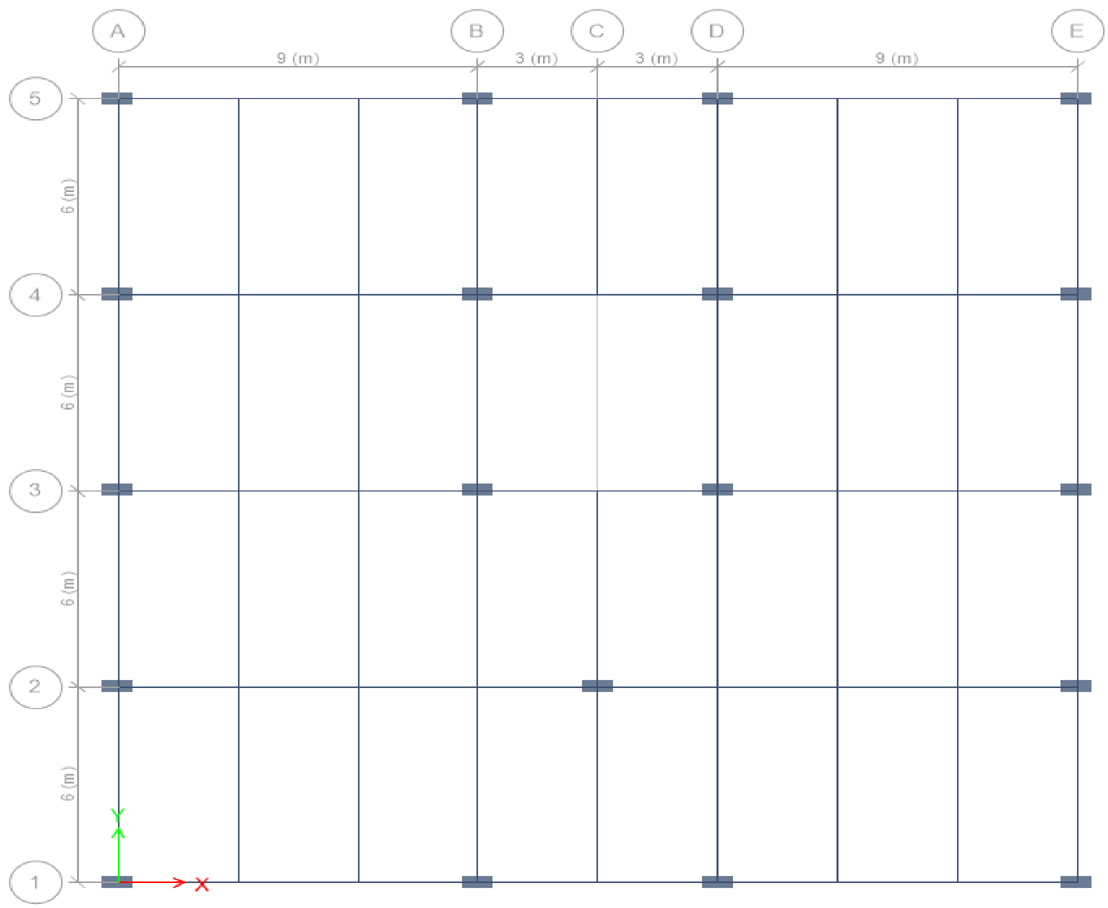



**Figure 28 Top view for Roof**

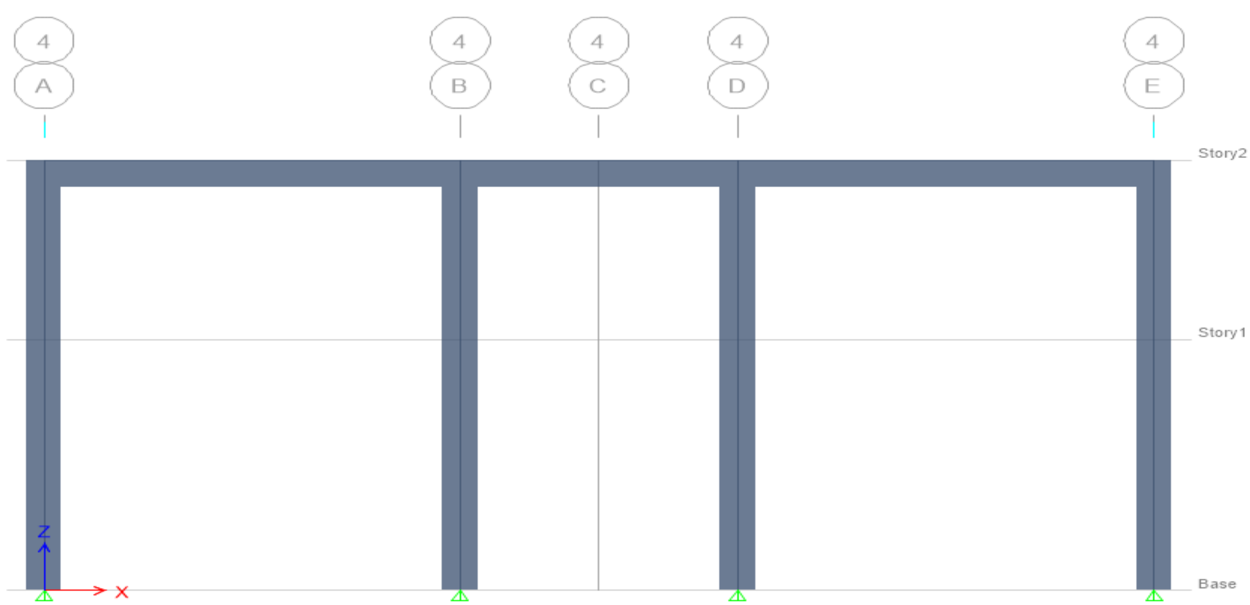



**Figure 29 Front view**

# Chapter 3: Steel

Symbols:

| SHORTS | MEANING |
|---|---|
| $A_g$ | Gross Area of Section |
| $A_w$ | Web Area |
| $C_b$ | Lateral Torsional buckling |
| $C_v$ | Web Shear |
| $C_w$ | Warping Constant |
| $E$ | Modulus of Elasticity |
| $F_{cr}$ | Buckling Stress |
| $F_e$ | Elastic Critical Buckling Stress |
| $F_y$ | Yielding Stress |
| $I_x$ | Moment of inertia at X- axis |
| $I_y$ | Moment of inertia at Y- axis |
| $J$ | Torsional Constant |
| $K$ | Effective Length |
| $K_v$ | Plate Buckling Coeffient |
| $L$ | Length |
| $L_b$ | Distance between Braces |
| $L_p$ , $L_r$ | Limiting Laterally Unbrace Length |
| $M_{max}$ | Maximum Moment |
| $M_n$ | Nominal Flexure Strength |
| $M_p$ | Plastic Bending Moment |
| $M_u$ | Ultimate Moment |

| | |
|---|---|
| $\mathbf{P_n}$ | Nominal Axial Strength |
| $\mathbf{S_x}$ | Elastic modulus at X-axis |
| $\mathbf{S_y}$ | Elastic modulus at Y-axis |
| $\mathbf{V_n}$ | Nominal Shear Strength |
| $\mathbf{V_u}$ | Ultimate Shear Strength |
| $\mathbf{Z_x}$ | Plastic modulus at X-axis |
| $\mathbf{Z_y}$ | Plastic modulus at Y-axis |
| **b** | Width |
| **C** | Coefficient |
| **d** | Depth of Web |
| **h** | Overall height |
| $\mathbf{h_o}$ | Distance between Flange Centroid |
| $\mathbf{r_x}$ | Governing radius at X-axis |
| $\mathbf{r_y}$ | Governing radius at Y-axis |
| $\mathbf{t_w}$ | Thickness of Web |
| $\mathbf{t_f}$ | Thickness of Flange |
| $\emptyset_{\mathbf{b}}$ | Resistance Factor for Flexure |
| $\emptyset_{\mathbf{c}}$ | Resistance Factor for Compression |
| $\emptyset_{\mathbf{v}}$ | Resistance Factor for Shear |
| $\Delta_{\mathbf{max}}$ | Maximum Deflection |
| $\Delta_{\mathbf{act}}$ | Actual Deflection |

## 3.1 Introduction:

Steel buildings first gained popularity in the early 20th century. Their use became more widespread during World War II and significantly expanded after the war when steel became more available. Steel buildings have been widely accepted, in part due to cost efficiency. The range of application has expanded with improved materials, products and design capabilities with the availability of computer aided design software.

Steel provides several advantages over other building materials, such as wood:

- Steel is a "green" product; it is structurally sound and manufactured to strict specifications and tolerances. It is also energy efficient. Any excess material is 100% recyclable.
- Steel does not easily warp, buckle, twist or bend, and is therefore easy to modify and offers design flexibility. Steel is also easy to install.
- Steel is cost effective and rarely fluctuates in price.
- Steel allows for improved quality of construction and less maintenance, while offering improved safety and resistance.
- With the propagation of mold and mildew in residential buildings, using steel minimizes these infestations. Mold needs moist, porous material to grow. Steel studs do not have those problems.

This chapter will include three main sections: Design information, Structural analysis, and Design process.

## 3.2 Design Information:

This section will provide the applicable design codes, material specifications, and load combinations.

### 3.2.1 Applicable Design Codes:

The loads as described in the design summary sheet have been applied on the structure in accordance with:

- American Institute of Steel Construction (AISC) Manual of Steel Construction/ Load and Resistance Factor Design (LRFD).
- American Institute of Steel Construction (AISC) Manual of Steel Construction/ Allowable Strength Design (ASD).
- Saudi Building Code (SBC) For Minimum Design Loads for Buildings and Other Structures (301).
- European Specifications Beams (EURONORM) 53-62.
- Egyptian Building Code (EBS) For Connection design.

### 3.2.2 Material specifications:

The following is the list of the material standards and specifications for which the building components have been designed:

**Table 6 Material specifications**

| No. | Materials | Specifications | Minimum Strength |
|---|---|---|---|
| **1** | Steel Members | ASTM A 992M Grade 345 Type 1 | $F_y = 34.5$ kN/cm$^2$<br>$F_u = 47.0$ kN/cm$^2$ |
| **2** | Bolts | ASTM A 36M Hot Dip Galvanized Class C | $F_y = 36.0$ kN/cm$^2$<br>$F_u = 52.0$ kN/cm$^2$ |
| **3** | Plates | ASTM A 992M Grade 345 Type 1 | $F_y = 34.5$ kN/cm$^2$<br>$F_u = 47.0$ kN/cm$^2$ |

### 3.2.3 Load Combinations:

This section will provide the calculation of the load combinations. These loads are calculated in accordance with Saudi Building Code (SBC) 301. These loads will be used in the structural analysis of the mezzanine beam and roof beam. Dome beam needs to calculate the dome weight and added to the load combination to do the structural analysis and dome beam design.

**1- Dead Loads:**

**Table 7 Dead load**

| No. | Types of Dead load | Value | Note |
|---|---|---|---|
| **1** | Slab thickness | 0.25 m | According to slab calculation in concrete section. |
| **2** | Beam weight | --- | Steel section table. |
| **3** | Ceramic tiles | 0.15 t/m | Saudi Building Code (SBC) – 301 |
| **4** | Dome weight | 20.52 t/m | According to dome calculation in concrete section. |

Note: Slab is concrete-base. Density of Concrete 2.5 $t/m^3$

**2- Live Loads:**

**Table 8 Live load**

| No. | Types of Live load | Value | Note |
|---|---|---|---|
| **1** | Mezzanine | 0.5 $t/m^2$ | Saudi Building Code (SBC) – 301 |
| **2** | Roof | 0.25 $t/m^2$ | Saudi Building Code (SBC) – 301 |

**Ultimate Load**

**Table 9 Ultimate load**

| No. | Types of Load | Factor of Safety | Note |
|---|---|---|---|
| 1 | Dead | 1.2 | Saudi Building Code (SBC) – 301 |
| 2 | Live | 1.6 | Saudi Building Code (SBC) – 301 |

## 3.3 Structural Analysis:

As indicated previously, the mosque contains a ground floor and a mezzanine floor. The structural analysis will determine the critical reactions, maximum moment, and maximum shear.

Ultimate load that has been applied is taken according to the Saudi Building Code (SBC) as mentioned previously. The applied ultimate load is distributed load along the beam.

American Institute of Steel Construction (AISC) Manual of Steel Construction, Load and Resistance Factor Design (LRFD) will be applied on moment equations and shear equations. Moreover, SkyCiv software program will be used in addition to the calculations to determine the structural analysis.

In this section, a structural analysis will show the reactions on the mezzanine floor beam, the roof slab beam, dome beam, and column. Each beam will contain the input data and the output data. SkyCiv program will also show the moment diagram, shear diagram, and reaction forces. The results will be used during the design phase in the following section.

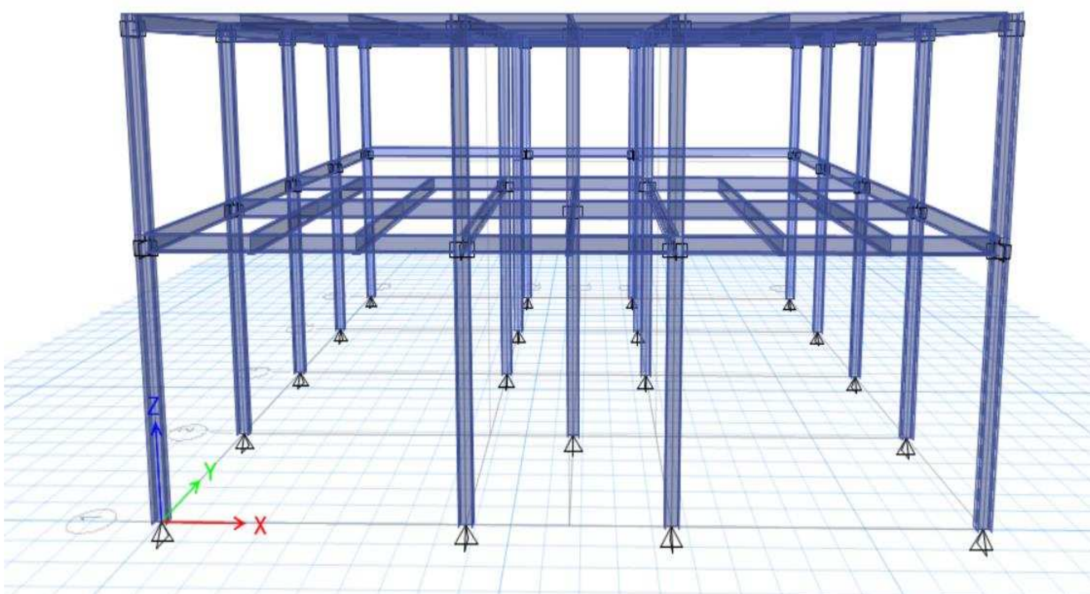



**Figure 30 ETABS 3D drawing**

### 3.3.1 Mezzanine floor beam (Girder 2-2):

- **Input:**

Ultimate load is applied on the mezzanine floor beam using calculation and SkyCiv program. The beam is fixed at both ends. The length of the beam is
12.00 meters. The drawing shows the length of the beam, applied load, and supports.

**Table 10 Mezzanine floor analysis**

| | | | | | |
|---|---|---|---|---|---|
| A- Dead Load | | | | | |
| A- Slab Weight = | Thickness × | Space | × Density = | | Total |
| | 0.25 m | 6 m | 2.5 | $t/m^3$ | 3.75 t/m |
| B- Beam Weight = | | | | | 0.178 t/m |
| C- Ceramic Tile = | | | | | 0.15 t/m |
| **Total Dead Load =** | | | | | 4.078 t/m |
| B- Live Load | | | | | |
| Live Load= | Load × | Space = | | | Total |
| | $0.5\ t/m^2$ | 6 m | | | 3.0 t/m |
| C- Ultimated Load | | | | | |
| Wu = | 1.2 × D.L + 1.6 × L.L = | | | | Total |
| | | | | | 9.69 t/m |

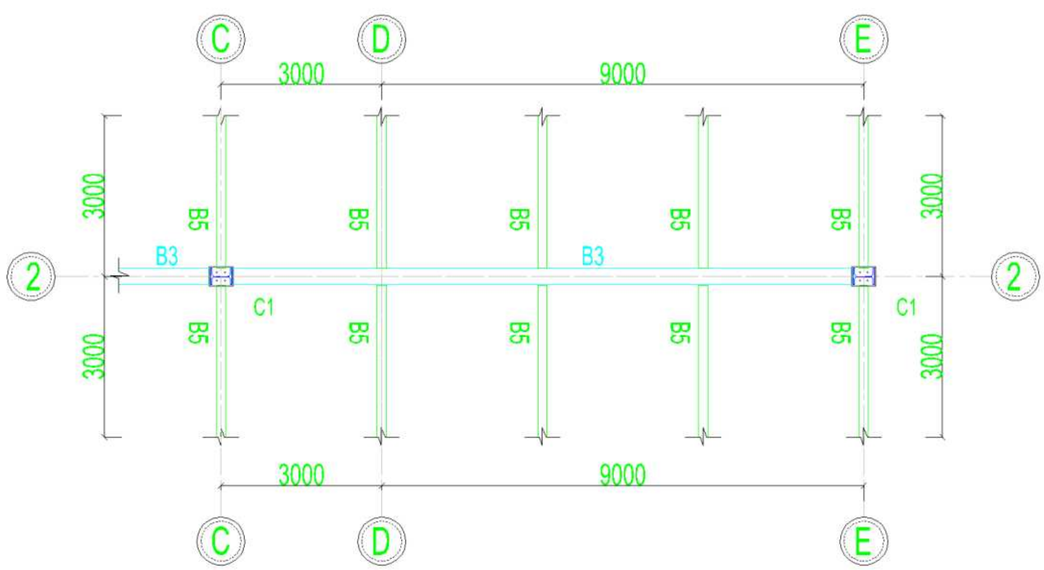



**Figure 31 AutoCAD Drawing, for Grade 2 - Level 7m**

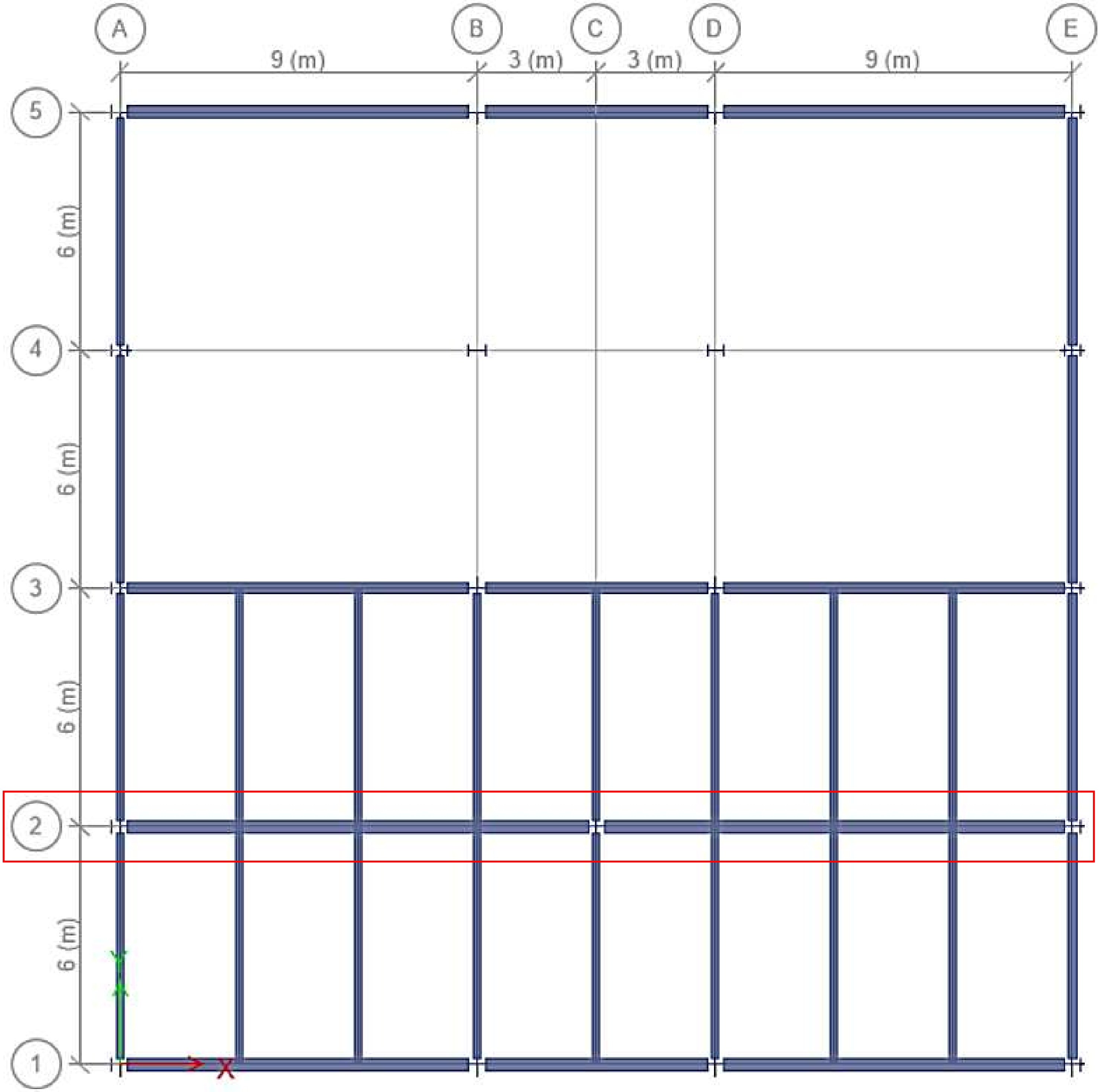



**Figure 32 ETABS drawing, mezzanine floor, Girder 2**

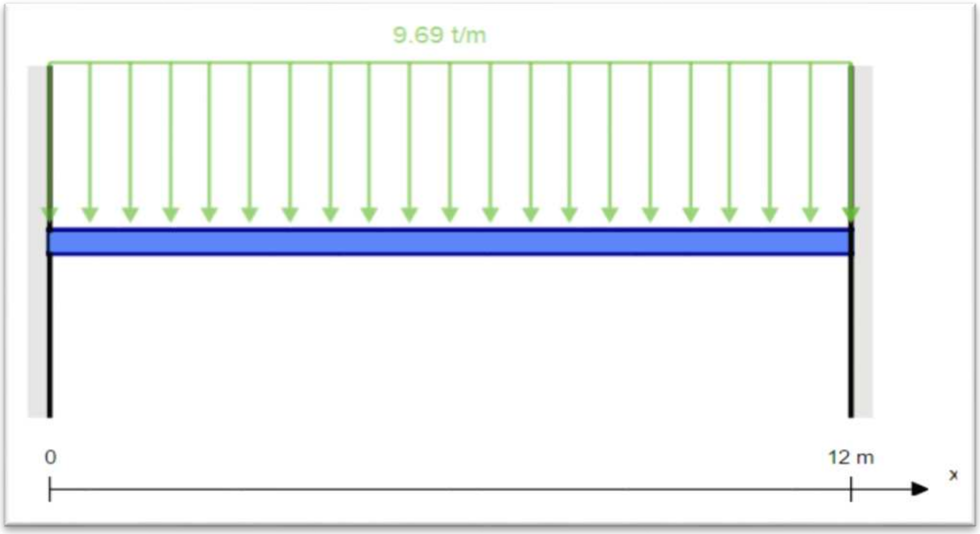



**Figure 33 Input data from SkyCiv program for mezzanine floor beam**

- **Output:**

Output data are the results of the structural analysis of the mezzanine floor beam using calculation and SkyCiv program. The following tables summarize the calculations of the reactions forces of the beam.

**Table 11 Structural analysis for girder 2**

| Calculate for Moment |
| --- |
| $M_{max(End)} = \frac{W_u \times L^2}{12} = \frac{9.69 \times 12^2}{12} = 116.32$ t-m |
| $M_{max(Cen)} = \frac{W_u \times L^2}{24} = \frac{9.69 \times 12^2}{24} = 58.16$ t-m |

| Calculate for Maximum Shear ($V_{max}$) |
| --- |
| $V_{max} = \frac{W_u \times L}{2} = \frac{9.69 \times 12}{2} = 58.16$ t |

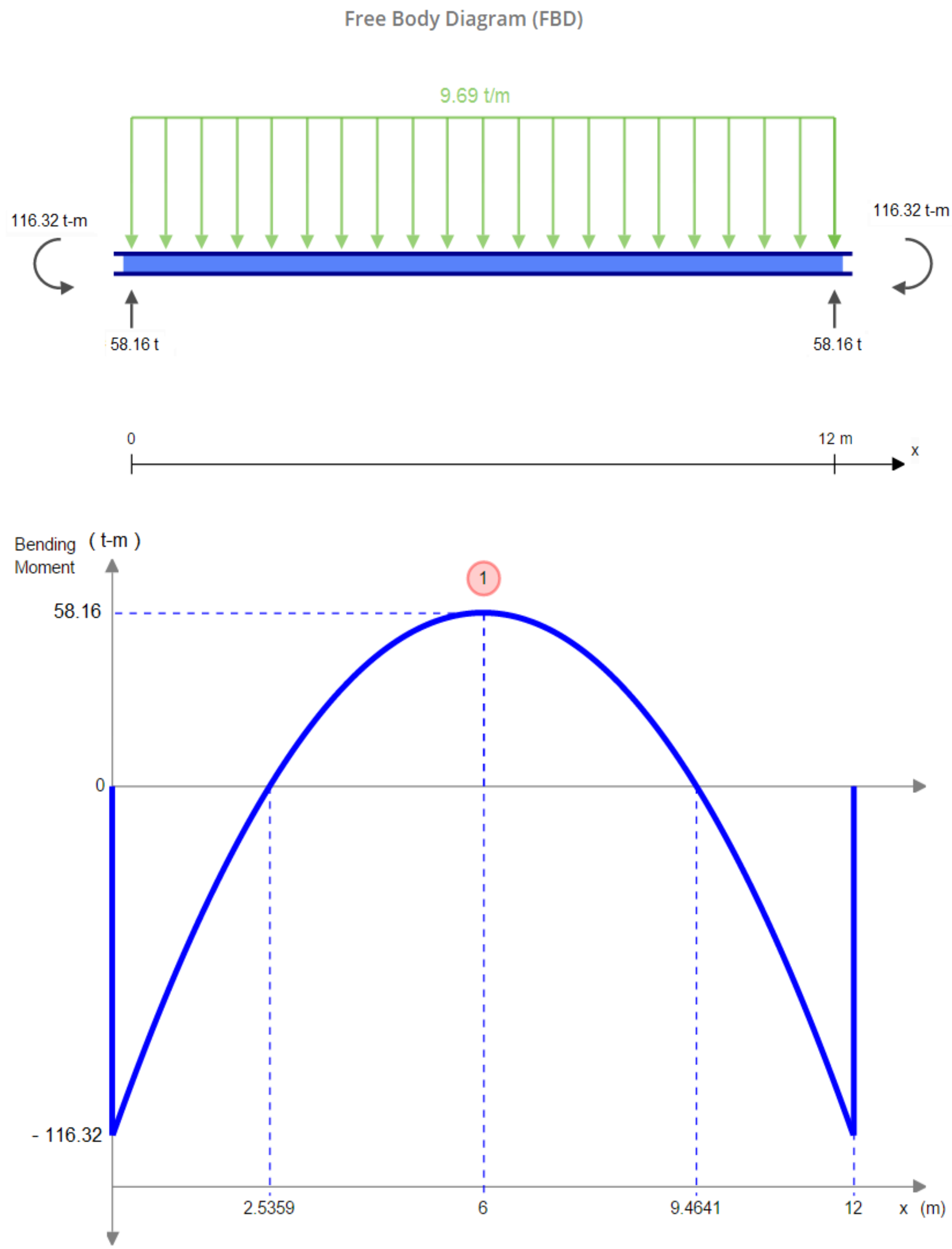



**Figure 34 Output data from SkyCiv program (moment)**

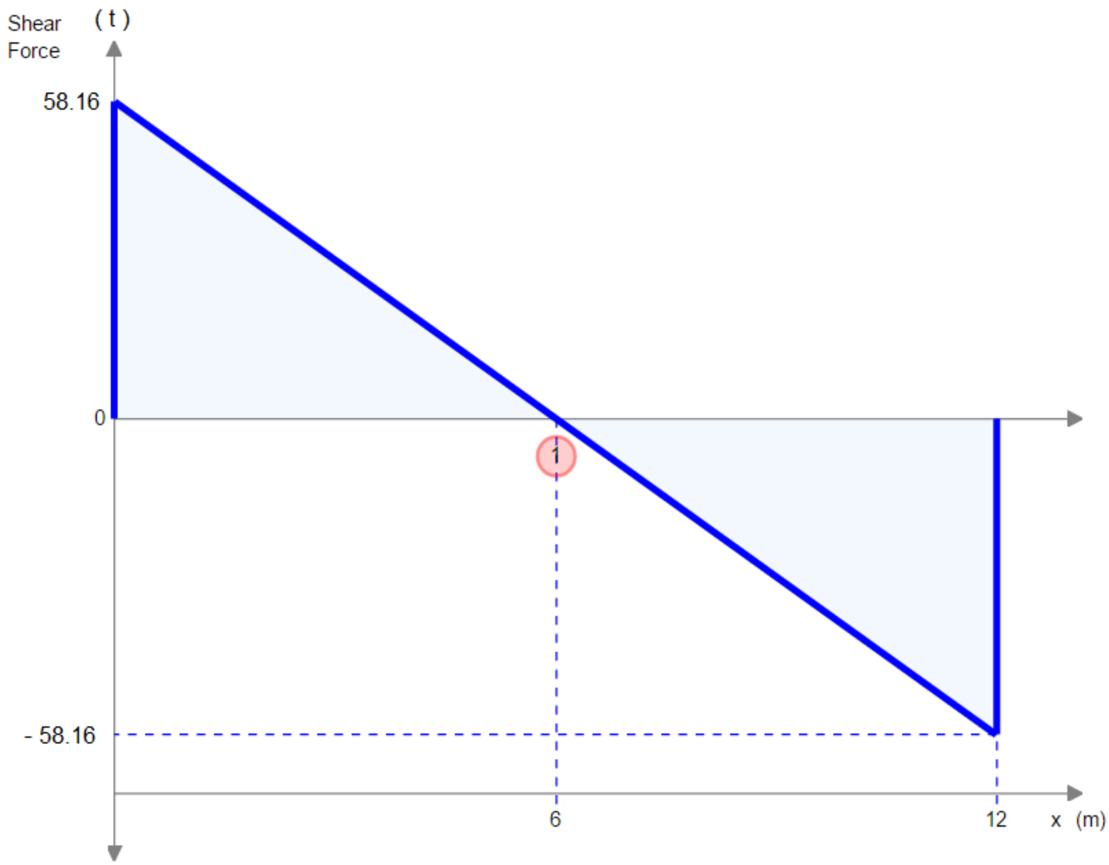



**Figure 35 Output data from SkyCiv program (shear)**

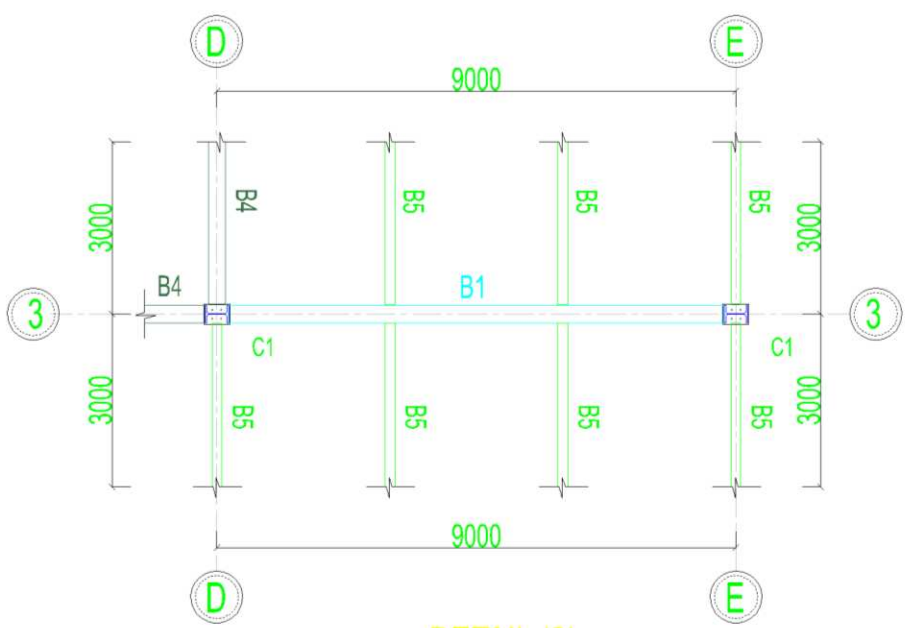
### 3.3.2 Roof beam (3DE):

- **Input:**

Ultimate load is applied on the roof beam using calculation and SkyCiv program. The beam is fixed at both ends. The length of the beam is 9.00 meters. The drawing shows the length of the beam, applied load, and supports.

**Table 12 Roof analysis for 3DE beam**

| | | | | | |
|---|---|---|---|---|---|
| A- Dead Load | | | | | |
| A- Slab Weight = | Thickness × | Space | × Density = | | Total |
| | 0.25 m | 6 m | 2.5 | $t/m^3$ | 3.75 t/m |
| B- Beam Weight = | | | | | 0.125 t/m |
| C- Ceramic Tile = | | | | | 0.15 t/m |
| **Total Dead Load =** | | | | | 4.025 t/m |
| B- Live Load | | | | | |
| Live Load= | Load × | Space = | | | Total |
| | 0.25 $t/m^2$ | 6 m | | | 1.5 t/m |
| C- Ultimated Load | | | | | |
| Wu = | 1.2 × D.L + 1.6 × L.L = | | | | Total |
| | | | | | 7.23 t/m |



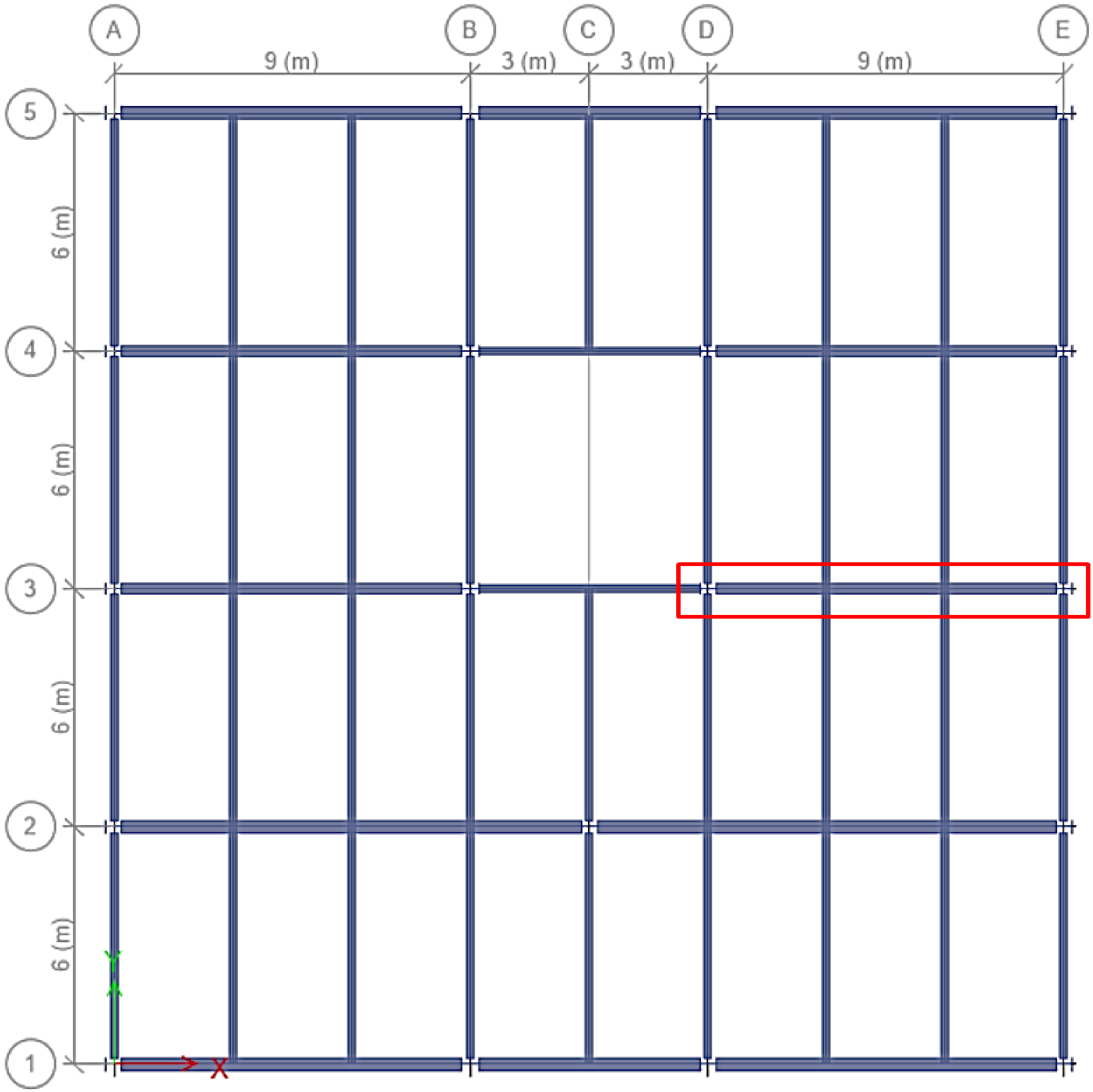



**Figure 36 ETABS drawing, Roof, 3DE.**

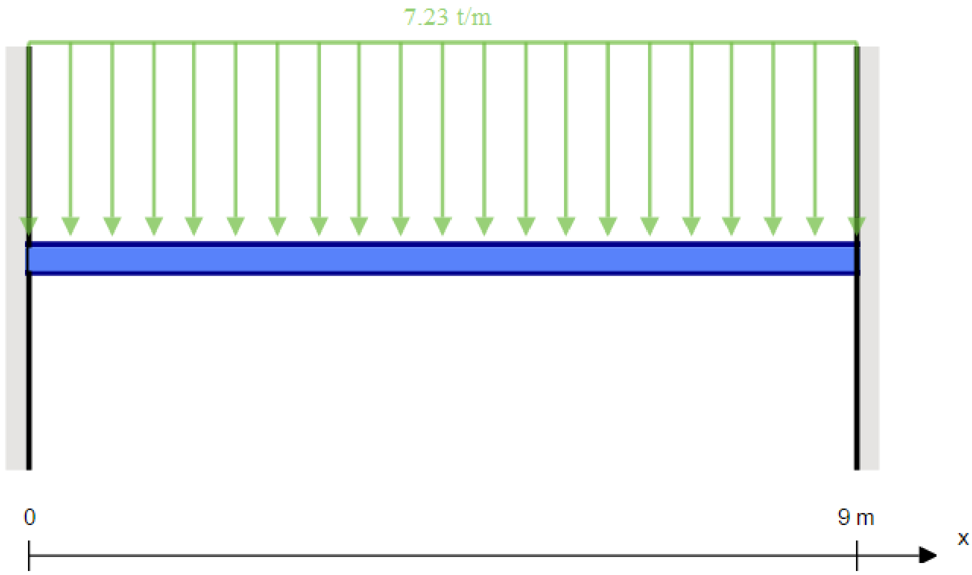



**Figure 37 Input data from SkyCiv program for Roof beam**

- **Output:**

Output data are the results of the structural analysis of the roof beam using calculation and SkyCiv program. The following tables summarize the calculations of the reactions forces of the beam.

**Table 13 Structural analysis for 3DE beam**

**Calculate for Moment**

$$M_{max(End)} = \frac{W_u \times L^2}{12} = \frac{7.23 \times 9^2}{12} = 48.80 \text{ t-m}$$

$$M_{max(Cen)} = \frac{W_u \times L^2}{24} = \frac{7.23 \times 9^2}{24} = 24.40 \text{ t-m}$$

**Calculate for Maximum Shear ($V_{max}$)**

$$V_{max} = \frac{W_u \times L}{2} = \frac{7.23 \times 9}{2} = 32.54 \text{ t}$$

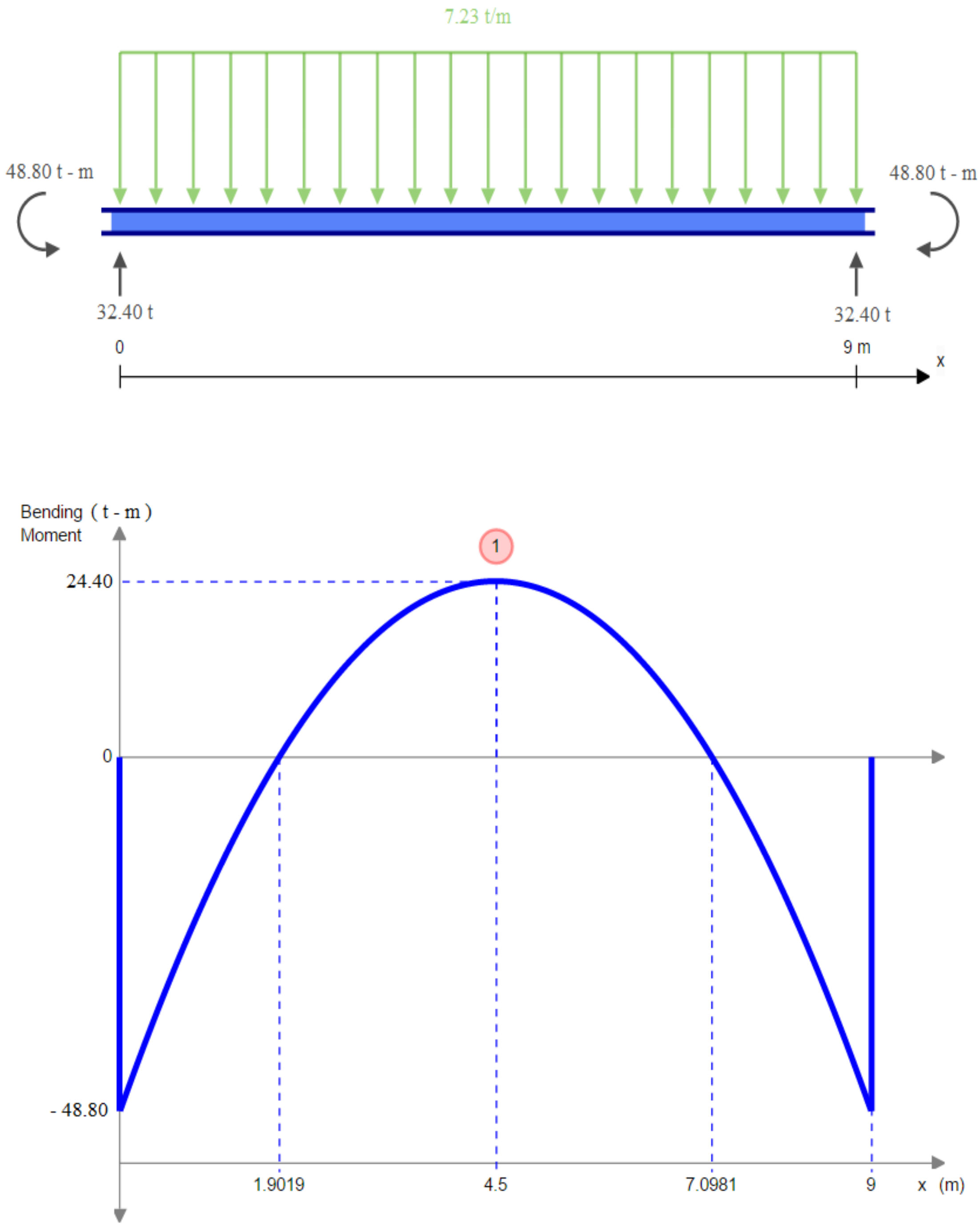



**Figure 38 Output data from SkyCiv program (moment)**


**Figure 39 Output data from SkyCiv program (shear)**

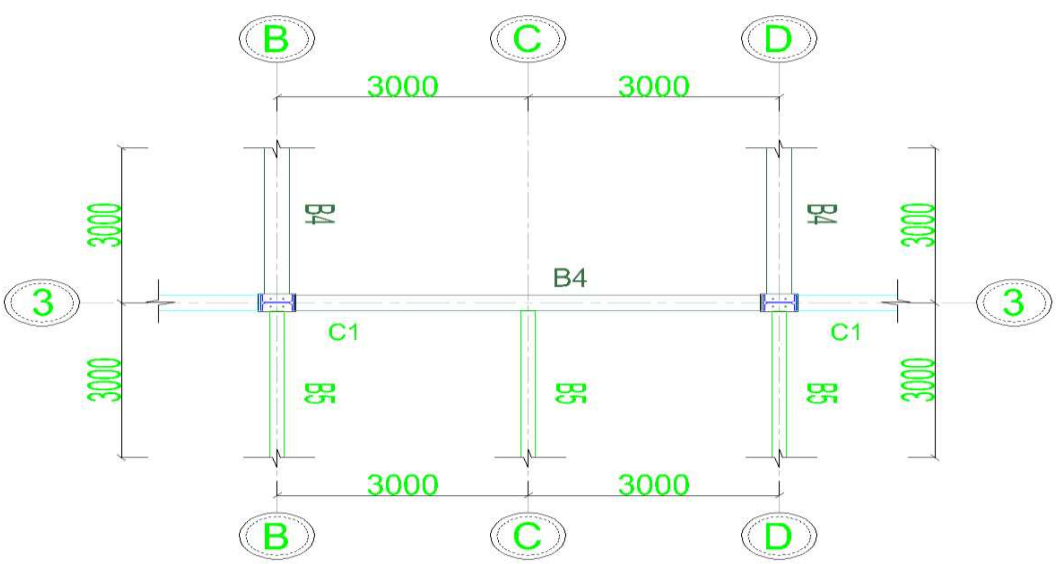
### 3.3.3 Dome beam (3BD):

- **Input:**

Ultimate load is applied on the dome beam using calculation and SkyCiv program. The beam is fixed at both ends. The length of the beam is 6.00 meters. The drawing shows the length of the beam, applied load, and supports. The dome weight is calculated and applied in the beam structural analysis to be used in dome beam design.

**Table 14 Roof analysis for dome beam**

| A- Dead Load (Distrubuded Load) | | | | |
|---|---|---|---|---|
| A- Slab Weight = | Thickness × | Space × | Density = | Total |
| | 0.25 m | 3 m | 2.5 $t/m^3$ | 1.88 t/m |
| B- Beam Weight = | | | | 0.078 t/m |
| **Total Dead Load =** | | | | 1.953 t/m |
| **Dome Load** | | | | |
| Dome Weight (P.L) = 1.2 × Weight = | | | | **Total** |
| P.L = | 1.2 | 17.1 t/m | | 20.52 t/m |
| **The Weight of Dome are Dead Point Load Acting on the Centar of The Beam** | | | | |
| B- Live Load | | | | |
| Live Load= | Load × | Space = | | Total |
| | 0.25 $t/m^2$ | 3 m | | 0.8 t/m |
| C- Ultimated Load | | | | |
| Wu = | 1.2 × D.L + 1.6 × L.L = | | | Total |
| | | | | 3.54 t/m |



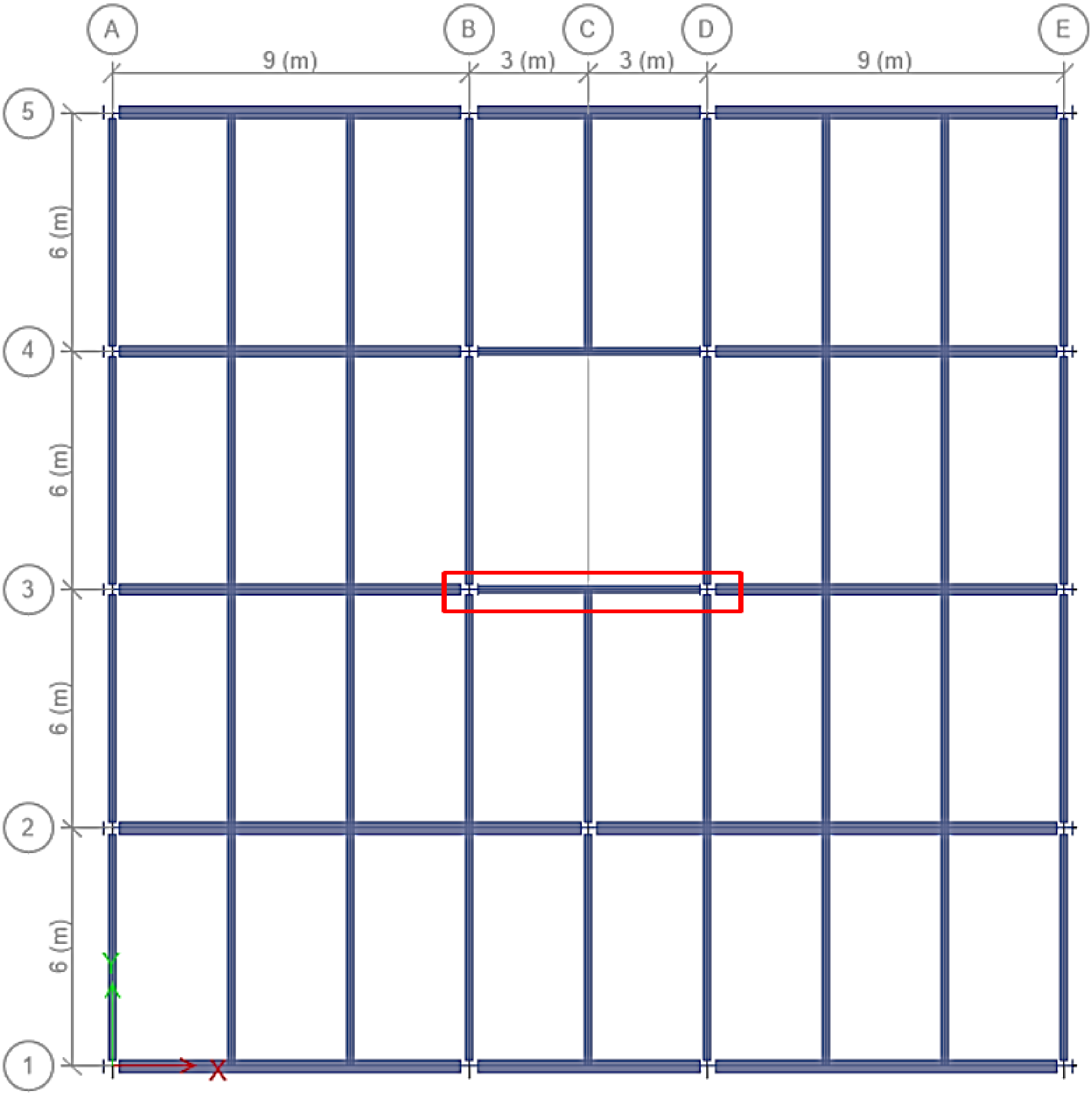



**Figure 40 ETABS drawing, Dome Beam (3BD).**

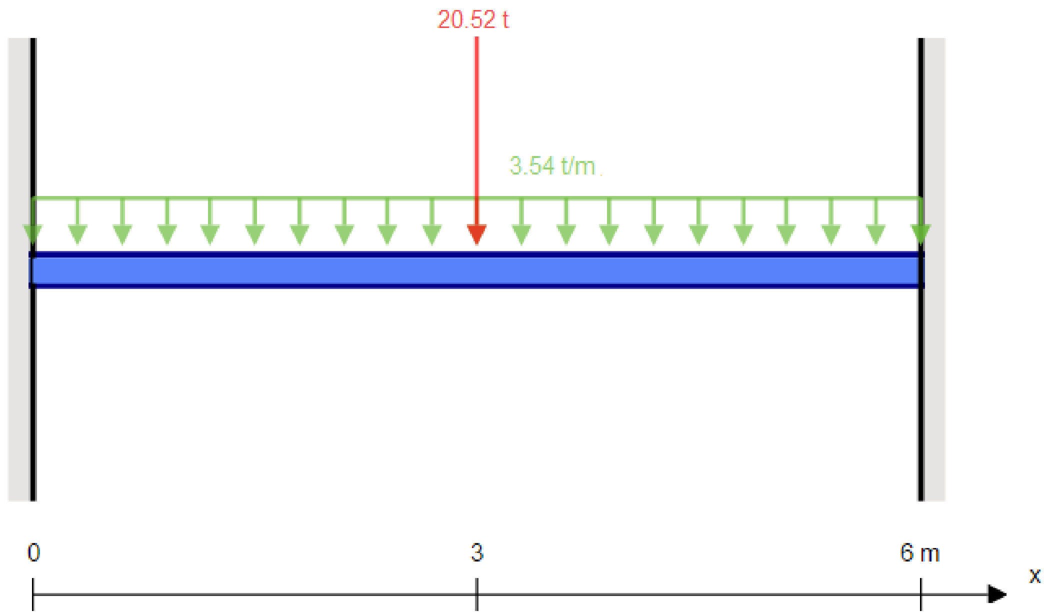



**Figure 41 Input data from SkyCiv program for Dome beam**

- **Output:**

Output data are the results of the structural analysis of the dome beam using calculation and SkyCiv program. The following tables summarize the calculations of the reactions forces of the beam.

**Table 15 Structural analysis for 3BD beam**

| | |
|---|---|
| $R_1 = \frac{W_u \times L}{2} + \frac{P.L}{2} =$ <br> $R_1 = \frac{3.54 \times 6}{2} + \frac{20.52}{2} = 20.88\ t$ | $R_2 = \frac{W_u \times L}{2} + \frac{P.L}{2} =$ <br> $R_2 = \frac{3.54 \times 6}{2} + \frac{20.52}{2} = 20.88\ t$ |

$$M_{max(both\ Ends)} = \frac{W_u \times L^2}{12} + \frac{P.L \times L}{8} =$$

$$M_{max(both\ Ends)} = \frac{3.54 \times 6^2}{12} + \frac{20.52 \times 3}{8} = 26.01\ (t - m)$$

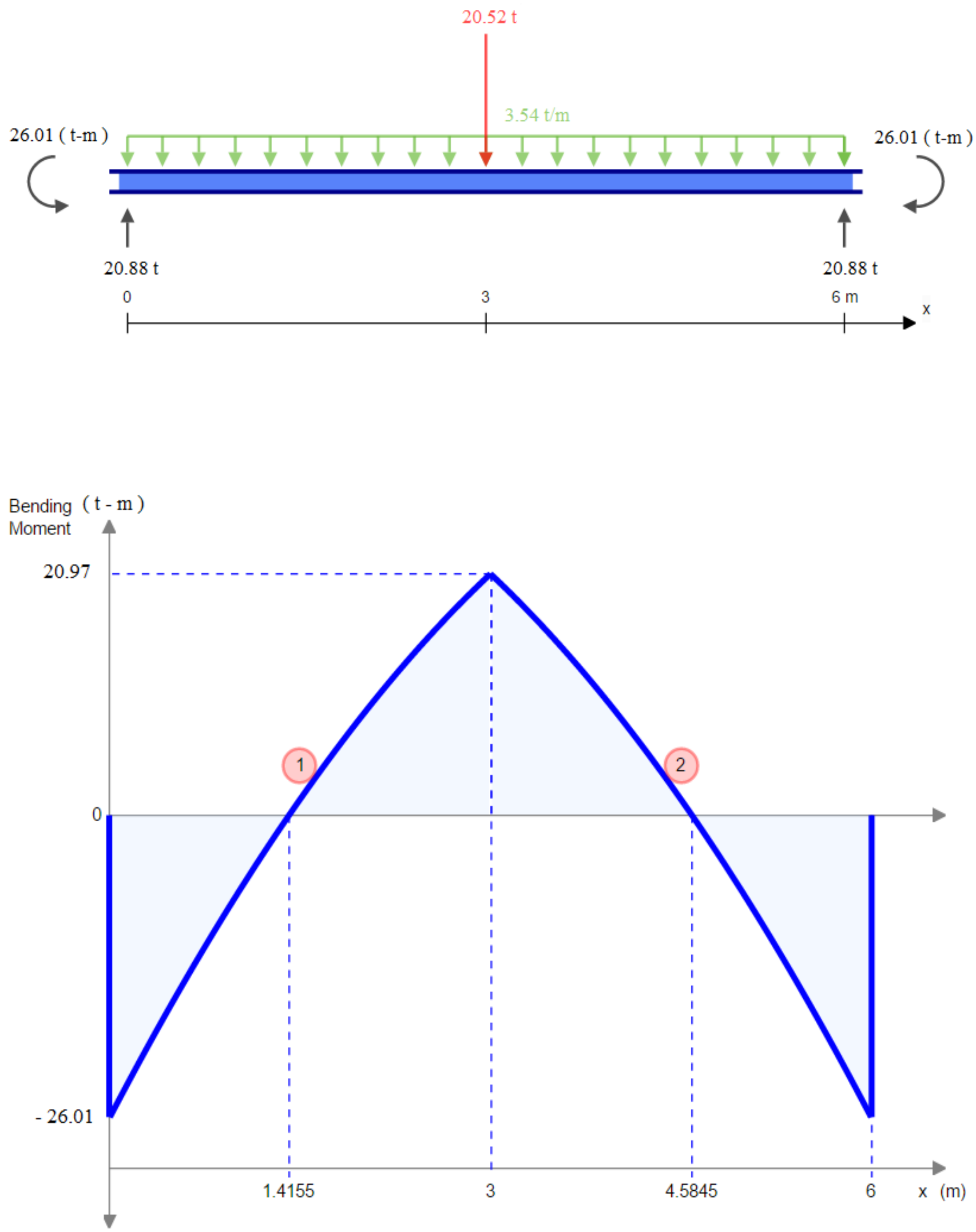



**Figure 42 Output data from SkyCiv program (moment)**

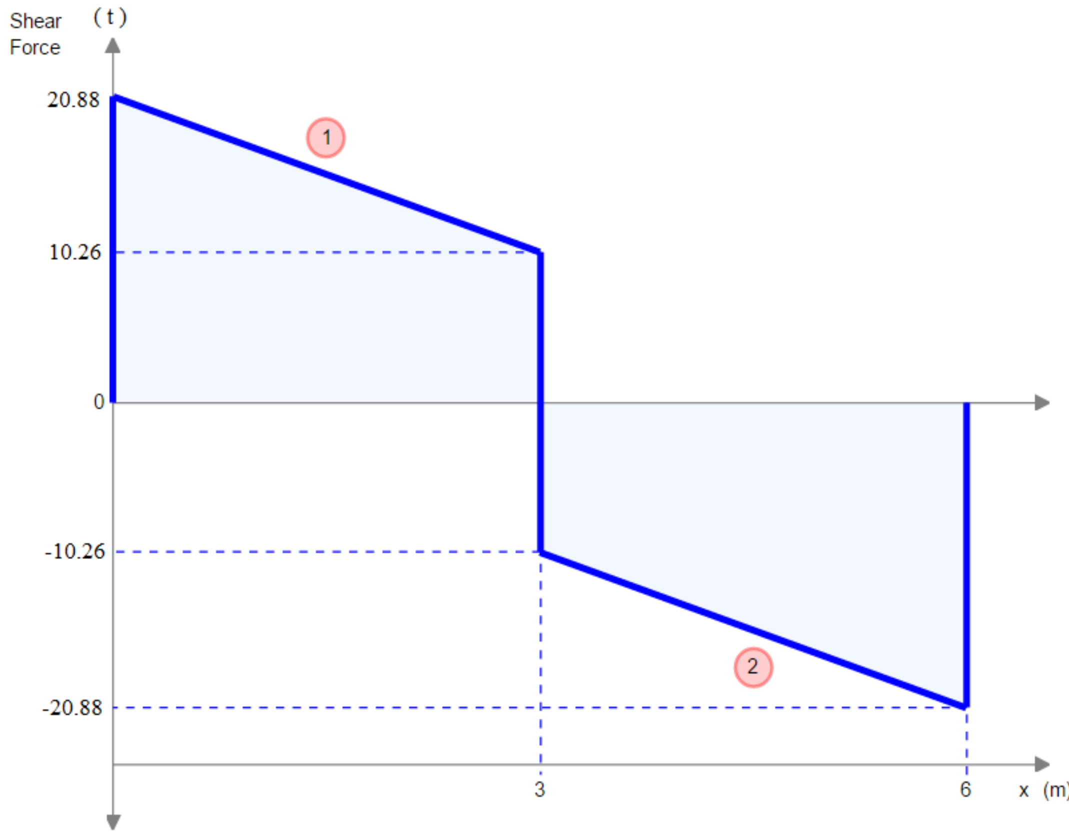



**Figure 43 Output data from SkyCiv program (shear)**

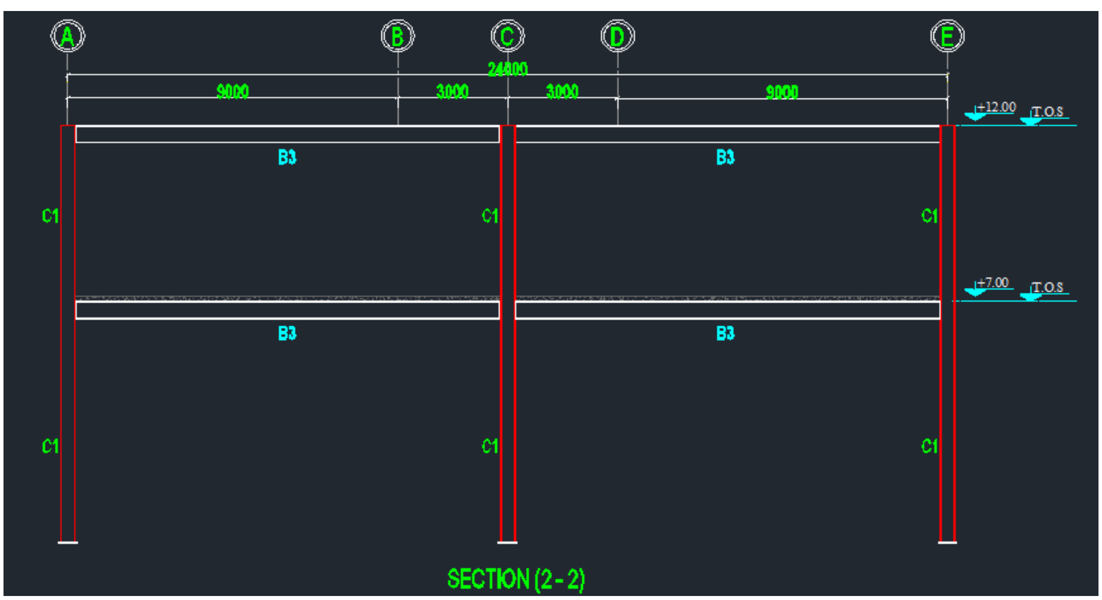
**3.3.4 Column (2C):**

- **Input:**

For column analysis, calculation would include the load combination. Using these load combinations, they will be used to do the moment calculation and shear.

**Table 16 Structural analysis for column**

| A- Dead Load | | |
|---|---|---|
| A- Slab Weight = | Thickness × Area × Density = | Total |
| | = 0.25 m × ( 6 m × 3 m ) × 2.5 $t/m^3$ = | 11 t |
| B- Beam Weight = | 0.187 t/m × ( 6 m + 3 m ) = | 1.68 t |
| C- Column Weight = | 0.125 t/m × (6 m + 3 m ) = | 1.13 t |
| D- Ceramic Tile = | 0.15 t/m × (6 m + 3 m ) = | 1.35 t |
| **Total Dead Load** = | | 15.41 t |
| B- Live Load | | |
| Live Load= | Load × Area = | Total |
| | = 0.5 $t/m^2$ × ( 6 m × 3 m ) = | 9.0 t |
| C- Ultimated Load | | |
| Wu = | 1.2 × D.L + 1.6 × L.L = | Total |
| | | 32.89 t |
| Combined the Point Load From First Floor and Roof | | |
| First Floor Load + Roof Load = | | Total |
| | = 32.89 t + 32.89 t = | 65.78 t |



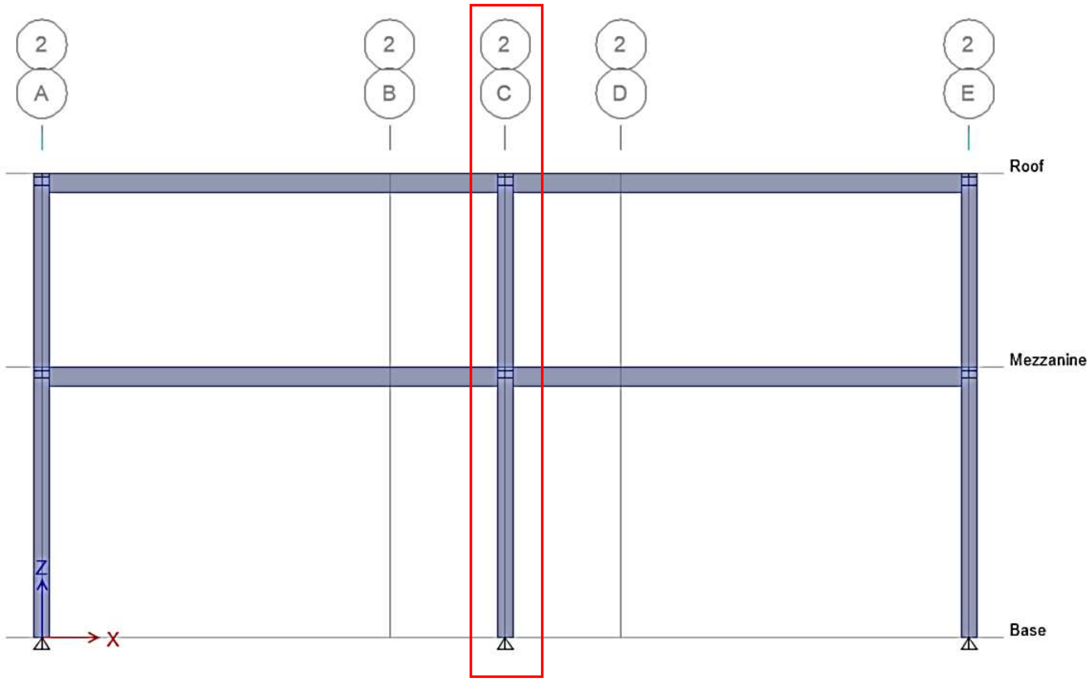



**Figure 44 ETABS drawing, Column 2C**

**Output:**

Output data are the results of the structural analysis of Column calculation. The following tables summarize the calculations of the reactions forces of the column.

**Table 17 Structural analysis for column**

| Calculate for Moment |
| --- |
| $M_{max(End)} = \dfrac{W_u \times L^2}{12} = \dfrac{65.78 \times 7^2}{12} =$ 268.60 t-m |
| $M_{max(Cen)} = \dfrac{W_u \times L^2}{24} = \dfrac{65.78 \times 7^2}{24} =$ 134.30 t-m |

| Calculate for Maximum Shear ($V_{max}$) |
| --- |
| $V_{max} = \dfrac{W_u \times L}{2} = \dfrac{7.23 \times 9}{2} =$ 32.54 t |

## 3.4 Design:

According to the structural analysis, the most critical analysis is the mezzanine floor beam analysis. Its moment, shear, and reactions are the most critical. These data will be used to design the mezzanine and roof beam. Dome beam and column have its own different output reaction.

The steel sections are taken from European Specifications Beams (EURONORM). For moment, shear, and deflection calculations, American Institute of Steel Construction (AISC) Manual of Steel Construction for Load and Resistance Factor Design (LRFD) is applied.

Selected steel section is shown in tables. These data are taken from the steel section properties, ETABS software during the modeling phase, and the American Building code equations. ETABS output will be included with the appendix.

Moment design will include the lateral-torsional buckling design. Accordingly, this will check the beam length if it is safe against lateral-torsional buckling. In next step, the nominal moment for the selected steel section will be calculated to check and compare it with the ultimate moment from the structural analysis.

Shear design will include web condition of the section. This will determine the web shearing strength of the selected section. Next step, the area for the web will be calculated to use it in nominal shear equation. In nominal shear, it will show the capability of the selected steel section against the ultimate shear that has been mentioned previously in the structural analysis.

Deflection check is the check of the maximum allowable deflection of the structure according to the building code with the deflection of the selected steel section. The deflection limit is calculated according to the American Building Code.

This section will show four designs: Mezzanine beam, roof beam, dome beam, and column. Each member will have the properties of the selected steel section, moment design, shear design, and deflection check.

### 3.4.1 Mezzanine beam design (Girder 2-2):

- **Properties:**

**Table 18 Steel section properties (HE 500 B)**

| Section Property (HE500B) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $A_g$ | 0.0239 | $m^2$ | $S_x$ | 0.004287 | $m^3$ | $C_w$ | 0.000007018 | $m^6$ |
| b | 0.3 | M | $S_y$ | 0.000842 | $m^3$ | J | 0.000005 | $m^4$ |
| d | 0.39 | M | $r_x$ | 0.212 | m | L | 12 | m |
| $h_o$ | 0.42 | M | $r_y$ | 0.0727 | m | $Ø_b$ | 0.9 | - |
| h | 0.5 | M | $t_w$ | 0.0145 | m | $Ø_v$ | 1 | - |
| $I_x$ | 0.001072 | $m^4$ | $t_f$ | 0.028 | m | $K_v$ | 5 | - |
| $I_y$ | 0.000126 | $m^4$ | $F_y$ | 35153.48 | $t/m^2$ | $C_b$ | 1.14 | - |
| $Z_x$ | 0.004815 | $m^3$ | $F_u$ | 45700 | $t/m^2$ | $C_v$ | 1 | - |
| $Z_y$ | 0.001292 | $m^3$ | E | 20389019.16 | $t/m^2$ | C | 1 | - |
| Steel Section | | | ETABS Software | | | Building Code | | |

**Note:**

The date that are shown in blue are taken from the steel section properties, European Specifications Beams (EURONORM).

The data that are shown in green are taken from the ETABS software during the modeling phase.

The data that are shown in red are taken from the American Building Code according to its equations.

- **Moment design:**

**1- Lateral-Torsional Buckling**

$$L_b = L - [(0.211 \times L) \times 2]$$

$$L_b = 12 - [(0.211 \times 12) \times 2] = 6.94 \text{ m}$$

$$L_p = 1.76 \times r_y \times \sqrt{\frac{E}{F_y}}$$

$$L_p = 1.76 \times 0.0727 \times \sqrt{\frac{20389019.2}{35153.48}} = 3.08 \text{ m}$$

$$L_r = 1.95 \times r_{ts} \times \frac{E}{0.7 \times F_y} \times \sqrt{\frac{J \times c}{S_x \times h_o} + \sqrt{\left(\frac{J \times c}{S_x \times h_o}\right)^2 + 6.76\left(\frac{0.7 \times F_y}{E}\right)^2}}$$

$$L_r = 1.95 \times 0.083 \times \frac{20389019.16}{0.7 \times 35153.48} \times \sqrt{\frac{0.000005 \times 1}{0.004287 \times 0.42} + \sqrt{\left(\frac{0.000005 \times 1}{0.004287 \times 0.42}\right)^2 + 6.76\left(\frac{0.7 \times 35153.48}{20389019.16}\right)^2}} = 11.24 \text{ m}$$

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} =$$

$$r_{ts} = \sqrt{\frac{\sqrt{0.000126 \times 0.000007018}}{0.004287}} = 0.083 \text{ m}$$

**2- The Case for Lateral-Torsional Bukling**

| | | | | | | |
|---|---|---|---|---|---|---|
| a) $L_b \le L_p$ | 6.94 m | $\le$ | 3.08 m | | | ✘ |
| b) $L_p < L_b \le L_r$ | 3.08 m | $<$ | 6.94 m | $\le$ | 11.24 m | ✔ |
| C) $L_b > L_r$ | 6.94 m | $>$ | 11.24 m | | | ✘ |

| 3- Value Of Plastic Bending Moment and Nominal Flexure Strength |
|---|
| $M_p = F_y \times Z_x = 35153.48 \times 0.004815 = 169.26$ t/m |
| $M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$ <br> $M_n = 1.14 \times \left[169.26 - (169.26 - 0.7 \times 35153.48 \times 0.004287)\left(\frac{6.94 - 3.08}{11.24 - 3.08}\right)\right] = 158.60$ t/m |

| 4- Check for the Moment | | | | |
|---|---|---|---|---|
| $M_u =$ 116.3 t/m <br> $\emptyset_b M_n =$ 142.74 t/m | $M_u$ <br> 116.3 t/m | $\leq$ <br> $\leq$ | $\emptyset_b M_n$ <br> 142.74 t/m | ✓ $M_u \leq \emptyset_b M_n$ |

- **Shear design:**

1- Check for Web Shear Conditions ($C_v$)

$$\frac{h}{t_w} = \frac{0.5}{0.0145} = 34.48$$

$$1.10\sqrt{\frac{k_v \times E}{F_y}}$$

$$1.10 \times \sqrt{\frac{5 \times 20389019.2}{35153.48}} = 59.24$$

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v \times E}{F_y}}$$

$$34.48 \leq 59.24 \quad ✓$$

| 2- Find Web Area ($A_w$) |
|---|
| $A_w = d \times t_w = 0.39 \times 0.0145 = 0.00566$ $m^2$ |

| 3- Find Nominal Shear ($V_n$) |
|---|
| $V_n = 0.6 \times F_y \times A_w \times C_v$ |
| $V_n = 0.6 \times 35153.48 \times 0.00566 \times 1 = 119.28$ t |

| 4- Check for Shear | | | |
|---|---|---|---|
| $\emptyset_v V_n =$ | 119.28 t | | |
| $V_u$ | $\leq$ | $\emptyset_v V_n$ | ✓ |
| 58.16 t | $\leq$ | 119.28 t | |

- **Deflection Check:**

| Check for Deflection (Δ) |
|---|
| $\Delta_{max} = \frac{L}{240} = \frac{12000}{240} = $ 50 mm |
| $\Delta_{act} = \frac{W_u \times L^4}{384 \times E \times I_x} = \frac{9.69 \times 12^4}{384 \times 20389019.16 \times 0.001072} = $ 24 mm |
| $\Delta_{act} < \Delta_{max}$ |
| 24 mm < 50 mm ✓ |

Accordingly, the selected steel section for the mezzanine beam is HE 500 B. This section is safe against moment, shear, and deflection limit.

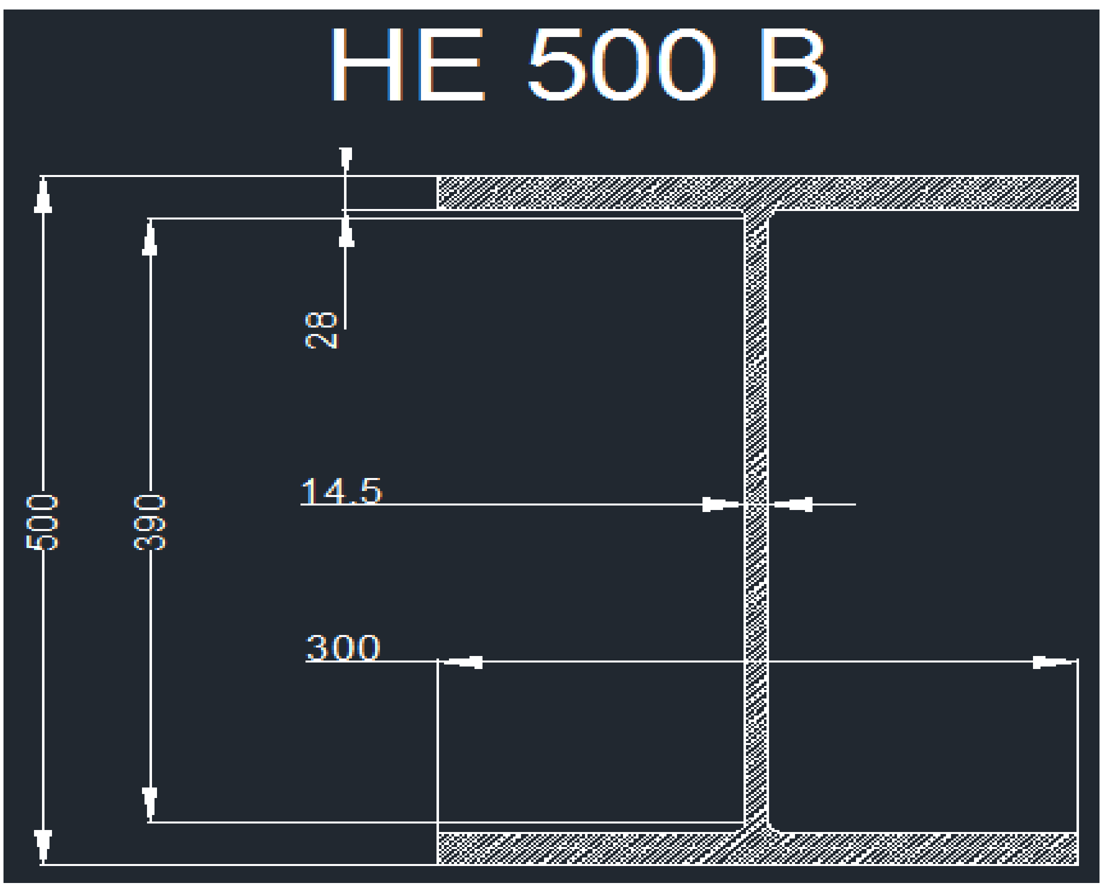



**Figure 45 Steel section properties (HE 500 B)**

## ETABS 2015 Steel Frame Design

## (Mezzanine floor beam)

## AISC 360-10 Steel Section Check (Strength Summary)

**Element Details**

| Level | Element | Location (m) | Combo | Element Type | Section | Classification |
|---|---|---|---|---|---|---|
| Mezzanine | B60 | 11.8 | DStlS2 | Special Moment Frame | HE500B | Compact |

**LLRF and Demand/Capacity Ratio**

| L (m) | LLRF | Stress Ratio Limit |
|---|---|---|
| 12.00000 | 1 | 0.95 |

**Analysis and Design Parameters**

| Provision | Analysis | 2nd Order | Reduction |
|---|---|---|---|
| LRFD | Direct Analysis | General 2nd Order | Tau-b Fixed |

**Stiffness Reduction Factors**

| $\alpha P_r / P_y$ | $\alpha P_r / P_e$ | $\tau_b$ | EA factor | EI factor |
|---|---|---|---|---|
| 0 | 0 | 1 | 0.8 | 0.8 |

**Design Code Parameters**

| $\Phi_b$ | $\Phi_c$ | $\Phi_{TY}$ | $\Phi_{TF}$ | $\Phi_V$ | $\Phi_{V-RI}$ | $\Phi_{VT}$ |
|---|---|---|---|---|---|---|
| 0.9 | 0.9 | 0.9 | 0.75 | 0.9 | 1 | 1 |

**Section Properties**

| A ($m^2$) | J ($m^4$) | $I_{33}$ ($m^4$) | $I_{22}$ ($m^4$) | $A_{v3}$ ($m^2$) | $A_{v2}$ ($m^2$) |
|---|---|---|---|---|---|
| 0.0239 | 0.000005 | 0.001072 | 0.000126 | 0.0168 | 0.0073 |

**Design Properties**

| $S_{33}$ ($m^3$) | $S_{22}$ ($m^3$) | $Z_{33}$ ($m^3$) | $Z_{22}$ ($m^3$) | $r_{33}$ (m) | $r_{22}$ (m) | $C_w$ ($m^6$) |
|---|---|---|---|---|---|---|
| 0.004288 | 0.000841 | 0.004815 | 0.001292 | 0.21179 | 0.07267 | 7.018E-06 |

**Material Properties**

| E (tonf/m²) | $f_y$ (tonf/m²) | $R_y$ | α |
|---|---|---|---|
| 20389019.16 | 35153.48 | 1.1 | NA |

**Stress Check forces and Moments**

| Location (m) | $P_u$ (tonf) | $M_{u33}$ (tonf-m) | $M_{u22}$ (tonf-m) | $V_{u2}$ (tonf) | $V_{u3}$ (tonf) | $T_u$ (tonf-m) |
|---|---|---|---|---|---|---|
| 11.8 | 0 | -84.99 | 0 | 40 | 0 | 0 |

**Axial Force & Biaxial Moment Design Factors (H1-1b)**

| | L Factor | $K_1$ | $K_2$ | $B_1$ | $B_2$ | $C_m$ |
|---|---|---|---|---|---|---|
| Major Bending | 0.967 | 1 | 1 | 1 | 1 | 1 |
| Minor Bending | 0.25 | 1 | 1 | 1 | 1 | 1 |

**Parameters for Lateral Torsion Buckling**

| $L_{ltb}$ | $K_{ltb}$ | $C_b$ |
|---|---|---|
| 0.25 | 1 | 1.109 |

**Demand/Capacity (D/C) Ratio Eqn. (H1-1b)**

| D/C Ratio = | $(P_r /2P_c ) + (M_{r33} /M_{c33} ) + (M_{r22} /M_{c22} )$ |
|---|---|
| 0.558 = | 0 + 0.558 + 0 |

**Axial Force and Capacities**

| $P_u$ Force (tonf) | $\phi P_{nc}$ Capacity (tonf) | $\phi P_{nt}$ Capacity (tonf) |
|---|---|---|
| 0 | 607.22 | 756.15 |

**Moments and Capacities**

| | $M_u$ Moment (tonf-m) | $\phi M_n$ Capacity (tonf-m) | $\phi M_n$ No $L_{TBD}$ (tonf-m) |
|---|---|---|---|
| Major Bending | 84.99 | 152.34 | 152.34 |
| Minor Bending | 0 | 40.88 | |

**Shear Design**

| | $V_u$ Force (tonf) | $\phi V_n$ Capacity (tonf) | Stress Ratio |
|---|---|---|---|
| Major Shear | 40 | 152.92 | 0.262 |
| Minor Shear | 0 | 318.91 | 0 |

**End Reaction Major Shear Forces**

| Left End Reaction (tonf) | Load Combo | Right End Reaction (tonf) | Load Combo |
|---|---|---|---|
| 15.58 | DStlS2 | 40 | DStlS2 |

**DEFLECTION DESIGN (Combo DStlD2)**

| Type | Consider | Deflection m | Limit m | Ratio | Status |
|---|---|---|---|---|---|
| Dead Load | Yes | 0.00775 | 0.1 | 0.077 | OK |
| Super DL + Live Load | Yes | 0.01544 | 0.1 | 0.154 | OK |
| Live Load | Yes | 0.01544 | 0.03333 | 0.463 | OK |
| Total Load | Yes | 0.02319 | 0.05 | 0.464 | OK |
| Total - Camber | Yes | 0.02319 | 0.05 | 0.464 | OK |

### 3.4.2 Roof beam design (3DE):

- **Properties:**

**Table 19 Steel section properties (HE 400 A)**

| Section Property (HE400A) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $A_g$ | 0.0159 | $m^2$ | $S_x$ | 0.002311 | $m^3$ | $C_w$ | 0.000000022 | $m^6$ |
| b | 0.3 | m | $S_y$ | 0.000571 | $m^3$ | J | 0.000002 | $m^4$ |
| d | 0.298 | m | $r_x$ | 0.168 | m | L | 9 | m |
| $h_o$ | 0.32 | m | $r_y$ | 0.0734 | m | $Ø_b$ | 0.9 | - |
| h | 0.39 | m | $t_w$ | 0.011 | m | $Ø_v$ | 1 | - |
| $I_x$ | 0.0004507 | $m^4$ | $t_f$ | 0.019 | m | $K_v$ | 5 | - |
| $I_y$ | 0.0000856 | $m^4$ | $F_y$ | 35153.48 | $t/m^2$ | $C_b$ | 1.14 | - |
| $Z_x$ | 0.002562 | $m^3$ | $F_u$ | 45700 | $t/m^2$ | $C_v$ | 1 | - |
| $Z_y$ | 0.00873 | $m^3$ | E | 20389019.16 | $t/m^2$ | C | 1 | - |
| Steel Section | | | ETABS Software | | | Building Code | | |

**Note:**

The date that are shown in blue are taken from the steel section properties, European Specifications Beams (EURONORM).

The data that are shown in green are taken from the ETABS software during the modeling phase.

The data that are shown in red are taken from the American Building Code according to its equations.

- **Moment design:**

**1- Lateral-Torsional Bukling**

$$L_b = L - [(0.211 \times L) \times 2]$$

$$L_b = 9 - [(0.211 \times 9) \times 2] = 5.20 \text{ m}$$

$$L_p = 1.76 \times r_y \times \sqrt{\frac{E}{F_y}}$$

$$L_p = 1.76 \times 0.0734 \times \sqrt{\frac{20389019}{35153.48}} = 3.11 \text{ m}$$

$$L_r = 1.95 \times r_{ts} \times \frac{E}{0.7 \times F_y} \times \sqrt{\frac{J \times c}{S_x \times h_o} + \sqrt{\left(\frac{J \times c}{S_x \times h_o}\right)^2 + 6.76\left(\frac{0.7 \times F_y}{E}\right)^2}}$$

$$L_r = 1.95 \times 0 \times \frac{20389019.16}{0.7 \times 35153.48} \times \sqrt{\frac{0.000002 \times 1}{0.002311 \times 0.3} + \sqrt{\left(\frac{0.000002 \times 1}{0.002311 \times 0.32}\right)^2 + 6.76\left(\frac{0.7 \times 35153.48}{20389019.16}\right)^2}} = 3.26 \text{ m}$$

$$r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} =$$

$$r_{ts} = \sqrt{\frac{\sqrt{0.0000856 \times 0.000000022}}{0.002311}} = 0.02 \text{ m}$$

**2- The Case for Lateral-Torsional Bukling**

| | | | | | | |
|---|---|---|---|---|---|---|
| a) $L_b \leq L_p$ | 5.20 m | ≤ | 3.11 m | | | ✘ |
| b) $L_p < L_b \leq L_r$ | 3.11 m | < | 5.20 m | ≤ | 3.26 m | ✘ |
| C) $L_b > L_r$ | 5.20 m | > | 3.26 m | | | ✔ |

## 3- Find Buckling Stress

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \times \sqrt{1 + 0.078 \times \frac{Jc}{S_x h_o} \times \left(\frac{L_b}{r_{ts}}\right)^2}$$

$$F_{cr} = \frac{1.14 \times 3.142^2 \times 20389019.16}{\left(\frac{5.20}{0.02}\right)^2} \times \sqrt{1 + 0.078 \times \frac{0.000002}{0.002311} \times \frac{1}{0.32} \times \left(\frac{5.20}{0.02}\right)^2} = 16399.60 \quad t/m^2$$

## 4- Nominal Moment

$$M_p = F_y \times Z_x = 35153.48 \times 0.00256 = 90.06 \quad t/m$$

$$M_n = F_{cr} \times S_x = 16399.60 \times 0.00231 = 37.90 \ t/m$$

## 5- Check for the Moment

| | | | | |
|---|---|---|---|---|
| $M_n \leq M_p$ | $M_n$ | $\leq$ | $M_p$ | |
| | 37.90 t/m | $\leq$ | 90 t/m | ✓ |

- **Shear design:**

## 1- Check for Web Shear Conditions ($C_v$)

$$\frac{h}{t_w} = \frac{0.39}{0.011} = 35.45$$

$$1.10\sqrt{\frac{k_v \times E}{F_y}}$$

$$1.10 \times \sqrt{\frac{5 \times 20389019.16}{35153.48}} = 59.24$$

$$\frac{h}{t_w} \leq 1.10\sqrt{\frac{k_v \times E}{F_y}}$$

$$35.45 \leq 59.24$$ ✓

## 2- Find Web Area ($A_w$)

$$A_w = d \times t_w = 0.298 \times 0.011 = 0.003278 \ m^2$$

## 3- Find Nominal Shear (Vn)

$$V_n = 0.6 \times F_y \times A_w \times C_v$$

$$V_n = 0.6 \times 35153.48 \times 0.003278 \times 1 = 69.14 \text{ t}$$

## 4- Check for Shear

$$\emptyset_v V_n = 69.14 \text{ t}$$

| $V_u$ | $\leq$ | $\emptyset_v V_n$ | |
|---|---|---|---|
| 32.54 t | $\leq$ | 69.14 t | ✓ |

- **Deflection Check:**

**Check for Deflection (Δ)**

$$\Delta_{max} = \frac{L}{240} = \frac{9}{240} = 38\ \text{mm}$$

$$\Delta_{act} = \frac{W_u \times L^4}{384 \times E \times I_x} = \frac{7.23 \times 9^4}{384 \times 20389019.16 \times 0.0004507} = 13\ \text{mm}$$

$$\Delta_{act} < \Delta_{max}$$

$$13\ \text{mm} < 38\ \text{mm}\ ✓$$

Accordingly, the selected steel section for the roof beam is HE 400 A. This section is safe against moment, shear, and deflection limit.

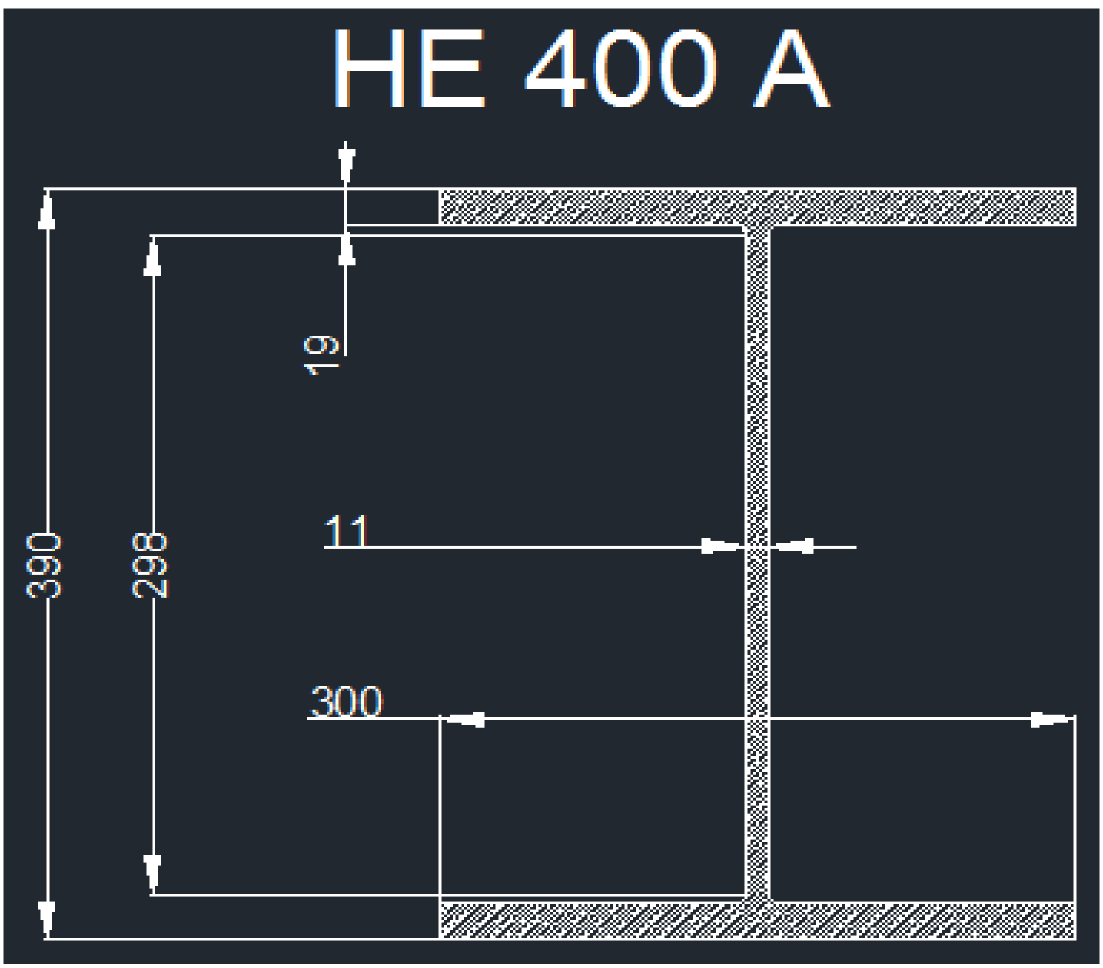



**Figure 46 Steel section properties (HE 400 A)**

## ETABS 2015 Steel Frame Design

## (Roof beam)

## AISC 360-10 Steel Section Check (Strength Summary)

**Element Details**

| Level | Element | Location (m) | Combo | Element Type | Section | Classification |
|---|---|---|---|---|---|---|
| Mezzanine | B51 | 8.8 | DStlS2 | Special Moment Frame | HE400A | Compact |

**LLRF and Demand/Capacity Ratio**

| L (m) | LLRF | Stress Ratio Limit |
|---|---|---|
| 9.00000 | 1 | 0.95 |

**Analysis and Design Parameters**

| Provision | Analysis | 2nd Order | Reduction |
|---|---|---|---|
| LRFD | Direct Analysis | General 2nd Order | Tau-b Fixed |

**Stiffness Reduction Factors**

| $\alpha P_r / P_y$ | $\alpha P_r / P_e$ | $\tau_b$ | EA factor | EI factor |
|---|---|---|---|---|
| 0 | 0 | 1 | 0.8 | 0.8 |

**Design Code Parameters**

| $\Phi_b$ | $\Phi_c$ | $\Phi_{TY}$ | $\Phi_{TF}$ | $\Phi_V$ | $\Phi_{V-RI}$ | $\Phi_{VT}$ |
|---|---|---|---|---|---|---|
| 0.9 | 0.9 | 0.9 | 0.75 | 0.9 | 1 | 1 |

**Section Properties**

| A (m²) | J (m⁴) | $I_{33}$ (m⁴) | $I_{22}$ (m⁴) | $A_{v3}$ (m²) | $A_{v2}$ (m²) |
|---|---|---|---|---|---|
| 0.0159 | 0.000002 | 0.000451 | 0.000086 | 0.0114 | 0.0043 |

**Design Properties**

| $S_{33}$ (m³) | $S_{22}$ (m³) | $Z_{33}$ (m³) | $Z_{22}$ (m³) | $r_{33}$ (m) | $r_{22}$ (m) | $C_w$ (m⁶) |
|---|---|---|---|---|---|---|
| 0.002311 | 0.000571 | 0.002562 | 0.000873 | 0.16836 | 0.07339 | 0 |

**Material Properties**

| E ($tonf/m^2$) | $f_y$ ($tonf/m^2$) | $R_y$ | α |
|---|---|---|---|
| 20389019.16 | 35153.48 | 1.1 | NA |

**Stress Check forces and Moments**

| Location (m) | $P_u$ (tonf) | $M_{u33}$ (tonf-m) | $M_{u22}$ (tonf-m) | $V_{u2}$ (tonf) | $V_{u3}$ (tonf) | $T_u$ (tonf-m) |
|---|---|---|---|---|---|---|
| 8.8 | 0 | -38.5 | 0 | 23.49 | 0 | 0 |

**Axial Force & Biaxial Moment Design Factors (H1-1b)**

| | L Factor | $K_1$ | $K_2$ | $B_1$ | $B_2$ | $C_m$ |
|---|---|---|---|---|---|---|
| Major Bending | 0.956 | 1 | 1 | 1 | 1 | 1 |
| Minor Bending | 0.333 | 1 | 1 | 1 | 1 | 1 |

**Parameters for Lateral Torsion Buckling**

| $L_{ltb}$ | $K_{ltb}$ | $C_b$ |
|---|---|---|
| 0.333 | 1 | 1.014 |

**Demand/Capacity (D/C) Ratio Eqn. (H1-1b)**

| D/C Ratio = | $(P_r / 2P_c) + (M_{r33} / M_{c33}) + (M_{r22} / M_{c22})$ |
|---|---|
| 0.475 = | 0 + 0.475 + 0 |

**Axial Force and Capacities**

| $P_u$ Force (tonf) | $\phi P_{nc}$ Capacity (tonf) | $\phi P_{nt}$ Capacity (tonf) |
|---|---|---|
| 0 | 415.68 | 503.05 |

**Moments and Capacities**

| | $M_u$ Moment (tonf-m) | $\phi M_n$ Capacity (tonf-m) | $\phi M_n$ No $L_{TBD}$ (tonf-m) |
|---|---|---|---|
| Major Bending | 38.5 | 81.06 | 81.06 |
| Minor Bending | 0 | 27.62 | |

**Shear Design**

| | $V_u$ Force (tonf) | $\phi V_n$ Capacity (tonf) | Stress Ratio |
|---|---|---|---|
| Major Shear | 23.49 | 90.49 | 0.26 |
| Minor Shear | 0 | 216.4 | 0 |

**End Reaction Major Shear Forces**

| Left End Reaction (tonf) | Load Combo | Right End Reaction (tonf) | Load Combo |
|---|---|---|---|
| 23.49 | DStlS2 | 23.49 | DStlS2 |

**DEFLECTION DESIGN (Combo DStlD2)**

| Type | Consider | Deflection m | Limit m | Ratio | Status |
|---|---|---|---|---|---|
| Dead Load | Yes | 0.00692 | 0.075 | 0.092 | OK |
| Super DL + Live Load | Yes | 0.00816 | 0.075 | 0.109 | OK |
| Live Load | Yes | 0.00816 | 0.025 | 0.327 | OK |
| Total Load | Yes | 0.01509 | 0.0375 | 0.402 | OK |
| Total - Camber | Yes | 0.01509 | 0.0375 | 0.402 | OK |

### 3.4.3 Dome beam design (3BD):

- **Properties:**

**Table 20 Steel section properties (IPE 450)**

| Section Property (IPE 450) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $A_g$ | 0.00988 | $m^2$ | $S_x$ | 0.0015 | $m^3$ | $C_w$ | 0.00000039 | $m^6$ |
| b | 0.19 | m | $S_y$ | 0.000176 | $m^3$ | J | 0.000001 | $m^4$ |
| d | 0.379 | m | $r_x$ | 0.185 | m | L | 6 | m |
| $h_o$ | 0.394 | m | $r_y$ | 0.0412 | m | $Ø_b$ | 0.9 | - |
| h | 0.45 | m | $t_w$ | 0.0094 | m | $Ø_v$ | 1 | - |
| $I_x$ | 0.0003374 | $m^4$ | $t_f$ | 0.0146 | m | $K_v$ | 5 | - |
| $I_y$ | 0.00001676 | $m^4$ | $F_y$ | 35153.48 | $t/m^2$ | $C_b$ | 1.195 | - |
| $Z_x$ | 0.001702 | $m^3$ | $F_u$ | 45700 | $t/m^2$ | $C_v$ | 0.43 | - |
| $Z_y$ | 0.000276 | $m^3$ | E | 20389019.16 | $t/m^2$ | C | 1 | - |
| | Steel Section | | | ETABS Software | | | Building Code | |

**Note:**

The date that are shown in blue are taken from the steel section properties, European Specifications Beams (EURONORM).

The data that are shown in green are taken from the ETABS software during the modeling phase.

The data that are shown in red are taken from the American Building Code according to its equations.

- **Moment design:**

| 1- Lateral-Torsional Bukling |
|---|
| $L_b = L - [(0.211 \times L) \times 2]$<br>$L_b = 6 - [(0.211 \times 6) \times 2] = 3.47\ m$ |
| $L_p = 1.76 \times r_y \times \sqrt{\frac{E}{F_y}}$<br>$L_p = 1.76 \times 0.0412 \times \sqrt{\frac{20389019}{35153.48}} = 1.75\ m$ |
| $L_r = 1.95 \times r_{ts} \times \frac{E}{0.7 \times F_y} \times \sqrt{\frac{J \times c}{S_x \times h_o} + \sqrt{\left(\frac{J \times c}{S_x \times h_o}\right)^2 + 6.76\left(\frac{0.7 \times F_y}{E}\right)^2}}$<br>$L_r = 1.95 \times 0.041 \times \frac{20389019.16}{0.7 \times 35153.48} \times \sqrt{\frac{1E\text{-}06 \times 1}{0.0015 \times 0.4} + \sqrt{\left(\frac{1E\text{-}06 \times 1}{0.0015 \times 0.394}\right)^2 + 6.76\left(\frac{0.7 \times 35153.48}{20389019.16}\right)^2}} = 4.84\ m$ |
| $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} =$<br>$r_{ts} = \sqrt{\frac{\sqrt{1.7E\text{-}05 \times 0.00000039}}{0.0015}} = 0.041\ m$ |

| 2- The Case for Lateral-Torsional Bukling | | | | | | |
|---|---|---|---|---|---|---|
| a) $L_b \leq L_p$ | 3.47 m | $\leq$ | 1.75 m | | | ✘ |
| b) $L_p < L_b \leq L_r$ | 1.75 m | $<$ | 3.47 m | $\leq$ | 4.84 m | ✔ |
| C) $L_b > L_r$ | 3.47 m | $>$ | 4.84 m | | | ✘ |

| 3- Nominal Moment |
|---|

$M_p = F_y \times Z_x = 35153.48 \times 0.0017 = 59.83 \quad t/m$

$\boxed{M_n \le M_p}$

$$M_n = C_b\left[M_p - (M_p - 0.7F_yS_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right]$$

$$M_n = 1.20 \times \left[59.83 - (59.83 - 0.7 \times 35153.48 \times 0.0015)\left(\frac{3.47 - 1.75}{4.84 - 1.75}\right)\right] = 56.24 \ t/m$$

| $M_n$ | $\le$ | $M_p$ | |
|---|---|---|---|
| 56.24 | $\le$ | 59.83 | ✓ |

| 4- Check for the Moment | | | | |
|---|---|---|---|---|
| $M_u =$ 26.0 t/m | $M_u$ | $\le$ | $\emptyset_b M_n$ | $\boxed{M_u \le \emptyset_b M_n}$ |
| $\emptyset_b M_n =$ 50.62 t/m | 26.0 t/m | $\le$ | 50.62 t/m | ✓ |

- **Shear design:**

| 1- Check for Web Shear Conditions ($C_v$) |
| --- |
| $$\frac{h}{t_w} = \frac{0.45}{0.0094} = 47.87$$ $$1.37 \times \sqrt{\frac{k_v \times E}{F_y}}$$ $$1.37 \times \sqrt{\frac{5 \times 20389019.16}{35153.48}} = 73.78$$ $$\frac{h}{t_w} > 1.37\sqrt{\frac{k_v \times E}{F_y}}$$ $$47.87 > 73.78$$ ✓ |

| 2- Find the Value of Web Shear According to the Conditions ($C_v$) |
| --- |
| $$C_v = \frac{1.51 \times E \times K_v}{\left(\frac{h}{t_w}\right)^2 \times F_y} =$$ $$C_v = \frac{1.51 \times 20389019.16 \times 5}{\left(\frac{0.45}{0.0094}\right)^2 \times 35153.48} = 1.91$$ |

## 3- Find Web Area ($A_w$)

$$A_w = d \times t_w = 0.379 \times 0.0094 = 0.00356 \quad m^2$$

## 4- Find Nominal Shear ($V_n$)

$$V_n = 0.6 \times F_y \times A_w \times \left( C_v + \frac{1 - C_v}{1.15 \times \sqrt{1 + \left(\frac{a}{h}\right)^2}} \right) =$$

$$V_n = 0.6 \times 35153.48 \times 0.00356 \times \left( 0.43 + \frac{0.57}{1.15 \times \sqrt{1 + \left(\frac{0}{0.45}\right)^2}} \right) = 69.56 \text{ t}$$

## 5- Check for Shear

$$\emptyset_v \times V_n = 1 \times 69.56 = 69.56 \text{ t}$$

| | | | |
|---|---|---|---|
| $V_u$ | $\leq$ | $\emptyset_v V_n$ | ✓ |
| 20.88 t | $\leq$ | 69.56 t | |

- **Deflection Check:**

Check for Deflection (Δ)

$$\Delta_{max} = \frac{L}{240} = \frac{6000}{240} = 25 \text{ mm}$$

$$\Delta_{act} = \frac{W_u \times L^4}{384 \times E \times I_x} + \frac{P \times L^3}{192 \times E \times I_x}$$

$$\Delta_{act} = \frac{3.54 \times 6^4}{384 \times 20389019.16 \times 0.0002313} + \frac{20.52 \times 6^3}{192 \times 20389019.16 \times 0.0002313} = 7.4 \text{ mm}$$

$$\Delta_{act} < \Delta_{max}$$

$$7.4 \text{ mm} < 25 \text{ mm}$$

Accordingly, the selected steel section for the dome beam is IPE 450. This section is safe against moment, shear, and deflection limit.

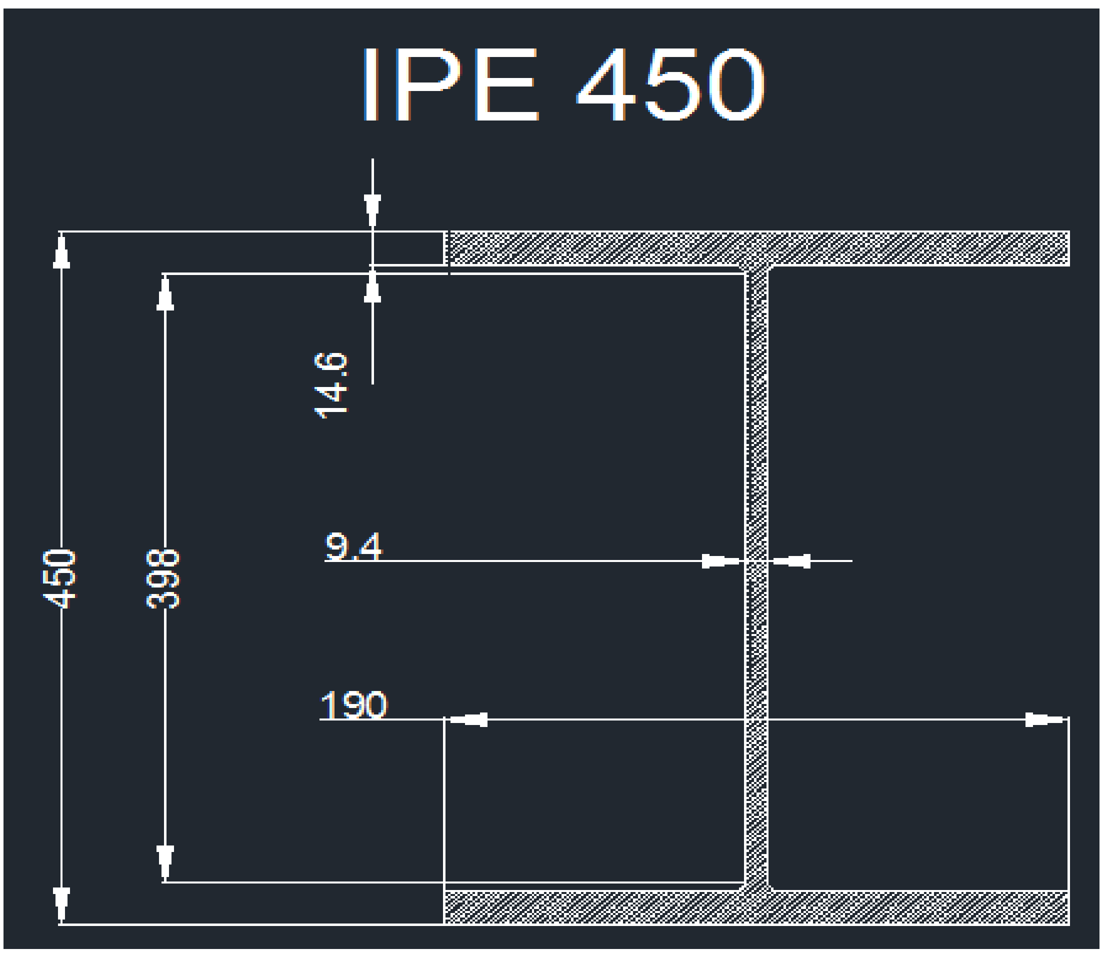



**Figure 47 Steel section properties (IPE 450)**

**ETABS 2015 Steel Frame Design**

**(Dome beam)**

**AISC 360-10 Steel Section Check (Strength Summary)**

**Element Details**

| Level | Element | Location (m) | Combo | Element Type | Section | Classification |
|---|---|---|---|---|---|---|
| Mezzanine | B39 | 3 | DStlS2 | Special Moment Frame | IPE450 | Compact |

**LLRF and Demand/Capacity Ratio**

| L (m) | LLRF | Stress Ratio Limit |
|---|---|---|
| 6.00000 | 1 | 0.95 |

**Analysis and Design Parameters**

| Provision | Analysis | 2nd Order | Reduction |
|---|---|---|---|
| LRFD | Direct Analysis | General 2nd Order | Tau-b Fixed |

**Stiffness Reduction Factors**

| $\alpha P_r / P_y$ | $\alpha P_r / P_e$ | $\tau_b$ | EA factor | EI factor |
|---|---|---|---|---|
| 0 | 0 | 1 | 0.8 | 0.8 |

**Design Code Parameters**

| $\Phi_b$ | $\Phi_c$ | $\Phi_{TY}$ | $\Phi_{TF}$ | $\Phi_V$ | $\Phi_{V-RI}$ | $\Phi_{VT}$ |
|---|---|---|---|---|---|---|
| 0.9 | 0.9 | 0.9 | 0.75 | 0.9 | 1 | 1 |

**Section Properties**

| A ($m^2$) | J ($m^4$) | $I_{33}$ ($m^4$) | $I_{22}$ ($m^4$) | $A_{v3}$ ($m^2$) | $A_{v2}$ ($m^2$) |
|---|---|---|---|---|---|
| 0.0099 | 0.000001 | 0.000337 | 0.000017 | 0.0055 | 0.0042 |

**Design Properties**

| $S_{33}$ ($m^3$) | $S_{22}$ ($m^3$) | $Z_{33}$ ($m^3$) | $Z_{22}$ ($m^3$) | $r_{33}$ (m) | $r_{22}$ (m) | $C_w$ ($m^6$) |
|---|---|---|---|---|---|---|
| 0.0015 | 0.000176 | 0.001702 | 0.000276 | 0.1848 | 0.04119 | 0 |

**Material Properties**

| E (tonf/$m^2$) | $f_y$ (tonf/$m^2$) | $R_y$ | α |
|---|---|---|---|
| 20389019.16 | 35153.48 | 1.1 | NA |

**Stress Check forces and Moments**

| Location (m) | $P_u$ (tonf) | $M_{u33}$ (tonf-m) | $M_{u22}$ (tonf-m) | $V_{u2}$ (tonf) | $V_{u3}$ (tonf) | $T_u$ (tonf-m) |
|---|---|---|---|---|---|---|
| 3 | 0 | 22.18 | 0 | 0 | 0 | 1.497E-03 |

**Axial Force & Biaxial Moment Design Factors (H1.3b, H1-2)**

| | L Factor | $K_1$ | $K_2$ | $B_1$ | $B_2$ | $C_m$ |
|---|---|---|---|---|---|---|
| Major Bending | 1 | 1 | 1 | 1 | 1 | 1 |
| Minor Bending | 1 | 1 | 1 | 1 | 1 | 1 |

**Parameters for Lateral Torsion Buckling**

| $L_{ltb}$ | $K_{ltb}$ | $C_b$ |
|---|---|---|
| 1 | 1 | 1.136 |

**Demand/Capacity (D/C) Ratio Eqn. (H1.3b, H1-2)**

| D/C Ratio = | $(P_r / P_c) + (M_{r33} / M_{c33})^2 + (M_{r22} / M_{c22})$ |
|---|---|
| 0.521 = | 0 + 0.521 + 0 |

**Axial Force and Capacities**

| $P_u$ Force (tonf) | $\phi P_{nc}$ Capacity (tonf) | $\phi P_{nt}$ Capacity (tonf) |
|---|---|---|
| 0 | 73.95 | 312.58 |

**Moments and Capacities**

| | $M_u$ Moment (tonf-m) | $\phi M_n$ Capacity (tonf-m) | $\phi M_n$ No $L_{TBD}$ (tonf-m) |
|---|---|---|---|
| Major Bending | 22.18 | 30.74 | 53.85 |
| Minor Bending | 0 | 8.73 | |

**Shear Design**

| | $V_u$ Force (tonf) | $\phi V_n$ Capacity (tonf) | Stress Ratio |
|---|---|---|---|
| Major Shear | 0 | 89.22 | 0 |
| Minor Shear | 0 | 105.32 | 0 |

**End Reaction Major Shear Forces**

| Left End Reaction (tonf) | Load Combo | Right End Reaction (tonf) | Load Combo |
|---|---|---|---|
| 14.79 | DStlS2 | 14.79 | DStlS2 |

**DEFLECTION DESIGN (Combo DStlD2)**

| Type | Consider | Deflection m | Limit m | Ratio | Status |
|---|---|---|---|---|---|
| Dead Load | Yes | 0.00514 | 0.05 | 0.103 | OK |
| Super DL + Live Load | Yes | 0.00366 | 0.05 | 0.073 | OK |
| Live Load | Yes | 0.00366 | 0.01667 | 0.22 | OK |
| Total Load | Yes | 0.0088 | 0.025 | 0.352 | OK |
| Total - Camber | Yes | 0.0088 | 0.025 | 0.352 | OK |

### 3.4.4 Column design (2C):

- **Properties:**

**Table 21 Steel section properties (HE 400 B)**

| Section Property (HE400B) | | | | | | | | |
|---|---|---|---|---|---|---|---|---|
| $A_g$ | 0.0198 | $m^2$ | $S_x$ | 0.002884 | $m^3$ | $C_w$ | 0.000000022 | $m^6$ |
| b | 0.3 | m | $S_y$ | 0.000721 | $m^3$ | J | 0.000002 | $m^4$ |
| d | 0.298 | m | $r_x$ | 0.171 | m | L | 7 | m |
| $h_o$ | 0.32 | m | $r_y$ | 0.074 | m | $Ø_b$ | 0.9 | - |
| h | 0.39 | m | $t_w$ | 0.0135 | m | $Ø_v$ | 1 | - |
| $I_x$ | 0.0005768 | $m^4$ | $t_f$ | 0.024 | m | $K_v$ | 5 | - |
| $I_y$ | 0.00001082 | $m^4$ | $F_y$ | 35153.48 | $t/m^2$ | $C_b$ | 1.14 | - |
| $Z_x$ | 0.003232 | $m^3$ | $F_u$ | 45700 | $t/m^2$ | K | 0.65 | - |
| $Z_y$ | 0.001104 | $m^3$ | E | 20389019.16 | $t/m^2$ | C | 1 | - |
| | Steel Section | | | Etabs Software | | | Building Code | |

**Note:**

The date that are shown in blue are taken from the steel section properties, European Specifications Beams (EURONORM).

The data that are shown in green are taken from the ETABS 2015 software during the modeling phase.

The data that are shown in red are taken from the American Building Code according to its equations.

- **Moment design:**

| 1- Lateral-Torsional Bukling |
|---|
| $L_b = L - [(0.211 \times L) \times 2]$<br>$L_b = 7 - [(0.211 \times 7) \times 2] = 4.05\ m$ |
| $L_p = 1.76 \times r_y \times \sqrt{\frac{E}{F_y}}$<br>$L_p = 1.76 \times 0.074 \times \sqrt{\frac{20389019}{35153.48}} = 3.14\ m$ |
| $L_r = 1.95 \times r_{ts} \times \frac{E}{0.7 \times F_y} \times \sqrt{\frac{J \times c}{S_x \times h_o} + \sqrt{\left(\frac{J \times c}{S_x \times h_o}\right)^2 + 6.76\left(\frac{0.7 \times F_y}{E}\right)^2}}$<br>$L_r = 1.95 \times 0 \times \frac{20389019.16}{0.7 \times 35153.48} \times \sqrt{\frac{0.000002 \times 1}{0.002884 \times 0.3} + \sqrt{\left(\frac{0.000002 \times 1}{0.002884 \times 0.32}\right)^2 + 6.76\left(\frac{0.7 \times 35153.48}{20389019.16}\right)^2}} = 1.63\ m$ |
| $r_{ts} = \sqrt{\frac{\sqrt{I_y C_w}}{S_x}} =$<br>$r_{ts} = \sqrt{\frac{\sqrt{1.082E\text{-}05 \times 0.000000022}}{0.002884}} = 0.01\ m$ |

| 2- The Case for Lateral-Torsional Bukling | | | | | | |
|---|---|---|---|---|---|---|
| $a)\ L_b \le L_p$ | 4.05 m | $\leq$ | 3.14 m | | | ✘ |
| $b)\ L_p < L_b \le L_r$ | 3.14 m | $<$ | 4.05 m | $\leq$ | 1.63 m | ✘ |
| $C)\ L_b > L_r$ | 4.05 m | $>$ | 1.63 m | | | ✓ |

| 3- Find Buckling Stress |
|---|

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2} \times \sqrt{1 + 0.078 \times \frac{Jc}{S_x h_o} \times \left(\frac{L_b}{r_{ts}}\right)^2}$$

$$F_{cr} = \frac{1.14 \times 3.142^2 \times 20389019.16}{\left(\frac{4.05}{0.01}\right)^2} \times \sqrt{1 + 0.078 \times \frac{0.000002 \times 1}{0.002884 \times 0.32} \times \left(\frac{4.05}{0.01}\right)^2} = 9876.82 \quad t/m^2$$

| 4- Nominal Moment |
|---|

$$M_p = F_y \times Z_x = 35153.48 \times 0.00323 = 113.62 \quad t/m$$

$$M_n = F_{cr} \times S_x = 9876.82 \times 0.00288 = 28.48\ t/m$$

| 5- Check for the Moment | | | | |
|---|---|---|---|---|
| $\emptyset_b \times M_n \leq M_p$ | $\emptyset_b \times M_n$ | $\leq$ | $M_p$ | ✓ |
| | 0.9 × 37.90 | $\leq$ | 113.62 | |
| | 25.64 | $\leq$ | 113.62 | |

Compression buckling design is required for the column to know the bearing capacity of column.

## 1- Cases for Flexural Buckling Stress

$$1)\ \frac{KL}{r_x} \leq 4.71 \times \sqrt{\frac{E}{F_y}}$$

$$2)\ \frac{KL}{r_x} > 4.71 \times \sqrt{\frac{E}{F_y}}$$

## 2- Caculate Buckling Stress

$$\frac{KL}{r_x} = \frac{0.65 \times 7}{0.171} = 26.61$$

$$4.71 \times \sqrt{\frac{E}{F_y}}$$

$$4.71 \times \sqrt{\frac{20389019.16}{35153.48}} = 113.43$$

## 3- Check for Buckling Stress

$$\frac{KL}{r_x} \leq 4.71 \times \sqrt{\frac{E}{F_y}}$$

$$26.61 \leq 113.43$$

**So It's the first case**

## 4- Elastic Critical Buckling Stress

$$F_e = \frac{\pi^2 \times E}{\left(\frac{KL}{r_x}\right)^2} = \frac{(\ 3.142\ )^2 \times\ 20389019.16}{\left(\frac{0.65\ \times\ 7}{0.171}\right)^2} = \quad 284228 \quad t/m^2$$

## 5- Buckling Stress

$$F_{cr} = \left[0.658^{\frac{F_y}{F_e}}\right] \times F_y =$$

$$F_{cr} = \left[0.658^{\frac{35153.48}{284228.44}}\right] \times \ 35153.48 \ = \ 33380.01 \quad t/m^2$$

## 6- Nominal Compressive Strength

$$P_n = F_{cr} \times A_g = \ 33380.01 \times 0.0198 \ = \ 660.92 \text{ t}$$

$$V_{max} \leq P_n$$

$$230.23 \leq 660.92$$

- **Deflection:**

Check for Deflection ($\Delta$)

$$\Delta_{max} = \frac{L}{240} = \frac{7}{240} = \quad 29.17 \text{ mm}$$

$$\Delta_{act} = \frac{W_u \times L^4}{384 \times E \times I_x} = \frac{32.89 \quad \times \quad 7^{\;4}}{384 \times 20389019.16 \times 0.000451} = \quad 22.38 \text{ mm}$$

$$\Delta_{act} \quad < \quad \Delta_{max}$$

$$22.38 \text{ mm} \quad < \quad 29.17 \text{ mm} \quad ✓$$

Accordingly, the selected steel section for the roof beam is HE 400 B. This section is safe against moment, shear, and deflection limit.

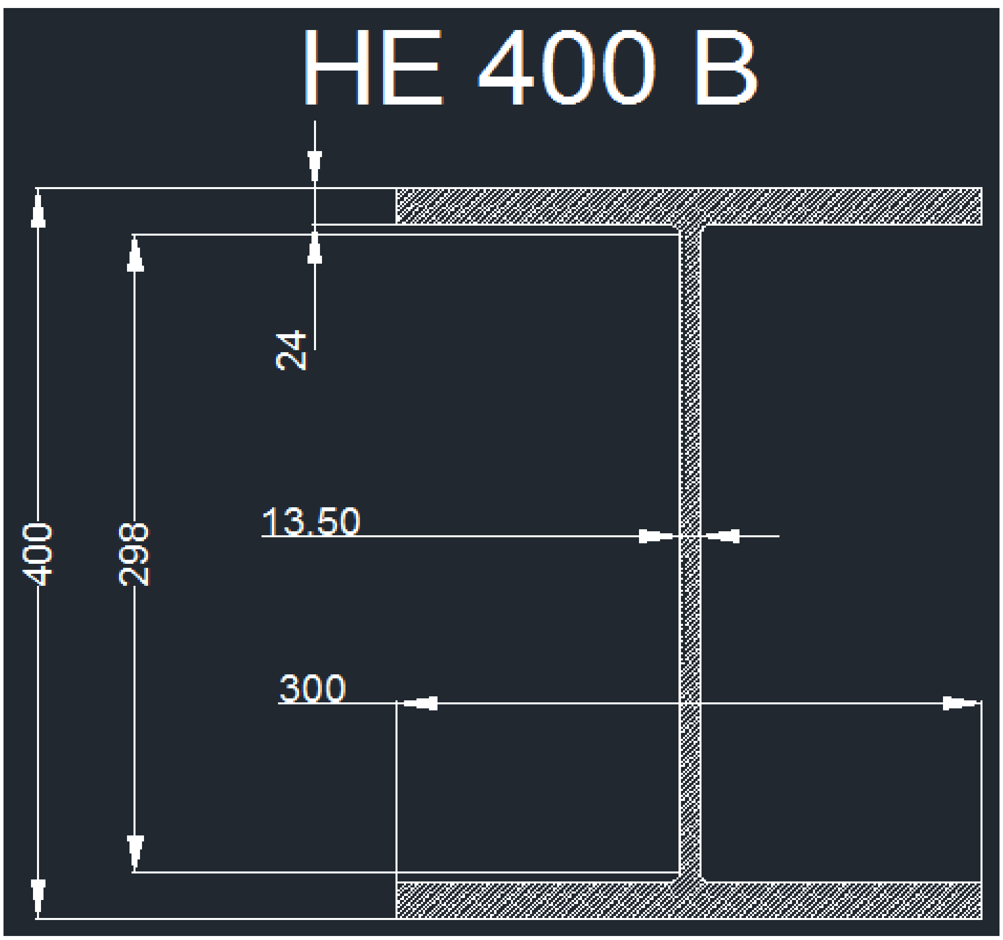



**Figure 48 Steel section properties (HE 400 B)**

**ETABS 2015 Steel Frame Design**

**(Column)**

**AISC 360-10 Steel Section Check (Strength Summary)**

**Element Details**

| Level | Element | Location (m) | Combo | Element Type | Section | Classification |
|---|---|---|---|---|---|---|
| Mezzanine | C28 | 6.5 | DStlS2 | Special Moment Frame | HE400B | Compact |

**LLRF and Demand/Capacity Ratio**

| L (m) | LLRF | Stress Ratio Limit |
|---|---|---|
| 7.00000 | 1 | 0.95 |

**Analysis and Design Parameters**

| Provision | Analysis | 2nd Order | Reduction |
|---|---|---|---|
| LRFD | Direct Analysis | General 2nd Order | Tau-b Fixed |

**Stiffness Reduction Factors**

| $\alpha P_r / P_y$ | $\alpha P_r / P_e$ | $\tau_b$ | EA factor | EI factor |
|---|---|---|---|---|
| 0 | 0 | 1 | 0.8 | 0.8 |

**Design Code Parameters**

| $\Phi_b$ | $\Phi_c$ | $\Phi_{TY}$ | $\Phi_{TF}$ | $\Phi_V$ | $\Phi_{V-RI}$ | $\Phi_{VT}$ |
|---|---|---|---|---|---|---|
| 0.9 | 0.9 | 0.9 | 0.75 | 0.9 | 1 | 1 |

**Section Properties**

| A ($m^2$) | J ($m^4$) | $I_{33}$ ($m^4$) | $I_{22}$ ($m^4$) | $A_{v3}$ ($m^2$) | $A_{v2}$ ($m^2$) |
|---|---|---|---|---|---|
| 0.0198 | 0.000004 | 0.000577 | 0.000108 | 0.0144 | 0.0054 |

**Design Properties**

| $S_{33}$ ($m^3$) | $S_{22}$ ($m^3$) | $Z_{33}$ ($m^3$) | $Z_{22}$ ($m^3$) | $r_{33}$ (m) | $r_{22}$ (m) | $C_w$ ($m^6$) |
|---|---|---|---|---|---|---|
| 0.002884 | 0.000721 | 0.003232 | 0.001104 | 0.17068 | 0.07392 | 0 |

**Material Properties**

| E (tonf/$m^2$) | $f_y$ (tonf/$m^2$) | $R_y$ | α |
|---|---|---|---|
| 20389019.16 | 35153.48 | 1.1 | NA |

**Stress Check forces and Moments**

| Location (m) | $P_u$ (tonf) | $M_{u33}$ (tonf-m) | $M_{u22}$ (tonf-m) | $V_{u2}$ (tonf) | $V_{u3}$ (tonf) | $T_u$ (tonf-m) |
|---|---|---|---|---|---|---|
| 6.5 | 0 | 0 | 0 | 0 | 0 | 0 |

**Axial Force & Biaxial Moment Design Factors (H1-1b)**

| | L Factor | $K_1$ | $K_2$ | $B_1$ | $B_2$ | $C_m$ |
|---|---|---|---|---|---|---|
| Major Bending | 0.929 | 1 | 1 | 1 | 1 | 1 |
| Minor Bending | 0.929 | 1 | 1 | 1 | 1 | 1 |

**Parameters for Lateral Torsion Buckling**

| $L_{ltb}$ | $K_{ltb}$ | $C_b$ |
|---|---|---|
| 0.929 | 1 | 1 |

**Demand/Capacity (D/C) Ratio Eqn. (H1-1b)**

| D/C Ratio = | $(P_r /2P_c ) + (M_{r33} /M_{c33} ) + (M_{r22} /M_{c22} )$ |
|---|---|
| 0 = | 0 + 0 + 0 |

**Axial Force and Capacities**

| $P_u$ Force (tonf) | $\phi P_{nc}$ Capacity (tonf) | $\phi P_{nt}$ Capacity (tonf) |
|---|---|---|
| 0 | 355.93 | 626.44 |

**Moments and Capacities**

| | $M_u$ Moment (tonf-m) | $\phi M_n$ Capacity (tonf-m) | $\phi M_n$ No $L_{TBD}$ (tonf-m) |
|---|---|---|---|
| Major Bending | 0 | 87.78 | 102.25 |
| Minor Bending | 0 | 34.93 | |

**Shear Design**

| | $V_u$ Force (tonf) | $\phi V_n$ Capacity (tonf) | Stress Ratio |
|---|---|---|---|
| Major Shear | 0 | 113.9 | 0 |
| Minor Shear | 0 | 273.35 | 0 |

**Joint Design**

| Continuity Plate Area ($m^2$) | Load Combo | Doubler (m) | Load Combo |
|---|---|---|---|
| 0.0026 | DStlS2 | 0.01476 | DStlS2 |

### 3.4.5 ETABS drawings:

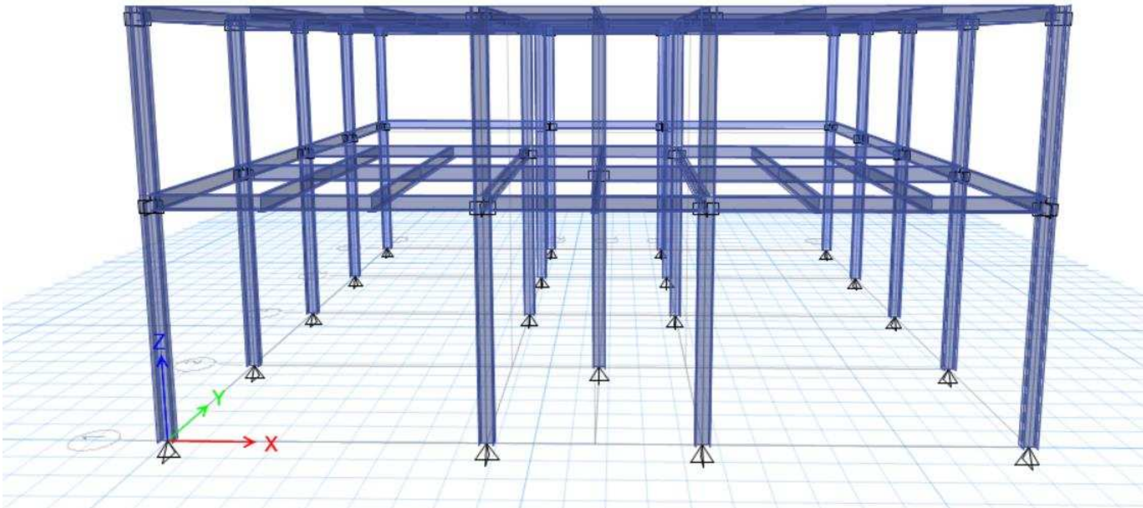



**Figure 49 ETABS 3D model**

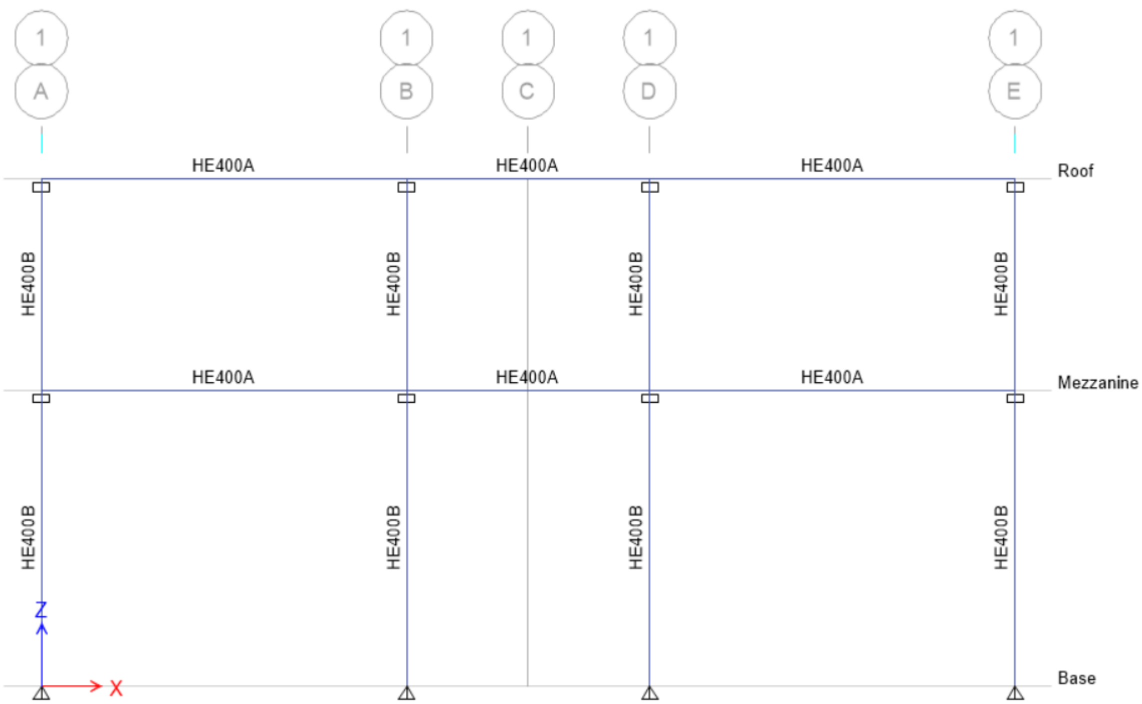



**Figure 50 Side view**

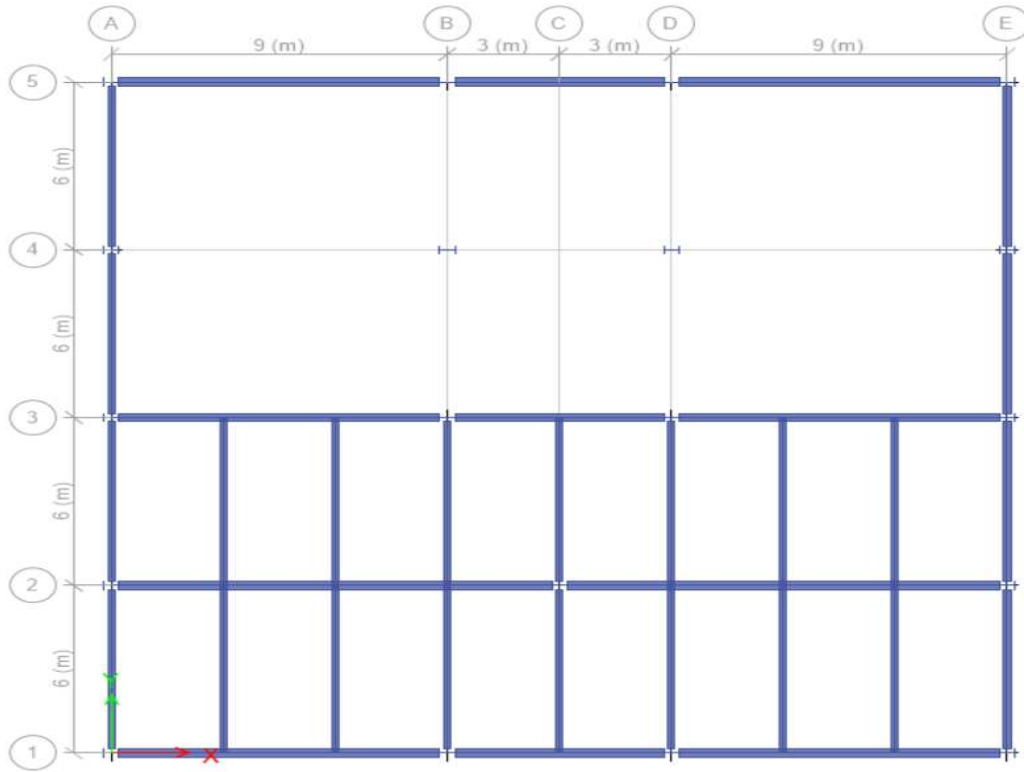



**Figure 51 Top view for mezzanine floor**

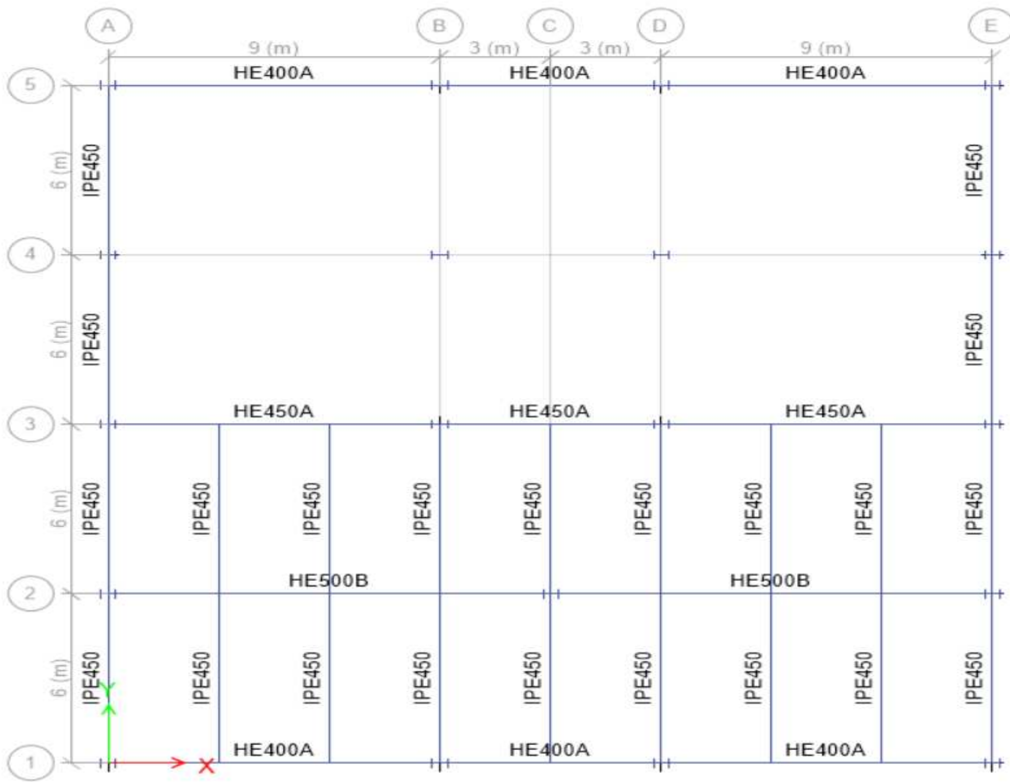



**Figure 52 Top view of sections for mezzanine floor**

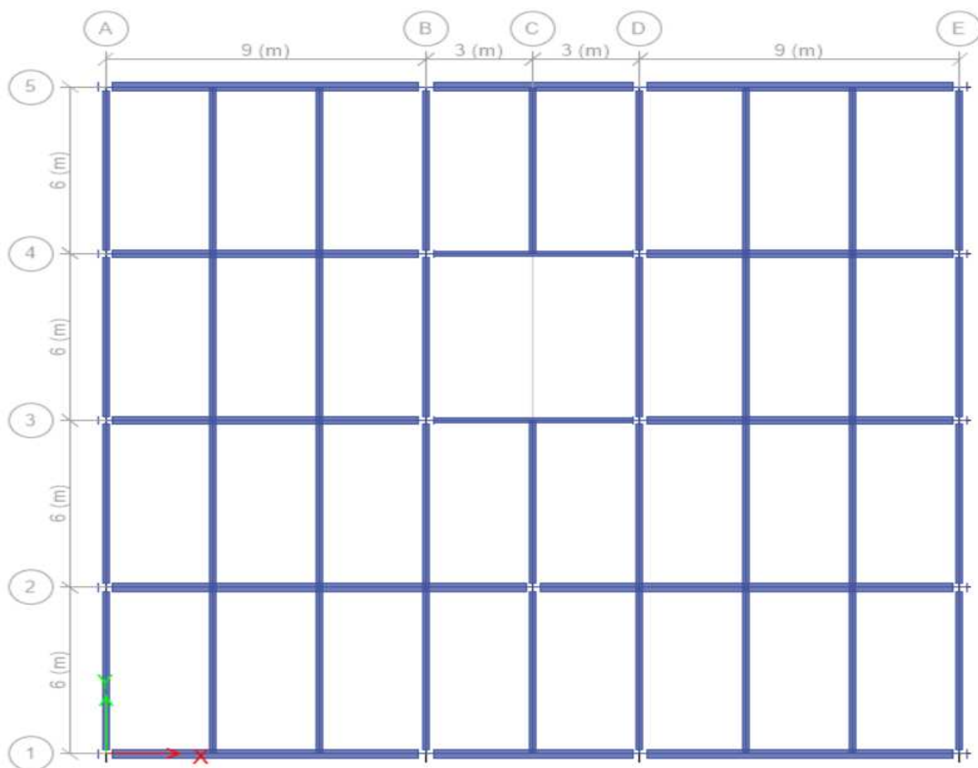



**Figure 53 Top view for roof**

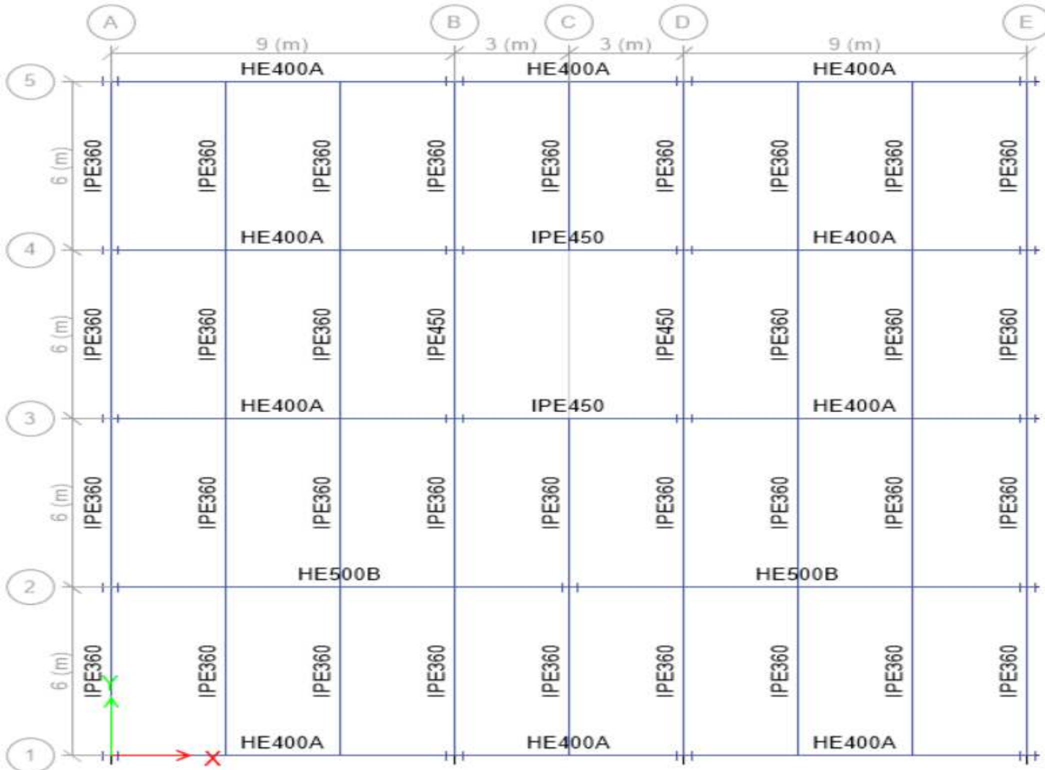



**Figure 54 Top view of sections for roof**

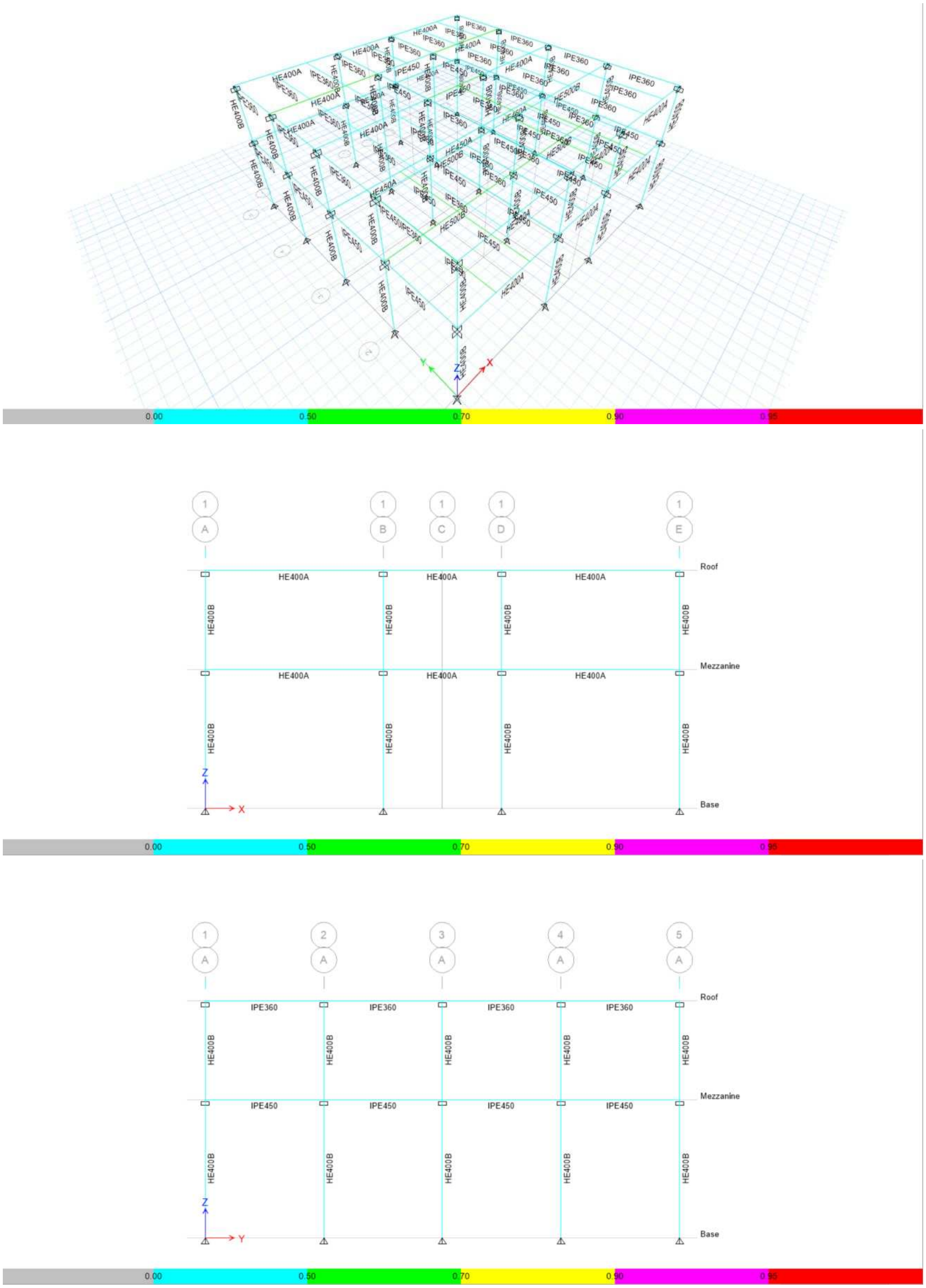



**Figure 55 Steel stress ratio (column)**

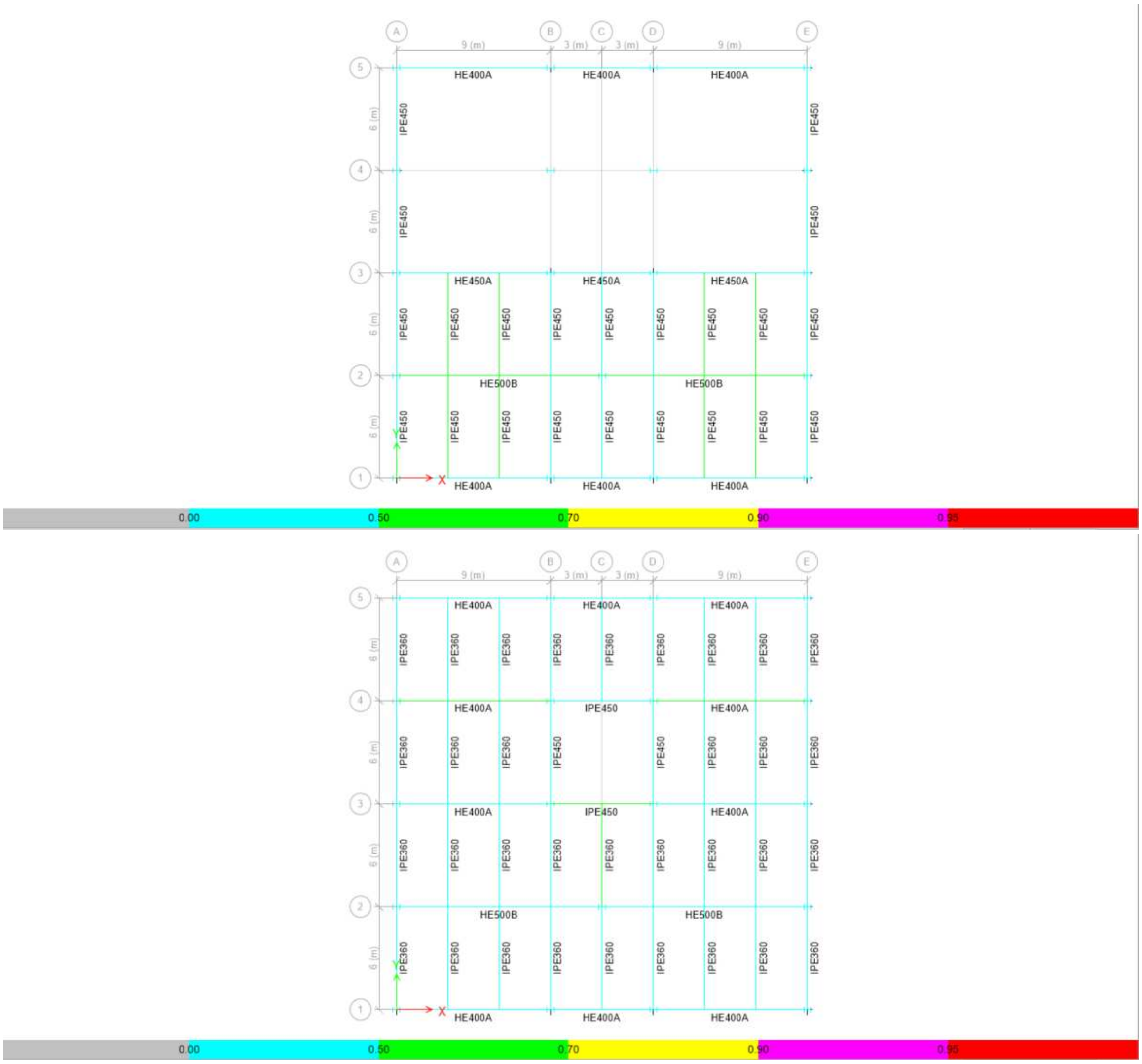



**Figure 56 Steel stress ratio (mezzanine and roof)**

### 3.5 Connection design:

Steel moment frames have been in use for more than one hundred years, dating to the earliest use of structural steel in building construction. Steel building construction with the frame carrying the vertical loads initiated with the Home Insurance Building in Chicago, a 10 story structure constructed in 1884 with a height of 45 meter often credited with being the first skyscraper.

This and other tall buildings in Chicago spawned an entire generation of tall buildings, constructed with load bearing steel frames supporting concrete floors and non-load bearing, unreinforced masonry infill walls at their perimeters. Framing in these early structures typically utilized "H" shapes built up from plates, and "L" and "Z" sections.

Moment-resisting frames are rectilinear assemblages of beams and columns, with the beams rigidly connected to the columns. Resistance to lateral forces is provided primarily by rigid frame action, by the development of bending moment and shear force in the frame members and joints.

Through virtue of the rigid beam-column connections, a moment frame cannot displace laterally without bending the beams or columns depending on the geometry of the connection. The bending rigidity and strength of the frame members is therefore the primary source of lateral stiffness and strength for the entire frame.

Two combined connection type will be used in this design section, Bolted connection and Welded connection. In this section, connection design for the steel structure frame will be applied to calculate the needed numbers of the bolts and weld length. American Institute of Steel Construction (AISC) Manual of Steel Construction/ Allowable Strength Design (ASD) is applied. In addition to calculations, LimCon software will be used for connection design. LimCon output will be included with the appendix.

### 3.5.1 Connection of Mezzanine Floor:

#### 3.5.1.1 Moment Connection (column HEB 400 with beam HEB500):

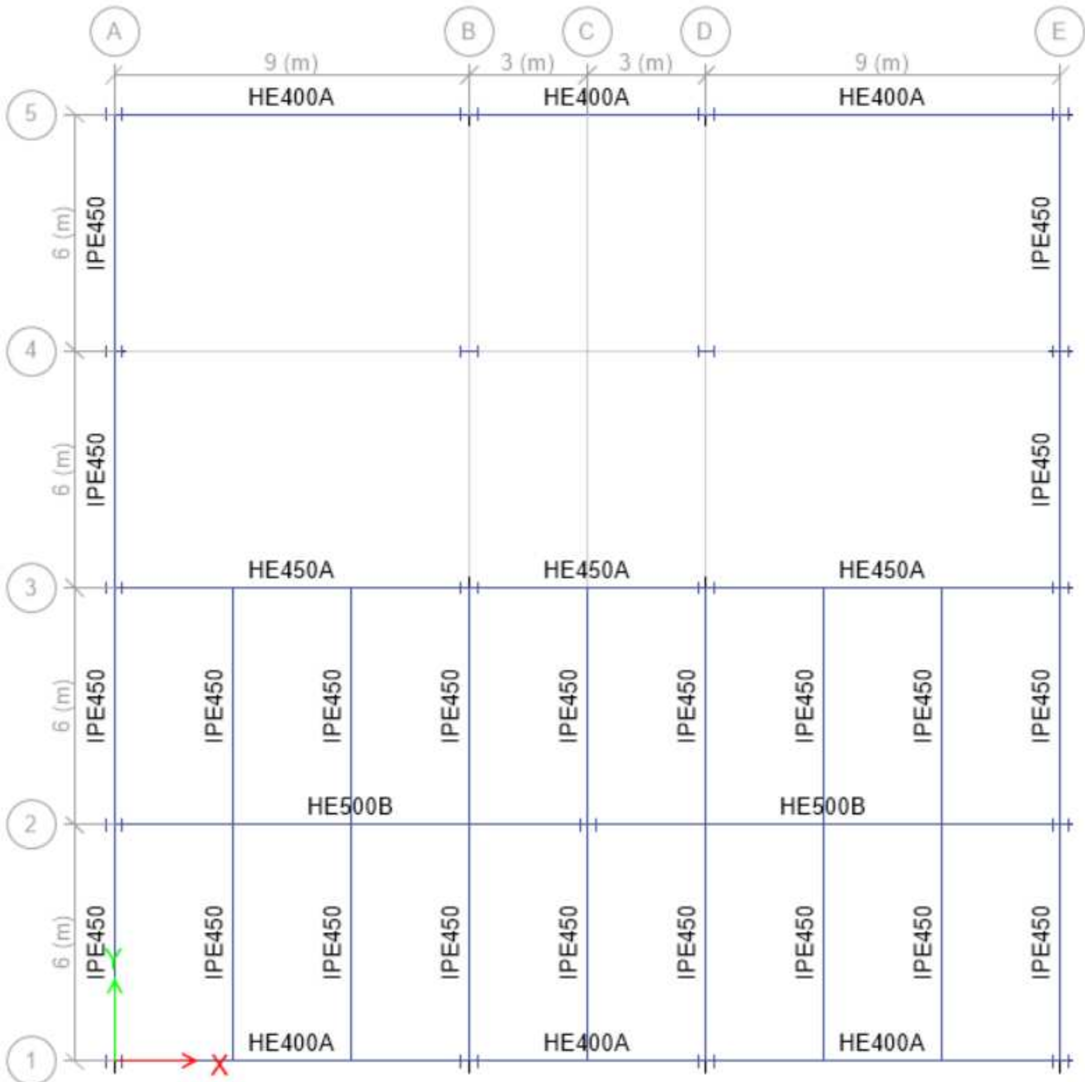



**Figure 57 Mezzanine Floor plan - Connection**

**Material specefication:**

| Material: steel geade (36) | | |
|---|---|---|
| $F_y$ | 3.6 | t/cm$^2$ |
| $F_u$ | 5.2 | t/cm$^2$ |

| Bolts | | |
|---|---|---|
| D | 30 | mm |
| Grade | 10.9 | - |
| $A_s$ | 5.61 | cm$^2$ |
| T | 35.34 | t |
| $P_s$ | 16.86 | t |

**Section properties:**

| Column Section (HEB400) | | |
|---|---|---|
| $h$ | 400 | mm |
| $h_i$ | 352 | mm |
| $b_f$ | 300 | mm |
| $t_w$ | 13.5 | mm |
| $t_f$ | 24 | mm |
| $r$ | 27 | mm |
| $A$ | 198 | $cm^2$ |
| $I_x$ | 57680 | $cm^4$ |

| Beam Section (HEB500) | | |
|---|---|---|
| $h$ | 500 | mm |
| $h_i$ | 444 | mm |
| $b_f$ | 300 | mm |
| $t_w$ | 14.5 | mm |
| $t_f$ | 28 | mm |
| $r$ | 27 | mm |
| $A$ | 239 | $cm^2$ |
| $I_x$ | 107200 | $cm^4$ |

**Loading:**

| Analysis of Moment and Shear (ASD Method) | | | | | |
|---|---|---|---|---|---|
| Dead Load = | 4.078 | t-m | | | |
| Live Load = | 3 | t-m | | | |
| Ultimated Load = | | DL | + | LL | |
| | | 4.08 | + | 3 | = 7.078 t-m |

Calculating for Moment and Shear

$$M_{max} = \frac{W_u \times L^2}{12} = \frac{7.078 \times 12^2}{12} = 84.94 \text{ t-m}$$

$$V_{max} = \frac{W_u \times L}{2} = \frac{7.078 \times 12}{2} = 42.47 \text{ t}$$

Combined Connection Bolts and Welds

$$M_{b\text{ and }w} = \frac{M_{max}}{2} = \frac{84.94}{2} = 42.47 \text{ t-m}$$

$$V_{b\text{ and }w} = \frac{V_{max}}{2} = \frac{42.47}{2} = 21.23 \text{ t}$$

## Design of welding:

**1- Length of weld lines around the flange for Beam**

$L_f = b_f + (b_f - t_w - 2 \times t_f) =$

$L_f = \mathbf{30} + (\mathbf{30} - \mathbf{1.45} - \mathbf{2} \times \mathbf{2.8}) =$ **52.95 cm**

**2 -Welding size around the flange**

$T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{\mathbf{42.47 \times 100}}{\mathbf{50 - 2.8}} =$ **89.97 t**

**3 -The Stress of Welding**

$F_{pw} = 0.2 \times F_u =$

$F_{pw} = \mathbf{0.2} \times \mathbf{5.2} =$ **1.04 $t/cm^2$**

**4 -The Thickness of welding around the Flange**

$S_w = \dfrac{T_b}{L_f \times F_{pw}} = \dfrac{\mathbf{89.97}}{\mathbf{52.95 \times 1.04}} =$ **1.63 cm**

**Take $S_w$= 2 cm**

**5 - Length of weld lines around the Web of Beam**

$L_w = 2 \times (b_f - 2 \times t_f) =$

$L_w = \mathbf{2} \times (\mathbf{30} - \mathbf{2} \times \mathbf{2.8}) =$ **48.8 cm**

**6 - The Thickness of welding around the Web**

$S_w = \dfrac{V_b}{L_w \times F_{pw}} = \dfrac{\mathbf{21.23}}{\mathbf{48.8 \times 1.04}} =$ **0.42 cm**

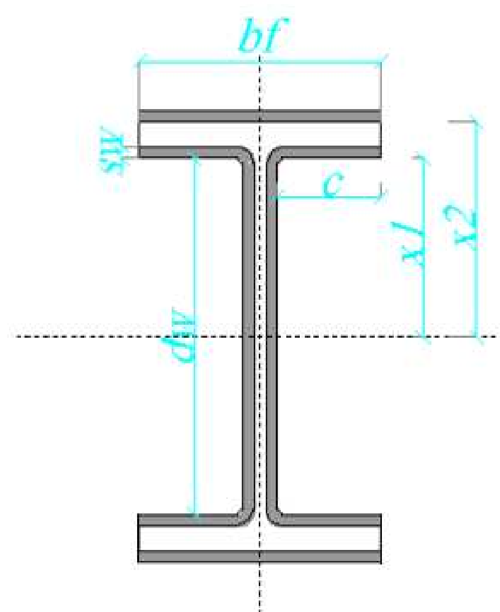
**Take $S_w$= 2 cm**

**The Total Thickness of Welding around Web and Flange**

**$S_w$ = 2 cm**

## Properties of new beam section:

| |
|---|
| $S_w$ = 20 mm |
| |
| $C = b_f/2 - S_w - (t_w/2) =$ |
| C = 300 / 2 - 20 -( 14.5 / 2 ) = 122.75 mm |
| |
| $d_w = h_i -( S_w \times 2 ) =$ |
| $d_w$ = 444 -( 20 × 2 ) = 404 mm |
| |
| $X_1 = d_w / 2 + S_w =$ |
| $X_1$ = 404 / 2 + 20 = 232 mm |
| |
| $X_2 = X_1 + t_f =$ |
| $X_2$ = 232 + 28 = 260 mm |



**Checking the safety of welding:**

**1- Area of Welding**

$$A_w = (2 \times d_w \times S_w) + (2 \times d_f \times S_w) + (4 \times C \times S_w) =$$

$$A_w = (\ 2 \times 40 \times 2\ ) + (\ 2 \times 30 \times 2\ ) + (\ 4 \times 12.28 \times 2\ ) = \quad 379.8 \quad cm^2$$

**2- Inertia Moment of Welding Section**

$$I_{xw} = \left(\frac{d_w{}^3 \times S_w}{12} \times 2\right) + \left(\frac{C \times S_w{}^3}{12} \times 4\right) + (C \times S_w \times X_1{}^2 \times 4) + \left(\frac{b_f \times S_w{}^3}{12} \times 2\right) + (b_f \times S_w \times X_2{}^2 \times 4) =$$

$$I_{xw} = \left(\frac{131879}{12} \times 2\right) + \left(\frac{98.2}{12} \times 4\right) + (\ 12.28 \times 2 \times 538.2 \times 4\ ) + \left(\frac{240}{12} \times 2\right) + (\ 30 \times 2 \times 6760 \times 4\ ) = 1697307.66 \quad cm^4$$

**3- Tensile Stress**

$$F_t = \frac{M_{max} \times y}{I_{xw}} = \frac{8494 \times 25}{1697307.66} = \quad 0.12510 \quad t/cm^2$$

**4- Gross Effective Area**

$$q = \frac{V}{2 \times S_w \times d_w} = \frac{42.5}{2 \times 2 \times 40.4} = \quad 0.263 \quad t/cm^2$$

**5- Equivalent Stress**

$$F_{eq} = \sqrt{F_t{}^2 + 3q^2} = \sqrt{0.12510^2 + 3 \times 0.263^2} = \quad 0.332 \quad t/cm^2$$

**6- Allowable Stress**

$$F_{all_w} = 1.1 \times F_{pw} = 1.1 \times 1 = \quad 1.1 \quad t/cm^2$$

**7- Check the safety of welding**

$$F_{eq} < F_{all_w}$$

$$0.332 < 1.14$$

✓

**Design of Bolts:**

| 1- Number of Bolts | | |
|---|---|---|
| $T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{42.47 \times 100}{50 \quad - 2.8} = \quad 89.97 \quad t$ | | |
| $N_b = \dfrac{T_b}{T} = \dfrac{89.97}{35.34} = 2.55$ | | |
| The Number of Bolts in one Row = | 5 | Bolts |
| The Number of Boltsin Plate = | 10 | Bolts |

| 2- Tensile Force |
|---|
| $Text_b = \dfrac{T_b}{\text{Number of Tension bolt}} = \dfrac{42}{6} = \quad 7.1 \quad t$ |

| 3- Prying Force |
|---|
| $P = \left[ \dfrac{0.5 \; - \left[ \dfrac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s} \right]}{\left(\dfrac{3 \times a}{4 \times b}\right) \times \left(\dfrac{a}{4 \times b} + 1\right) + \left[ \dfrac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s} \right]} \right] \times Text_b =$ |
| $P = \left[ \dfrac{0.5 \quad - \left[ \dfrac{224}{13237.5} \right]}{\left(\dfrac{15}{20}\right) \times (0.25 \quad + 1 \quad ) + \left[ \dfrac{224}{13237.5} \right]} \right] \times \; 7$ |
| $P = \quad 2.41 \quad t$ |

**Checking the safety of bolts:**

$$Text_b + P \le 0.8 \times T$$

$$7.1 + 2.41 \le 0.8 \times 35.34$$

$$9.48 \le 28.27 \quad \text{O.K.}$$

**Checking for shear:**

$$V_b = \frac{V_{max}}{n} = \frac{21.2}{10} = 2.1 \ t$$

$$V_b < P_s$$

$$2.12 < 16.86 \quad \text{O.K.}$$

**Design of end plates:**

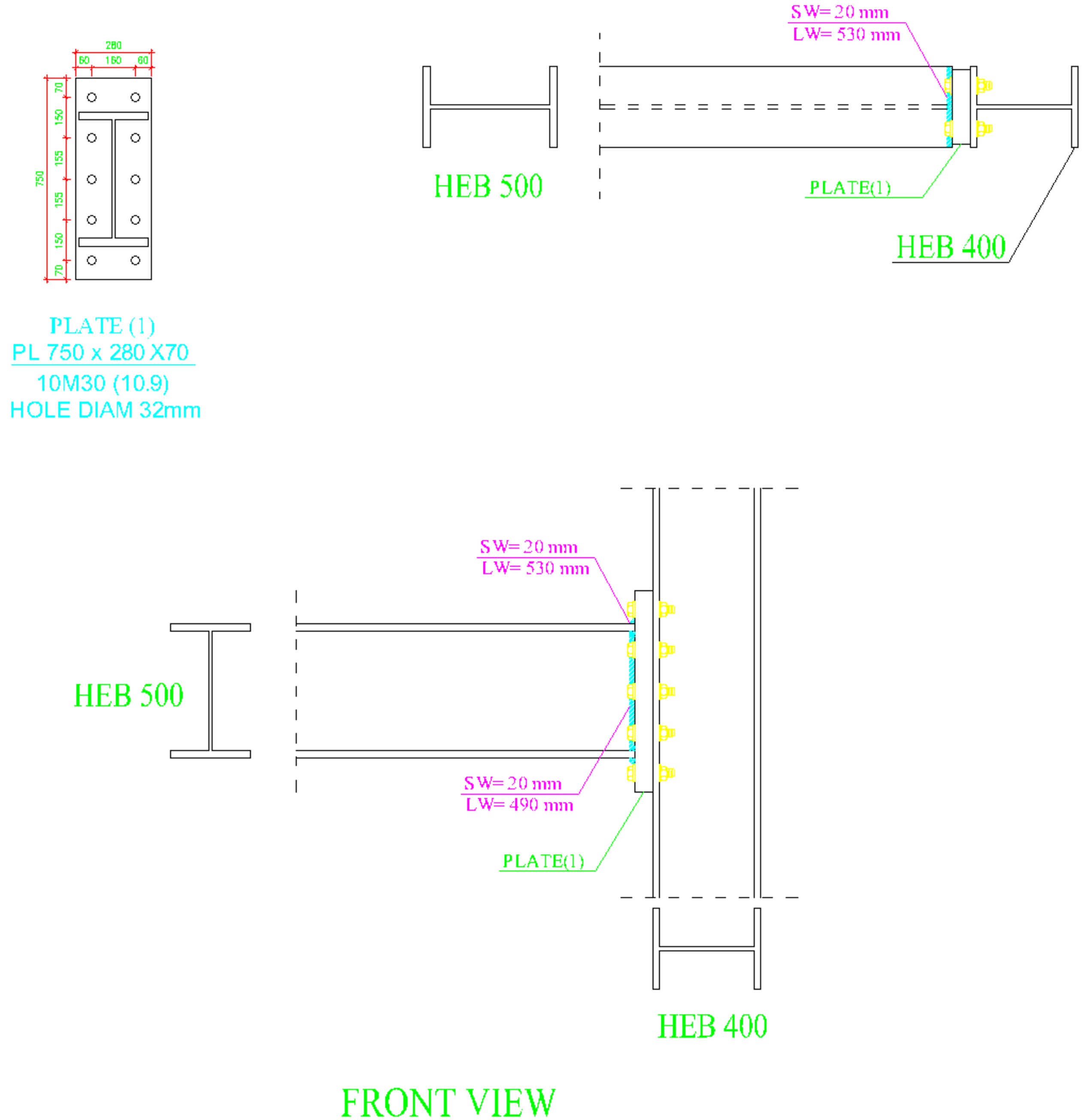
| Allowable Stress in Bending |
|---|
| $F_c = 0.64 \times F_y = 0.64 \times 3.60 = 2.3\ t/cm^2$ |

| Size around the flange |
|---|
| $T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{84.94 \times 100}{50 - 2.8} = 60.7\ t$ |

| Moment |
|---|
| $M = \dfrac{T_p \times L}{8} = \dfrac{60.7 \times 75}{8} = 569\ t\text{-}cm$ |

| Thickness of Plate |
|---|
| $t_p = \sqrt{\dfrac{6 \times M}{2 \times w \times F_c}} = \sqrt{\dfrac{6 \times 569}{2 \times 15 \times 2.30}} = 7\ cm$ |

| Plate Size | |
|---|---|
| Thickness of Plate = | 7 cm |
| Length of Plate = | 75 cm |
| Width of Plate = | 28 cm |




AutoCad Connection Drawing for Moment Connection in Mezzanine Floor

**3.5.1.2 Simple connection (beam HEA 400 with beam IPE 450):**

**Material specefication:**

| Material: steel geade (36) | | |
|---|---|---|
| $F_y$ | 3.6 | $t/cm^2$ |
| $F_u$ | 5.2 | $t/cm^2$ |

| Bolts | | |
|---|---|---|
| D | 30 | mm |
| Grade | 10.9 | - |
| $A_s$ | 5.61 | $cm^2$ |
| T | 35.34 | t |
| $P_s$ | 14.13 | t |

**Section properties:**

| Beam Section (IPE 450) | | |
|---|---|---|
| h | 450 | mm |
| $h_i$ | 420.8 | mm |
| $b_f$ | 190 | mm |
| $t_w$ | 9.8 | mm |
| $t_f$ | 14.6 | mm |
| r | 21 | mm |
| A | 98.8 | $cm^2$ |
| $I_x$ | 33740 | $cm^4$ |

| Beam Section (HEA 400) | | |
|---|---|---|
| h | 390 | mm |
| $h_i$ | 352 | mm |
| $b_f$ | 300 | mm |
| $t_w$ | 11 | mm |
| $t_f$ | 24 | mm |
| r | 27 | mm |
| A | 195 | $cm^2$ |
| $I_x$ | 45070 | $cm^4$ |

**Loading:**

| Analysis of Moment and Shear (ASD Method) | | | | | |
|---|---|---|---|---|---|
| Dead Load = | 4.078 | t-m | | | |
| Live Load = | 3 | t-m | | | |
| Ultimated Load = | | DL | + | LL | |
| | | 2.11 | + | 1.5 | = 3.610 t-m |

**Calculating for Moment and Shear**

$$M_{max} = \frac{W_u \times L^2}{12} = \frac{3.61 \times 12^2}{12} = 43.32 \text{ t-m}$$

$$V_{max} = \frac{W_u \times L}{2} = \frac{3.61 \times 12}{2} = 21.66 \text{ t}$$

**Design of welding:**

| 1- Length of weld lines around the flange for Beam |
|---|
| $L_f = b_f + (b_f - t_w - 2 \times t_f) =$ |
| $L_f = 19 + (19 - 0.98 - 2 \times 1.46) =$ **34.1 cm** |

| 2 -Welding size around the flange |
|---|
| $T_b = \frac{M_{max(ASD)}}{h - t_f} = \frac{43.32 \times 100}{45 - 1.46} =$ **99.49 t** |

| 3 -The Stress of Welding |
|---|
| $F_{pw} = 0.2 \times F_u =$ |
| $F_{pw} = 0.2 \times 5.2 =$ **1.04 $t/cm^2$** |

| 4 -The Thickness of welding around the Flange |
|---|
| $S_w = \frac{T_b}{L_f \times F_{pw}} = \frac{99.49}{34.10 \times 1.04} =$ **2.81 cm** |
| **Take $S_w$= 3 cm** |

| 5 - Length of weld lines around the Web of Beam |
|---|
| $L_w = 2 \times (b_f - 2 \times t_f) =$ |
| $L_w = 2 \times (19 - 2 \times 1.46) =$ **32.2 cm** |

| 6 - The Thickness of welding around the Web |
|---|
| $S_w = \frac{V_b}{L_w \times F_{pw}} = \frac{21.66}{32.2 \times 1.04} =$ **0.65 cm** |
| **Take $S_w$= 1 cm** |

| The Total Thickness of Welding around Web and Flange |
|---|
| $S_w$ = **3 cm** |

**Design of bolts:**

| 1- Prestressing for angle Plate Double Shear |
|---|
| $T_d = \frac{T}{2} = \frac{35.34}{2} = 17.67$ t |

| 2- Number of Bolts | | |
|---|---|---|
| $T_b = \frac{M_{max(ASD)}}{h - t_f} = \frac{43.32 \times 100}{45 - 1.46} =$ 99.49 t | | |
| $N_b = \frac{T_b}{T_d} = \frac{99.49}{17.67} = 5.63$ | | |
| The Number of Bolts in one Row = | 6 | Bolts |
| The Number of Boltsin Plate = | 12 | Bolts |

**3- Tensile Force**

$$Text_b = \frac{T_b}{\text{Number of Tension bolt}} = \frac{99.49}{6} = 17 \quad t$$

**4- Prying Force**

$$P = \left[ \frac{0.5 - \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]}{\left(\frac{3 \times a}{4 \times b}\right) \times \left(\frac{a}{4 \times b} + 1\right) + \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]} \right] \times Text_b =$$

$$P = \left[ \frac{0.5 - \left[\frac{224}{13237.5}\right]}{\left(\frac{15}{20}\right) \times (0.25 + 1) + \left[\frac{224}{13237.5}\right]} \right] \times 17$$

$$P = 5.64 \quad t$$

**Check of bolts:**

$$Text_b + P \leq 0.8 \times T$$

$$16.6 + 5.64 \leq 0.8 \times 35.34$$

$$22.22 \leq 28.27$$ O.K.

**Check of shear:**

$$V_b = \frac{V_{max}}{n} = \frac{21.7}{12} = 1.8 \quad t$$

$$V_b < P_s$$

$$1.81 < 14.13$$ O.K.

**Design of end plates:**

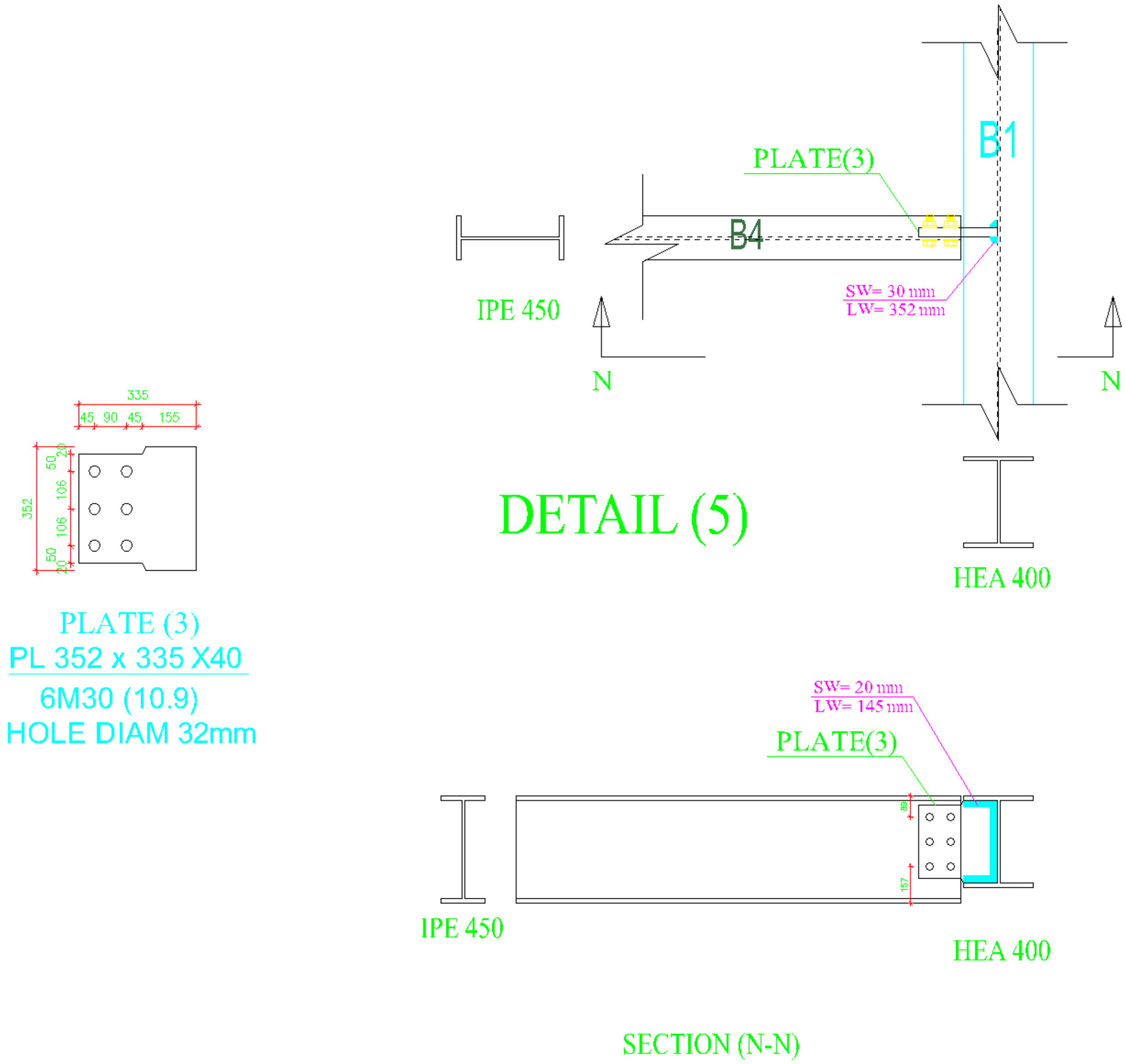
| Allowable Stress in Bending |
|---|
| $F_c = 0.64 \times F_y = 0.64 \times 3.60 =$ 2.3 t/cm$^2$ |

| Size around the flange |
|---|
| $T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{43.32 \times 100}{45 - 1.46} =$ 65.9 t |

| Moment |
|---|
| $M = \dfrac{T_p \times L}{8} = \dfrac{65.9 \times 27}{8} =$ 223 t-cm |

| Thickness of Plate |
|---|
| $t_p = \sqrt{\dfrac{6 \times M}{2 \times w \times F_c}} = \sqrt{\dfrac{6 \times 223}{2 \times 15 \times 2.30}} =$ 4 cm |

| Plate Size | |
|---|---|
| Thickness of Plate = | 4 cm |
| Length of Plate = | 27 cm |
| Wedith of Plate = | 20 cm |




AutoCad Connection Drawing for Simple Connection in Mezzanine Floor

### 3.5.2 Connection of Roof:

### 3.5.2.1 Moment Connection (column HEB 400 with beam IPE360):


**Figure 58 Roof plan - Connection**

**Material specefication:**

| Material: steel geade (36) | | |
|---|---|---|
| $F_y$ | 3.6 | t/cm$^2$ |
| $F_u$ | 5.2 | t/cm$^2$ |

| Bolts | | |
|---|---|---|
| D | 24 | mm |
| Grade | 10.9 | - |
| $A_s$ | 3.53 | cm$^2$ |
| T | 22.23 | t |
| $P_s$ | 10.6 | t |

**Section properties:**

| Column Section (HEB400) | | |
|---|---|---|
| $h$ | 400 | mm |
| $h_i$ | 352 | mm |
| $b_f$ | 300 | mm |
| $t_w$ | 13.5 | mm |
| $t_f$ | 24 | mm |
| $r$ | 27 | mm |
| $A$ | 198 | $cm^2$ |
| $I_x$ | 57680 | $cm^4$ |

| Beam Section (IPE360) | | |
|---|---|---|
| $h$ | 360 | mm |
| $h_i$ | 334.6 | mm |
| $b_f$ | 170 | mm |
| $t_w$ | 8 | mm |
| $t_f$ | 12.7 | mm |
| $r$ | 18 | mm |
| $A$ | 72.7 | $cm^2$ |
| $I_x$ | 16270 | $cm^4$ |

**Loading** :

| Analysis of Moment and Shear (ASD Method) | | | | | |
|---|---|---|---|---|---|
| Dead Load = | 4.078 | t-m | | | |
| Live Load = | 3 | t-m | | | |
| Ultimated Load = | | DL | + | LL | |
| | | 1.05 | + | 0.38 = | 1.430 t-m |

**Calculating for Moment and Shear**

$$M_{max} = \frac{W_u \times L^2}{12} = \frac{1.43 \times 12^2}{12} = 17.16 \text{ t-m}$$

$$V_{max} = \frac{W_u \times L}{2} = \frac{1.43 \times 12}{2} = 8.58 \text{ t}$$

**Combined Connection Bolts and Welds**

$$M_{b \text{ and } w} = \frac{M_{max}}{2} = \frac{17.16}{2} = 8.58 \text{ t-m}$$

$$V_{b \text{ and } w} = \frac{V_{max}}{2} = \frac{8.58}{2} = 4.29 \text{ t}$$

## Design of welding:

| 1- Length of weld lines around the flange for Beam |
|---|
| $L_f = b_f + (b_f - t_w - 2 \times t_f) =$ |
| $L_f =$ **17 +(17 − 0.8 − 2 ×1.27) = 30.66 cm** |

| 2 -Welding size around the flange |
|---|
| $T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{8.58 \times 100}{36 - 1.27} =$ **24.70 t** |

| 3 -The Stress of Welding |
|---|
| $F_{pw} = 0.2 \times F_u =$ |
| $F_{pw} = 0.2 \times 5.2 = 1.04\ t/cm^2$ |

| 4 -The Thickness of welding around the Flange |
|---|
| $S_w = \dfrac{T_b}{L_f \times F_{pw}} = \dfrac{24.70}{30.66 \times 1.04} =$ **0.77 cm** |
| **Take $S_w$= 1 cm** |

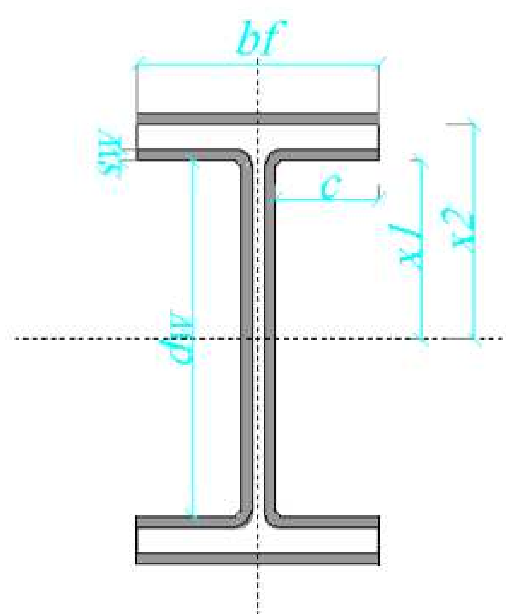
| 5 - Length of weld lines around the Web of Beam |
|---|
| $L_w = 2 \times (b_f - 2 \times t_f) =$ |
| $L_w =$ **2 × (17 −2 ×1.27 ) = 28.9 cm** |

| 6 - The Thickness of welding around the Web |
|---|
| $S_w = \dfrac{V_b}{L_w \times F_{pw}} = \dfrac{4.29}{28.9 \times 1.04} =$ **0.14 cm** |
| **Take $S_w$= 1 cm** |

| The Total Thickness of Welding around Web and Flange |
|---|
| **$S_w$ = 1 cm** |

## Properties of new beam section

| | | | | | | | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|---|
| $S_w$ | = | 10 | mm | | | | | | | | | | |
| $C = b_f/2 - S_w - (t_w/2) =$ | | | | | | | | | | | | | |
| C | = | 300 | / | 2 | - | 10 | -( | 8.0 | / | 2 | ) | = | 136 mm |
| $d_w = h_i$ -( $S_w \times 2$ ) = | | | | | | | | | | | | | |
| $d_w$ | = | 335 | -( | 10 | × | 2 | ) | = | 315 mm | | | | |
| $X_1 = d_w / 2 + S_w =$ | | | | | | | | | | | | | |
| $X_1$ | = | 315 | / | 2 | + | 10 | = | 187 | mm | | | | |
| $X_2 = X_1 + t_f =$ | | | | | | | | | | | | | |
| $X_2$ | = | 187 | + | 13 | = | 200 | mm | | | | | | |



**Checking the safety of welding:**

**1- Area of Welding**

$$A_w = (2 \times d_w \times S_w) + (2 \times d_f \times S_w) + (4 \times C \times S_w) =$$

$$A_w = (\ 2 \times\ 31\ \times 1\ ) + (\ 2\ \times 30\ \times\ 1\ ) + (\ 4\ \times 13.60 \times 1\ ) = \ 177.32\ \ cm^2$$

**2- Inertia Moment of Welding Section**

$$I_{xw} = \left(\frac{d_w{}^3 \times S_w}{12} \times 2\right) + \left(\frac{C \times S_w{}^3}{12} \times 4\right) + (C \times S_w \times X_1{}^2 \times 4) + \left(\frac{b_f \times S_w{}^3}{12} \times 2\right) + (b_f \times S_w \times X_2{}^2 \times 4) =$$

$$I_{xw} = \left(\frac{31137}{12} \times 2\right) + \left(\frac{13.6}{12} \times 4\right) + (13.60 \times 1 \times 350.8 \times 4\ ) + \left(\frac{30}{12} \times 2\right) + (\ 30 \times 1 \times\ 4000 \times 4\ ) = \ 504283.25\ \ cm^4$$

**3- Tensile Stress**

$$F_t = \frac{M_{max} \times y}{I_{xw}} = \frac{1716\ \times\ \ 25}{504283.25} = \ \ 0.08507\ \ t/cm^2$$

**4- Gross Effective Area**

$$q = \frac{V}{2 \times S_w \times d_w} = \frac{8.6}{2 \times\ \ 1\ \times 31.5} = \ \ 0.136\ \ t/cm^2$$

**5- Equivalent Stress**

$$F_{eq} = \sqrt{F_t{}^2 + 3q^2} = \sqrt{0.08507^2 + 3\ \times 0.136^2} = \ \ 0.141\ \ t/cm^2$$

**6- Allowable Stress**

$$F_{all_w} = 1.1 \times F_{pw} = \ 1.1 \times\ 1\ \ = \ \ 1.1\ \ t/cm^2$$

| 7- Check the safety of welding | | |
|---|---|---|
| $F_{e_q}$ | $<$ | $F_{all_w}$ |
| 0.141 | $<$ | 1.14 |

**Design of Bolts:**

### 1- Number of Bolts

$$T_b = \frac{M_{max(ASD)}}{h - t_f} = \frac{8.58 \times 100}{36 - 1.27} = 24.70 \quad t$$

$$N_b = \frac{T_b}{T} = \frac{24.70}{22.23} = 1.11$$

| | | |
|---|---|---|
| The Number of Bolts in one Row = | 3 | Bolts |
| The Number of Boltsin Plate = | 6 | Bolts |

### 2- Tensile Force

$$Text_b = \frac{T_b}{\text{Number of Tension bolt}} = \frac{9}{4} = 2.1 \quad t$$

### 3- Prying Force

$$P = \left[\frac{0.5 - \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]}{\left(\frac{3 \times a}{4 \times b}\right) \times \left(\frac{a}{4 \times b} + 1\right) + \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]}\right] \times Text_b =$$

$$P = \left[\frac{0.5 - \left[\frac{224}{13237.5}\right]}{\left(\frac{15}{20}\right) \times (0.25 + 1) + \left[\frac{224}{13237.5}\right]}\right] \times 2$$

$$P = 0.73 \quad t$$

**Checking of Bolts:**

$$Text_b + P \leq 0.8 \times T$$

$$2.1+0.73 \leq 0.8 \times 22.23$$

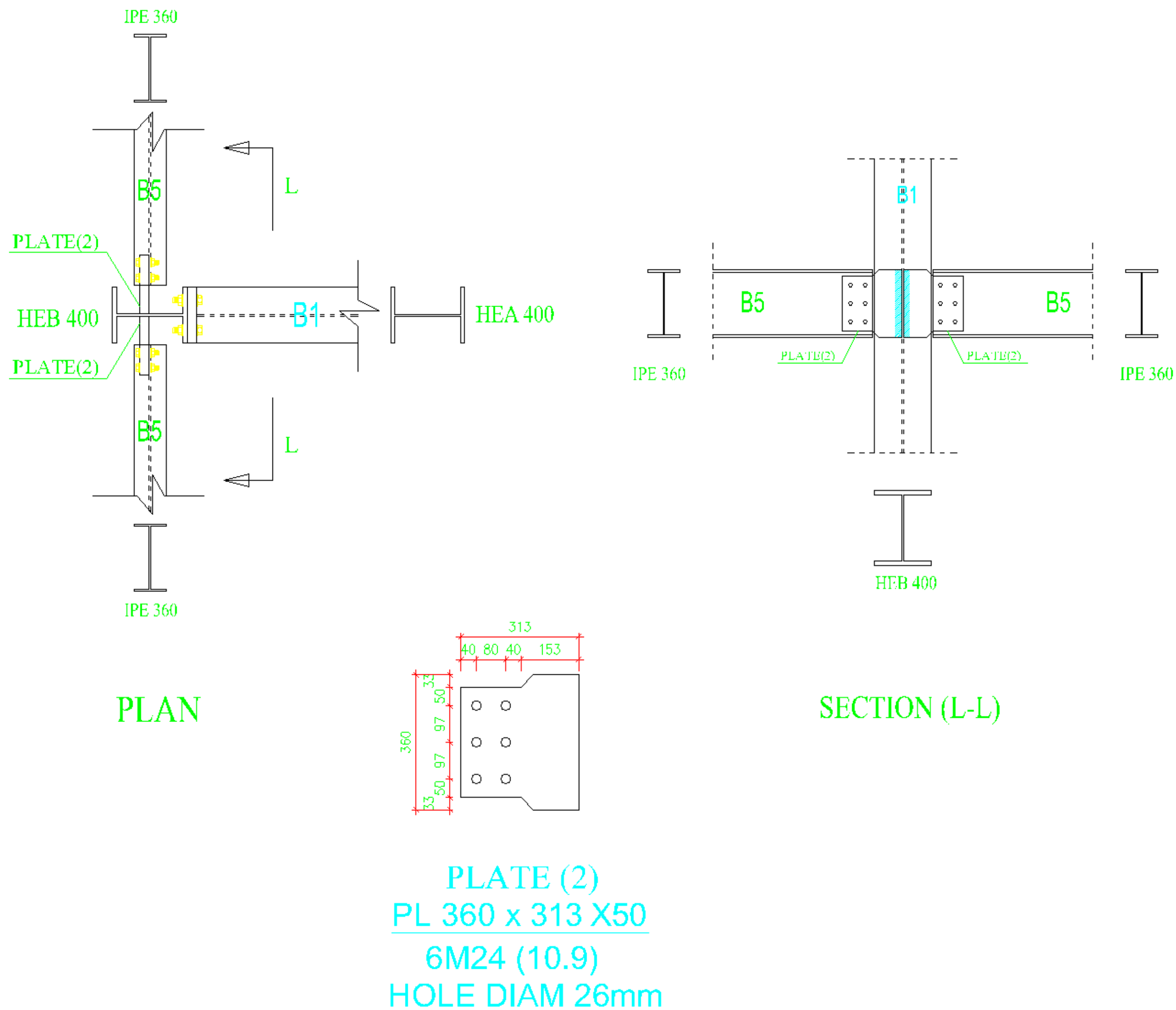
$$2.87 \leq 17.78 \quad \text{O.K.}$$

**Design of Shear:**

$$V_b = \frac{V_{max}}{n} = \frac{4.3}{6} = 0.7 \ \text{t}$$

$$V_b < P_s$$

$$0.72 < 10.60 \quad \text{O.K.}$$




AutoCad Connection Drawing for Moment Connection in Roof

### 3.5.2.2 Simple Connection of Roof (beam HEA 400 with IPE 360):

**Material Specification**

| Material: steel geade (36) | | |
|---|---|---|
| $F_y$ | 3.6 | $t/cm^2$ |
| $F_u$ | 5.2 | $t/cm^2$ |

| Bolts | | |
|---|---|---|
| D | 30 | mm |
| Grade | 10.9 | - |
| $A_s$ | 5.61 | $cm^2$ |
| T | 35.34 | t |
| $P_s$ | 14.13 | t |

**Section Properties**

| Beam Section (HEA 400) | | |
|---|---|---|
| h | 390 | mm |
| $h_i$ | 352 | mm |
| $b_f$ | 300 | mm |
| $t_w$ | 11 | mm |
| $t_f$ | 24 | mm |
| r | 27 | mm |
| A | 195 | $cm^2$ |
| $I_x$ | 45070 | $cm^4$ |

| Beam Section (IPE 360) | | |
|---|---|---|
| h | 360 | mm |
| $h_i$ | 334.6 | mm |
| $b_f$ | 170 | mm |
| $t_w$ | 8 | mm |
| $t_f$ | 12.7 | mm |
| r | 18 | mm |
| A | 72.7 | $cm^2$ |
| $I_x$ | 16270 | $cm^4$ |

**Loading**

| Analysis of Moment and Shear (ASD Method) | | | | | |
|---|---|---|---|---|---|
| Dead Load = | 4.078 | t-m | | | |
| Live Load = | 3 | t-m | | | |
| Ultimated Load = | | DL | + | LL | |
| | | 2.11 | + | 0.75 = | 2.860 t-m |

**Calculating for Moment and Shear**

$$M_{max} = \frac{W_u \times L^2}{12} = \frac{2.86 \times 12^2}{12} = 34.32 \text{ t-m}$$

$$V_{max} = \frac{W_u \times L}{2} = \frac{2.86 \times 12}{2} = 17.16 \text{ t}$$

**Design of welding:**

**1- Length of weld lines around the flange for Beam**

$$L_f = b_f + (b_f - t_w - 2 \times t_f) =$$

$$L_f = 17 + (17 - 0.8 - 2 \times 1.27) = 30.66 \text{ cm}$$

**2 -Welding size around the flange**

$$T_b = \frac{M_{max(ASD)}}{h - t_f} = \frac{34.32 \times 100}{36 - 1.27} = 98.82 \text{ t}$$

**3 -The Stress of Welding**

$$F_{pw} = 0.2 \times F_u =$$

$$F_{pw} = 0.2 \times 5.2 = 1.04 \text{ t/cm}^2$$

**4 -The Thickness of welding around the Flange**

$$S_w = \frac{T_b}{L_f \times F_{pw}} = \frac{98.82}{30.66 \times 1.04} = 3 \text{ cm}$$

Take $S_w$= 3 cm

**5 - Length of weld lines around the Web of Beam**

$$L_w = 2 \times (b_f - 2 \times t_f) =$$

$$L_w = 2 \times (17 - 2 \times 1.27) = 28.9 \text{ cm}$$

**6 - The Thickness of welding around the Web**

$$S_w = \frac{V_b}{L_w \times F_{pw}} = \frac{17.16}{28.9 \times 1.04} = 0.57 \text{ cm}$$

Take $S_w$= 1 cm

**The Total Thickness of Welding around Web and Flange**

$S_w$ = 3 cm

**Design of Bolts:**

**1- Prestressing for angle Plate Double Shear**

$$T_d = \frac{T}{2} = \frac{35.34}{2} = 17.67 \text{ t}$$

**2- Number of Bolts**

$$T_b = \frac{M_{max(ASD)}}{h - t_f} = \frac{34.32 \times 100}{36 - 1.27} = 98.82 \text{ t}$$

$$N_b = \frac{T_b}{T_d} = \frac{98.82}{17.67} = 5.59$$

| | | |
|---|---|---|
| The Number of Bolts in one Row = | 6 | Bolts |
| The Number of Boltsin Plate = | 12 | Bolts |

| 3- Tensile Force |
|---|
| $Text_b = \frac{T_b}{\text{Number of Tension bolt}} = \frac{98.82}{6} = 16 \quad t$ |

| 4- Prying Force |
|---|
| $P = \left[\frac{0.5 - \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]}{\left(\frac{3 \times a}{4 \times b}\right) \times \left(\frac{a}{4 \times b} + 1\right) + \left[\frac{W \times T_b{}^4}{30 \times a \times b^2 \times A_s}\right]}\right] \times Text_b =$ |
| $P = \left[\frac{0.5 - \left[\frac{224}{13237.5}\right]}{\left(\frac{15}{20}\right) \times (0.25 + 1) + \left[\frac{224}{13237.5}\right]}\right] \times 16$ |
| $P = 5.60 \quad t$ |

**Checking for Bolts:**

$$Text_b + P \leq 0.8 \times T$$

$$16.5 + 5.60 \leq 0.8 \times 35.34$$

$$22.07 \leq 28.27 \quad \text{O.K.}$$

**Checking for Shear:**

$$V_b = \frac{V_{max}}{n} = \frac{17.2}{12} = 1.4 \quad t$$

$$V_b < P_s$$

$$1.43 < 14.13 \quad \text{O.K.}$$

**Design of end plates:**

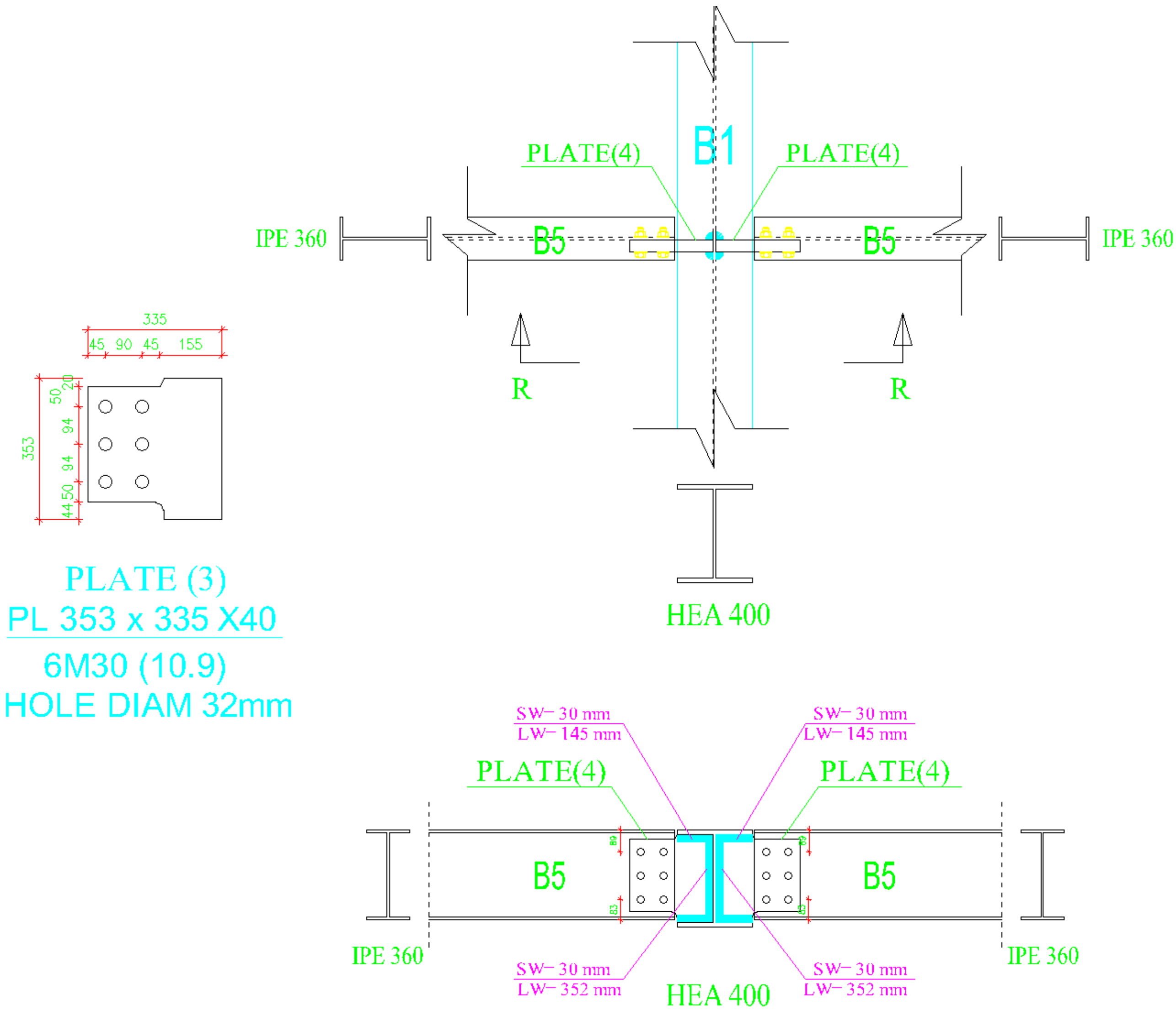
| Allowable Stress in Bending |
| --- |
| $F_c = 0.64 \times F_y = 0.64 \times 3.60 = 2.3\ t/cm^2$ |

| Size around the flange |
| --- |
| $T_b = \dfrac{M_{max(ASD)}}{h - t_f} = \dfrac{34.32 \times 100}{36 - 1.27} = 75.1\ t$ |

| Moment |
| --- |
| $M = \dfrac{T_p \times L}{8} = \dfrac{75.1 \times 25}{8} = 235\ \text{t-cm}$ |

| Thickness of Plate |
| --- |
| $t_p = \sqrt{\dfrac{6 \times M}{2 \times w \times F_c}} = \sqrt{\dfrac{6 \times 235}{2 \times 15 \times 2.30}} = 5\ cm$ |

| Plate Size | |
| --- | --- |
| Thickness of Plate = | 5 cm |
| Length of Plate = | 25 cm |
| Width of Plate = | 20 cm |




AutoCad Connection Drawing for Simple Connection in Roof

**Summary for Connection in Mezzanine Floor:**

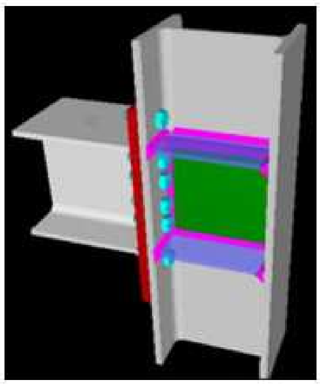
Connection of Mezzanine Floor:

| Connection | Detail | | Type of Calculation | Repeated |
|---|---|---|---|---|
| | Beam | Column | | |
| 1 | HEB 500 | HEB 400 | Excel Sheet | Along Grade 2 |
| 2 | HEA 400 | HEB 400 | Limcon Software | Along Grade 1 and 5 |
| 3 | IPE 450 | HEB 400 | Limcon Software | Along Grade A and E |
| 4 | HEA 450 | HEB 400 | Limcon Software | Along Grade 3 |

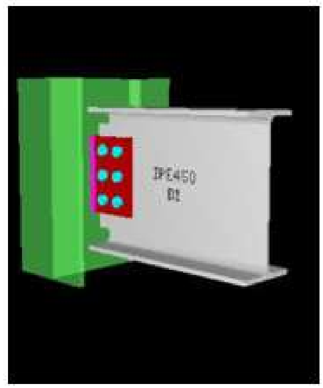
Excel Sheet
Calculation for
Moment
Connection

HEB 500
HEB 400

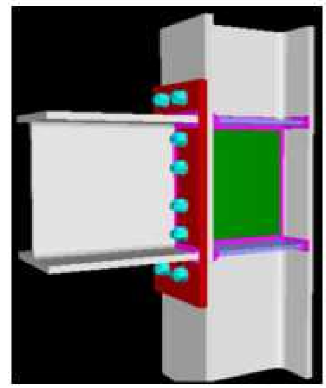
Connection 1



Connection 2



Connection 3



Connection 4

Connection of Sub-beam in Mezzanine Floor:

| Connection | Detail | | Type of Calculation | Repeated |
|---|---|---|---|---|
| | Beam | Beam | | |
| 5 | HEA 400 | IPE 450 | Limcon Software | Along Grade 1 and 3 |

Note: The connection of sub-beam applied to the beam HEB 500 to be symmetrical.

Excel Sheet
Calculation for
Simple
Connection

HEA 400
IPE 450

Connection 5

**Summary for Connection in Roof:**

Connection of Roof:

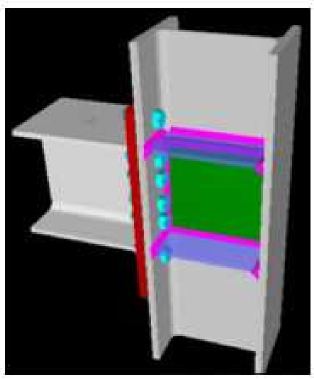
| Connection | Detail | | Type of Calculation | Repeated |
|---|---|---|---|---|
| | Beam | Column | | |
| 1 | HEB 500 | HEB 400 | Excel Sheet | Along Grade 2 |
| 2 | HEA 400 | HEB 400 | Limcon Software | Along Grade 1-3-4 and 5 |
| 3 | IPE 450 | HEB 400 | Limcon Software | 3B - 3D - 4B 4D |
| 4 | IPE 360 | HEB 400 | Excel Sheet | Along Grade A-B-D and E |

Note: Connection 2 and 4 it's applied without the connection in grade 3B - 3D - 4B and 4D (the area of Dome)

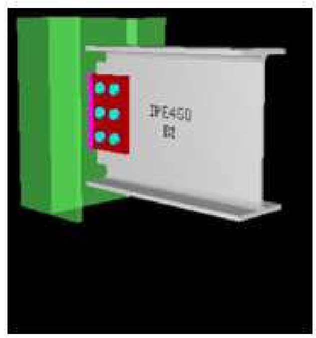
Excel Sheet Calculation for Moment Connection

HEB 500
HEB 400

Connection 1



Connection 2




Connection 3

Excel Sheet Calculation for Moment Connection

IPE 360
HEB 400

Connection 4

Connection of Sub-beam in Roof:

| Connection | Detail | | Type of Calculation | Repeated |
|---|---|---|---|---|
| | Beam | Beam | | |
| 6 | HEA 400 | IPE 360 | Excel Sheet | Along Grade 1-2-3-4 and 5 |

Note: The connection of sub-beam applied to beam HEB 500 to be symmetrical.

Excel Sheet Calculation for Simple Connection

HEA 400
IPE 360

Connection 6

# Chapter 4: Foundation

## 4.1 Introduction

A foundation is the element of an architectural structure which connects it to the ground, and transfers loads from the structure to the ground. Foundations are generally considered either shallow or deep. This chapter is made for the mosque foundation design and a shallow foundation is going to be used for this project. This section includes the calculations and verification of the suitable foundation system for different elements of the mosque. We will first check for isolated foundation. If it does not work, then we will go directly for Raft foundation. In case of using raft foundation settlements calculation is required.

## 4.2 Isolated foundation

Isolated foundations (footings) are standalone families that are part of the structural foundation category. Several types of isolated foundations can be loaded from the family library, including pile caps with multiple piles, rectangular piles, and single piles.

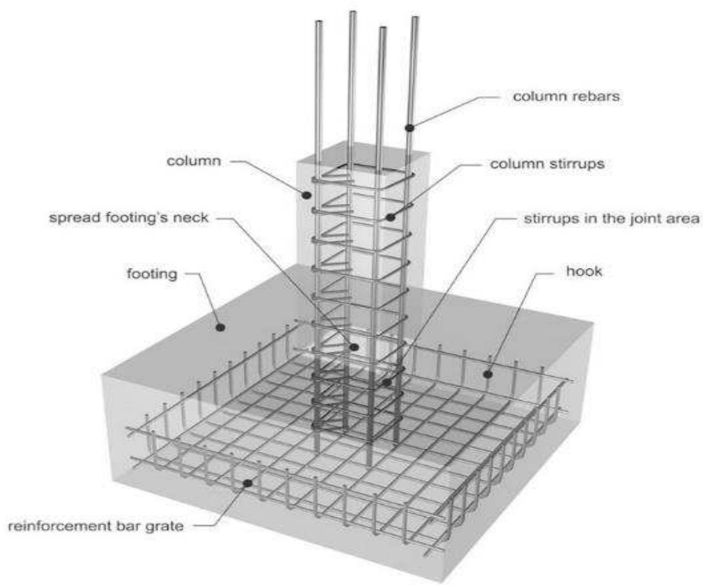



**Figure 59 Isolated Foundation**

## 4.3 Design of Isolated Foundation:

Analysis of Loading: defined the dead load and live load

| 1- Dead Load | |
|---|---|
| Slab Weight = Thickness × Area × Density | Total |
| 0.25 m × ( 12 m × 6 m) × 2.5 $t/m^3$ | 45 ton |
| Beam Weight = Weight × Area | |
| 0.187 t/m × ( 12 m + 6 m) | 3.37 ton |
| Sub-beam Weight = Weight × Area | |
| 0.089 t/m × ( 6 m × 4 m) | 2.14 ton |
| Column Weight = 0.125 t/m × 12 m | 1.5 ton |
| Ceramic Weight = 0.15 t/m × (12 m × 6 m) | 2.7 ton |
| **Total Dead Load =** | 54.70 ton |
| **2- Live Load** | |
| Live Load = | Total |
| 0.5 t/m × (12 m × 6 m) | 36 ton |

Properties and analysis to design the Foundation:

| | Total |
|---|---|
| (A) Dead Load $_{\text{(roof+1st floor)}}$ = | 109 ton |
| (B) Live Load $_{\text{(roof+1st floor)}}$ = | 72 ton |
| (C) $\gamma_s = \frac{2}{1+\beta}$ $\frac{2}{1 + 0.85}$ | 1.081 t/m^3 |
| (D) Fy = | 42,000 ton/m^2 |
| (E) $\gamma_c$ = | 2.50 ton/m^3 |
| (F) F'c = 30 MPa | 2500 ton/m^2 |
| (G) $\lambda = \frac{Kl}{r} =$ | 1 |
| (H) $\beta = \rightarrow$ For concrete Fy = 40 MPa | 0.85 |
| (I) Ø $\text{Steel}_{\text{Diameter}}$ 1.60 cm | 0.016 m |
| (J) Ø $\text{Steel}_{\text{area}}$ = | 0.0126 m^2 |
| $\text{Steel}_{\text{Diameter}}$ According to the steel table from the European specification (53-62) | |

Before proceeding a 90cm thick foundation-base is assumed:

| Assuming the footing thickness: | | | | |
|---|---|---|---|---|
| Isolated Footing Thickness (90 cm) | | | | |
| | (hc) width = | 90.0 cm | $\longrightarrow$ | 0.90 m |
| Total depth → | (df) depth = | 200.0 cm | $\longrightarrow$ | 2.00 m |

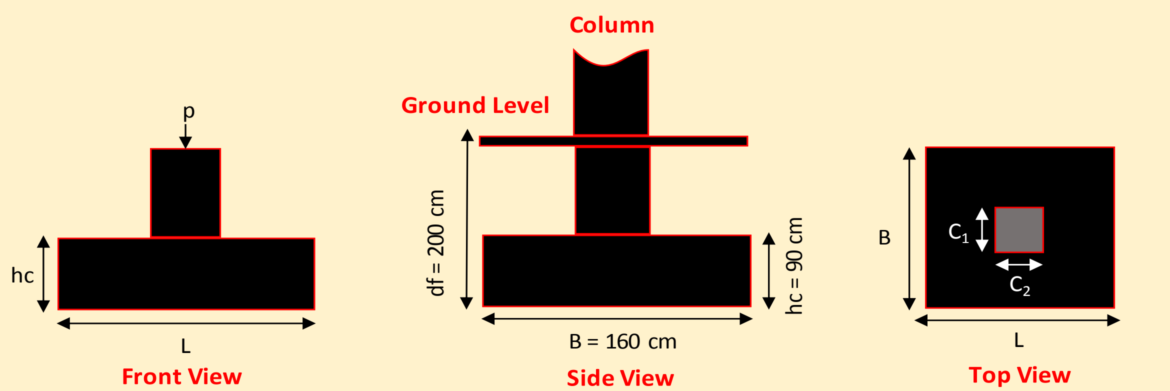


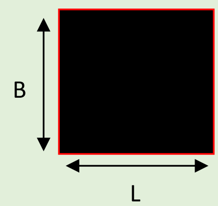

**Figure 60 Foundation thickness and height assumption**

The calculation are made based on ACI code and all the details are included in the Excel sheet in order to design the foundation:

### 1- Gross soil pressure

Assuming the area:

Area = (L x B) $(0.7\ m \times 0.7\ m)$ — Total: 0.49 m^2




Use one way slab : $\frac{L}{B} \leq 2$

$Saftey\ Factor : \text{Ø} = 2.5$

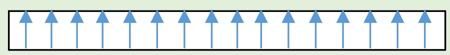
$$q_{all(gross)} = \frac{p}{\text{Ø} \times A} = \frac{90.702}{2.5 \times 0.49}$$

Total: 74.04 ton/m^2

According to ACI code saftey factor can be used Ø 2.5 - 3.0

### 2- Allowable net soil

$q_{all\ (net)} = q_{all\ (gross)} - \gamma_c\ (hc) - \gamma_s\ (d_f - h_c)$



$q_{all\ (net)} = 74.04 - 2.5 \times (0.9) - 1.081 \times (2.00 - 0.9)$ — 70.60 ton/m^2

### 3- Checking the Area

$$A_{req} = \frac{P_D + P_L}{q_{all\ (net)}}$$

$$A_{req} = \frac{109 + 72}{70.60}$$

Total: 2.57 m^2

$B = \sqrt{A_{req}}$

$B = \sqrt{2.57}$ Because its one way slab then B = L — Total: 1.6 m

### 4- Soil Pressure

$P_u =$ 1.2 x Dead Load + 1.6 x Live Load

$P_u = 1.2 \times 109.4 + 1.6 \times 72.0$ — Total: 246.48 ton

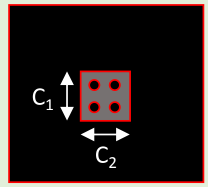
$$q_u\ (net) = \frac{1.2_{PD} + 1.6_{PL}}{L \times B} = \frac{246.48}{2.57}$$

Total: 95.93 ton/m^2

## 5- Checking the thickness for Punctuing shear

| Diameter: | Total |
|---|---|
| $Diameter_{avg} = h_c - 7.5cm - Bardiameter$ | |
| $Diameter_{avg} = 90 - 7.5 - 0.02$ | 82 cm |
| | 0.825 m |

| Bar Diameter: | |
|---|---|
| $Bar_{diameter} = C + d_{avg}$ | |
| 1- 0.400 + 0.825 | 1.225 m |
| 2- 0.400 + 0.825 | 1.225 m |




According to the steel table from the European specification (53-62)

### Actual Area of Steel

Ht = Hc

$$Asfy = (0.85\,f'c\,a \times b)$$

$$As = \left(\frac{0.85\,f'c\,a \times b}{fy}\right)$$

$$As = \frac{0.85 \times 2500.00 \times 0.49 \times 1.60}{42000}$$ 0.0397 m^2

→ 1.562 in^2

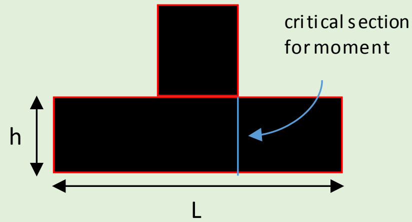
Table 4. Areas of Multiple of Reinforcing Bars ($in^2$)

| Number of bars | Bar number #3 | #4 | $5 | #6 | #7 | #8 | #9 | #10 | #11 |
|---|---|---|---|---|---|---|---|---|---|
| 1 | 0.11 | 0.20 | 0.31 | 0.44 | 0.60 | 0.79 | 1.00 | 1.27 | 1.56 |
| 2 | 0.22 | 0.40 | 0.62 | 0.88 | 1.20 | 1.58 | 2.00 | 2.54 | 3.12 |
| 3 | 0.33 | 0.60 | 0.93 | 1.32 | 1.80 | 2.37 | 3.00 | 3.81 | 4.68 |
| 4 | 0.44 | 0.80 | 1.24 | 1.76 | 2.40 | 3.16 | 4.00 | 5.08 | 6.24 |
| 5 | 0.55 | 1.00 | 1.55 | 2.20 | 3.00 | 3.95 | 5.00 | 6.35 | 7.80 |
| 6 | 0.66 | 1.20 | 1.86 | 2.64 | 3.60 | 4.74 | 6.00 | 7.62 | 9.36 |
| 7 | 0.77 | 1.40 | 2.17 | 3.08 | 4.20 | 5.53 | 7.00 | 8.89 | 10.92 |
| 8 | 0.88 | $\boxed{1.60}$ | 2.48 | 3.52 | 4.80 | 6.32 | 8.00 | 10.16 | 12.48 |
| 9 | 0.99 | 1.80 | 2.79 | 3.96 | 5.40 | 7.11 | 9.00 | 11.43 | 14.04 |
| 10 | 1.10 | 2.00 | 3.10 | 4.40 | 6.00 | 7.90 | 10.00 | 12.70 | 15.60 |

Take 8#4 Ø → 8 No. 4 bars

**Shear:**

Formula for concrete column we use:




Ultimate shear ($V_u$) :

$V_u = q_u\,(net) \times [(L) \times B - (C_1 + d) \times (C_2 + d)]$

$V_u = (95.93) \times [(1.60) \times 1.60 - (1.225) \times (1.225)]$ — 102.56 ton

**($B_0$) Checking:**

$B_0 = 4 \times C + davg$

$B_0 = 4 \times (0.4 + 0.825)$ — 4.90 m

**Concrete shear ($V_c$) :**

$V_c = 0.53 \times \sqrt{f'c} \times \left(1 + \dfrac{2}{\beta}\right) \times \lambda \times b_0 \times d$ ---------→ ❶

$0.53 \times \sqrt{2500} \times 1 + \left(\dfrac{2}{0.85}\right) \times 1 \times 4.90 \times 0.825$ — 359.07 ton

$V_c = \lambda \times \sqrt{f'c} \times b_0 \times d$ ---------→ ❷

$1 \times \sqrt{2500} \times 4.90 \times 0.825$ — 202.06 ton

$V_c = 0.27 \times \left(\dfrac{A_s \times d}{b_0} + 2\right) x\,\lambda \times \sqrt{f'c} \times b_0 \times d$ ---------→ ❸

$0.27 \times \left(\dfrac{0.0126 \times 0.825}{4.90} + 2\right) \times 1 \times \sqrt{2500} \times 4.90 \times 0.825$ — 109.23 ton

**Check:**

| Vc | > | Vu |
|---|---|---|
| 109.23 | > | 102.56 |

Safe

## 6- Checking for beam shear

| | Total |
|---|---|
| $ØV_c = 0.75 \times 0.53 \times \sqrt{f'c} \times L \times d$ | |
| $ØV_c = 0.75 \times 0.53 \times \sqrt{2500} \times 1.60 \times 0.825$ | 26.28 ton |
| $V_u = q_u(net)\, L \left[\left(\frac{B - C_1}{2}\right) - d\right]$ | |
| $V_u = 95.9 \times 1.60 \left[\frac{1.60 - 0.400}{2} - 0.825\right]$ | -34.35 ton |

Check:

| $ØV_c$ | > | $V_u$ |
|---|---|---|
| 26.28 | > | -34.35 |

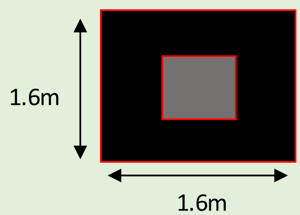
Safe

## 7- Compute the area of flexural reinforcement in each direction

| | Total |
|---|---|
| a- Reinforcement in the long direction: | |
| $Mu = qu(net)\frac{B}{2} \times \left(\frac{L - C_2}{2}^2\right)$ | |
| $95.93 \times \left(\frac{1.60}{2}\right) \times \left(\frac{1.60 - 0.40}{2}^2\right)$ | 117.09 ton-m^2 |
| b- Reinforcement in the short direction: | |
| $Mu = qu(net)\frac{L}{2} \times \left(\frac{B - C_1}{2}^2\right)$ | |
| $95.93 \times \left(\frac{1.60}{2}\right) \times \left(\frac{1.60 - 0.40}{2}^2\right)$ | 117.09 ton-m^2 |

## 8- Check for bearing strength of column and footing concrete

| For supported column, the bearing capacity $\phi P_n$ is: | Total |
|---|---|
| $\phi Pn = \phi(0.85\, f'c\, A_1)$ | |
| $\phi = 0.65$ | |
| $A_1$= Cross sectional area of column | |
| Critical column area = 159 cm^2 → 1.59 m^2 | |
| $0.65 \times (0.85 \times 2500 \times 1.59)$ | 2196.2 ton/m^2 |
| $\phi Pn = 2\phi(0.85\, f'c\, A_1)$ | |
| $2 \times 0.65 \times (0.85 \times 2500 \times 1.59)$ | 4392.4 ton/m^2 |
| $\phi$ = Strength reduction factor for bearing = 0.65 according to ACI code | |
| **For a supporting footing:** | |
| A2= Cross sectional area of lower base: | |
| A2 = 2.6 m^2 | |
| $\phi Pn = \phi(0.85\, f'c\, A_1)\sqrt{\frac{A_2}{A_1}} \leq 2.0\phi\,(0.85\, f'c\, A_1)$ | |
| $0.65 \times (0.85 \times 2500 \times 1.59) \times \sqrt{\frac{2.6}{1.59}}$ | 2791.78 ton/m^2 |
| **Check:** | |
| $\phi P_n$ column > $\phi P_n$ footing | |
| 4392.38 > 2791.78 | |
| **For columns, minimum dowel reinforcement is given by ACI Code 15.8.2.1 as:** | |
| $A_{s,\min} = 0.005\, Ag$ | |
| $0.005 \times 159$ cm^2 | 0.795 cm^2 |
| → 1.59 m^2 | 0.00795 m^2 |
| **Required dowel reinforcement is give by:** | |
| $A_{s,req} = \frac{(\phi P_n - Pu)}{\phi\, fy}$ | |
| $\frac{(2791.78 - 246.48)}{0.65 \times 2500}$ | 1.57 m^2 |



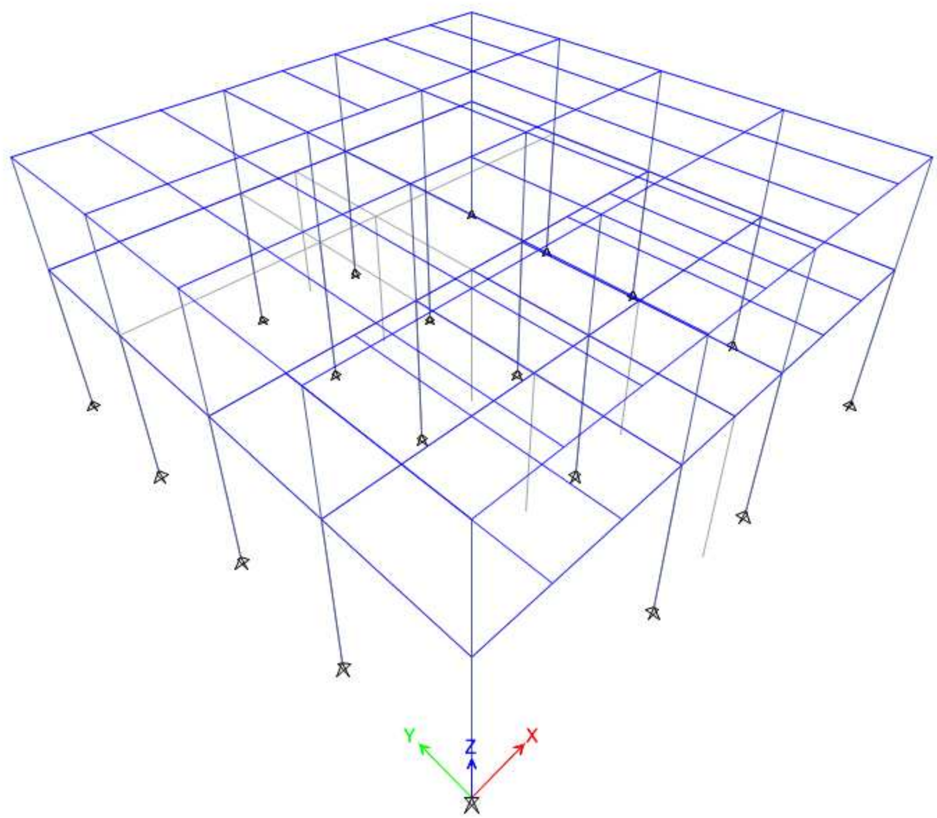
# Appendix

## Appendix A: Analysis & design report using ETABS 2015

ETABS® 2015

Integrated Building Design Software

---



### 2.6.1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

**a) Story Data**

**Story Data**

| Name | Height mm | Elevation mm | Master Story | Similar To | Splice Story |
|---|---|---|---|---|---|
| Story2 | 5000 | 12000 | Yes | None | No |
| Story1 | 7000 | 7000 | No | Story2 | No |
| Base | 0 | 0 | No | None | No |

**b) Grid Data**

**Grid Systems**

| Name | Type | Story Range | X Origin m | Y Origin m | Rotation deg | Bubble Size mm | Color |
|---|---|---|---|---|---|---|---|
| G1 | Cartesian | Default | 0 | 0 | 0 | 1250 | |

**Table 1.3 - Grid Lines**

| Grid System | Grid Direction | Grid ID | Visible | Bubble Location | Ordinate m |
|---|---|---|---|---|---|
| G1 | X | A | Yes | End | 0 |
| G1 | X | B | Yes | End | 9 |
| G1 | X | C | Yes | End | 12 |
| G1 | X | D | Yes | End | 15 |
| G1 | X | E | Yes | End | 24 |
| G1 | Y | 1 | Yes | Start | 0 |
| G1 | Y | 2 | Yes | Start | 6 |
| G1 | Y | 3 | Yes | Start | 12 |
| G1 | Y | 4 | Yes | Start | 18 |
| G1 | Y | 5 | Yes | Start | 24 |

c) **Point Coordinates**

**Joint Coordinates Data**

| Label | X mm | Y mm | ΔZ Below mm |
|---|---|---|---|
| 1 | 24000 | 0 | 0 |
| 2 | 12000 | 0 | 0 |
| 3 | 0 | 0 | 0 |
| 4 | 24000 | 6000 | 0 |
| 5 | 12000 | 6000 | 0 |
| 6 | 0 | 6000 | 0 |
| 7 | 24000 | 12000 | 0 |
| 9 | 0 | 12000 | 0 |
| 10 | 24000 | 18000 | 0 |
| 12 | 0 | 18000 | 0 |
| 13 | 24000 | 24000 | 0 |
| 14 | 12000 | 24000 | 0 |
| 15 | 0 | 24000 | 0 |
| 16 | 15000 | 12000 | 0 |
| 17 | 9000 | 12000 | 0 |
| 8 | 15000 | 18000 | 0 |
| 18 | 9000 | 18000 | 0 |
| 11 | 12000 | 12000 | 0 |
| 19 | 12000 | 18000 | 0 |
| 20 | 3000 | 0 | 0 |
| 21 | 6000 | 0 | 0 |
| 22 | 9000 | 0 | 0 |
| 23 | 15000 | 0 | 0 |
| 24 | 18000 | 0 | 0 |
| 25 | 21000 | 0 | 0 |
| 26 | 15000 | 6000 | 0 |
| 27 | 18000 | 6000 | 0 |
| 28 | 21000 | 6000 | 0 |
| 29 | 3000 | 6000 | 0 |
| 30 | 6000 | 6000 | 0 |
| 31 | 9000 | 6000 | 0 |
| 32 | 21000 | 12000 | 0 |
| 33 | 18000 | 12000 | 0 |
| 34 | 6000 | 12000 | 0 |

| Label | X mm | Y mm | ΔZ Below mm |
|---|---|---|---|
| 35 | 3000 | 12000 | 0 |
| 36 | 21000 | 18000 | 0 |
| 37 | 18000 | 18000 | 0 |
| 38 | 6000 | 18000 | 0 |
| 39 | 3000 | 18000 | 0 |
| 40 | 21000 | 24000 | 0 |
| 41 | 18000 | 24000 | 0 |
| 42 | 6000 | 24000 | 0 |
| 43 | 3000 | 24000 | 0 |
| 44 | 15000 | 24000 | 0 |
| 45 | 9000 | 24000 | 0 |

**d) line Connectivity**

**Column Connectivity Data**

| Column | I-End Point | J-End Point | I-End Story |
|---|---|---|---|
| C1 | 1 | 1 | Below |
| C3 | 3 | 3 | Below |
| C4 | 4 | 4 | Below |
| C5 | 5 | 5 | Below |
| C6 | 6 | 6 | Below |
| C7 | 7 | 7 | Below |
| C9 | 9 | 9 | Below |
| C10 | 10 | 10 | Below |
| C12 | 12 | 12 | Below |
| C13 | 13 | 13 | Below |
| C15 | 15 | 15 | Below |
| C16 | 16 | 16 | Below |
| C17 | 17 | 17 | Below |
| C8 | 8 | 8 | Below |
| C18 | 18 | 18 | Below |
| C11 | 44 | 44 | Below |
| C21 | 45 | 45 | Below |
| C14 | 23 | 23 | Below |
| C19 | 22 | 22 | Below |

**Beam Connectivity Data**

| Beam | I-End Point | J-End Point | Curve Type |
|---|---|---|---|
| B1 | 1 | 4 | None |
| B2 | 4 | 7 | None |
| B3 | 7 | 10 | None |
| B4 | 10 | 13 | None |
| B7 | 12 | 15 | None |
| B8 | 9 | 12 | None |
| B9 | 6 | 9 | None |
| B10 | 3 | 6 | None |
| B13 | 2 | 5 | None |
| B14 | 5 | 4 | None |
| B15 | 6 | 5 | None |
| B16 | 16 | 7 | None |
| B17 | 17 | 16 | None |
| B18 | 9 | 17 | None |
| B19 | 5 | 11 | None |
| B20 | 8 | 10 | None |
| B21 | 18 | 8 | None |
| B22 | 12 | 18 | None |
| B23 | 19 | 14 | None |
| B24 | 16 | 8 | None |
| B25 | 17 | 18 | None |
| B42 | 25 | 28 | None |
| B43 | 24 | 27 | None |
| B44 | 23 | 26 | None |
| B45 | 22 | 31 | None |
| B46 | 21 | 30 | None |
| B47 | 20 | 29 | None |
| B48 | 28 | 32 | None |
| B49 | 27 | 33 | None |
| B50 | 26 | 16 | None |
| B51 | 31 | 17 | None |
| B52 | 30 | 34 | None |
| B53 | 29 | 35 | None |
| B26 | 32 | 36 | None |
| B27 | 33 | 37 | None |
| B28 | 34 | 38 | None |
| B29 | 35 | 39 | None |
| B30 | 36 | 40 | None |

| Beam | I-End Point | J-End Point | Curve Type |
|---|---|---|---|
| B31 | 37 | 41 | None |
| B32 | 38 | 42 | None |
| B33 | 39 | 43 | None |
| B34 | 8 | 44 | None |
| B35 | 18 | 45 | None |
| B36 | 44 | 13 | None |
| B37 | 14 | 44 | None |
| B39 | 45 | 14 | None |
| B40 | 15 | 45 | None |
| B5 | 45 | 44 | None |
| B60 | 3 | 22 | None |
| B62 | 23 | 1 | None |
| B63 | 22 | 23 | None |

### e) Mass

**Mass Source**

| Name | Include Elements | Include Added Mass | Include Loads | Include Lateral | Include Vertical | Lump at Stories | IsDefault |
|---|---|---|---|---|---|---|---|
| MsSrc1 | Yes | Yes | No | Yes | No | Yes | Yes |

### f) Groups

**Group Definitions**

| Name | Color |
|---|---|
| All | Yellow |

### 2.6.2 Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

#### a) Materials

**Material Properties - Summary**

| Name | Type | E MPa | ν | Unit Weight $kN/m^3$ | Design Strengths |
|---|---|---|---|---|---|
| 4000Psi | Concrete | 24855.58 | 0.2 | 23.5631 | Fc=27.58 MPa |
| A615Gr60 | Rebar | 199947.98 | 0.3 | 76.9729 | Fy=413.69 MPa, Fu=620.53 MPa |

#### b) Frame Sections

**Frame Sections - Summary**

| Name | Material | Shape |
|---|---|---|
| Column | 4000Psi | Concrete Rectangular |
| Girder | 4000Psi | Concrete Rectangular |
| Sub-beam | 4000Psi | Concrete Rectangular |

#### c) Reinforcement Sizes

**Reinforcing Bar Sizes**

| Name | Diameter mm | Area $mm^2$ |
|---|---|---|
| 10 | 10 | 79 |
| 20 | 20 | 314 |

### 2.6.3 Assignments

This chapter provides a listing of the assignments applied to the model.

#### a) Joint Assignments

**Joint Assignments - Summary**

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Story2 | 1 | 3 | From Area | |
| Story2 | 2 | 6 | From Area | |
| Story2 | 3 | 9 | From Area | |
| Story2 | 4 | 14 | From Area | |
| Story2 | 5 | 16 | From Area | |
| Story2 | 6 | 18 | From Area | |
| Story2 | 7 | 23 | From Area | |
| Story2 | 9 | 27 | From Area | |
| Story2 | 10 | 32 | From Area | |
| Story2 | 12 | 36 | From Area | |
| Story2 | 13 | 41 | From Area | |
| Story2 | 14 | 43 | From Area | |
| Story2 | 15 | 45 | From Area | |
| Story2 | 16 | 48 | From Area | |
| Story2 | 17 | 51 | From Area | |
| Story2 | 8 | 52 | From Area | |
| Story2 | 18 | 54 | From Area | |
| Story2 | 11 | 33 | From Area | |
| Story2 | 19 | 34 | From Area | |
| Story2 | 20 | 81 | From Area | |
| Story2 | 21 | 79 | From Area | |
| Story2 | 22 | 77 | From Area | |
| Story2 | 23 | 75 | From Area | |
| Story2 | 24 | 73 | From Area | |
| Story2 | 25 | 71 | From Area | |
| Story2 | 26 | 76 | From Area | |
| Story2 | 27 | 74 | From Area | |
| Story2 | 28 | 72 | From Area | |
| Story2 | 29 | 82 | From Area | |
| Story2 | 30 | 80 | From Area | |
| Story2 | 31 | 78 | From Area | |
| Story2 | 32 | 83 | From Area | |
| Story2 | 33 | 84 | From Area | |

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Story2 | 34 | 85 | From Area | |
| Story2 | 35 | 86 | From Area | |
| Story2 | 36 | 87 | From Area | |
| Story2 | 37 | 88 | From Area | |
| Story2 | 38 | 89 | From Area | |
| Story2 | 39 | 90 | From Area | |
| Story2 | 40 | 91 | From Area | |
| Story2 | 41 | 92 | From Area | |
| Story2 | 42 | 93 | From Area | |
| Story2 | 43 | 94 | From Area | |
| Story2 | 44 | 95 | From Area | |
| Story2 | 45 | 96 | From Area | |
| Story1 | 1 | 2 | From Area | |
| Story1 | 2 | 5 | From Area | |
| Story1 | 3 | 8 | From Area | |
| Story1 | 4 | 13 | From Area | |
| Story1 | 5 | 15 | From Area | |
| Story1 | 6 | 17 | From Area | |
| Story1 | 7 | 22 | From Area | |
| Story1 | 9 | 26 | From Area | |
| Story1 | 10 | 31 | From Area | |
| Story1 | 12 | 35 | From Area | |
| Story1 | 13 | 40 | From Area | |
| Story1 | 15 | 44 | From Area | |
| Story1 | 16 | 47 | From Area | |
| Story1 | 17 | 50 | From Area | |
| Story1 | 8 | 25 | From Area | |
| Story1 | 18 | 53 | From Area | |
| Story1 | 11 | 29 | From Area | |
| Story1 | 20 | 55 | From Area | |
| Story1 | 21 | 56 | From Area | |
| Story1 | 22 | 57 | From Area | |
| Story1 | 23 | 58 | From Area | |
| Story1 | 24 | 59 | From Area | |
| Story1 | 25 | 60 | From Area | |
| Story1 | 26 | 61 | From Area | |
| Story1 | 27 | 62 | From Area | |
| Story1 | 28 | 63 | From Area | |
| Story1 | 29 | 64 | From Area | |

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Story1 | 30 | 65 | From Area | |
| Story1 | 31 | 66 | From Area | |
| Story1 | 32 | 67 | From Area | |
| Story1 | 33 | 68 | From Area | |
| Story1 | 34 | 69 | From Area | |
| Story1 | 35 | 70 | From Area | |
| Story1 | 44 | 38 | From Area | |
| Story1 | 45 | 98 | From Area | |
| Base | 1 | 1 | From Area | UX; UY; UZ |
| Base | 3 | 7 | From Area | UX; UY; UZ |
| Base | 4 | 10 | From Area | UX; UY; UZ |
| Base | 5 | 11 | From Area | UX; UY; UZ |
| Base | 6 | 12 | From Area | UX; UY; UZ |
| Base | 7 | 19 | From Area | UX; UY; UZ |
| Base | 9 | 21 | From Area | UX; UY; UZ |
| Base | 10 | 28 | From Area | UX; UY; UZ |
| Base | 12 | 30 | From Area | UX; UY; UZ |
| Base | 13 | 37 | From Area | UX; UY; UZ |
| Base | 15 | 39 | From Area | UX; UY; UZ |
| Base | 16 | 46 | From Area | UX; UY; UZ |
| Base | 17 | 49 | From Area | UX; UY; UZ |
| Base | 8 | 20 | From Area | UX; UY; UZ |
| Base | 18 | 24 | From Area | UX; UY; UZ |
| Base | 22 | 97 | From Area | UX; UY; UZ |
| Base | 23 | 42 | From Area | UX; UY; UZ |
| Base | 44 | 100 | From Area | UX; UY; UZ |
| Base | 45 | 107 | From Area | UX; UY; UZ |

## b) Frame Assignments

**Frame Assignments - Summary**

| Story | Label | Unique Name | Design Type | Length mm | Analysis Section | Design Section | Max Station Spacing mm | Min Number Stations |
|---|---|---|---|---|---|---|---|---|
| Story2 | C1 | 2 | Column | 5000 | Column | Column | | 3 |
| Story2 | C3 | 6 | Column | 5000 | Column | Column | | 3 |
| Story2 | C4 | 8 | Column | 5000 | Column | Column | | 3 |
| Story2 | C5 | 10 | Column | 5000 | Column | Column | | 3 |
| Story2 | C6 | 12 | Column | 5000 | Column | Column | | 3 |
| Story2 | C7 | 14 | Column | 5000 | Column | Column | | 3 |
| Story2 | C9 | 18 | Column | 5000 | Column | Column | | 3 |
| Story2 | C10 | 20 | Column | 5000 | Column | Column | | 3 |
| Story2 | C12 | 24 | Column | 5000 | Column | Column | | 3 |
| Story2 | C13 | 26 | Column | 5000 | Column | Column | | 3 |
| Story2 | C15 | 30 | Column | 5000 | Column | Column | | 3 |
| Story2 | C16 | 32 | Column | 5000 | Column | Column | | 3 |
| Story2 | C17 | 34 | Column | 5000 | Column | Column | | 3 |
| Story2 | C8 | 16 | Column | 5000 | Column | Column | | 3 |
| Story2 | C18 | 36 | Column | 5000 | Column | Column | | 3 |
| Story2 | C11 | 117 | Column | 5000 | Column | Column | | 3 |
| Story2 | C21 | 129 | Column | 5000 | Column | Column | | 3 |
| Story2 | C14 | 40 | Column | 5000 | Column | Column | | 3 |
| Story2 | C19 | 119 | Column | 5000 | Column | Column | | 3 |
| Story1 | C1 | 1 | Column | 7000 | Column | Column | | 3 |
| Story1 | C3 | 5 | Column | 7000 | Column | Column | | 3 |
| Story1 | C4 | 7 | Column | 7000 | Column | Column | | 3 |
| Story1 | C5 | 9 | Column | 7000 | Column | Column | | 3 |
| Story1 | C6 | 11 | Column | 7000 | Column | Column | | 3 |
| Story1 | C7 | 13 | Column | 7000 | Column | Column | | 3 |
| Story1 | C9 | 17 | Column | 7000 | Column | Column | | 3 |
| Story1 | C10 | 19 | Column | 7000 | Column | Column | | 3 |
| Story1 | C12 | 23 | Column | 7000 | Column | Column | | 3 |
| Story1 | C13 | 25 | Column | 7000 | Column | Column | | 3 |
| Story1 | C15 | 29 | Column | 7000 | Column | Column | | 3 |
| Story1 | C16 | 31 | Column | 7000 | Column | Column | | 3 |
| Story1 | C17 | 33 | Column | 7000 | Column | Column | | 3 |
| Story1 | C8 | 15 | Column | 7000 | Column | Column | | 3 |

| Story | Label | Unique Name | Design Type | Length mm | Analysis Section | Design Section | Max Station Spacing mm | Min Number Stations |
|---|---|---|---|---|---|---|---|---|
| Story1 | C18 | 35 | Column | 7000 | Column | Column | | 3 |
| Story1 | C11 | 116 | Column | 7000 | Column | Column | | 3 |
| Story1 | C21 | 128 | Column | 7000 | Column | Column | | 3 |
| Story1 | C14 | 39 | Column | 7000 | Column | Column | | 3 |
| Story1 | C19 | 118 | Column | 7000 | Column | Column | | 3 |
| Story2 | B1 | 54 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B2 | 55 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B3 | 56 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B4 | 57 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B7 | 60 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B8 | 61 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B9 | 62 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B10 | 63 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B13 | 66 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B14 | 67 | Beam | 12000 | Girder | Girder | 500 | |
| Story2 | B15 | 68 | Beam | 12000 | Girder | Girder | 500 | |
| Story2 | B16 | 69 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B17 | 70 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B18 | 71 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B19 | 72 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B20 | 73 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B21 | 74 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B22 | 75 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B23 | 76 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B24 | 77 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B25 | 78 | Beam | 6000 | Girder | Girder | 500 | |
| Story2 | B42 | 45 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B43 | 46 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B44 | 48 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B45 | 49 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B46 | 80 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |

| Story | Label | Unique Name | Design Type | Length mm | Analysis Section | Design Section | Max Station Spacing mm | Min Number Stations |
|---|---|---|---|---|---|---|---|---|
| Story2 | B47 | 81 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B48 | 82 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B49 | 84 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B50 | 85 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B51 | 86 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B52 | 88 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B53 | 89 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B26 | 90 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B27 | 92 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B28 | 93 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B29 | 94 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B30 | 107 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B31 | 108 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B32 | 109 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B33 | 110 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B34 | 111 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B35 | 112 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story2 | B36 | 114 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B37 | 59 | Beam | 3000 | Girder | Girder | 500 | |
| Story2 | B39 | 58 | Beam | 3000 | Girder | Girder | 500 | |

| Story | Label | Unique Name | Design Type | Length mm | Analysis Section | Design Section | Max Station Spacing mm | Min Number Stations |
|---|---|---|---|---|---|---|---|---|
| Story2 | B40 | 115 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B60 | 120 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B62 | 125 | Beam | 9000 | Girder | Girder | 500 | |
| Story2 | B63 | 123 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B1 | 21 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B2 | 22 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B3 | 37 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B4 | 38 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B7 | 41 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B8 | 42 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B9 | 43 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B10 | 44 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B13 | 47 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B14 | 87 | Beam | 12000 | Girder | Girder | 500 | |
| Story1 | B15 | 91 | Beam | 12000 | Girder | Girder | 500 | |
| Story1 | B16 | 50 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B17 | 51 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B18 | 52 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B19 | 53 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B42 | 95 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B43 | 96 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B44 | 97 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B45 | 98 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B46 | 99 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B47 | 100 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B48 | 101 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B49 | 102 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B50 | 103 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |

| Story | Label | Unique Name | Design Type | Length mm | Analysis Section | Design Section | Max Station Spacing mm | Min Number Stations |
|---|---|---|---|---|---|---|---|---|
| Story1 | B51 | 104 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B52 | 105 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B53 | 106 | Beam | 6000 | Sub-beam | Sub-beam | 500 | |
| Story1 | B36 | 27 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B40 | 113 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B5 | 28 | Beam | 6000 | Girder | Girder | 500 | |
| Story1 | B60 | 130 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B62 | 135 | Beam | 9000 | Girder | Girder | 500 | |
| Story1 | B63 | 133 | Beam | 6000 | Girder | Girder | 500 | |

### 2.6.4 Loads

This chapter provides loading information as applied to the model.

#### a) Load Patterns

**Load Patterns**

| Name | Type | Self-Weight Multiplier |
|---|---|---|
| Dead | Dead | 1 |
| Live | Live | 0 |

#### b) Applied Loads

**Frame Loads - Point**

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance | Absolute Distance mm | Force kN |
|---|---|---|---|---|---|---|---|---|---|
| Story2 | B17 | 70 | Beam | Dead | Force | Gravity | 0.5 | 3000 | 61.5 |
| Story2 | B21 | 74 | Beam | Dead | Force | Gravity | 0.5 | 3000 | 61.5 |
| Story2 | B24 | 77 | Beam | Dead | Force | Gravity | 0.5 | 3000 | 61.5 |
| Story2 | B25 | 78 | Beam | Dead | Force | Gravity | 0.5 | 3000 | 61.5 |

**Frame Loads - Distributed**

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start mm | Absolute Distance End mm | Force at Start kN/m | Force at End kN/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story2 | B1 | 54 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B2 | 55 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B3 | 56 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B4 | 57 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B7 | 60 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B8 | 61 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B9 | 62 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B10 | 63 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B13 | 66 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B19 | 72 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B23 | 76 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B24 | 77 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |
| Story2 | B25 | 78 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 9.197 | 9.197 |

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start mm | Absolute Distance End mm | Force at Start kN/m | Force at End kN/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story2 | B42 | 45 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B43 | 46 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B44 | 48 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B45 | 49 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B46 | 80 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B47 | 81 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B48 | 82 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B49 | 84 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B50 | 85 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B51 | 86 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B52 | 88 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B53 | 89 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B26 | 90 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B27 | 92 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B28 | 93 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B29 | 94 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B30 | 107 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B31 | 108 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B32 | 109 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B33 | 110 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B34 | 111 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story2 | B35 | 112 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 18.394 | 18.394 |
| Story1 | B1 | 21 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 11.004 | 11.004 |
| Story1 | B2 | 22 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 11.004 | 11.004 |
| Story1 | B3 | 37 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 0.5 | 0.5 |
| Story1 | B4 | 38 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 0.5 | 0.5 |
| Story1 | B7 | 41 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 0.5 | 0.5 |
| Story1 | B8 | 42 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 0.5 | 0.5 |
| Story1 | B9 | 43 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 11.004 | 11.004 |
| Story1 | B10 | 44 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 11.004 | 11.004 |
| Story1 | B13 | 47 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B16 | 50 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9000 | 0.5 | 0.5 |
| Story1 | B17 | 51 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 0.5 | 0.5 |
| Story1 | B18 | 52 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9000 | 0.5 | 0.5 |
| Story1 | B19 | 53 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B42 | 95 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B43 | 96 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B44 | 97 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B45 | 98 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start mm | Absolute Distance End mm | Force at Start kN/m | Force at End kN/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story1 | B46 | 99 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B47 | 100 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B48 | 101 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B49 | 102 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B50 | 103 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B51 | 104 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B52 | 105 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B53 | 106 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6000 | 20.509 | 20.509 |
| Story1 | B60 | 130 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3000 | 0.5 | 0.5 |
| Story1 | B60 | 130 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3000 | 6000 | 0.5 | 0.5 |
| Story1 | B60 | 130 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6000 | 9000 | 0.5 | 0.5 |
| Story1 | B62 | 135 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3000 | 0.5 | 0.5 |
| Story1 | B62 | 135 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3000 | 6000 | 0.5 | 0.5 |
| Story1 | B62 | 135 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6000 | 9000 | 0.5 | 0.5 |
| Story1 | B63 | 133 | Beam | Dead | Force | Gravity | 0 | 0.5 | 0 | 3000 | 0.5 | 0.5 |
| Story1 | B63 | 133 | Beam | Dead | Force | Gravity | 0.5 | 1 | 3000 | 6000 | 0.5 | 0.5 |
| Story2 | B1 | 54 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B2 | 55 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B3 | 56 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B4 | 57 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B7 | 60 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B8 | 61 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B9 | 62 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B10 | 63 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B13 | 66 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B17 | 70 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B19 | 72 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B21 | 74 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B23 | 76 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B24 | 77 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B25 | 78 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 3.75 | 3.75 |
| Story2 | B42 | 45 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B43 | 46 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start mm | Absolute Distance End mm | Force at Start kN/m | Force at End kN/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Story2 | B44 | 48 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B45 | 49 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B46 | 80 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B47 | 81 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B48 | 82 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B49 | 84 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B50 | 85 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B51 | 86 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B52 | 88 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B53 | 89 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B26 | 90 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B27 | 92 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B28 | 93 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B29 | 94 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B30 | 107 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B31 | 108 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B32 | 109 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B33 | 110 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B34 | 111 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story2 | B35 | 112 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story1 | B1 | 21 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story1 | B2 | 22 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story1 | B9 | 43 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story1 | B10 | 44 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 7.5 | 7.5 |
| Story1 | B13 | 47 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B19 | 53 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B42 | 95 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B43 | 96 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B44 | 97 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B45 | 98 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B46 | 99 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B47 | 100 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B48 | 101 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B49 | 102 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B50 | 103 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B51 | 104 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B52 | 105 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |
| Story1 | B53 | 106 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6000 | 15 | 15 |

## c) Load Cases

**Load Cases - Summary**

| Name | Type |
|---|---|
| Dead | Linear Static |
| Live | Linear Static |

## d) Load Combinations

**Load Combinations**

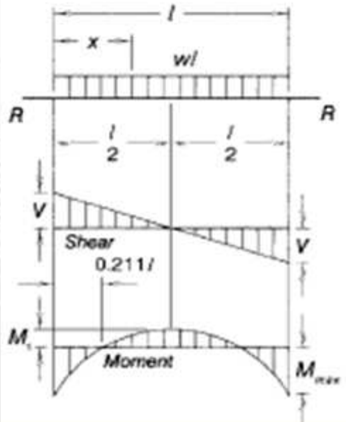
| Name | Load Case/Combo | Scale Factor | Type | Auto |
|---|---|---|---|---|
| dead+live 2015 | Dead | 1.2 | Linear Add | No |
| dead+live 2015 | Live | 1.6 | | No |
| DCon1 | Dead | 1.4 | Linear Add | Yes |
| DCon2 | Dead | 1.2 | Linear Add | Yes |
| DCon2 | Live | 1.6 | | No |

TABLE 4-1:
MINIMUM UNIFORMLY DISTRIBUTED LIVE LOADS, $L_o$, AND MINIMUM CONCENTRATED LIVE LOADS

| Occupancy or Use | Uniform $kN/m^2$ | Conc. kN |
|---|---|---|
| Apartments (see residential) | | |
| Access floor systems<br>Office use<br>Computer use | <br>2.5<br>5 | <br>9<br>9 |
| Armories and drill rooms | 7.5 | |
| Assembly areas and theaters<br>• Fixed seats (fastened to floor)<br>• Lobbies<br>• Movable seats<br>• Platforms (assembly)<br>• Stage floors | <br>3<br>5<br>5<br>5<br>7.5 | |
| Balconies (exterior)<br>On one- and two-family residences only, and not exceeding 10 $m^2$ | 5<br>3 | |
| Bowling alleys, poolrooms, and similar recreational areas | 4 | |
| Catwalks for maintenance access | 2 | 1.5 |
| Corridors<br>First floor<br>Other floors, same as occupancy served except as indicated | <br>5 | |
| Mosques | 5 | |
| Decks (patio and roof)<br>Same as area served, or for the type of occupancy accommodated | | |
| Dining rooms and restaurants | 5 | |
| Dwellings (see residential) | | |
| Elevator machine room grating (on area of 2500 $mm^2$) | | 1.5 |
| Finish light floor plate construction (on area of 650 $mm^2$) | | 1 |
| Fire escapes | 5 | |
| Fixed ladders | See Section 4.4 | |
| Garages (passenger vehicles only)<br>Trucks and buses | 2<br>Note (2) | Note (1)<br>Note (2) |
| Grandstands (see stadium and arena bleachers) | | |
| Gymnasiums, main floors, and balconies | 5 Note (4) | |
| Handrails, guardrails, and grab bars | See Section 4.4 | |
| Hospitals<br>• Operating rooms, laboratories<br>• Private rooms<br>• Wards<br>• Corridors above first floor | <br>3<br>2<br>2<br>4 | <br>4.5<br>4.5<br>4.5<br>4.5 |
| Hotels (see residential) | | |
| Libraries<br>• Reading rooms<br>• Stack rooms<br>• Corridors above first floor | <br>3<br>7.5 Note (3)<br>4 | <br>4.5<br>4.5<br>4.5 |
| Manufacturing<br>• Light<br>• Heavy | <br>6<br>12 | <br>9<br>13.5 |
| Marquees and canopies | 4 | |
| Office buildings<br>• File and computer rooms shall be designed for heavier loads based on anticipated occupancy:<br>• Lobbies and first floor corridors<br>• Offices<br>• Corridors above first floor | <br><br>5<br>2.5<br>4.0 | <br><br>9<br>9<br>9 |
| Penal institutions<br>Cell blocks<br>Corridors | <br>2<br>5 | |

## Table 3–23 (continued)
# Shears, Moments, and Deflections

### 15. BEAM FIXED AT BOTH ENDS — UNIFORMLY DISTRIBUTED LOADS




Total Equiv. Uniform Load ........ $= \frac{2wl}{3}$

$R = V$ ........ $= \frac{wl}{2}$

$V_x$ ........ $= w\left(\frac{l}{2} - x\right)$

$M_{max}$ (at ends) ........ $= \frac{wl^2}{12}$

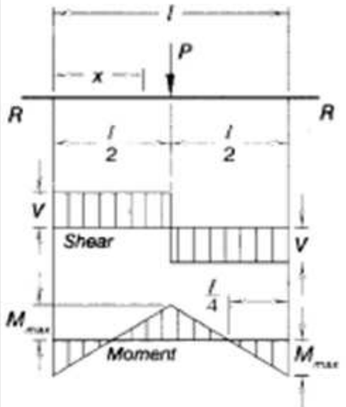
$M_1$ (at center) ........ $= \frac{wl^2}{24}$

$M_x$ ........ $= \frac{w}{12}\left(6lx - l^2 - 6x^2\right)$

$\Delta_{max}$ (at center) ........ $= \frac{wl^4}{384\,EI}$

$\Delta_x$ ........ $= \frac{wx^2}{24\,EI}(l - x)^2$

### 16. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT CENTER




Total Equiv. Uniform Load ........ $= P$

$R = V$ ........ $= \frac{P}{2}$

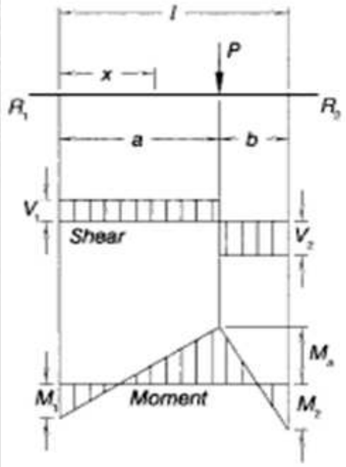
$M_{max}$ (at center and ends) ........ $= \frac{Pl}{8}$

$M_x$ (when $x < \frac{l}{2}$) ........ $= \frac{P}{8}(4x - l)$

$\Delta_{max}$ (at center) ........ $= \frac{Pl^3}{192\,EI}$

$\Delta_x$ (when $x < \frac{l}{2}$) ........ $= \frac{Px^2}{48\,EI}(3l - 4x)$

### 17. BEAM FIXED AT BOTH ENDS — CONCENTRATED LOAD AT ANY POINT




$R_1 = V_1$ ($= V_{max}$ when $a < b$) ........ $= \frac{Pb^2}{l^3}(3a + b)$

$R_2 = V_2$ ($= V_{max}$ when $a > b$) ........ $= \frac{Pa^2}{l^3}(a + 3b)$

$M_1$ ($= M_{max}$ when $a < b$) ........ $= \frac{Pab^2}{l^2}$

$M_2$ ($= M_{max}$ when $a > b$) ........ $= \frac{Pa^2b}{l^2}$

$M_a$ (at point of load) ........ $= \frac{2Pa^2b^2}{l^3}$

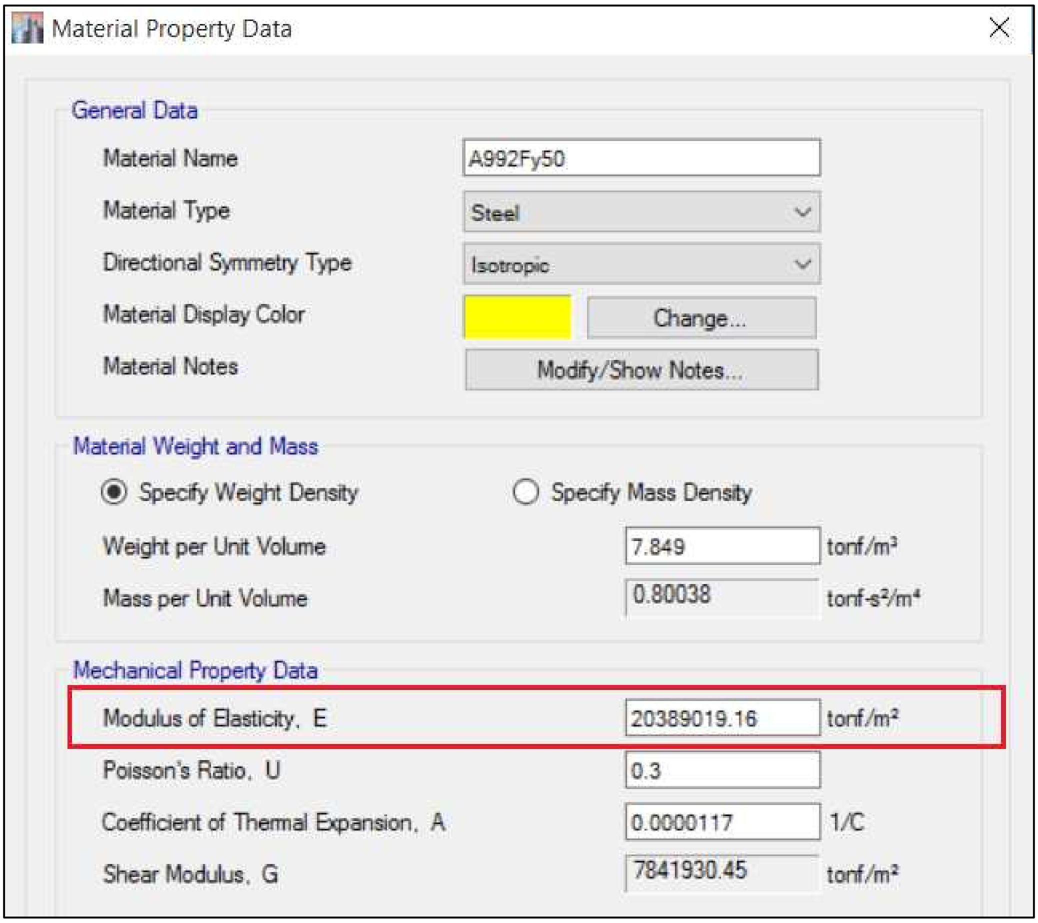
$M_x$ (when $x < a$) ........ $= R_1x - \frac{Pab^2}{l^2}$

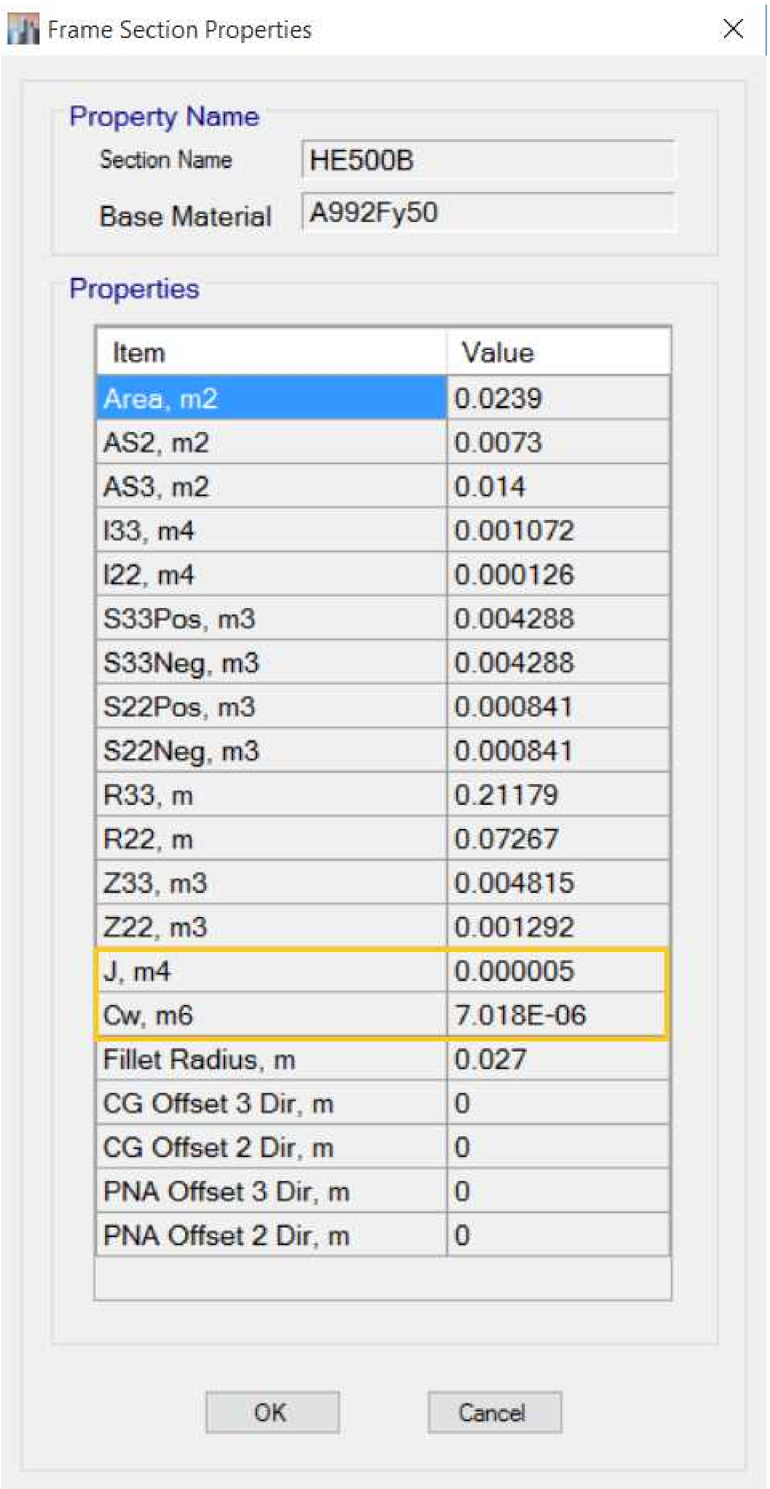
$\Delta_{max}$ (when $a > b$ at $x = \frac{2al}{3a+b}$) ........ $= \frac{2Pa^3b^2}{3EI(3a+b)^2}$

$\Delta_a$ (at point of load) ........ $= \frac{Pa^3b^3}{3EIl^3}$

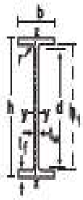
$\Delta_x$ (when $x < a$) ........ $= \frac{Pb^2x^2}{6EIl^3}(3al - 3ax - bx)$


Material Property Design Data
Material Name and Type
Material Name
A992Fy50
Material Type
Steel, Isotropic
Design Properties for Steel Materials
Minimum Yield Stress, Fy
35153.48
tonf/m²
Minimum Tensile Strength, Fu
45699.53
tonf/m²
Effective Yield Stress, Fye
38668.83
tonf/m²
Effective Tensile Strength, Fue
50269.48
tonf/m²


Material Property Data
General Data
Material Name
A992Fy50
Material Type
Steel
Directional Symmetry Type
Isotropic
Material Display Color
Change...
Material Notes
Modify/Show Notes...
Material Weight and Mass
Specify Weight Density
Specify Mass Density
Weight per Unit Volume
7.849
tonf/m³
Mass per Unit Volume
0.80038
tonf-s²/m⁴
Mechanical Property Data
Modulus of Elasticity, E
20389019.16
tonf/m²
Poisson's Ratio, U
0.3
Coefficient of Thermal Expansion, A
0.0000117
1/C
Shear Modulus, G
7841930.45
tonf/m²


Frame Section Properties
Property Name
Section Name HE500B
Base Material A992Fy50
Properties
Item Value
Area, m2 0.0239
AS2, m2 0.0073
AS3, m2 0.014
I33, m4 0.001072
I22, m4 0.000126
S33Pos, m3 0.004288
S33Neg, m3 0.004288
S22Pos, m3 0.000841
S22Neg, m3 0.000841
R33, m 0.21179
R22, m 0.07267
Z33, m3 0.004815
Z22, m3 0.001292
J, m4 0.000005
Cw, m6 7.018E-06
Fillet Radius, m 0.027
CG Offset 3 Dir, m 0
CG Offset 2 Dir, m 0
PNA Offset 3 Dir, m 0
PNA Offset 2 Dir, m 0
OK
Cancel

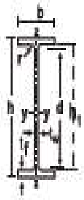
**EUROPEAN SPECIFICATION BEAMS WITH PARALLEL FLANGES IN ACCORDANCE WITH EURONORM 19-57**



| Designation | G | h | b | $t_w$ | $t_f$ | r | A | $h_1$ | d |
|---|---|---|---|---|---|---|---|---|---|
| | kg/m | mm | mm | mm | mm | mm | $cm^2$ | mm | mm |
| IPE 270 A | 30.7 | 267 | 135 | 5.5 | 8.7 | 15 | 39.1 | 249.6 | 219.6 |
| IPE 270 | 36.1 | 270 | 135 | 6.6 | 10.2 | 15 | 45.9 | 249.6 | 219.6 |
| IPE 270 O | 42.3 | 274 | 136 | 7.5 | 12.2 | 15 | 53.8 | 249.6 | 219.6 |
| IPE 270 R | 44.0 | 276 | 133 | 7.7 | 13.1 | 15 | 56.0 | 249.8 | 219.8 |
| IPE 300 A | 36.5 | 297 | 150 | 6.1 | 9.2 | 15 | 46.5 | 278.6 | 248.6 |
| IPE 300 | 42.2 | 300 | 150 | 7.1 | 10.7 | 15 | 53.8 | 278.6 | 248.6 |
| IPE 300 O | 49.3 | 304 | 152 | 8.0 | 12.7 | 15 | 62.8 | 278.6 | 248.6 |
| IPE 300 R | 51.7 | 306 | 147 | 8.5 | 13.7 | 15 | 65.9 | 278.6 | 248.6 |
| IPE 330 A | 43.0 | 327 | 160 | 6.5 | 10.0 | 18 | 54.7 | 307.0 | 271.0 |
| IPE 330 | 49.1 | 330 | 160 | 7.5 | 11.5 | 18 | 62.6 | 307.0 | 271.0 |
| IPE 330 O | 57.0 | 334 | 162 | 8.5 | 13.5 | 18 | 72.6 | 307.0 | 271.0 |
| IPE 330 R | 60.3 | 336 | 158 | 9.2 | 14.5 | 18 | 76.8 | 307.0 | 271.0 |
| IPE 360 A | 50.2 | 357.6 | 170 | 6.6 | 11.5 | 18 | 64.0 | 334.6 | 298.6 |
| IPE 360 | 57.1 | 360 | 170 | 8.0 | 12.7 | 18 | 72.7 | 334.6 | 298.6 |
| IPE 360 O | 66.0 | 364 | 172 | 9.2 | 14.7 | 18 | 84.1 | 334.6 | 298.6 |
| IPE 360 R | 70.3 | 366 | 168 | 9.9 | 16.0 | 18 | 89.6 | 334.0 | 298.0 |
| IPE 400 A | 57.4 | 397 | 180 | 7.0 | 12.0 | 21 | 73.1 | 373.0 | 331.0 |
| IPE 400 | 66.3 | 400 | 180 | 8.6 | 13.5 | 21 | 84.5 | 373.0 | 331.0 |
| IPE 400 O | 75.7 | 404 | 182 | 9.7 | 15.5 | 21 | 96.4 | 373.0 | 331.0 |
| IPE 400 R | 81.5 | 407 | 178 | 10.6 | 17.0 | 21 | 104 | 373.0 | 331.0 |
| IPE 400 V | 84.0 | 408 | 182 | 10.6 | 17.5 | 21 | 107 | 373.0 | 331.0 |
| IPE 450 A | 67.2 | 447 | 190 | 7.6 | 13.1 | 21 | 85.5 | 420.8 | 378.8 |
| IPE 450 | 77.6 | 450 | 190 | 9.4 | 14.6 | 21 | 98.8 | 420.8 | 378.8 |
| IPE 450 O | 92.4 | 456 | 192 | 11.0 | 17.6 | 21 | 118 | 420.8 | 378.8 |
| IPE 450 R | 95.2 | 458 | 188 | 11.3 | 18.6 | 21 | 121 | 420.8 | 378.8 |
| IPE 450 V | 104 | 460 | 194 | 12.4 | 19.6 | 21 | 132 | 420.8 | 378.8 |

38

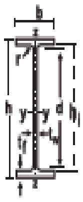
**EUROPEAN SPECIFICATION BEAMS WITH PARALLEL FLANGES IN ACCORDANCE WITH EURONORM 19-57**



| $I_y$ | $I_z$ | $i_y$ | $i_z$ | $W_y$ | $W_z$ | $W_{ply}$ | $W_{plz}$ | $I_w$ | $I_T$ |
|---|---|---|---|---|---|---|---|---|---|
| $cm^4$ | $cm^4$ | cm | cm | $cm^3$ | $cm^3$ | $cm^3$ | $cm^3$ | $dm^6$ | $cm^4$ |
| 4917 | 358 | 11.2 | 3.02 | 368 | 53.0 | 412 | 82.3 | 0.060 | 10.4 |
| 5790 | 420 | 11.2 | 3.02 | 429 | 62.2 | 484 | 97.0 | 0.071 | 15.9 |
| 6947 | 513 | 11.4 | 3.09 | 507 | 75.5 | 575 | 118 | 0.088 | 25.0 |
| 7312 | 516 | 11.4 | 3.03 | 530 | 77.6 | 602 | 121 | 0.089 | 29.1 |
| 7173 | 519 | 12.4 | 3.34 | 483 | 69.2 | 542 | 107 | 0.107 | 13.3 |
| 8356 | 604 | 12.5 | 3.35 | 557 | 80.5 | 628 | 125 | 0.126 | 19.9 |
| 9994 | 746 | 12.6 | 3.45 | 658 | 98.1 | 744 | 153 | 0.158 | 31.0 |
| 10500 | 728 | 12.6 | 3.32 | 686 | 99.0 | 780 | 155 | 0.155 | 37.0 |
| 10230 | 685 | 13.7 | 3.54 | 626 | 85.6 | 702 | 133 | 0.172 | 19.6 |
| 11770 | 788 | 13.7 | 3.55 | 713 | 98.5 | 804 | 154 | 0.200 | 28.1 |
| 13910 | 960 | 13.8 | 3.64 | 833 | 119 | 943 | 185 | 0.247 | 42.2 |
| 14690 | 958 | 13.8 | 3.53 | 874 | 121 | 995 | 190 | 0.247 | 50.6 |
| 14520 | 944 | 15.1 | 3.84 | 812 | 111 | 907 | 172 | 0.283 | 27.4 |
| 16270 | 1043 | 15.0 | 3.79 | 904 | 123 | 1019 | 191 | 0.315 | 37.4 |
| 19050 | 1251 | 15.0 | 3.86 | 1047 | 145 | 1186 | 227 | 0.382 | 55.7 |
| 20290 | 1270 | 15.0 | 3.76 | 1109 | 151 | 1262 | 236 | 0.389 | 68.7 |
| 20290 | 1171 | 16.7 | 4.00 | 1022 | 130 | 1144 | 202 | 0.434 | 36.2 |
| 23130 | 1318 | 16.5 | 3.95 | 1156 | 146 | 1307 | 229 | 0.492 | 51.3 |
| 26750 | 1564 | 16.7 | 4.03 | 1324 | 172 | 1502 | 269 | 0.590 | 73.3 |
| 28860 | 1606 | 16.7 | 3.93 | 1418 | 180 | 1618 | 284 | 0.611 | 92.5 |
| 30140 | 1766 | 16.8 | 4.06 | 1477 | 194 | 1681 | 304 | 0.673 | 99.6 |
| 29760 | 1502 | 18.7 | 4.19 | 1331 | 158 | 1494 | 246 | 0.707 | 47.1 |
| 33740 | 1676 | 18.5 | 4.12 | 1500 | 176 | 1702 | 276 | 0.794 | 66.7 |
| 40920 | 2085 | 18.6 | 4.21 | 1795 | 217 | 2046 | 341 | 1.00 | 109 |
| 42400 | 2070 | 18.7 | 4.13 | 1851 | 220 | 2115 | 346 | 0.999 | 123 |
| 46200 | 2397 | 18.7 | 4.26 | 2009 | 247 | 2301 | 389 | 1.16 | 149 |

39

## EUROPEAN SPECIFICATION BEAMS WITH PARALLEL FLANGES IN ACCORDANCE WITH EURONORM 53-62




| Designation | G | h | b | $t_w$ | $t_f$ | r | A | $h_i$ | d |
|---|---|---|---|---|---|---|---|---|---|
| | kg/m | mm | mm | mm | mm | mm | $cm^2$ | mm | mm |
| HE 280 AA | 61.2 | 264 | 280 | 7.0 | 10.0 | 24 | 78 | 244 | 196 |
| HE 280 A | 76.4 | 270 | 280 | 8.0 | 13.0 | 24 | 97.3 | 244 | 196 |
| HE 280 B | 103 | 280 | 280 | 10.5 | 18.0 | 24 | 131 | 244 | 196 |
| HE 280 M | 189 | 310 | 288 | 18.5 | 33.0 | 24 | 240 | 244 | 196 |
| HE 300 AA | 69.8 | 283 | 300 | 7.5 | 10.5 | 27 | 88.9 | 262 | 208 |
| HE 300 A | 88.3 | 290 | 300 | 8.5 | 14.0 | 27 | 113 | 262 | 208 |
| HE 300 B | 117 | 300 | 300 | 11.0 | 19.0 | 27 | 149 | 262 | 208 |
| HE 300 C | 177 | 320 | 305 | 16.0 | 29.0 | 27 | 225 | 262 | 208 |
| HE 300 M | 238 | 340 | 310 | 21.0 | 39.0 | 27 | 303 | 262 | 208 |
| HE 320 AA | 74.2 | 301 | 300 | 8.0 | 11.0 | 27 | 94.6 | 279 | 225 |
| HE 320 A | 97.6 | 310 | 300 | 9.0 | 15.5 | 27 | 124 | 279 | 225 |
| HE 320 B | 127 | 320 | 300 | 11.5 | 20.5 | 27 | 161 | 279 | 225 |
| HE 320 M | 245 | 359 | 309 | 21.0 | 40.0 | 27 | 312 | 279 | 225 |
| HE 340 AA | 78.9 | 320 | 300 | 8.5 | 11.5 | 27 | 101 | 297 | 243 |
| HE 340 A | 105 | 330 | 300 | 9.5 | 16.5 | 27 | 133 | 297 | 243 |
| HE 340 B | 134 | 340 | 300 | 12.0 | 21.5 | 27 | 171 | 297 | 243 |
| HE 340 M | 248 | 377 | 309 | 21.0 | 40.0 | 27 | 316 | 297 | 243 |
| HE 360 AA | 83.7 | 339 | 300 | 9.0 | 12.0 | 27 | 107 | 315 | 261 |
| HE 360 A | 112 | 350 | 300 | 10.0 | 17.5 | 27 | 143 | 315 | 261 |
| HE 360 B | 142 | 360 | 300 | 12.5 | 22.5 | 27 | 181 | 315 | 261 |
| HE 360 M | 250 | 395 | 308 | 21.0 | 40.0 | 27 | 319 | 315 | 261 |
| HE 400 AA | 92.4 | 378 | 300 | 9.5 | 13.0 | 27 | 118 | 352 | 298 |
| HE 400 x 107 | 107 | 384 | 297 | 10.0 | 16.0 | 27 | 136 | 352 | 298 |
| HE 400 A | 125 | 390 | 300 | 11.0 | 19.0 | 27 | 159 | 352 | 298 |
| HE 400 B | 155 | 400 | 300 | 13.5 | 24.0 | 27 | 198 | 352 | 298 |
| HE 400 M | 256 | 432 | 307 | 21.0 | 40.0 | 27 | 326 | 352 | 298 |
| HE 450 AA | 100 | 425 | 300 | 10.0 | 13.5 | 27 | 127 | 398 | 344 |
| HE 450 x 123 | 123 | 435 | 300 | 10.2 | 18.5 | 27 | 158 | 398 | 344 |
| HE 450 A | 140 | 440 | 300 | 11.5 | 21.0 | 27 | 178 | 398 | 344 |
| HE 450 B | 171 | 450 | 300 | 14.0 | 26.0 | 27 | 218 | 398 | 344 |
| HE 450 M | 263 | 478 | 307 | 21.0 | 40.0 | 27 | 335 | 398 | 344 |
| HE 500 AA | 107 | 472 | 300 | 10.5 | 14.0 | 27 | 137 | 444 | 390 |
| HE 500 A | 155 | 490 | 300 | 12.0 | 23.0 | 27 | 198 | 444 | 390 |
| HE 500 B | 187 | 500 | 300 | 14.5 | 28.0 | 27 | 239 | 444 | 390 |
| HE 500 M | 270 | 524 | 306 | 21.0 | 40.0 | 27 | 344 | 444 | 390 |

32


## EUROPEAN SPECIFICATION BEAMS WITH PARALLEL FLANGES IN ACCORDANCE WITH EURONORM 53-62



| $I_y$ | $I_z$ | $i_y$ | $i_z$ | $W_y$ | $W_z$ | $W_{ply}$ | $W_{plz}$ | $I_w$ | $I_T$ |
|---|---|---|---|---|---|---|---|---|---|
| $cm^4$ | $cm^4$ | cm | cm | $cm^3$ | $cm^3$ | $cm^3$ | $cm^3$ | $dm^6$ | $cm^4$ |
| 10560 | 3664 | 11.6 | 6.85 | 800 | 262 | 873 | 399 | 0.591 | 35.5 |
| 13670 | 4763 | 11.9 | 7.00 | 1013 | 340 | 1112 | 518 | 0.786 | 63.5 |
| 19270 | 6595 | 12.1 | 7.09 | 1376 | 471 | 1534 | 718 | 1.13 | 146 |
| 39550 | 13160 | 12.8 | 7.40 | 2551 | 914 | 2966 | 1397 | 2.52 | 807 |
| 13800 | 4734 | 12.5 | 7.30 | 976 | 316 | 1065 | 482 | 0.879 | 47.8 |
| 18260 | 6310 | 12.7 | 7.49 | 1260 | 421 | 1383 | 641 | 1.20 | 87.8 |
| 25170 | 8563 | 13.0 | 7.58 | 1678 | 571 | 1869 | 870 | 1.69 | 189 |
| 40950 | 13740 | 13.5 | 7.80 | 2559 | 901 | 2927 | 1374 | 2.91 | 604 |
| 59200 | 19400 | 14.0 | 8.00 | 3482 | 1252 | 4078 | 1913 | 4.39 | 1411 |
| 16450 | 4959 | 13.2 | 7.24 | 1093 | 331 | 1196 | 506 | 1.04 | 53.6 |
| 22930 | 6985 | 13.6 | 7.49 | 1479 | 466 | 1628 | 710 | 1.51 | 112 |
| 30820 | 9239 | 13.8 | 7.57 | 1926 | 616 | 2149 | 939 | 2.07 | 230 |
| 68130 | 19710 | 14.8 | 7.95 | 3796 | 1276 | 4435 | 1951 | 5.01 | 1506 |
| 19550 | 5185 | 13.9 | 7.18 | 1222 | 346 | 1341 | 529 | 1.23 | 60.0 |
| 27690 | 7436 | 14.4 | 7.46 | 1678 | 496 | 1850 | 756 | 1.83 | 131 |
| 36660 | 9690 | 14.6 | 7.53 | 2156 | 646 | 2408 | 986 | 2.46 | 263 |
| 76370 | 19710 | 15.6 | 7.90 | 4052 | 1276 | 4718 | 1953 | 5.60 | 1512 |
| 23040 | 5410 | 14.7 | 7.12 | 1359 | 361 | 1495 | 553 | 1.45 | 67.1 |
| 33090 | 7887 | 15.2 | 7.43 | 1891 | 526 | 2088 | 802 | 2.18 | 153 |
| 43190 | 10140 | 15.5 | 7.49 | 2400 | 676 | 2683 | 1032 | 2.89 | 298 |
| 84870 | 19520 | 16.3 | 7.83 | 4297 | 1268 | 4989 | 1942 | 6.15 | 1513 |
| 31250 | 5861 | 16.3 | 7.06 | 1654 | 391 | 1824 | 600 | 1.95 | 81.3 |
| 37640 | 6998 | 16.6 | 7.20 | 1960 | 471 | 2165 | 721 | 2.37 | 126 |
| 45070 | 8564 | 16.8 | 7.34 | 2311 | 571 | 2562 | 873 | 2.95 | 193 |
| 57680 | 10820 | 17.1 | 7.40 | 2884 | 721 | 3232 | 1104 | 3.82 | 361 |
| 104100 | 19340 | 17.9 | 7.70 | 4820 | 1260 | 5571 | 1934 | 7.43 | 1520 |
| 41890 | 6088 | 18.2 | 6.92 | 1971 | 406 | 2183 | 624 | 2.58 | 91.4 |
| 55860 | 8338 | 18.8 | 7.27 | 2568 | 556 | 2836 | 850 | 3.62 | 178 |
| 63720 | 9465 | 18.9 | 7.29 | 2896 | 631 | 3216 | 966 | 4.15 | 250 |
| 79890 | 11720 | 19.1 | 7.33 | 3551 | 781 | 3982 | 1198 | 5.27 | 448 |
| 131500 | 19340 | 19.8 | 7.59 | 5501 | 1260 | 6331 | 1939 | 9.28 | 1534 |
| 54640 | 6314 | 20.0 | 6.79 | 2315 | 421 | 2576 | 649 | 3.31 | 103 |
| 86970 | 10370 | 21.0 | 7.24 | 3550 | 691 | 3949 | 1059 | 5.65 | 318 |
| 107200 | 12620 | 21.2 | 7.27 | 4287 | 842 | 4815 | 1292 | 7.03 | 548 |
| 161900 | 19150 | 21.7 | 7.46 | 6180 | 1252 | 7094 | 1932 | 11.2 | 1544 |

33

$C_b$ is permitted to be conservatively taken as 1.0 for all cases. For cantilevers or overhangs where the free end is unbraced, $C_b = 1.0$.

**User Note:** For doubly symmetric members with no transverse loading between brace points, Equation F1-1 reduces to 2.27 for the case of equal end moments of opposite sign and to 1.67 when one end moment equals zero.

## F2. DOUBLY SYMMETRIC COMPACT I-SHAPED MEMBERS AND CHANNELS BENT ABOUT THEIR MAJOR AXIS

This section applies to doubly symmetric I-shaped members and channels bent about their major axis, having compact webs and compact flanges as defined in Section B4.

**User Note:** All current ASTM A6 W, S, M, C and MC shapes except W21×48, W14×99, W14×90, W12×65, W10×12, W8×31, W8×10, W6×15, W6×9, W6×8.5, and M4×6 have compact flanges for $F_y = 50$ ksi (345 MPa); all current ASTM A6 W, S, M, HP, C and MC shapes have compact webs at $F_y \le 65$ ksi (450 MPa).

The *nominal flexural strength*, $M_n$, shall be the lower value obtained according to the *limit states* of *yielding* (*plastic moment*) and *lateral-torsional buckling*.

### 1. Yielding

$$M_n = M_p = F_y Z_x \tag{F2-1}$$

where

$F_y$ = *specified minimum yield stress* of the type of steel being used, ksi (MPa)
$Z_x$ = plastic section modulus about the x-axis, in.$^3$ (mm$^3$)

### 2. Lateral-Torsional Buckling

(a) When $L_b \le L_p$, the limit state of lateral-torsional buckling does not apply.
(b) When $L_p < L_b \le L_r$

$$M_n = C_b \left[ M_p - (M_p - 0.7F_y S_x)\left(\frac{L_b - L_p}{L_r - L_p}\right)\right] \le M_p \tag{F2-2}$$

(c) When $L_b > L_r$

$$M_n = F_{cr} S_x \le M_p \tag{F2-3}$$

where

$L_b$ = length between points that are either braced against lateral displacement of compression flange or braced against twist of the cross section, in. (mm)

$$F_{cr} = \frac{C_b \pi^2 E}{\left(\frac{L_b}{r_{ts}}\right)^2}\sqrt{1 + 0.078\frac{Jc}{S_x h_o}\left(\frac{L_b}{r_{ts}}\right)^2} \tag{F2-4}$$

and where

$E$ = modulus of elasticity of steel = 29,000 ksi (200 000 MPa)
$J$ = torsional constant, in.$^4$ (mm$^4$)
$S_x$ = elastic section modulus taken about the x-axis, in.$^3$ (mm$^3$)

**User Note:** The square root term in Equation F2-4 may be conservatively taken equal to 1.0.

The limiting lengths $L_p$ and $L_r$ are determined as follows:

$$L_p = 1.76 r_y \sqrt{\frac{E}{F_y}} \tag{F2-5}$$

$$L_r = 1.95 r_{ts} \frac{E}{0.7F_y} \sqrt{\frac{Jc}{S_x h_o}} \sqrt{1 + \sqrt{1 + 6.76 \left( \frac{0.7F_y}{E} \frac{S_x h_o}{Jc} \right)^2}} \tag{F2-6}$$

where

$$r_{ts}^2 = \frac{\sqrt{I_y C_w}}{S_x} \tag{F2-7}$$

and

For a doubly symmetric I-shape: $c = 1$ (F2-8a)

For a channel: $$c = \frac{h_o}{2} \sqrt{\frac{I_y}{C_w}} \tag{F2-8b}$$

where

$h_o$ = distance between the flange centroids, in. (mm)

**User Note:** If the square root term in Equation F2-4 is conservatively taken equal to 1, Equation F2-6 becomes

$$L_r = \pi r_{ts} \sqrt{\frac{E}{0.7F_y}}$$

Note that this approximation can be extremely conservative.

For doubly symmetric I-shapes with rectangular flanges, $C_w = \frac{I_y h_o^2}{4}$ and thus Equation F2-7 becomes

$$r_{ts}^2 = \frac{I_y h_o}{2S_x}$$

$r_{ts}$ may be approximated accurately and conservatively as the radius of gyration of the compression flange plus one-sixth of the web:

$$r_{ts} = \frac{b_f}{\sqrt{12\left(1 + \frac{1}{6}\frac{h t_w}{b_f t_f}\right)}}$$

# CHAPTER G

# DESIGN OF MEMBERS FOR SHEAR

This chapter addresses webs of singly or doubly symmetric members subject to shear in the plane of the web, single angles and *HSS* sections, and shear in the weak direction of singly or doubly symmetric shapes.

The chapter is organized as follows:

G1. General Provisions
G2. Members with Unstiffened or Stiffened Webs
G3. Tension Field Action
G4. Single Angles
G5. Rectangular HSS and Box Members
G6. Round HSS
G7. Weak Axis Shear in Singly and Doubly Symmetric Shapes
G8. Beams and Girders with Web Openings

**User Note:** For applications not included in this chapter, the following sections apply:

- H3.3 Unsymmetric sections.
- J4.2 Shear strength of connecting elements.
- J10.6 Web panel zone shear.

## G1. GENERAL PROVISIONS

Two methods of calculating shear strength are presented below. The method presented in Section G2 does not utilize the post *buckling strength* of the member (*tension field action*). The method presented in Section G3 utilizes tension field action.

The *design shear strength*, $\phi_v V_n$, and the *allowable shear strength*, $V_n/\Omega_v$, shall be determined as follows.

For all provisions in this chapter except Section G2.1a:

$$\phi_v = 0.90\,\text{(LRFD)} \qquad \Omega_v = 1.67\,\text{(ASD)}$$

## G2. MEMBERS WITH UNSTIFFENED OR STIFFENED WEBS

### 1. Nominal Shear Strength

This section applies to webs of singly or doubly symmetric members and channels subject to shear in the plane of the web.

The *nominal shear strength*, $V_n$, of unstiffened or stiffened webs, according to the *limit states* of *shear yielding* and *shear buckling*, is

$$V_n = 0.6 F_y A_w C_v \tag{G2-1}$$

(a) For webs of rolled I-shaped members with $h/t_w \le 2.24\sqrt{E/F_y}$:

$$\phi_v = 1.00\,(\text{LRFD}) \qquad \Omega_v = 1.50\,(\text{ASD})$$

and

$$C_v = 1.0 \tag{G2-2}$$

**User Note:** All current ASTM A6 W, S and HP shapes except W44×230, W40×149, W36×135, W33×118, W30×90, W24×55, W16×26 and W12×14 meet the criteria stated in Section G2.1(a) for $F_y = 50$ ksi (345 MPa).

(b) For webs of all other doubly symmetric shapes and singly symmetric shapes and channels, except round *HSS*, the web shear coefficient, $C_v$, is determined as follows:

(i) For $h/t_w \le 1.10\sqrt{k_v E/F_y}$

$$C_v = 1.0 \tag{G2-3}$$

(ii) For $1.10\sqrt{k_v E/F_y} < h/t_w \le 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.10\sqrt{k_v E/F_y}}{h/t_w} \tag{G2-4}$$

(iii) For $h/t_w > 1.37\sqrt{k_v E/F_y}$

$$C_v = \frac{1.51 E k_v}{(h/t_w)^2 F_y} \tag{G2-5}$$

where

$A_w$ = the overall depth times the web thickness, $dt_w$, in.$^2$ (mm$^2$)

The web plate buckling coefficient, $k_v$, is determined as follows:

(i) For unstiffened webs with $h/t_w < 260$, $k_v = 5$ except for the stem of tee shapes where $k_v = 1.2$.

(ii) For stiffened webs,

$$k_v = 5 + \frac{5}{(a/h)^2}$$
$$= 5 \text{ when } a/h > 3.0 \text{ or } a/h > \left[\frac{260}{(h/t_w)}\right]^2$$

where

$a$ = clear distance between transverse *stiffeners*, in. (mm)
$h$ = for rolled shapes, the clear distance between flanges less the fillet or corner radii, in. (mm)

axes, respectively, considering the member loaded in bending only as prescribed in Clause 2.6.5.

**In addition, the compressive bending stress alone shall be checked against the lateral torsional buckling stress.**

2.6.8 Equivalent Stress $f_e$

Whenever the material is subjected to axial and shear stresses, the equivalent stress (**$f_e$**) must not exceed the permitted stresses given in this code plus **10%**, and the equivalent stress shall be calculated as follows:

$$f_e = \sqrt{f^2 + 3q^2} \leq 1.1\ F_{all} \quad \text{.................................} \quad \mathbf{2.39}$$

## 2.7 EFFECTIVE AREAS

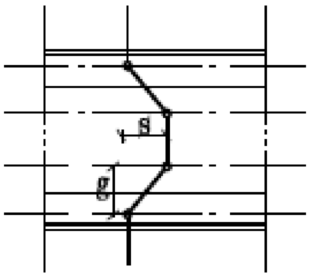
### 2.7.1 Effective Net Area

The effective net sectional-area of a tension member shall be used. This area is the sum of the products of the thickness and net width of each element as measured normal to the axis of the member. For a chain of holes extending across a part in any diagonal or zigzag line, the net width of the parts shall be obtained by deducting from the gross width the sum of the diameters of all holes in the chain and adding, for each gauge space in the chain, the quantity **($s^2/4g$)**.

Where:

**s =** The staggered pitch, i.e., the distance, measured parallel to the direction of stress in the member, centre to centre of holes in consecutive lines.

**t =** The thickness of the material.




**g =** The gauge, i.e., the distance, measured at right angles to the direction of stress in the member, centre to centre of holes in consecutive lines.

### 6.5.2.6 Design Strength in Connections Subjected to Combined Shear and Bending Moment

In moment connections of the type shown in Fig. 6.6, the loss of clamping forces in region "A" is always coupled with a corresponding increase in contact pressure in region "B". The clamping force remains unchanged and there is no decrease of the frictional resistance as given by the following :-

$$P_S = \mu T / \gamma \quad \ldots\ldots\ldots\ldots\ldots \quad 6.14$$

The induced maximum tensile force $T_{(ext,b,M)}$ due to the applied moment (M) in addition to the prying force P that may occur, must not exceed the pretension force as follows:-

$$T_{(ext,b,M)} + P \leq 0.8T \quad \ldots\ldots\ldots\ldots\ldots \quad 6.15$$

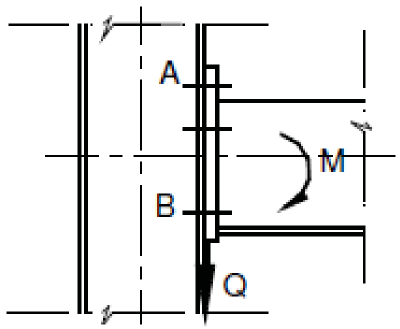



**Figure (6.6) Connections Subjected to Combined Shear and Bending Moment**

### 6.5.2.7 Design Strength in Connections Subjected to Combined Shear, Tension, and Bending Moment

When the connection is subjected to shearing force (Q), a tension force ($T_{ext}$) and a bending moment (M), the design strength per bolt is to be according to the following formulae:-

### 6.9.2 Determination of The Prying Force P

In order to determine the prying force P, the connection is to be transformed to an equivalent Tee stub connection as shown in Fig. 6.11. The prying force P can be determined using the following relation :-

$$P = \left[ \frac{\dfrac{1}{2} - \dfrac{w t_p^{\,4}}{30ab^2 A_s}}{\left(\dfrac{3a}{4}\right)\left(\dfrac{a}{4b}+1\right) + \dfrac{w t_p^{\,4}}{30ab^2 A_s}} \right] . T_{ext,b,M} \text{ or } T_{ext,b} \quad \ldots \quad 6.24$$

Where :

| | | |
|---|---|---|
| a,b | = | Bolt outer overhanging and inner bolt dimension with respect to the stem Tee stub respectively in cm. |
| w | = | Flange Tee stub breadth with respect to one column of bolts. |
| $A_s$ | = | Bolt stress area. |
| $T_{ext,b}$ , $T_{ext,b,M}$ | = | Applied external tension force on one bolt column due to either an applied external tension force $T_{ext}$ (Fig. 6.10a) or due to the replacement of the applied moment (M) by two equal external and opposite forces $T_b = C_b = \dfrac{M}{d_b}$ (Fig. 6.10b) or due to an exact analysis of an end plate moment connection (Fig. 6.10c) |

Where $T_{ext,b} = \dfrac{T_{ext}}{4}$ ( Fig. 6.10a)

$$T_{ext,b,M} = \frac{T_b}{4} \quad \text{(Fig. 6.10b)}$$

Where

$\overline{w}$ = Half breadth of end plate = half breadth of Tee-stub flange.

w = $\overline{w}$ for case of two columns of bolts .

$$T_{ext,b,M} + P \leq 0.8\ T \qquad \text{..........................} \qquad 6.28$$

**6.9.4 Safety Requirements for Beam to Column Connections**

**i.** Column web at the vicinity of the compression beam flange " crippling of the column web " :

Crippling of the column web is prevented if :

$$t_{wc} \geq \frac{b_b t_b}{t_b + 2t_p + 5k} \qquad \text{.................................} \qquad 6.29$$

If Equation 6.29 is not satisfied, use a pair of horizontal stiffeners fulfilling the following condition :

$$2b_{st}t_{st} \geq b_b t_b - (t_b + 2t_p + 5k)t_{wc} \qquad \text{...................} \qquad 6.30$$

In order to prevent the local buckling of these stiffeners:

$$b_{st}\ /\ t_{st} \ \leq 25\ /\ \sqrt{F_y} \qquad \text{...................} \qquad 6.31$$

**ii.** Column flange at the location of the tension beam flange " bending of the column flange ":
Bending of the column flange is prevented if:

$$\text{.................................} \qquad 6.32$$

If Equation 6.32 is not satisfied, use a pair of horizontal stiffeners fulfilling the condition of Equation 6.30:

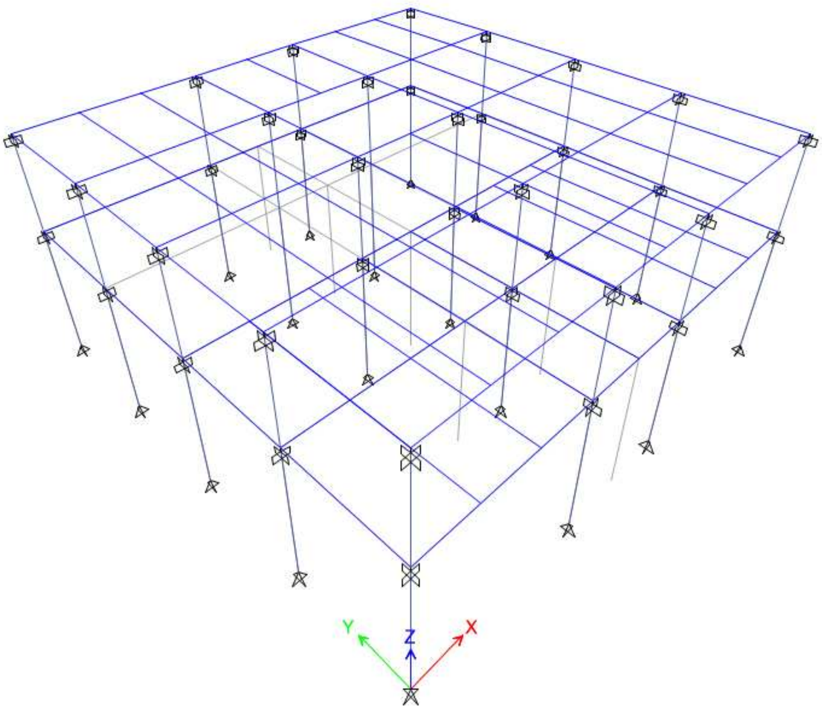
Table (6.3) Properties and Strength of High Strength Bolts (Grade 10.9*)

| Bolt Diameter (d) mm | Bolt Area (A) $cm^2$ | Stress Area ($A_s$) $cm^2$ | Pretension Force (T) tons | Required Torque ($M_a$) kg.m | Permissible Friction Load of One Bolt Per One Friction Surface ($P_s$) tons | | | | | | | |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| | | | | | Ordinary Steel Work | | | | Bridges and Cranes | | | |
| | | | | | St. 37&42-44 ($\mu$=0.4) | | St. 50-55 ($\mu$=0.5) | | St. 37&42-44 ($\mu$=0.4) | | St. 50-55 ($\mu$=0.5) | |
| | | | | | Cases of Loading | | | | Cases of Loading | | | |
| | | | | | I | II | I | II | I | II | I | II |
| M12 | 1.13 | 0.84 | 5.29 | 12 | 1.69 | 2.01 | 2.11 | 2.52 | 1.32 | 1.56 | 1.65 | 1.95 |
| M16 | 2.01 | 1.57 | 9.89 | 31 | 3.16 | 3.37 | 3.95 | 4.71 | 2.47 | 2.92 | 3.09 | 3.66 |
| M20 | 3.14 | 2.45 | 15.43 | 62 | 4.93 | 5.90 | 6.17 | 7.36 | 3.85 | 4.56 | 4.82 | 5.71 |
| M22 | 3.80 | 3.03 | 19.08 | 84 | 6.10 | 7.27 | 7.63 | 9.10 | 4.77 | 5.65 | 5.96 | 7.06 |
| M24 | 4.52 | 3.53 | 22.23 | 107 | 7.11 | 8.45 | 8.89 | 10.60 | 5.55 | 6.58 | 6.94 | 8.22 |
| M27 | 5.73 | 4.59 | 28.91 | 157 | 9.25 | 11.03 | 11.56 | 13.78 | 7.22 | 8.55 | 9.03 | 10.70 |
| M30 | 7.06 | 5.61 | 35.34 | 213 | 11.30 | 13.48 | 14.13 | 16.86 | 8.83 | 10.46 | 11.04 | 13.07 |
| M36 | 10.18 | 8.17 | 51.47 | 372 | 16.47 | 19.64 | 20.58 | 24.55 | 12.86 | 15.24 | 16.08 | 19.05 |

$T = (0.7)\,F_{yb}.A_s \quad M_a = 0.2\,d.T. \qquad P_s = \mu\,T/\gamma$

* For HSB grade 8.8 , the above values shall be reduced by 30%

ETABS® 2015
Integrated Building Design Software




**Project Report: Steel design for mosque**

# 1 Structure Data

This chapter provides model geometry information, including items such as story levels, point coordinates, and element connectivity.

## 1.1 Story Data

**Story Data**

| Name | Height m | Elevation m | Master Story | Similar To | Splice Story |
|---|---|---|---|---|---|
| Roof | 5 | 12 | No | None | No |
| Mezzanine | 7 | 7 | No | None | No |
| Base | 0 | 0 | No | None | No |

## 1.2 Grid Data

**Grid Systems**

| Name | Type | Story Range | X Origin m | Y Origin m | Rotation deg | Bubble Size m | Color |
|---|---|---|---|---|---|---|---|
| G1 | Cartesian | Default | 0 | 0 | 0 | 1.25 | ffa0a0a0 |

**Grid Lines**

| Grid System | Grid Direction | Grid ID | Visible | Bubble Location | Ordinate m |
|---|---|---|---|---|---|
| G1 | X | A | Yes | End | 0 |
| G1 | X | B | Yes | End | 9 |
| G1 | X | C | Yes | End | 12 |
| G1 | X | D | Yes | End | 15 |
| G1 | X | E | Yes | End | 24 |
| G1 | Y | 1 | Yes | Start | 0 |
| G1 | Y | 2 | Yes | Start | 6 |
| G1 | Y | 3 | Yes | Start | 12 |
| G1 | Y | 4 | Yes | Start | 18 |
| G1 | Y | 5 | Yes | Start | 24 |

### 1.3 Point Coordinates

**Joint Coordinates Data**

| Label | X<br>m | Y<br>m | ΔZ<br>Below<br>m |
|---|---|---|---|
| 1 | 24 | 18 | 0 |
| 2 | 24 | 12 | 0 |
| 5 | 0 | 6 | 0 |
| 6 | 0 | 12 | 0 |
| 7 | 0 | 18 | 0 |
| 8 | 0 | 24 | 0 |
| 9 | 0 | 0 | 0 |
| 10 | 24 | 6 | 0 |
| 11 | 24 | 24 | 0 |
| 12 | 24 | 0 | 0 |
| 13 | 12 | 6 | 0 |
| 14 | 12 | 12 | 0 |
| 16 | 12 | 24 | 0 |
| 17 | 12 | 0 | 0 |
| 23 | 15 | 12 | 0 |
| 24 | 18 | 12 | 0 |
| 25 | 21 | 12 | 0 |
| 26 | 3 | 0 | 0 |
| 27 | 6 | 0 | 0 |
| 28 | 9 | 0 | 0 |
| 29 | 15 | 0 | 0 |
| 30 | 18 | 0 | 0 |
| 31 | 21 | 0 | 0 |
| 32 | 3 | 6 | 0 |
| 33 | 6 | 6 | 0 |
| 34 | 9 | 6 | 0 |
| 35 | 15 | 6 | 0 |
| 36 | 18 | 6 | 0 |
| 37 | 21 | 6 | 0 |
| 38 | 3 | 12 | 0 |
| 39 | 6 | 12 | 0 |
| 40 | 9 | 12 | 0 |
| 4 | 15 | 18 | 0 |
| 18 | 9 | 18 | 0 |
| 15 | 12 | 18 | 0 |
| 19 | 9 | 24 | 0 |
| 20 | 6 | 18 | 0 |
| 21 | 6 | 24 | 0 |

| Label | X<br>m | Y<br>m | ΔZ Below<br>m |
|---|---|---|---|
| 22 | 3 | 18 | 0 |
| 41 | 3 | 24 | 0 |
| 42 | 21 | 18 | 0 |
| 43 | 21 | 24 | 0 |
| 44 | 18 | 18 | 0 |
| 45 | 18 | 24 | 0 |
| 46 | 15 | 24 | 0 |
| 47 | 0 | 6 | 1 |
| 48 | 0 | 12 | 1 |
| 49 | 0 | 18 | 1 |
| 50 | 0 | 24 | 1 |
| 51 | 12 | 0 | 1 |
| 52 | 12 | 6 | 1 |
| 53 | 12 | 12 | 1 |
| 54 | 12 | 24 | 1 |
| 56 | 24 | 6 | 1 |
| 57 | 24 | 12 | 1 |
| 58 | 24 | 18 | 1 |
| 59 | 24 | 24 | 1 |
| 61 | 9 | 18 | 1 |
| 62 | 15 | 12 | 1 |
| 63 | 9 | 12 | 1 |

### 1.4 Line Connectivity

**Column Connectivity Data**

| Column | I-End Point | J-End Point | I-End Story |
|---|---|---|---|
| C16 | 9 | 9 | Below |
| C23 | 5 | 5 | Below |
| C24 | 6 | 6 | Below |
| C25 | 7 | 7 | Below |
| C26 | 8 | 8 | Below |
| C28 | 13 | 13 | Below |
| C32 | 12 | 12 | Below |
| C33 | 10 | 10 | Below |
| C34 | 2 | 2 | Below |
| C35 | 1 | 1 | Below |
| C36 | 11 | 11 | Below |
| C2 | 4 | 4 | Below |
| C3 | 18 | 18 | Below |

| Column | I-End Point | J-End Point | I-End Story |
|---|---|---|---|
| C4 | 23 | 23 | Below |
| C5 | 40 | 40 | Below |
| C1 | 29 | 29 | Below |
| C6 | 28 | 28 | Below |
| C7 | 19 | 19 | Below |
| C8 | 46 | 46 | Below |

**Beam Connectivity Data**

| Beam | I-End Point | J-End Point | Curve Type |
|---|---|---|---|
| B1 | 31 | 37 | None |
| B2 | 30 | 36 | None |
| B3 | 29 | 35 | None |
| B4 | 12 | 10 | None |
| B5 | 17 | 13 | None |
| B6 | 28 | 34 | None |
| B7 | 27 | 33 | None |
| B8 | 26 | 32 | None |
| B9 | 9 | 5 | None |
| B10 | 37 | 25 | None |
| B39 | 36 | 24 | None |
| B40 | 35 | 23 | None |
| B41 | 10 | 2 | None |
| B42 | 13 | 14 | None |
| B43 | 34 | 40 | None |
| B44 | 33 | 39 | None |
| B45 | 32 | 38 | None |
| B46 | 5 | 6 | None |
| B52 | 5 | 13 | None |
| B60 | 13 | 10 | None |
| B15 | 6 | 7 | None |
| B17 | 2 | 1 | None |
| B18 | 7 | 8 | None |
| B20 | 1 | 11 | None |
| B11 | 23 | 2 | None |
| B12 | 4 | 1 | None |
| B16 | 18 | 4 | None |
| B19 | 7 | 18 | None |
| B21 | 6 | 40 | None |
| B22 | 40 | 23 | None |
| B23 | 23 | 4 | None |
| B24 | 40 | 18 | None |

| Beam | I-End Point | J-End Point | Curve Type |
|---|---|---|---|
| B25 | 15 | 16 | None |
| B26 | 18 | 19 | None |
| B27 | 20 | 21 | None |
| B28 | 22 | 41 | None |
| B29 | 42 | 43 | None |
| B30 | 44 | 45 | None |
| B31 | 4 | 46 | None |
| B32 | 25 | 42 | None |
| B33 | 24 | 44 | None |
| B34 | 39 | 20 | None |
| B35 | 38 | 22 | None |
| B49 | 29 | 12 | None |
| B51 | 9 | 28 | None |
| B53 | 28 | 29 | None |
| B57 | 8 | 19 | None |
| B59 | 46 | 11 | None |
| B61 | 19 | 46 | None |

### 1.5 Mass

**Mass Source**

| Name | Include Elements | Include Added Mass | Include Loads | Include Lateral | Include Vertical | Lump at Stories | IsDefault |
|---|---|---|---|---|---|---|---|
| MsSrc1 | Yes | Yes | No | Yes | No | Yes | Yes |

**Mass Summary by Story**

| Story | UX tonf-s²/m | UY tonf-s²/m | UZ tonf-s²/m |
|---|---|---|---|
| Roof | 3.52277 | 3.52277 | 0 |
| Mezzanine | 4.17327 | 4.17327 | 0 |
| Base | 1.05386 | 1.05386 | 0 |

### 1.6 Groups

**Group Definitions**

| Name | Color |
|---|---|
| All | Yellow |

## 2 Properties

This chapter provides property information for materials, frame sections, shell sections, and links.

### 2.1 Materials

**Material Properties - Summary**

| Name | Type | E tonf/m² | ν | Unit Weight tonf/m³ | Design Strengths |
|---|---|---|---|---|---|
| A615Gr60 | Rebar | 20389019.16 | 0.3 | 7.849 | Fy=42184.18 tonf/m², Fu=63276.27 tonf/m² |
| A992Fy50 | Steel | 20389019.16 | 0.3 | 7.849 | Fy=35153.48 tonf/m², Fu=45699.53 tonf/m² |

### 2.2 Frame Sections

**Frame Sections - Summary**

| Name | Material | Shape |
|---|---|---|
| HE400A | A992Fy50 | Steel I/Wide Flange |
| HE400B | A992Fy50 | Steel I/Wide Flange |
| HE450A | A992Fy50 | Steel I/Wide Flange |
| HE500B | A992Fy50 | Steel I/Wide Flange |
| IPE360 | A992Fy50 | Steel I/Wide Flange |
| IPE450 | A992Fy50 | Steel I/Wide Flange |

### 2.3 Reinforcement Sizes

**Reinforcing Bar Sizes**

| Name | Diameter m | Area m² |
|---|---|---|
| 10 | 0.01 | 7.9E-05 |
| 20 | 0.02 | 0.000314 |

## 3 Assignments

This chapter provides a listing of the assignments applied to the model.

### 3.1 Joint Assignments

**Joint Assignments - Summary**

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Roof | 1 | 109 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 2 | 105 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 5 | 61 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 6 | 65 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 7 | 69 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 8 | 73 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 9 | 57 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 10 | 101 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 11 | 113 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 12 | 97 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 13 | 81 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 14 | 85 | From Area | |
| Roof | 16 | 93 | From Area | |
| Roof | 17 | 77 | From Area | |
| Roof | 23 | 12 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 24 | 30 | From Area | |
| Roof | 25 | 29 | From Area | |
| Roof | 26 | 27 | From Area | |
| Roof | 27 | 25 | From Area | |
| Roof | 28 | 23 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 29 | 21 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 30 | 19 | From Area | |
| Roof | 31 | 17 | From Area | |
| Roof | 32 | 28 | From Area | |
| Roof | 33 | 26 | From Area | |
| Roof | 34 | 22 | From Area | |
| Roof | 35 | 16 | From Area | |
| Roof | 36 | 20 | From Area | |
| Roof | 37 | 18 | From Area | |
| Roof | 38 | 32 | From Area | |
| Roof | 39 | 31 | From Area | |
| Roof | 40 | 15 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 4 | 5 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 18 | 9 | From Area | UX; UY; UZ; RX; RY; RZ |
| Roof | 15 | 24 | From Area | |
| Roof | 19 | 33 | From Area | UX; UY; UZ; RX; RY; RZ |

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Roof | 20 | 34 | From Area | |
| Roof | 21 | 35 | From Area | |
| Roof | 22 | 36 | From Area | |
| Roof | 41 | 37 | From Area | |
| Roof | 42 | 38 | From Area | |
| Roof | 43 | 39 | From Area | |
| Roof | 44 | 40 | From Area | |
| Roof | 45 | 41 | From Area | |
| Roof | 46 | 42 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 1 | 108 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 2 | 104 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 5 | 60 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 6 | 64 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 7 | 68 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 8 | 72 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 9 | 56 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 10 | 100 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 11 | 112 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 12 | 96 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 13 | 80 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 14 | 84 | From Area | |
| Mezzanine | 16 | 92 | From Area | |
| Mezzanine | 17 | 76 | From Area | |
| Mezzanine | 23 | 134 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 24 | 135 | From Area | |
| Mezzanine | 25 | 136 | From Area | |
| Mezzanine | 26 | 119 | From Area | |
| Mezzanine | 27 | 120 | From Area | |
| Mezzanine | 28 | 121 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 29 | 122 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 30 | 123 | From Area | |
| Mezzanine | 31 | 124 | From Area | |
| Mezzanine | 32 | 125 | From Area | |
| Mezzanine | 33 | 126 | From Area | |
| Mezzanine | 34 | 53 | From Area | |
| Mezzanine | 35 | 52 | From Area | |
| Mezzanine | 36 | 129 | From Area | |
| Mezzanine | 37 | 130 | From Area | |
| Mezzanine | 38 | 131 | From Area | |
| Mezzanine | 39 | 132 | From Area | |
| Mezzanine | 40 | 133 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 4 | 4 | From Area | |

| Story | Label | Unique Name | Diaphragm | Restraints |
|---|---|---|---|---|
| Mezzanine | 18 | 8 | From Area | |
| Mezzanine | 19 | 48 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 46 | 51 | From Area | UX; UY; UZ; RX; RY; RZ |
| Mezzanine | 47 | 59 | From Area | |
| Mezzanine | 48 | 63 | From Area | |
| Mezzanine | 49 | 67 | From Area | |
| Mezzanine | 50 | 71 | From Area | |
| Mezzanine | 51 | 75 | From Area | |
| Mezzanine | 52 | 79 | From Area | |
| Mezzanine | 53 | 83 | From Area | |
| Mezzanine | 54 | 91 | From Area | |
| Mezzanine | 56 | 99 | From Area | |
| Mezzanine | 57 | 103 | From Area | |
| Mezzanine | 58 | 107 | From Area | |
| Mezzanine | 59 | 111 | From Area | |
| Mezzanine | 61 | 7 | From Area | |
| Mezzanine | 62 | 11 | From Area | |
| Mezzanine | 63 | 14 | From Area | |
| Base | 1 | 106 | From Area | UX; UY; UZ |
| Base | 2 | 102 | From Area | UX; UY; UZ |
| Base | 5 | 58 | From Area | UX; UY; UZ |
| Base | 6 | 62 | From Area | UX; UY; UZ |
| Base | 7 | 66 | From Area | UX; UY; UZ |
| Base | 8 | 70 | From Area | UX; UY; UZ |
| Base | 9 | 54 | From Area | UX; UY; UZ |
| Base | 10 | 98 | From Area | UX; UY; UZ |
| Base | 11 | 110 | From Area | UX; UY; UZ |
| Base | 12 | 94 | From Area | UX; UY; UZ |
| Base | 13 | 78 | From Area | UX; UY; UZ |
| Base | 23 | 10 | From Area | UX; UY; UZ |
| Base | 28 | 44 | From Area | UX; UY; UZ |
| Base | 29 | 1 | From Area | UX; UY; UZ |
| Base | 40 | 13 | From Area | UX; UY; UZ |
| Base | 4 | 2 | From Area | UX; UY; UZ |
| Base | 18 | 6 | From Area | UX; UY; UZ |
| Base | 19 | 46 | From Area | UX; UY; UZ |
| Base | 46 | 49 | From Area | UX; UY; UZ |

### 3.2 Frame Assignments

Table 3.2 - Frame Assignments - Summary

| Story | Label | Unique Name | Design Type | Length m | Analysis Section | Design Section | Max Station Spacing m | Min Number Stations | Releases |
|---|---|---|---|---|---|---|---|---|---|
| Roof | C16 | 36 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C23 | 39 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C24 | 42 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C25 | 45 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C26 | 48 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C28 | 54 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C32 | 66 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C33 | 69 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C34 | 72 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C35 | 75 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C36 | 78 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C2 | 33 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C3 | 59 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C4 | 80 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C5 | 83 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C1 | 132 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C6 | 135 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C7 | 50 | Column | 5 | HE400B | HE400B | | 3 | No |
| Roof | C8 | 94 | Column | 5 | HE400B | HE400B | | 3 | No |
| Mezzanine | C16 | 34 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C23 | 37 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C24 | 40 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C25 | 43 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C26 | 46 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C28 | 52 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C32 | 64 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C33 | 67 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C34 | 70 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C35 | 73 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C36 | 76 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C2 | 31 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C3 | 57 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C4 | 60 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C5 | 81 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C1 | 2 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C6 | 133 | Column | 7 | HE400B | HE400B | | 3 | No |

| Story | Label | Unique Name | Design Type | Length m | Analysis Section | Design Section | Max Station Spacing m | Min Number Stations | Releases |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | C7 | 22 | Column | 7 | HE400B | HE400B | | 3 | No |
| Mezzanine | C8 | 51 | Column | 7 | HE400B | HE400B | | 3 | No |
| Roof | B1 | 109 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B2 | 110 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B3 | 61 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B4 | 25 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B5 | 95 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B6 | 63 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B7 | 113 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B8 | 114 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B9 | 92 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B10 | 115 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B39 | 116 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B40 | 62 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B41 | 28 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B42 | 104 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B43 | 85 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B44 | 119 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B45 | 120 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B46 | 91 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B52 | 107 | Beam | 12 | HE500B | HE500B | 0.5 | | No |
| Roof | B60 | 108 | Beam | 12 | HE500B | HE500B | 0.5 | | No |
| Roof | B15 | 88 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B17 | 30 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B18 | 87 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B20 | 84 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B11 | 96 | Beam | 9 | HE400A | HE400A | 0.5 | | No |
| Roof | B12 | 97 | Beam | 9 | HE400A | HE400A | 0.5 | | No |
| Roof | B16 | 98 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Roof | B19 | 99 | Beam | 9 | HE400A | HE400A | 0.5 | | No |
| Roof | B21 | 100 | Beam | 9 | HE400A | HE400A | 0.5 | | No |
| Roof | B22 | 101 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Roof | B23 | 102 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Roof | B24 | 103 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Roof | B25 | 86 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B26 | 121 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |

| Story | Label | Unique Name | Design Type | Length m | Analysis Section | Design Section | Max Station Spacing m | Min Number Stations | Releases |
|---|---|---|---|---|---|---|---|---|---|
| Roof | B27 | 122 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B28 | 123 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B29 | 124 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B30 | 125 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B31 | 126 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B32 | 127 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B33 | 128 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B34 | 129 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B35 | 130 | Beam | 6 | IPE360 | IPE360 | 0.5 | | No |
| Roof | B49 | 143 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Roof | B51 | 138 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Roof | B53 | 141 | Beam | 6 | HE400 A | HE400 A | 0.5 | | No |
| Roof | B57 | 153 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Roof | B59 | 150 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Roof | B61 | 149 | Beam | 6 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B1 | 1 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B2 | 3 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B3 | 89 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Mezzanine | B4 | 5 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B5 | 6 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B6 | 105 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Mezzanine | B7 | 8 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B8 | 9 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B9 | 10 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B10 | 11 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B39 | 12 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B40 | 90 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Mezzanine | B41 | 14 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B42 | 15 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B43 | 111 | Beam | 6 | IPE450 | IPE450 | 0.5 | | No |
| Mezzanine | B44 | 17 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B45 | 18 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B46 | 19 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B52 | 20 | Beam | 12 | HE500B | HE500B | 0.5 | | No |

| Story | Label | Unique Name | Design Type | Length m | Analysis Section | Design Section | Max Station Spacing m | Min Number Stations | Releases |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | B60 | 21 | Beam | 12 | HE500B | HE500B | 0.5 | | No |
| Mezzanine | B15 | 24 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B17 | 26 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B18 | 27 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B20 | 29 | Beam | 6 | IPE450 | IPE450 | 0.5 | | Yes |
| Mezzanine | B11 | 106 | Beam | 9 | HE450 A | HE450 A | 0.5 | | No |
| Mezzanine | B21 | 55 | Beam | 9 | HE450 A | HE450 A | 0.5 | | No |
| Mezzanine | B22 | 56 | Beam | 6 | HE450 A | HE450 A | 0.5 | | No |
| Mezzanine | B49 | 137 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B51 | 23 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B53 | 136 | Beam | 6 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B57 | 139 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B59 | 146 | Beam | 9 | HE400 A | HE400 A | 0.5 | | No |
| Mezzanine | B61 | 144 | Beam | 6 | HE400 A | HE400 A | 0.5 | | No |

## 4 Loads

This chapter provides loading information as applied to the model.

### 4.1 Load Patterns

**Load Patterns**

| Name | Type | Self-Weight Multiplier |
|---|---|---|
| Dead | Dead | 0 |
| Live | Live | 0 |

### 4.2 Applied Loads

#### 4.2.1 Line Loads

**Frame Loads - Point**

| Story | Label | Unique Name | Design Type | Load Pattern | Load Type | Direction | Relative Distance | Absolute Distance m | Force tonf |
|---|---|---|---|---|---|---|---|---|---|
| Roof | B16 | 98 | Beam | Dead | Force | Gravity | 0.5 | 3 | 17.1 |
| Roof | B22 | 101 | Beam | Dead | Force | Gravity | 0.5 | 3 | 17.1 |
| Roof | B23 | 102 | Beam | Dead | Force | Gravity | 0.5 | 3 | 17.1 |
| Roof | B24 | 103 | Beam | Dead | Force | Gravity | 0.5 | 3 | 17.1 |

**Frame Loads - Distributed**

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B1 | 109 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B2 | 110 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B4 | 25 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B5 | 95 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B7 | 113 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B8 | 114 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B9 | 92 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B10 | 115 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B39 | 116 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B41 | 28 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B42 | 104 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B44 | 119 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B45 | 120 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B46 | 91 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B52 | 107 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 12 | 0.19 | 0.19 |
| Roof | B60 | 108 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 12 | 0.19 | 0.19 |
| Roof | B15 | 88 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B17 | 30 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B18 | 87 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B20 | 84 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.05 | 1.05 |
| Roof | B11 | 96 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 0.13 | 0.13 |
| Roof | B12 | 97 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 0.13 | 0.13 |
| Roof | B16 | 98 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.02 | 1.02 |
| Roof | B19 | 99 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 0.13 | 0.13 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B21 | 100 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 0.13 | 0.13 |
| Roof | B22 | 101 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.02 | 1.02 |
| Roof | B23 | 102 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.02 | 1.02 |
| Roof | B24 | 103 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.02 | 1.02 |
| Roof | B26 | 121 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B27 | 122 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B28 | 123 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B29 | 124 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B30 | 125 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B31 | 126 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B32 | 127 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B33 | 128 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B34 | 129 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B35 | 130 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.1 | 2.1 |
| Roof | B49 | 143 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.19 | 0.19 |
| Roof | B49 | 143 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.19 | 0.19 |
| Roof | B49 | 143 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.19 | 0.19 |
| Roof | B51 | 138 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.19 | 0.19 |
| Roof | B51 | 138 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.19 | 0.19 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B51 | 138 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.19 | 0.19 |
| Roof | B53 | 141 | Beam | Dead | Force | Gravity | 0 | 0.5 | 0 | 3 | 0.19 | 0.19 |
| Roof | B53 | 141 | Beam | Dead | Force | Gravity | 0.5 | 1 | 3 | 6 | 0.19 | 0.19 |
| Roof | B57 | 153 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.13 | 0.13 |
| Roof | B57 | 153 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.13 | 0.13 |
| Roof | B57 | 153 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.13 | 0.13 |
| Roof | B59 | 150 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.13 | 0.13 |
| Roof | B59 | 150 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.13 | 0.13 |
| Roof | B59 | 150 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.13 | 0.13 |
| Roof | B61 | 149 | Beam | Dead | Force | Gravity | 0.5 | 1 | 3 | 6 | 0.13 | 0.13 |
| Roof | B61 | 149 | Beam | Dead | Force | Gravity | 0 | 0.5 | 0 | 3 | 0.13 | 0.13 |
| Mezzanine | B1 | 1 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B2 | 3 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B4 | 5 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.55 | 1.55 |
| Mezzanine | B5 | 6 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B7 | 8 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B8 | 9 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B9 | 10 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.55 | 1.55 |
| Mezzanine | B10 | 11 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | B39 | 12 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B41 | 14 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.55 | 1.55 |
| Mezzanine | B42 | 15 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B44 | 17 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B45 | 18 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 2.11 | 2.11 |
| Mezzanine | B46 | 19 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.55 | 1.55 |
| Mezzanine | B52 | 20 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 12 | 0.19 | 0.19 |
| Mezzanine | B60 | 21 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 12 | 0.19 | 0.19 |
| Mezzanine | B15 | 24 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 0.58 | 0.58 |
| Mezzanine | B17 | 26 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 0.58 | 0.58 |
| Mezzanine | B18 | 27 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 0.58 | 0.58 |
| Mezzanine | B20 | 29 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 0.58 | 0.58 |
| Mezzanine | B11 | 106 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 1.58 | 1.58 |
| Mezzanine | B21 | 55 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 9 | 1.58 | 1.58 |
| Mezzanine | B22 | 56 | Beam | Dead | Force | Gravity | 0 | 1 | 0 | 6 | 1.58 | 1.58 |
| Mezzanine | B49 | 137 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 1.69 | 1.69 |
| Mezzanine | B49 | 137 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 1.69 | 1.69 |
| Mezzanine | B49 | 137 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 1.69 | 1.69 |
| Mezzanine | B51 | 23 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 1.69 | 1.69 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | B51 | 23 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 1.69 | 1.69 |
| Mezzanine | B51 | 23 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 1.69 | 1.69 |
| Mezzanine | B53 | 136 | Beam | Dead | Force | Gravity | 0.5 | 1 | 3 | 6 | 1.69 | 1.69 |
| Mezzanine | B53 | 136 | Beam | Dead | Force | Gravity | 0 | 0.5 | 0 | 3 | 1.69 | 1.69 |
| Mezzanine | B57 | 139 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.58 | 0.58 |
| Mezzanine | B57 | 139 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.58 | 0.58 |
| Mezzanine | B57 | 139 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.58 | 0.58 |
| Mezzanine | B59 | 146 | Beam | Dead | Force | Gravity | 0 | 0.333333 | 0 | 3 | 0.58 | 0.58 |
| Mezzanine | B59 | 146 | Beam | Dead | Force | Gravity | 0.333333 | 0.666667 | 3 | 6 | 0.58 | 0.58 |
| Mezzanine | B59 | 146 | Beam | Dead | Force | Gravity | 0.666667 | 1 | 6 | 9 | 0.58 | 0.58 |
| Mezzanine | B61 | 144 | Beam | Dead | Force | Gravity | 0 | 0.5 | 0 | 3 | 0.58 | 0.58 |
| Mezzanine | B61 | 144 | Beam | Dead | Force | Gravity | 0.5 | 1 | 3 | 6 | 0.58 | 0.58 |
| Roof | B1 | 109 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B2 | 110 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B4 | 25 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B5 | 95 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B7 | 113 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B8 | 114 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B9 | 92 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B10 | 115 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B39 | 116 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B41 | 28 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B42 | 104 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B44 | 119 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B45 | 120 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B46 | 91 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B15 | 88 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B17 | 30 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B18 | 87 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B20 | 84 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.38 | 0.38 |
| Roof | B16 | 98 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Roof | B22 | 101 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Roof | B23 | 102 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.13 | 1.13 |
| Roof | B24 | 103 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.13 | 1.13 |
| Roof | B26 | 121 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B27 | 122 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B28 | 123 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B29 | 124 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Roof | B30 | 125 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B31 | 126 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B32 | 127 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B33 | 128 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B34 | 129 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Roof | B35 | 130 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Mezzanine | B1 | 1 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B2 | 3 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B4 | 5 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Mezzanine | B5 | 6 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B7 | 8 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B8 | 9 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B9 | 10 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Mezzanine | B10 | 11 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B39 | 12 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B41 | 14 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |
| Mezzanine | B42 | 15 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B44 | 17 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |
| Mezzanine | B45 | 18 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 1.5 | 1.5 |

| Story | Label | Unique Name | Design Type | Load Pattern | LoadType | Direction | Relative Distance Start | Relative Distance End | Absolute Distance Start m | Absolute Distance End m | Force at Start tonf/m | Force at End tonf/m |
|---|---|---|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | B46 | 19 | Beam | Live | Force | Gravity | 0 | 1 | 0 | 6 | 0.75 | 0.75 |

### 4.3 Load Cases

**Load Cases - Summary**

| Name | Type |
|---|---|
| Dead | Linear Static |
| Live | Linear Static |

### 4.4 Load Combinations

**Load Combinations**

| Name | Load Case/Combo | Scale Factor | Type | Auto |
|---|---|---|---|---|
| D | Dead | 1.4 | Linear Add | No |
| D+L | Dead | 1.2 | Linear Add | No |
| D+L | Live | 1.6 | | No |
| DStlS1 | Dead | 1.4 | Linear Add | Yes |
| DStlS2 | Dead | 1.2 | Linear Add | Yes |
| DStlS2 | Live | 1.6 | | No |
| DStlD1 | Dead | 1 | Linear Add | Yes |
| DStlD2 | Dead | 1 | Linear Add | Yes |
| DStlD2 | Live | 1 | | No |
| DCon1 | Dead | 1.4 | Linear Add | Yes |
| DCon2 | Dead | 1.2 | Linear Add | Yes |
| DCon2 | Live | 1.6 | | No |

## 5 Analysis Results

This chapter provides analysis results.

### 5.1 Structure Results

**Base Reactions**

| Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m | X m | Y m | Z m |
|---|---|---|---|---|---|---|---|---|---|
| Dead | 0 | 0 | 685.72 | 7305.7 | -8228.59 | 0 | 0 | 0 | 0 |
| Live | 0 | 0 | 247.5 | 2443.5 | -2970 | 0 | 0 | 0 | 0 |
| D | 0 | 0 | 960 | 10227.97 | -11520.03 | 0 | 0 | 0 | 0 |
| D+L | 0 | 0 | 1218.86 | 12676.44 | -14626.31 | 0 | 0 | 0 | 0 |
| DStlS1 | 0 | 0 | 960 | 10227.97 | -11520.03 | 0 | 0 | 0 | 0 |
| DStlS2 | 0 | 0 | 1218.86 | 12676.44 | -14626.31 | 0 | 0 | 0 | 0 |
| DStlD1 | 0 | 0 | 685.72 | 7305.7 | -8228.59 | 0 | 0 | 0 | 0 |
| DStlD2 | 0 | 0 | 933.22 | 9749.2 | -11198.59 | 0 | 0 | 0 | 0 |
| DCon1 | 0 | 0 | 960 | 10227.97 | -11520.03 | 0 | 0 | 0 | 0 |
| DCon2 | 0 | 0 | 1218.86 | 12676.44 | -14626.31 | 0 | 0 | 0 | 0 |

## 5.2 Story Results

**Story Drifts**

| Story | Load Case/Combo | Label | Item | Drift | X m | Y m | Z m |
|---|---|---|---|---|---|---|---|
| Roof | Dead | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | Dead | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | Live | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | Live | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | D | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | D | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | D+L | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | D+L | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DStlS1 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DStlS1 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DStlS2 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DStlS2 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DStlD1 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DStlD1 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DStlD2 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DStlD2 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DCon1 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DCon1 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Roof | DCon2 | 46 | Max Drift X | 0 | 15 | 24 | 12 |
| Roof | DCon2 | 46 | Max Drift Y | 0 | 15 | 24 | 12 |
| Mezzanine | Dead | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | Dead | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | Live | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | Live | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | D | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | D | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | D+L | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | D+L | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DStlS1 | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | DStlS1 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DStlS2 | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | DStlS2 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DStlD1 | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | DStlD1 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DStlD2 | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | DStlD2 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DCon1 | 46 | Max Drift X | 0 | 15 | 24 | 7 |
| Mezzanine | DCon1 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |
| Mezzanine | DCon2 | 46 | Max Drift X | 0 | 15 | 24 | 7 |

| Story | Load Case/Combo | Label | Item | Drift | X m | Y m | Z m |
|---|---|---|---|---|---|---|---|
| Mezzanine | DCon2 | 46 | Max Drift Y | 0 | 15 | 24 | 7 |

**Story Forces**

| Story | Load Case/Combo | Location | P tonf | VX tonf | VY tonf | T tonf-m | MX tonf-m | MY tonf-m |
|---|---|---|---|---|---|---|---|---|
| Roof | Dead | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | Dead | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | Live | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | Live | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | D | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | D | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | D+L | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | D+L | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlS1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlS1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlS2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlS2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlD1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlD1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlD2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DStlD2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DCon1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DCon1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DCon2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Roof | DCon2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | Dead | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | Dead | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | Live | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | Live | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | D | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | D | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | D+L | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | D+L | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlS1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlS1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlS2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlS2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlD1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlD1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlD2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DStlD2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DCon1 | Top | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Load Case/Combo | Location | P tonf | VX tonf | VY tonf | T tonf-m | MX tonf-m | MY tonf-m |
|---|---|---|---|---|---|---|---|---|
| Mezzanine | DCon1 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DCon2 | Top | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | DCon2 | Bottom | 0 | 0 | 0 | 0 | 0 | 0 |

### 5.3 Point Results

**Joint Reactions**

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 1 | 109 | Dead | 0 | 0 | 20.8 | 0.00325 | 28.71 | 0 |
| Roof | 1 | 109 | Live | 0 | 0 | 7.24 | 0.001161 | 9.97 | 0 |
| Roof | 1 | 109 | D | 0 | 0 | 29.12 | 0.00455 | 40.19 | 0 |
| Roof | 1 | 109 | D+L | 0 | 0 | 36.55 | 0.01 | 50.4 | 0 |
| Roof | 1 | 109 | DStlS1 | 0 | 0 | 29.12 | 0.00455 | 40.19 | 0 |
| Roof | 1 | 109 | DStlS2 | 0 | 0 | 36.55 | 0.01 | 50.4 | 0 |
| Roof | 1 | 109 | DStlD1 | 0 | 0 | 20.8 | 0.00325 | 28.71 | 0 |
| Roof | 1 | 109 | DStlD2 | 0 | 0 | 28.04 | 0.004411 | 38.68 | 0 |
| Roof | 1 | 109 | DCon1 | 0 | 0 | 29.12 | 0.00455 | 40.19 | 0 |
| Roof | 1 | 109 | DCon2 | 0 | 0 | 36.55 | 0.01 | 50.4 | 0 |
| Roof | 2 | 105 | Dead | 0 | 0 | 19.02 | 0.001438 | 25.24 | 0 |
| Roof | 2 | 105 | Live | 0 | 0 | 6.6 | 0.0005294 | 8.72 | 0 |
| Roof | 2 | 105 | D | 0 | 0 | 26.63 | 0.002013 | 35.34 | 0 |
| Roof | 2 | 105 | D+L | 0 | 0 | 33.38 | 0.002572 | 44.24 | 0 |
| Roof | 2 | 105 | DStlS1 | 0 | 0 | 26.63 | 0.002013 | 35.34 | 0 |
| Roof | 2 | 105 | DStlS2 | 0 | 0 | 33.38 | 0.002572 | 44.24 | 0 |
| Roof | 2 | 105 | DStlD1 | 0 | 0 | 19.02 | 0.001438 | 25.24 | 0 |
| Roof | 2 | 105 | DStlD2 | 0 | 0 | 25.62 | 0.001967 | 33.96 | 0 |
| Roof | 2 | 105 | DCon1 | 0 | 0 | 26.63 | 0.002013 | 35.34 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 2 | 105 | DCon2 | 0 | 0 | 33.38 | 0.002572 | 44.24 | 0 |
| Roof | 5 | 61 | Dead | 0 | 0 | 25.92 | -0.01 | -45.56 | 0 |
| Roof | 5 | 61 | Live | 0 | 0 | 8.88 | -0.003282 | -15.53 | 0 |
| Roof | 5 | 61 | D | 0 | 0 | 36.29 | -0.01 | -63.78 | 0 |
| Roof | 5 | 61 | D+L | 0 | 0 | 45.32 | -0.02 | -79.52 | 0 |
| Roof | 5 | 61 | DStlS1 | 0 | 0 | 36.29 | -0.01 | -63.78 | 0 |
| Roof | 5 | 61 | DStlS2 | 0 | 0 | 45.32 | -0.02 | -79.52 | 0 |
| Roof | 5 | 61 | DStlD1 | 0 | 0 | 25.92 | -0.01 | -45.56 | 0 |
| Roof | 5 | 61 | DStlD2 | 0 | 0 | 34.81 | -0.01 | -61.09 | 0 |
| Roof | 5 | 61 | DCon1 | 0 | 0 | 36.29 | -0.01 | -63.78 | 0 |
| Roof | 5 | 61 | DCon2 | 0 | 0 | 45.32 | -0.02 | -79.52 | 0 |
| Roof | 6 | 65 | Dead | 0 | 0 | 19.02 | 0.001438 | -25.24 | 0 |
| Roof | 6 | 65 | Live | 0 | 0 | 6.6 | 0.0005294 | -8.72 | 0 |
| Roof | 6 | 65 | D | 0 | 0 | 26.63 | 0.002013 | -35.34 | 0 |
| Roof | 6 | 65 | D+L | 0 | 0 | 33.38 | 0.002572 | -44.24 | 0 |
| Roof | 6 | 65 | DStlS1 | 0 | 0 | 26.63 | 0.002013 | -35.34 | 0 |
| Roof | 6 | 65 | DStlS2 | 0 | 0 | 33.38 | 0.002572 | -44.24 | 0 |
| Roof | 6 | 65 | DStlD1 | 0 | 0 | 19.02 | 0.001438 | -25.24 | 0 |
| Roof | 6 | 65 | DStlD2 | 0 | 0 | 25.62 | 0.001967 | -33.96 | 0 |
| Roof | 6 | 65 | DCon1 | 0 | 0 | 26.63 | 0.002013 | -35.34 | 0 |
| Roof | 6 | 65 | DCon2 | 0 | 0 | 33.38 | 0.002572 | -44.24 | 0 |
| Roof | 7 | 69 | Dead | 0 | 0 | 20.8 | 0.00325 | -28.71 | 0 |
| Roof | 7 | 69 | Live | 0 | 0 | 7.24 | 0.001161 | -9.97 | 0 |
| Roof | 7 | 69 | D | 0 | 0 | 29.12 | 0.00455 | -40.19 | 0 |
| Roof | 7 | 69 | D+L | 0 | 0 | 36.55 | 0.01 | -50.4 | 0 |
| Roof | 7 | 69 | DStlS1 | 0 | 0 | 29.12 | 0.00455 | -40.19 | 0 |
| Roof | 7 | 69 | DStlS2 | 0 | 0 | 36.55 | 0.01 | -50.4 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 7 | 69 | DStlD1 | 0 | 0 | 20.8 | 0.00325 | -28.71 | 0 |
| Roof | 7 | 69 | DStlD2 | 0 | 0 | 28.04 | 0.004411 | -38.68 | 0 |
| Roof | 7 | 69 | DCon1 | 0 | 0 | 29.12 | 0.00455 | -40.19 | 0 |
| Roof | 7 | 69 | DCon2 | 0 | 0 | 36.55 | 0.01 | -50.4 | 0 |
| Roof | 8 | 73 | Dead | 0 | 0 | 8.82 | -3.17 | -11.08 | 0 |
| Roof | 8 | 73 | Live | 0 | 0 | 2.95 | -1.13 | -3.66 | 0 |
| Roof | 8 | 73 | D | 0 | 0 | 12.35 | -4.44 | -15.51 | 0 |
| Roof | 8 | 73 | D+L | 0 | 0 | 15.31 | -5.63 | -19.15 | 0 |
| Roof | 8 | 73 | DStlS1 | 0 | 0 | 12.35 | -4.44 | -15.51 | 0 |
| Roof | 8 | 73 | DStlS2 | 0 | 0 | 15.31 | -5.63 | -19.15 | 0 |
| Roof | 8 | 73 | DStlD1 | 0 | 0 | 8.82 | -3.17 | -11.08 | 0 |
| Roof | 8 | 73 | DStlD2 | 0 | 0 | 11.78 | -4.31 | -14.74 | 0 |
| Roof | 8 | 73 | DCon1 | 0 | 0 | 12.35 | -4.44 | -15.51 | 0 |
| Roof | 8 | 73 | DCon2 | 0 | 0 | 15.31 | -5.63 | -19.15 | 0 |
| Roof | 9 | 57 | Dead | 0 | 0 | 9.06 | 3.17 | -11.43 | 0 |
| Roof | 9 | 57 | Live | 0 | 0 | 2.94 | 1.13 | -3.63 | 0 |
| Roof | 9 | 57 | D | 0 | 0 | 12.68 | 4.44 | -16 | 0 |
| Roof | 9 | 57 | D+L | 0 | 0 | 15.56 | 5.62 | -19.52 | 0 |
| Roof | 9 | 57 | DStlS1 | 0 | 0 | 12.68 | 4.44 | -16 | 0 |
| Roof | 9 | 57 | DStlS2 | 0 | 0 | 15.56 | 5.62 | -19.52 | 0 |
| Roof | 9 | 57 | DStlD1 | 0 | 0 | 9.06 | 3.17 | -11.43 | 0 |
| Roof | 9 | 57 | DStlD2 | 0 | 0 | 11.99 | 4.31 | -15.06 | 0 |
| Roof | 9 | 57 | DCon1 | 0 | 0 | 12.68 | 4.44 | -16 | 0 |
| Roof | 9 | 57 | DCon2 | 0 | 0 | 15.56 | 5.62 | -19.52 | 0 |
| Roof | 10 | 101 | Dead | 0 | 0 | 25.92 | -0.01 | 45.56 | 0 |
| Roof | 10 | 101 | Live | 0 | 0 | 8.88 | -0.0032 | 15.53 | 0 |
| Roof | 10 | 101 | D | 0 | 0 | 36.29 | -0.01 | 63.78 | 0 |
| Roof | 10 | 101 | D+L | 0 | 0 | 45.32 | -0.02 | 79.52 | 0 |
| Roof | 10 | 101 | DStlS1 | 0 | 0 | 36.29 | -0.01 | 63.78 | 0 |
| Roof | 10 | 101 | DStlS2 | 0 | 0 | 45.32 | -0.02 | 79.52 | 0 |
| Roof | 10 | 101 | DStlD1 | 0 | 0 | 25.92 | -0.01 | 45.56 | 0 |
| Roof | 10 | 101 | DStlD2 | 0 | 0 | 34.81 | -0.01 | 61.09 | 0 |
| Roof | 10 | 101 | DCon1 | 0 | 0 | 36.29 | -0.01 | 63.78 | 0 |
| Roof | 10 | 101 | DCon2 | 0 | 0 | 45.32 | -0.02 | 79.52 | 0 |
| Roof | 11 | 113 | Dead | 0 | 0 | 8.82 | -3.17 | 11.08 | 0 |
| Roof | 11 | 113 | Live | 0 | 0 | 2.95 | -1.13 | 3.66 | 0 |
| Roof | 11 | 113 | D | 0 | 0 | 12.35 | -4.44 | 15.51 | 0 |
| Roof | 11 | 113 | D+L | 0 | 0 | 15.31 | -5.63 | 19.15 | 0 |
| Roof | 11 | 113 | DStlS1 | 0 | 0 | 12.35 | -4.44 | 15.51 | 0 |
| Roof | 11 | 113 | DStlS2 | 0 | 0 | 15.31 | -5.63 | 19.15 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 11 | 113 | DStlD1 | 0 | 0 | 8.82 | -3.17 | 11.08 | 0 |
| Roof | 11 | 113 | DStlD2 | 0 | 0 | 11.78 | -4.31 | 14.74 | 0 |
| Roof | 11 | 113 | DCon1 | 0 | 0 | 12.35 | -4.44 | 15.51 | 0 |
| Roof | 11 | 113 | DCon2 | 0 | 0 | 15.31 | -5.63 | 19.15 | 0 |
| Roof | 12 | 97 | Dead | 0 | 0 | 9.06 | 3.17 | 11.43 | 0 |
| Roof | 12 | 97 | Live | 0 | 0 | 2.94 | 1.13 | 3.63 | 0 |
| Roof | 12 | 97 | D | 0 | 0 | 12.68 | 4.44 | 16 | 0 |
| Roof | 12 | 97 | D+L | 0 | 0 | 15.56 | 5.62 | 19.52 | 0 |
| Roof | 12 | 97 | DStlS1 | 0 | 0 | 12.68 | 4.44 | 16 | 0 |
| Roof | 12 | 97 | DStlS2 | 0 | 0 | 15.56 | 5.62 | 19.52 | 0 |
| Roof | 12 | 97 | DStlD1 | 0 | 0 | 9.06 | 3.17 | 11.43 | 0 |
| Roof | 12 | 97 | DStlD2 | 0 | 0 | 11.99 | 4.31 | 15.06 | 0 |
| Roof | 12 | 97 | DCon1 | 0 | 0 | 12.68 | 4.44 | 16 | 0 |
| Roof | 12 | 97 | DCon2 | 0 | 0 | 15.56 | 5.62 | 19.52 | 0 |
| Roof | 13 | 81 | Dead | 0 | 0 | 33.74 | 1.13 | 0 | 0 |
| Roof | 13 | 81 | Live | 0 | 0 | 11.31 | 0.27 | 0 | 0 |
| Roof | 13 | 81 | D | 0 | 0 | 47.23 | 1.59 | 0 | 0 |
| Roof | 13 | 81 | D+L | 0 | 0 | 58.57 | 1.79 | 0 | 0 |
| Roof | 13 | 81 | DStlS1 | 0 | 0 | 47.23 | 1.59 | 0 | 0 |
| Roof | 13 | 81 | DStlS2 | 0 | 0 | 58.57 | 1.79 | 0 | 0 |
| Roof | 13 | 81 | DStlD1 | 0 | 0 | 33.74 | 1.13 | 0 | 0 |
| Roof | 13 | 81 | DStlD2 | 0 | 0 | 45.04 | 1.4 | 0 | 0 |
| Roof | 13 | 81 | DCon1 | 0 | 0 | 47.23 | 1.59 | 0 | 0 |
| Roof | 13 | 81 | DCon2 | 0 | 0 | 58.57 | 1.79 | 0 | 0 |
| Roof | 23 | 12 | Dead | 0 | 0 | 39.2 | 13.55 | -6.28 | 0 |
| Roof | 23 | 12 | Live | 0 | 0 | 13.38 | 2.59 | -3.09 | 0 |
| Roof | 23 | 12 | D | 0 | 0 | 54.88 | 18.97 | -8.79 | 0 |
| Roof | 23 | 12 | D+L | 0 | 0 | 68.44 | 20.4 | -12.48 | 0 |
| Roof | 23 | 12 | DStlS1 | 0 | 0 | 54.88 | 18.97 | -8.79 | 0 |
| Roof | 23 | 12 | DStlS2 | 0 | 0 | 68.44 | 20.4 | -12.48 | 0 |
| Roof | 23 | 12 | DStlD1 | 0 | 0 | 39.2 | 13.55 | -6.28 | 0 |
| Roof | 23 | 12 | DStlD2 | 0 | 0 | 52.58 | 16.14 | -9.37 | 0 |
| Roof | 23 | 12 | DCon1 | 0 | 0 | 54.88 | 18.97 | -8.79 | 0 |
| Roof | 23 | 12 | DCon2 | 0 | 0 | 68.44 | 20.4 | -12.48 | 0 |
| Roof | 28 | 23 | Dead | 0 | 0 | 9.69 | 2.39 | 7.46 | 0 |
| Roof | 28 | 23 | Live | 0 | 0 | 2.95 | 0.8 | 2.41 | 0 |
| Roof | 28 | 23 | D | 0 | 0 | 13.57 | 3.34 | 10.44 | 0 |
| Roof | 28 | 23 | D+L | 0 | 0 | 16.34 | 4.15 | 12.81 | 0 |
| Roof | 28 | 23 | DStlS1 | 0 | 0 | 13.57 | 3.34 | 10.44 | 0 |
| Roof | 28 | 23 | DStlS2 | 0 | 0 | 16.34 | 4.15 | 12.81 | 0 |
| Roof | 28 | 23 | DStlD1 | 0 | 0 | 9.69 | 2.39 | 7.46 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 28 | 23 | DStlD2 | 0 | 0 | 12.64 | 3.19 | 9.87 | 0 |
| Roof | 28 | 23 | DCon1 | 0 | 0 | 13.57 | 3.34 | 10.44 | 0 |
| Roof | 28 | 23 | DCon2 | 0 | 0 | 16.34 | 4.15 | 12.81 | 0 |
| Roof | 29 | 21 | Dead | 0 | 0 | 9.69 | 2.39 | -7.46 | 0 |
| Roof | 29 | 21 | Live | 0 | 0 | 2.95 | 0.8 | -2.41 | 0 |
| Roof | 29 | 21 | D | 0 | 0 | 13.57 | 3.34 | -10.44 | 0 |
| Roof | 29 | 21 | D+L | 0 | 0 | 16.34 | 4.15 | -12.81 | 0 |
| Roof | 29 | 21 | DStlS1 | 0 | 0 | 13.57 | 3.34 | -10.44 | 0 |
| Roof | 29 | 21 | DStlS2 | 0 | 0 | 16.34 | 4.15 | -12.81 | 0 |
| Roof | 29 | 21 | DStlD1 | 0 | 0 | 9.69 | 2.39 | -7.46 | 0 |
| Roof | 29 | 21 | DStlD2 | 0 | 0 | 12.64 | 3.19 | -9.87 | 0 |
| Roof | 29 | 21 | DCon1 | 0 | 0 | 13.57 | 3.34 | -10.44 | 0 |
| Roof | 29 | 21 | DCon2 | 0 | 0 | 16.34 | 4.15 | -12.81 | 0 |
| Roof | 40 | 15 | Dead | 0 | 0 | 39.2 | 13.55 | 6.28 | 0 |
| Roof | 40 | 15 | Live | 0 | 0 | 13.38 | 2.59 | 3.09 | 0 |
| Roof | 40 | 15 | D | 0 | 0 | 54.88 | 18.97 | 8.79 | 0 |
| Roof | 40 | 15 | D+L | 0 | 0 | 68.44 | 20.4 | 12.48 | 0 |
| Roof | 40 | 15 | DStlS1 | 0 | 0 | 54.88 | 18.97 | 8.79 | 0 |
| Roof | 40 | 15 | DStlS2 | 0 | 0 | 68.44 | 20.4 | 12.48 | 0 |
| Roof | 40 | 15 | DStlD1 | 0 | 0 | 39.2 | 13.55 | 6.28 | 0 |
| Roof | 40 | 15 | DStlD2 | 0 | 0 | 52.58 | 16.14 | 9.37 | 0 |
| Roof | 40 | 15 | DCon1 | 0 | 0 | 54.88 | 18.97 | 8.79 | 0 |
| Roof | 40 | 15 | DCon2 | 0 | 0 | 68.44 | 20.4 | 12.48 | 0 |
| Roof | 4 | 5 | Dead | 0 | 0 | 43.93 | -9.59 | -12.71 | 0 |
| Roof | 4 | 5 | Live | 0 | 0 | 15.08 | -1.12 | -5.43 | 0 |
| Roof | 4 | 5 | D | 0 | 0 | 61.5 | -13.43 | -17.79 | 0 |
| Roof | 4 | 5 | D+L | 0 | 0 | 76.85 | -13.31 | -23.93 | 0 |
| Roof | 4 | 5 | DStlS1 | 0 | 0 | 61.5 | -13.43 | -17.79 | 0 |
| Roof | 4 | 5 | DStlS2 | 0 | 0 | 76.85 | -13.31 | -23.93 | 0 |
| Roof | 4 | 5 | DStlD1 | 0 | 0 | 43.93 | -9.59 | -12.71 | 0 |
| Roof | 4 | 5 | DStlD2 | 0 | 0 | 59.02 | -10.72 | -18.13 | 0 |
| Roof | 4 | 5 | DCon1 | 0 | 0 | 61.5 | -13.43 | -17.79 | 0 |
| Roof | 4 | 5 | DCon2 | 0 | 0 | 76.85 | -13.31 | -23.93 | 0 |
| Roof | 18 | 9 | Dead | 0 | 0 | 43.93 | -9.59 | 12.71 | 0 |
| Roof | 18 | 9 | Live | 0 | 0 | 15.08 | -1.12 | 5.43 | 0 |
| Roof | 18 | 9 | D | 0 | 0 | 61.5 | -13.43 | 17.79 | 0 |
| Roof | 18 | 9 | D+L | 0 | 0 | 76.85 | -13.31 | 23.93 | 0 |
| Roof | 18 | 9 | DStlS1 | 0 | 0 | 61.5 | -13.43 | 17.79 | 0 |
| Roof | 18 | 9 | DStlS2 | 0 | 0 | 76.85 | -13.31 | 23.93 | 0 |
| Roof | 18 | 9 | DStlD1 | 0 | 0 | 43.93 | -9.59 | 12.71 | 0 |
| Roof | 18 | 9 | DStlD2 | 0 | 0 | 59.02 | -10.72 | 18.13 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Roof | 18 | 9 | DCon1 | 0 | 0 | 61.5 | -13.43 | 17.79 | 0 |
| Roof | 18 | 9 | DCon2 | 0 | 0 | 76.85 | -13.31 | 23.93 | 0 |
| Roof | 19 | 33 | Dead | 0 | 0 | 12.36 | -6.32 | 10.72 | 0 |
| Roof | 19 | 33 | Live | 0 | 0 | 4.08 | -2.26 | 3.67 | 0 |
| Roof | 19 | 33 | D | 0 | 0 | 17.3 | -8.85 | 15.01 | 0 |
| Roof | 19 | 33 | D+L | 0 | 0 | 21.36 | -11.21 | 18.73 | 0 |
| Roof | 19 | 33 | DStlS1 | 0 | 0 | 17.3 | -8.85 | 15.01 | 0 |
| Roof | 19 | 33 | DStlS2 | 0 | 0 | 21.36 | -11.21 | 18.73 | 0 |
| Roof | 19 | 33 | DStlD1 | 0 | 0 | 12.36 | -6.32 | 10.72 | 0 |
| Roof | 19 | 33 | DStlD2 | 0 | 0 | 16.44 | -8.58 | 14.38 | 0 |
| Roof | 19 | 33 | DCon1 | 0 | 0 | 17.3 | -8.85 | 15.01 | 0 |
| Roof | 19 | 33 | DCon2 | 0 | 0 | 21.36 | -11.21 | 18.73 | 0 |
| Roof | 46 | 42 | Dead | 0 | 0 | 12.36 | -6.32 | -10.72 | 0 |
| Roof | 46 | 42 | Live | 0 | 0 | 4.08 | -2.26 | -3.67 | 0 |
| Roof | 46 | 42 | D | 0 | 0 | 17.3 | -8.85 | -15.01 | 0 |
| Roof | 46 | 42 | D+L | 0 | 0 | 21.36 | -11.21 | -18.73 | 0 |
| Roof | 46 | 42 | DStlS1 | 0 | 0 | 17.3 | -8.85 | -15.01 | 0 |
| Roof | 46 | 42 | DStlS2 | 0 | 0 | 21.36 | -11.21 | -18.73 | 0 |
| Roof | 46 | 42 | DStlD1 | 0 | 0 | 12.36 | -6.32 | -10.72 | 0 |
| Roof | 46 | 42 | DStlD2 | 0 | 0 | 16.44 | -8.58 | -14.38 | 0 |
| Roof | 46 | 42 | DCon1 | 0 | 0 | 17.3 | -8.85 | -15.01 | 0 |
| Roof | 46 | 42 | DCon2 | 0 | 0 | 21.36 | -11.21 | -18.73 | 0 |
| Mezzanine | 1 | 108 | Dead | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | D | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | D+L | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DStlS1 | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DStlS2 | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DStlD1 | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DStlD2 | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DCon1 | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 1 | 108 | DCon2 | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 2 | 104 | Dead | 0 | 0 | 19.82 | -0.43 | 23.3 | 0 |
| Mezzanine | 2 | 104 | Live | 0 | 0 | 6.75 | -0.33 | 9 | 0 |
| Mezzanine | 2 | 104 | D | 0 | 0 | 27.75 | -0.6 | 32.62 | 0 |
| Mezzanine | 2 | 104 | D+L | 0 | 0 | 34.58 | -1.04 | 42.36 | 0 |
| Mezzanine | 2 | 104 | DStlS1 | 0 | 0 | 27.75 | -0.6 | 32.62 | 0 |
| Mezzanine | 2 | 104 | DStlS2 | 0 | 0 | 34.58 | -1.04 | 42.36 | 0 |
| Mezzanine | 2 | 104 | DStlD1 | 0 | 0 | 19.82 | -0.43 | 23.3 | 0 |
| Mezzanine | 2 | 104 | DStlD2 | 0 | 0 | 26.57 | -0.76 | 32.3 | 0 |
| Mezzanine | 2 | 104 | DCon1 | 0 | 0 | 27.75 | -0.6 | 32.62 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | 2 | 104 | DCon2 | 0 | 0 | 34.58 | -1.04 | 42.36 | 0 |
| Mezzanine | 5 | 60 | Dead | 0 | 0 | 26.98 | 0 | -40.8 | 0 |
| Mezzanine | 5 | 60 | Live | 0 | 0 | 16.29 | 0 | -27.51 | 0 |
| Mezzanine | 5 | 60 | D | 0 | 0 | 37.77 | 0 | -57.13 | 0 |
| Mezzanine | 5 | 60 | D+L | 0 | 0 | 58.43 | 0 | -92.99 | 0 |
| Mezzanine | 5 | 60 | DStlS1 | 0 | 0 | 37.77 | 0 | -57.13 | 0 |
| Mezzanine | 5 | 60 | DStlS2 | 0 | 0 | 58.43 | 0 | -92.99 | 0 |
| Mezzanine | 5 | 60 | DStlD1 | 0 | 0 | 26.98 | 0 | -40.8 | 0 |
| Mezzanine | 5 | 60 | DStlD2 | 0 | 0 | 43.26 | 0 | -68.32 | 0 |
| Mezzanine | 5 | 60 | DCon1 | 0 | 0 | 37.77 | 0 | -57.13 | 0 |
| Mezzanine | 5 | 60 | DCon2 | 0 | 0 | 58.43 | 0 | -92.99 | 0 |
| Mezzanine | 6 | 64 | Dead | 0 | 0 | 19.82 | -0.43 | -23.3 | 0 |
| Mezzanine | 6 | 64 | Live | 0 | 0 | 6.75 | -0.33 | -9 | 0 |
| Mezzanine | 6 | 64 | D | 0 | 0 | 27.75 | -0.6 | -32.62 | 0 |
| Mezzanine | 6 | 64 | D+L | 0 | 0 | 34.58 | -1.04 | -42.36 | 0 |
| Mezzanine | 6 | 64 | DStlS1 | 0 | 0 | 27.75 | -0.6 | -32.62 | 0 |
| Mezzanine | 6 | 64 | DStlS2 | 0 | 0 | 34.58 | -1.04 | -42.36 | 0 |
| Mezzanine | 6 | 64 | DStlD1 | 0 | 0 | 19.82 | -0.43 | -23.3 | 0 |
| Mezzanine | 6 | 64 | DStlD2 | 0 | 0 | 26.57 | -0.76 | -32.3 | 0 |
| Mezzanine | 6 | 64 | DCon1 | 0 | 0 | 27.75 | -0.6 | -32.62 | 0 |
| Mezzanine | 6 | 64 | DCon2 | 0 | 0 | 34.58 | -1.04 | -42.36 | 0 |
| Mezzanine | 7 | 68 | Dead | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | D | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | D+L | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DStlS1 | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DStlS2 | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DStlD1 | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DStlD2 | 0 | 0 | 3.47 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DCon1 | 0 | 0 | 4.86 | 0 | 0 | 0 |
| Mezzanine | 7 | 68 | DCon2 | 0 | 0 | 4.16 | 0 | 0 | 0 |
| Mezzanine | 8 | 72 | Dead | 0 | 0 | 4.33 | -0.25 | -3.9 | 0 |
| Mezzanine | 8 | 72 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 8 | 72 | D | 0 | 0 | 6.07 | -0.36 | -5.46 | 0 |
| Mezzanine | 8 | 72 | D+L | 0 | 0 | 5.2 | -0.3 | -4.68 | 0 |
| Mezzanine | 8 | 72 | DStlS1 | 0 | 0 | 6.07 | -0.36 | -5.46 | 0 |
| Mezzanine | 8 | 72 | DStlS2 | 0 | 0 | 5.2 | -0.3 | -4.68 | 0 |
| Mezzanine | 8 | 72 | DStlD1 | 0 | 0 | 4.33 | -0.25 | -3.9 | 0 |
| Mezzanine | 8 | 72 | DStlD2 | 0 | 0 | 4.33 | -0.25 | -3.9 | 0 |
| Mezzanine | 8 | 72 | DCon1 | 0 | 0 | 6.07 | -0.36 | -5.46 | 0 |
| Mezzanine | 8 | 72 | DCon2 | 0 | 0 | 5.2 | -0.3 | -4.68 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | 9 | 56 | Dead | 0 | 0 | 18.58 | 0.68 | -24.03 | 0 |
| Mezzanine | 9 | 56 | Live | 0 | 0 | 6.75 | 0.33 | -9 | 0 |
| Mezzanine | 9 | 56 | D | 0 | 0 | 26.01 | 0.95 | -33.65 | 0 |
| Mezzanine | 9 | 56 | D+L | 0 | 0 | 33.09 | 1.34 | -43.24 | 0 |
| Mezzanine | 9 | 56 | DStlS1 | 0 | 0 | 26.01 | 0.95 | -33.65 | 0 |
| Mezzanine | 9 | 56 | DStlS2 | 0 | 0 | 33.09 | 1.34 | -43.24 | 0 |
| Mezzanine | 9 | 56 | DStlD1 | 0 | 0 | 18.58 | 0.68 | -24.03 | 0 |
| Mezzanine | 9 | 56 | DStlD2 | 0 | 0 | 25.33 | 1.01 | -33.03 | 0 |
| Mezzanine | 9 | 56 | DCon1 | 0 | 0 | 26.01 | 0.95 | -33.65 | 0 |
| Mezzanine | 9 | 56 | DCon2 | 0 | 0 | 33.09 | 1.34 | -43.24 | 0 |
| Mezzanine | 10 | 100 | Dead | 0 | 0 | 26.98 | 0 | 40.8 | 0 |
| Mezzanine | 10 | 100 | Live | 0 | 0 | 16.29 | 0 | 27.51 | 0 |
| Mezzanine | 10 | 100 | D | 0 | 0 | 37.77 | 0 | 57.13 | 0 |
| Mezzanine | 10 | 100 | D+L | 0 | 0 | 58.43 | 0 | 92.99 | 0 |
| Mezzanine | 10 | 100 | DStlS1 | 0 | 0 | 37.77 | 0 | 57.13 | 0 |
| Mezzanine | 10 | 100 | DStlS2 | 0 | 0 | 58.43 | 0 | 92.99 | 0 |
| Mezzanine | 10 | 100 | DStlD1 | 0 | 0 | 26.98 | 0 | 40.8 | 0 |
| Mezzanine | 10 | 100 | DStlD2 | 0 | 0 | 43.26 | 0 | 68.32 | 0 |
| Mezzanine | 10 | 100 | DCon1 | 0 | 0 | 37.77 | 0 | 57.13 | 0 |
| Mezzanine | 10 | 100 | DCon2 | 0 | 0 | 58.43 | 0 | 92.99 | 0 |
| Mezzanine | 11 | 112 | Dead | 0 | 0 | 4.34 | -0.25 | 3.9 | 0 |
| Mezzanine | 11 | 112 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 11 | 112 | D | 0 | 0 | 6.07 | -0.36 | 5.46 | 0 |
| Mezzanine | 11 | 112 | D+L | 0 | 0 | 5.2 | -0.3 | 4.68 | 0 |
| Mezzanine | 11 | 112 | DStlS1 | 0 | 0 | 6.07 | -0.36 | 5.46 | 0 |
| Mezzanine | 11 | 112 | DStlS2 | 0 | 0 | 5.2 | -0.3 | 4.68 | 0 |
| Mezzanine | 11 | 112 | DStlD1 | 0 | 0 | 4.34 | -0.25 | 3.9 | 0 |
| Mezzanine | 11 | 112 | DStlD2 | 0 | 0 | 4.34 | -0.25 | 3.9 | 0 |
| Mezzanine | 11 | 112 | DCon1 | 0 | 0 | 6.07 | -0.36 | 5.46 | 0 |
| Mezzanine | 11 | 112 | DCon2 | 0 | 0 | 5.2 | -0.3 | 4.68 | 0 |
| Mezzanine | 12 | 96 | Dead | 0 | 0 | 18.58 | 0.68 | 24.03 | 0 |
| Mezzanine | 12 | 96 | Live | 0 | 0 | 6.75 | 0.33 | 9 | 0 |
| Mezzanine | 12 | 96 | D | 0 | 0 | 26.01 | 0.95 | 33.65 | 0 |
| Mezzanine | 12 | 96 | D+L | 0 | 0 | 33.09 | 1.34 | 43.24 | 0 |
| Mezzanine | 12 | 96 | DStlS1 | 0 | 0 | 26.01 | 0.95 | 33.65 | 0 |
| Mezzanine | 12 | 96 | DStlS2 | 0 | 0 | 33.09 | 1.34 | 43.24 | 0 |
| Mezzanine | 12 | 96 | DStlD1 | 0 | 0 | 18.58 | 0.68 | 24.03 | 0 |
| Mezzanine | 12 | 96 | DStlD2 | 0 | 0 | 25.33 | 1.01 | 33.03 | 0 |
| Mezzanine | 12 | 96 | DCon1 | 0 | 0 | 26.01 | 0.95 | 33.65 | 0 |
| Mezzanine | 12 | 96 | DCon2 | 0 | 0 | 33.09 | 1.34 | 43.24 | 0 |
| Mezzanine | 13 | 80 | Dead | 0 | 0 | 27.27 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | 13 | 80 | Live | 0 | 0 | 18.03 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | D | 0 | 0 | 38.18 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | D+L | 0 | 0 | 61.57 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DStlS1 | 0 | 0 | 38.18 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DStlS2 | 0 | 0 | 61.57 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DStlD1 | 0 | 0 | 27.27 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DStlD2 | 0 | 0 | 45.3 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DCon1 | 0 | 0 | 38.18 | 0 | 0 | 0 |
| Mezzanine | 13 | 80 | DCon2 | 0 | 0 | 61.57 | 0 | 0 | 0 |
| Mezzanine | 23 | 134 | Dead | 0 | 0 | 22.61 | -4.1 | -13.94 | 0 |
| Mezzanine | 23 | 134 | Live | 0 | 0 | 7.6 | -2.72 | -5.71 | 0 |
| Mezzanine | 23 | 134 | D | 0 | 0 | 31.65 | -5.74 | -19.52 | 0 |
| Mezzanine | 23 | 134 | D+L | 0 | 0 | 39.29 | -9.27 | -25.87 | 0 |
| Mezzanine | 23 | 134 | DStlS1 | 0 | 0 | 31.65 | -5.74 | -19.52 | 0 |
| Mezzanine | 23 | 134 | DStlS2 | 0 | 0 | 39.29 | -9.27 | -25.87 | 0 |
| Mezzanine | 23 | 134 | DStlD1 | 0 | 0 | 22.61 | -4.1 | -13.94 | 0 |
| Mezzanine | 23 | 134 | DStlD2 | 0 | 0 | 30.21 | -6.82 | -19.65 | 0 |
| Mezzanine | 23 | 134 | DCon1 | 0 | 0 | 31.65 | -5.74 | -19.52 | 0 |
| Mezzanine | 23 | 134 | DCon2 | 0 | 0 | 39.29 | -9.27 | -25.87 | 0 |
| Mezzanine | 28 | 121 | Dead | 0 | 0 | 23.43 | 4.1 | 14.35 | 0 |
| Mezzanine | 28 | 121 | Live | 0 | 0 | 7.6 | 2.72 | 5.71 | 0 |
| Mezzanine | 28 | 121 | D | 0 | 0 | 32.8 | 5.74 | 20.09 | 0 |
| Mezzanine | 28 | 121 | D+L | 0 | 0 | 40.27 | 9.27 | 26.36 | 0 |
| Mezzanine | 28 | 121 | DStlS1 | 0 | 0 | 32.8 | 5.74 | 20.09 | 0 |
| Mezzanine | 28 | 121 | DStlS2 | 0 | 0 | 40.27 | 9.27 | 26.36 | 0 |
| Mezzanine | 28 | 121 | DStlD1 | 0 | 0 | 23.43 | 4.1 | 14.35 | 0 |
| Mezzanine | 28 | 121 | DStlD2 | 0 | 0 | 31.03 | 6.82 | 20.06 | 0 |
| Mezzanine | 28 | 121 | DCon1 | 0 | 0 | 32.8 | 5.74 | 20.09 | 0 |
| Mezzanine | 28 | 121 | DCon2 | 0 | 0 | 40.27 | 9.27 | 26.36 | 0 |
| Mezzanine | 29 | 122 | Dead | 0 | 0 | 23.43 | 4.1 | -14.35 | 0 |
| Mezzanine | 29 | 122 | Live | 0 | 0 | 7.6 | 2.72 | -5.71 | 0 |
| Mezzanine | 29 | 122 | D | 0 | 0 | 32.8 | 5.74 | -20.09 | 0 |
| Mezzanine | 29 | 122 | D+L | 0 | 0 | 40.27 | 9.27 | -26.36 | 0 |
| Mezzanine | 29 | 122 | DStlS1 | 0 | 0 | 32.8 | 5.74 | -20.09 | 0 |
| Mezzanine | 29 | 122 | DStlS2 | 0 | 0 | 40.27 | 9.27 | -26.36 | 0 |
| Mezzanine | 29 | 122 | DStlD1 | 0 | 0 | 23.43 | 4.1 | -14.35 | 0 |
| Mezzanine | 29 | 122 | DStlD2 | 0 | 0 | 31.03 | 6.82 | -20.06 | 0 |
| Mezzanine | 29 | 122 | DCon1 | 0 | 0 | 32.8 | 5.74 | -20.09 | 0 |
| Mezzanine | 29 | 122 | DCon2 | 0 | 0 | 40.27 | 9.27 | -26.36 | 0 |
| Mezzanine | 40 | 133 | Dead | 0 | 0 | 22.61 | -4.1 | 13.94 | 0 |
| Mezzanine | 40 | 133 | Live | 0 | 0 | 7.6 | -2.72 | 5.71 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Mezzanine | 40 | 133 | D | 0 | 0 | 31.65 | -5.74 | 19.52 | 0 |
| Mezzanine | 40 | 133 | D+L | 0 | 0 | 39.29 | -9.27 | 25.87 | 0 |
| Mezzanine | 40 | 133 | DStlS1 | 0 | 0 | 31.65 | -5.74 | 19.52 | 0 |
| Mezzanine | 40 | 133 | DStlS2 | 0 | 0 | 39.29 | -9.27 | 25.87 | 0 |
| Mezzanine | 40 | 133 | DStlD1 | 0 | 0 | 22.61 | -4.1 | 13.94 | 0 |
| Mezzanine | 40 | 133 | DStlD2 | 0 | 0 | 30.21 | -6.82 | 19.65 | 0 |
| Mezzanine | 40 | 133 | DCon1 | 0 | 0 | 31.65 | -5.74 | 19.52 | 0 |
| Mezzanine | 40 | 133 | DCon2 | 0 | 0 | 39.29 | -9.27 | 25.87 | 0 |
| Mezzanine | 19 | 48 | Dead | 0 | 0 | 4.34 | 0 | 2.17 | 0 |
| Mezzanine | 19 | 48 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 19 | 48 | D | 0 | 0 | 6.07 | 0 | 3.03 | 0 |
| Mezzanine | 19 | 48 | D+L | 0 | 0 | 5.2 | 0 | 2.6 | 0 |
| Mezzanine | 19 | 48 | DStlS1 | 0 | 0 | 6.07 | 0 | 3.03 | 0 |
| Mezzanine | 19 | 48 | DStlS2 | 0 | 0 | 5.2 | 0 | 2.6 | 0 |
| Mezzanine | 19 | 48 | DStlD1 | 0 | 0 | 4.34 | 0 | 2.17 | 0 |
| Mezzanine | 19 | 48 | DStlD2 | 0 | 0 | 4.34 | 0 | 2.17 | 0 |
| Mezzanine | 19 | 48 | DCon1 | 0 | 0 | 6.07 | 0 | 3.03 | 0 |
| Mezzanine | 19 | 48 | DCon2 | 0 | 0 | 5.2 | 0 | 2.6 | 0 |
| Mezzanine | 46 | 51 | Dead | 0 | 0 | 4.33 | 0 | -2.17 | 0 |
| Mezzanine | 46 | 51 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Mezzanine | 46 | 51 | D | 0 | 0 | 6.07 | 0 | -3.03 | 0 |
| Mezzanine | 46 | 51 | D+L | 0 | 0 | 5.2 | 0 | -2.6 | 0 |
| Mezzanine | 46 | 51 | DStlS1 | 0 | 0 | 6.07 | 0 | -3.03 | 0 |
| Mezzanine | 46 | 51 | DStlS2 | 0 | 0 | 5.2 | 0 | -2.6 | 0 |
| Mezzanine | 46 | 51 | DStlD1 | 0 | 0 | 4.33 | 0 | -2.17 | 0 |
| Mezzanine | 46 | 51 | DStlD2 | 0 | 0 | 4.33 | 0 | -2.17 | 0 |
| Mezzanine | 46 | 51 | DCon1 | 0 | 0 | 6.07 | 0 | -3.03 | 0 |
| Mezzanine | 46 | 51 | DCon2 | 0 | 0 | 5.2 | 0 | -2.6 | 0 |
| Base | 1 | 106 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 1 | 106 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | D | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Base | 2 | 102 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 2 | 102 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 5 | 58 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 6 | 62 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 7 | 66 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Base | 8 | 70 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 8 | 70 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 9 | 54 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 10 | 98 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 11 | 110 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Base | 12 | 94 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 12 | 94 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 13 | 78 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 23 | 10 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 28 | 44 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Base | 29 | 1 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 29 | 1 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 40 | 13 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 4 | 2 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 18 | 6 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |

| Story | Joint Label | Unique Name | Load Case/Combo | FX tonf | FY tonf | FZ tonf | MX tonf-m | MY tonf-m | MZ tonf-m |
|---|---|---|---|---|---|---|---|---|---|
| Base | 19 | 46 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 19 | 46 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | Dead | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | Live | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | D | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | D+L | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DStlS1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DStlS2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DStlD1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DStlD2 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DCon1 | 0 | 0 | 0 | 0 | 0 | 0 |
| Base | 46 | 49 | DCon2 | 0 | 0 | 0 | 0 | 0 | 0 |

### 5.4 Modal Results

**Modal Periods and Frequencies**

| Case | Mode | Period sec | Frequency cyc/sec | Circular Frequency rad/sec | Eigenvalue rad²/sec² |
|---|---|---|---|---|---|
| Modal | 1 | 0.165 | 6.069 | 38.133 | 1454.124 |
| Modal | 2 | 0.165 | 6.069 | 38.133 | 1454.124 |
| Modal | 3 | 0.073 | 13.652 | 85.7782 | 7357.8989 |
| Modal | 4 | 0.073 | 13.652 | 85.7782 | 7357.8989 |
| Modal | 5 | 0.058 | 17.222 | 108.2112 | 11709.6541 |
| Modal | 6 | 0.058 | 17.222 | 108.2112 | 11709.6541 |
| Modal | 7 | 0.057 | 17.665 | 110.9925 | 12319.3297 |
| Modal | 8 | 0.057 | 17.665 | 110.9925 | 12319.3297 |
| Modal | 9 | 0.031 | 32.324 | 203.0949 | 41247.5509 |

| Case | Mode | Period sec | Frequency cyc/sec | Circular Frequency rad/sec | Eigenvalue rad²/sec² |
|---|---|---|---|---|---|
| Modal | 10 | 0.026 | 38.823 | 243.9311 | 59502.382 |
| Modal | 11 | 0.026 | 38.823 | 243.9311 | 59502.382 |
| Modal | 12 | 0.025 | 39.734 | 249.6573 | 62328.7548 |

**Modal Participating Mass Ratios (Part 1 of 2)**

| Case | Mode | Period sec | UX | UY | UZ | Sum UX | Sum UY | Sum UZ |
|---|---|---|---|---|---|---|---|---|
| Modal | 1 | 0.165 | 0 | 0.0812 | 0 | 0 | 0.0812 | 0 |
| Modal | 2 | 0.165 | 0 | 0.0543 | 0 | 0 | 0.1354 | 0 |
| Modal | 3 | 0.073 | 0.0003 | 0 | 0 | 0.0003 | 0.1354 | 0 |
| Modal | 4 | 0.073 | 0.1351 | 0 | 0 | 0.1354 | 0.1354 | 0 |
| Modal | 5 | 0.058 | 0 | 0.1359 | 0 | 0.1354 | 0.2713 | 0 |
| Modal | 6 | 0.058 | 0 | 0.1356 | 0 | 0.1354 | 0.4069 | 0 |
| Modal | 7 | 0.057 | 0 | 0.1911 | 0 | 0.1354 | 0.598 | 0 |
| Modal | 8 | 0.057 | 0 | 0.2079 | 0 | 0.1354 | 0.8059 | 0 |
| Modal | 9 | 0.031 | 0 | 0.0249 | 0 | 0.1354 | 0.8308 | 0 |
| Modal | 10 | 0.026 | 0 | 2.489E-05 | 0 | 0.1354 | 0.8308 | 0 |
| Modal | 11 | 0.026 | 0 | 0.0002 | 0 | 0.1354 | 0.831 | 0 |
| Modal | 12 | 0.025 | 0 | 0.0001 | 0 | 0.1354 | 0.8311 | 0 |

**Modal Participating Mass Ratios (Part 2 of 2)**

| Case | Mode | RX | RY | RZ | Sum RX | Sum RY | Sum RZ |
|---|---|---|---|---|---|---|---|
| Modal | 1 | 0.014 | 0 | 0.0054 | 0.014 | 0 | 0.0054 |
| Modal | 2 | 0.0093 | 0 | 0.008 | 0.0233 | 0 | 0.0134 |
| Modal | 3 | 0 | 0.0001 | 0.0002 | 0.0233 | 0.0001 | 0.0136 |
| Modal | 4 | 0 | 0.0232 | 0.0678 | 0.0233 | 0.0233 | 0.0814 |
| Modal | 5 | 0.0234 | 0 | 0.0826 | 0.0467 | 0.0233 | 0.1639 |
| Modal | 6 | 0.0233 | 0 | 0.0827 | 0.07 | 0.0233 | 0.2467 |
| Modal | 7 | 0.3523 | 0 | 0.1273 | 0.4223 | 0.0233 | 0.374 |
| Modal | 8 | 0.3832 | 0 | 0.117 | 0.8055 | 0.0233 | 0.491 |
| Modal | 9 | 0.0458 | 0 | 0 | 0.8513 | 0.0233 | 0.491 |

| Case | Mode | RX | RY | RZ | Sum RX | Sum RY | Sum RZ |
|---|---|---|---|---|---|---|---|
| Modal | 10 | 4.282E-06 | 0 | 0.0077 | 0.8513 | 0.0233 | 0.4987 |
| Modal | 11 | 2.891E-05 | 0 | 0.0011 | 0.8513 | 0.0233 | 0.4998 |
| Modal | 12 | 0.0002 | 0 | 4.579E-05 | 0.8516 | 0.0233 | 0.4999 |

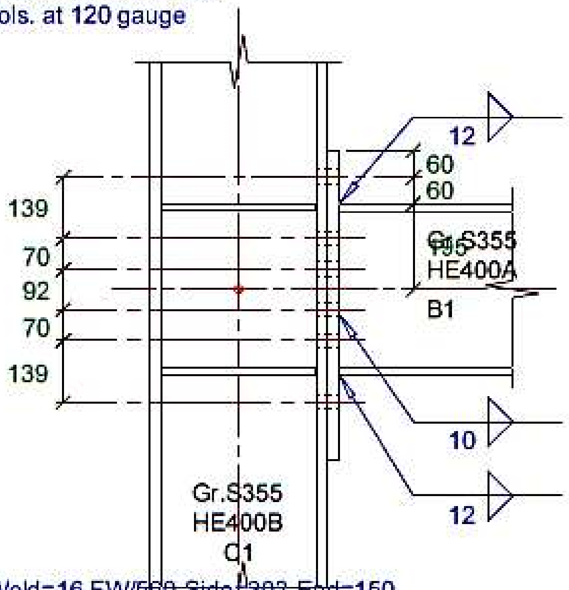
**Modal Load Participation Ratios**

| Case | Item Type | Item | Static % | Dynamic % |
|---|---|---|---|---|
| Modal | Acceleration | UX | 98.22 | 13.54 |
| Modal | Acceleration | UY | 99.91 | 83.11 |
| Modal | Acceleration | UZ | 0 | 0 |

**Modal Direction Factors**

| Case | Mode | Period sec | UX | UY | UZ | RZ |
|---|---|---|---|---|---|---|
| Modal | 1 | 0.165 | 0 | 1 | 0 | 0 |
| Modal | 2 | 0.165 | 0 | 1 | 0 | 0 |
| Modal | 3 | 0.073 | 1 | 0 | 0 | 0 |
| Modal | 4 | 0.073 | 1 | 0 | 0 | 0 |
| Modal | 5 | 0.058 | 0 | 0.782 | 0 | 0.218 |
| Modal | 6 | 0.058 | 0 | 0.781 | 0 | 0.219 |
| Modal | 7 | 0.057 | 0 | 0.742 | 0 | 0.258 |
| Modal | 8 | 0.057 | 0 | 0.773 | 0 | 0.227 |
| Modal | 9 | 0.031 | 0 | 1 | 0 | 0 |
| Modal | 10 | 0.026 | 0 | 0 | 0 | 1 |
| Modal | 11 | 0.026 | 0 | 0.002 | 0 | 0.998 |
| Modal | 12 | 0.025 | 0 | 0.025 | 0 | 0.975 |

CONNECTION: CONN4.GRID(1-B) - Bolted Moment End Plate
End plate: 313x28x700 Gr.S355
Flange weld: 12 FW/560 - Web weld: 10 FW/560
Bolts: 12 x M30 FH/B/N in 2 cols. at 120 gauge




Stiffeners:
Top: 140x16x352 Gr.S355 - Weld=16 FW/560 Side=302 End=150
Web doubler both sides: 302x25x305 Gr.S355 CJPGW all around
Btm.: 140x16x352 Gr.S355 - Weld=16 FW/560 Side=302 End=150

Plates: 109.1 kg
Bolts: 19.3 kg
Welds: 16.8 kg

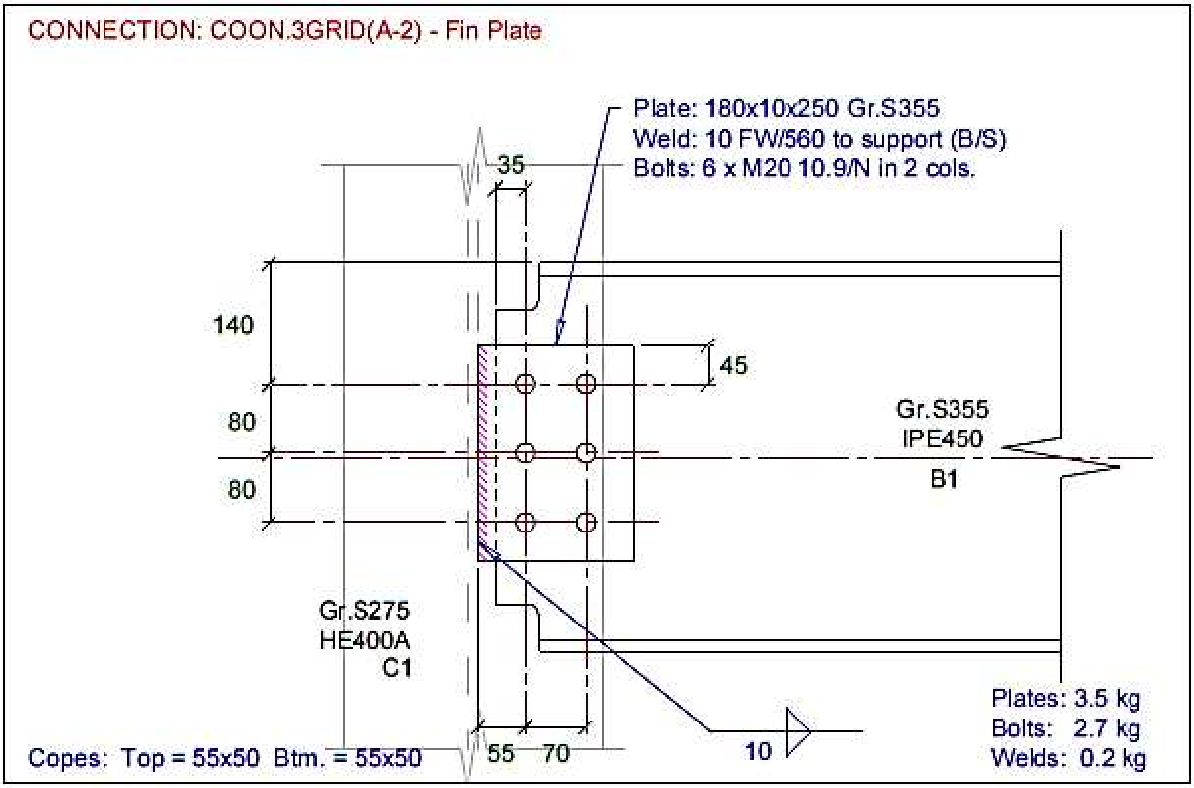
```
LIMCON V3.63.1.11 (0)                                                        10-DEC-15
                                                                              16:19:59
 Connection: CONN4.GRID(1-B)
       Type: Bolted Moment End Plate
             A - End plate square to beam
    Country: UK
      Units: SI metric
Design code: ANSI/AISC 360 (ASD)

Beam 1:  Mark=B1  Section=HE400A   Grade=S355   Angle= 0.00°
     d =     390 mm   Root rad. =     27 mm    Fyf =  355 N/mm2
     b =     300 mm        Area = 1.5900E+04   Fyw =  355 N/mm2
    tf =    19.0 mm          Sx = 2311000       Fu =  490 N/mm2
    tw =    11.0 mm          Zx = 2562000
 .Section moment strength  . . . .    544.6 kN.m                        Note 1
 .Section shear strength . . . . .    608.0 kN                          A360 (G2-1)
 .Section tension strength . . . .   3379.9 kN                          A360-(D2-1)
 .Section compression strength . .   3379.9 kN                          A360 E7

Column:  Mark=C1  Section=HE400B   Grade=S355
     d =     400 mm   Root rad. =     27 mm    Fyf =  355 N/mm2
     b =     300 mm        Area = 1.9800E+04   Fyw =  355 N/mm2
    tf =    24.0 mm          Sx = 2884000       Fu =  490 N/mm2
    tw =    13.5 mm          Zx = 3232000
 .Section moment strength  . . . .    687.0 kN.m                        Note 1
 .Section shear strength . . . . .    765.3 kN                          A360 (G2-1)
 .Section compression strength . .   4209.0 kN                          A360 E7

End plate:
 700x313x28  Gr./Fy/Fu=S355/35/490N/mm2
 Beam to end plate angle . . . . .    90.00°

End plate welds:
 12 FW/560N/mm2 flanges.
 10 FW/560N/mm2 web.

Bolts:
 6 x M30 FH/B/N top flange, 120 gauge.
 6 x M30 FH/B/N btm. flange, 120 gauge.

Stiffeners:  Gr./Fy/Fu=S355/35/490N/mm2   Welds fu=560N/mm2
```

```
 2/140 x16 top, 16 FW 302 at midpoint and across ends.
 2/140 x16 btm., 16 FW 302 at midpoint and across ends.

Web doubler plates:
 305x302x25  Gr./Fy/Fu=S355/35/490N/mm2
 No. web doubler plates  . . . . .       2
 Weld CJPGW/560N/mm2 all around.

BILL OF MATERIALS
Plates:
 1 no. - 700x313x28 Grade=S355 . . . . . . .   48.2 kg
 4 no. - 352x140x16 Grade=S355 . . . . . . .   24.8 kg
 2 no. - 305x302x25 Grade=S355 . . . . . . .   36.2 kg
 Total mass of plates  . . . . . . . . . . .  109.1 kg
Bolts:
 12 no. - M30 FH/B/N x 110 long  . . . . . .   19.3 kg
Welds:
 1200 mm - FW 12 fu=560N/mm2 . . . . . . . .    0.7 kg
 704 mm - FW 10 fu=560N/mm2  . . . . . . . .    0.3 kg
 4816 mm - FW 16 fu=560N/mm2 . . . . . . . .    4.8 kg
 1220 mm - CJPGW 25x25x85% fu=560N/mm2 . . .    5.1 kg
 1208 mm - CJPGW 25x25x100% fu=560N/mm2  . .    5.9 kg
 Total mass of welds . . . . . . . . . . . .   16.8 kg

MINIMUM ACTION CHECK
(Minima are based on section capacity, not member capacity.)
Specified minimum design actions:
 Bending        50% of Ms/Ω (  544.6)  =  272.3 kN.m
 Shear           0% of Vs/Ω (  609.2)  =    0.0 kN
                                          40.0 kN
 Tension         0% of Ns/Ω ( 3379.9)  =    0.0 kN
 Compression     0% of Nc/Ω ( 3379.9)  =    0.0 kN
 NOTE: Input design actions are not automatically increased if they are less than
       the specified minimum actions. Minimum actions may be set in any load case.
       This check warns if any design action is less than the specified minimum
       for all load cases.

INPUT DESIGN ACTIONS
 Beam 1:  Moment, M  . . . . . . .   386.6 kN.m
          Shear, V . . . . . . . .   235.0 kN
          Axial, P . . . . . . . .     0.0 kN
 Column:  Shear, Vc  . . . . . . .     0.0 kN
          Compression, Pc  . . . .     0.0 kN

SECTION ANALYSIS RESULT
Simplified analysis:
 Beam 1...              Pft   =  1042.1t   Pfc  =  1042.1c
                        Pwt   =     0.0    Pwc  =     0.0
                        Mw    =    0.00
                        Vw    =   235.0
Elastic analysis:
 Beam 1...              Pft   =   949.7t   Pfc  =   949.7c
                        Pwt   =     0.0    Pwc  =     0.0
                        Mw    =    34.3
                        Vw    =   235.0
Plastic analysis:
 Beam 1...              Pft   =   907.8t   Pfc  =   907.8c
                        Pwt   =     0.0    Pwc  =     0.0
                        Mw    =    49.8
                        Vw    =   235.0

NOTE: Simplified analysis results used.

BOLT ARRANGEMENT
 6-Bolt 2/4 Extended End Plate
 Connection checked for tension at top flange.

Using AISC/ASI model...

 Ref. 18: Steel Design Guide 16 (SDG16)
          Flush and Extended Multiple Row Moment End Plate Connections
          T.M. Murray & W.L. Shoemaker - AISC - 2002
 Ref. 19: Steel Design Guide 4 (SDG4)
          Extended End Plate Moment Connections - Seismic and Wind Applications
          T.M. Murray & E.A. Sumner - AISC - 2004
 Ref. 43: Design Guide 12 - Bolted End Plate to Column Moment Connections (DG12)
          T.J. Hogan & N. van der Kreek - ASI - 2009

GEOMETRY CHECKS
CHECK 1 - Detailing Requirements:
 Bolt UTS  . . . . . . . . . . . .     1000          ≥     800                Yes
```

```
 End plate width, bi . . . . . . .      313          ≥     313             Yes
                                        313          ≤     325             Yes
  NOTE: Clearances should be checked in virtual reality view.

ASD STRENGTH CHECKS
                                                  Strength ratio
                                                  Required strength
                                                  ASD strength                            Reference

Section Bending/Axial:
 Flange tension yield strength . . . . . . .  1211.7  ≥  Pft    =  907.8  1.33  Pass  Manual p.55
 Flange tension rupture strength . . . . . .  1396.5  ≥  Pft    =  907.8  1.54  Pass
 Flange compression strength . . . . . . . .  1211.7  ≥  Pft    =  907.8  1.33  Pass

CHECK 2 - Flange Welds:
 Flange width  . . . . . . . . .     300 mm
 Total fillet weld length  . . . .   589 mm                                         SDG4 p.38
 For 12 FW/560N/mm2 both sides...
  NOTE: Using 50% directional strength increase.                                     A360 (J2-5)
  Fillet weld strength . . . . . .  2.138 kN/mm
 Flange fillet weld tension strength . . . .  1259.5  ≥  Pft    = 1042.1  1.21  Pass
 Flange fillet weld compression strength . .  1259.5  ≥  Pfc    = 1042.1  1.21  Pass

CHECK 3 - Web Welds:
 Web shear force . . . . . . . . .  235.0 kN
 Web axial force . . . . . . . . .    0.0 kN
 Web bending moment  . . . . . . .   0.00 kN.m
 Length of web weld  . . . . . . .    352 mm
  NOTE: This check uses method from AISC SDG 16 (Ref. 18).
 Length for shear resistance . . .    176 mm
 For 10 FW/560N/mm2 both sides...
 Web fillet weld shear strength (2 sides)  .   418.2  ≥  V      =  235.0  1.78  Pass
 End plate design moment, Meq  . .  386.6 kN.m
 Section moment strength . . . . .  544.6 kN.m
 Beam moment utilization ratio . .    71%
 Beam web axial strength . . . . .  2.338 kN/mm
 » Web weld design force . . . . .  1.660 kN/mm
  NOTE: Using 50% directional strength increase.                                     A360 (J2-5)
  Fillet weld strength . . . . . .  1.782 kN/mm
 Web fillet weld axial strength (2 sides)  .   3.564  ≥  pw     =  1.660  2.15  Pass

CHECK 4 - Bolts at Tension Flange:
 Single bolt tension strength  . .  275.7 kN                                        A360-T:J3.2
 No. bolts effective at flange . .      6
 Sum of bolt lever arms  . . . . .    974 mm
 End plate design moment, Meq  . .  386.6 kN.m
 Tension bolt moment strength, Mnp/Ω . . . .   536.8  ≥  Meq    =  386.6  1.39  Pass

CHECK 5 - Bolts in Shear:
 Total shear resisted by bolts . .  235.0 kN
 Single bolt shear strength  . . .  161.5 kN                                        A360-T:J3.2
 No. bolts effective in shear  . .      6
 Bolt shear strength, Rn/Ω . . . . . . . . .   969.2  ≥  Vb     =  235.0  4.12  Pass  SDG4 (3.17)
 End plate bolt bearing strength, Rn/Ω . . .  2963.5  ≥  Vb     =  235.0  12.6  Pass  SDG4 (3.18)

CHECK 6 - End Plate in Bending:
 End plate design moment, Meq  . .  386.6 kN.m
 Yield line parameter, Yp  . . . .   3298 mm
 End plate yield strength, Mpl/Ω . . . . . .   549.6  ≥  Meq    =  386.6  1.42  Pass
 Bolt moment strength, Mbt/Ω . . .  536.8 kN.m
 » No prying strength, Mmin  . . .  595.8 kN.m
 » End plate no prying thickness .     29 mm                                        SDG4 (3.10)
 Is plate strong enough for no prying? . . .   826.0  ≥   893.7                No
 » Bolt prying does occur.
 Single bolt tension strength  . .  275.7 kN                                        A360-T:J3.2
 Specified minimum tension . . . .  370.0 kN
 Bolt max. prying force, Qmax,i  .   99.4 kN                                        SDG16 p.31
 Bolt max. prying force, Qmax,o  .  122.4 kN                                        SDG16 p.31
 Bolt rupture strength (prying)  . . . . . .   410.9  ≥  Meq    =  386.6  1.06  Pass
 Prying factor . . . . . . . . . .   0.42                                           Informative
 Bolt efficiency . . . . . . . . .    77%                                           Informative

CHECK 7 - End Plate in Shear:
 Horiz. shear  . . . . . . . . . .  347.4 kN
 Horiz. shear yield strength, Rn/Ω . . . . .  1244.5  ≥  Vh     =  347.4  3.58  Pass  SDG4 (3.12)
 Horiz. shear rupture strength, Rn/Ω . . . .  1000.2  ≥  Vh     =  347.4  2.88  Pass  SDG4 (3.13)

CHECK 8 - End Plate Stiffener:
 No stiffener.

CHECK 9 - End Plate Stiffener Welds:
```

```
 No stiffener.

COLUMN-SIDE CHECKS...
 Ref. 19: Steel Design Guide 4 (SDG4)
          Extended End Plate Moment Connections - Seismic and Wind Applications
          T.M. Murray & E.A. Sumner - AISC - 2004

CHECK 10/16 - Unstiffened Column Flange Bending at Beam Tension Flange:
  NOTE: This capacity is required for checking stiffeners.
 Using method from AISC SDG 4 (Ref.19)...
 End plate design moment, Meq  . .   386.6 kN.m
 Yield line parameter, Yc  . . . .    3557 mm
 Col. flange no prying thickness .      28 mm                                    SDG4 (3.20)
 Column flange strength  . . . . .   435.6 kN.m
 Equivalent flange force, Rn/Ω . .  1174.0 kN                                    SDG4 (3.22)
 End plate design moment, Meq  . .   386.6 kN.m
 No prying strength, Mmin  . . . .   594.6 kN.m
 Unstiffened col. flange strength, Mcf/Ω . .   435.6  ≥  Meq   =  386.6  1.13  Pass  Informative
 » Tension flange stiffeners may not be required.

CHECK 11 - Unstiffened Column Web Yielding at Beam Tension Flange:
  NOTE: This capacity is required for checking stiffeners.
 Unstiffened col. web yield strength, Rn/Ω .  1294.0  ≥  Pft   = 1042.1  1.24  Pass  Informative
 » Tension flange stiffeners may not be required.

CHECK 12 - Unstiffened Column Web Yielding at Beam Compression Flange:
  NOTE: This capacity is required for checking stiffeners.
 Unstiffened col. web yield strength, Rn/Ω .  1294.0  ≥  Pfc   = 1042.1  1.24  Pass  Informative
 » Compression flange stiffeners may not be required.

CHECK 13 - Unstiffened Column Web Crippling at Beam Compression Flange:
  NOTE: This check not required with compression flange stiffeners.
 Column web crippling strength, Rn/Ω . . . .  1075.4  ≥  Pfc   = 1042.1  1.03  Pass  Informative
 » Compression flange stiffeners may not be required.

CHECK 14 - Unstiffened Column Web Buckling at Beam Compression Flange:
  NOTE: This check not required with compression flange stiffeners.
 Column web buckling strength, Rn/Ω  . . . .   999.8  ≥  Pfc   = 1042.1  0.96  Fail  Informative

CHECK 15/21 - Unstiffened Column Web Panel in Shear:
 Web doubler thickness . . . . . .      25           ≥      14               Yes
 Total web doubler thickness . . .      50 mm
 Column nominal axial capacity, Py   14129 kN                                    Note 4
 Ratio Pu/Py . . . . . . . . . . .    0.00
 Column web panel shear  . . . . .  1042.1 kN
 Column web panel shear strength, Rv/Ω . . .  3239.6  ≥  Vp    = 1042.1  3.11  Pass  SDG13 (2.2-1)

CHECK 22 - Transverse Stiffeners at Beam Tension Flange:
 Stiffener width . . . . . . . . .     140           ≥     143               Yes
                                       140           ≤     143               Yes
 Stiffener effective width . . . .     125 mm
 Stiffener thickness . . . . . . .    16.0           ≥     9.5               Yes
 Column flange strength, Rft/Ω . .  1174.0 kN
 Column web yield strength, Rwt/Ω   1291.4 kN
 » Unstiffened column strength . .  1174.0 kN
 Flange tension  . . . . . . . . .  1042.1 kN
 » Check of stiffeners not required.

CHECK 22A - Stiffened Column Flange Bending at Beam Tension Flange:
 Using method from AISC SDG 4 (Ref.19)...
 Yield line parameter, Ycs . . . .    5113 mm
 Col. flange no prying thickness .      23 mm                                    SDG4 (3.20)
 Column flange strength  . . . . .   626.1 kN.m
 End plate design moment, Meq  . .   386.6 kN.m
 Stiffened col. flange strength, Mcts/Ω  . .   626.1  ≥  Meq   -  386.6  1.62  Pass
 No prying strength, Mmin  . . . .   594.6 kN.m
 Is col. flange strong enough for no prying?   941.0  ≥   893.7               Yes
 » Bolt prying may not occur.

CHECK 23 - Transverse Stiffeners at Beam Compression Flange:
 Stiffener width . . . . . . . . .     140           ≥     143               Yes
                                       140           ≤     143               Yes
 Stiffener thickness . . . . . . .    16.0           ≥     9.5               Yes
 Stiffener side weld . . . . . . .     302           ≥     302               Yes
 Column web yield strength . . . .  1291.4 kN
 Column web crippling strength . .  1073.3 kN
 Column web buckling strength  . .   999.8 kN
 » Unstiffened column strength . .   999.8 kN
 Flange compression  . . . . . . .  1042.1 kN
 » Stiffener design compression  .    42.3 kN
 Stiffener section yield...
```

```
  Stiffener yield strength, Rfcs/Ω . . . . .   850.3  ≥  Pcs   =   42.3  20.1  Pass
 Stiffener effective width . . . .      125 mm
 Stiffener section strong axis buckling...
  Stiffener buckling strength, Rfcb/Ω  . . .   895.5  ≥  Pcs   =   42.3  21.2  Pass
 Side welds...
  Total side weld length . . . . .     1208 mm
  Stiffener side weld strength, Rftw/Ω . . .  2296.0  ≥  Pcs   =   42.3  54.3  Pass

CHECK 24 - Diagonal Shear Stiffeners:
 No diagonal shear stiffeners.

NOTES:
  1. Flexural yield capacity ignoring section slenderness.
  4. Compression yield capacity of section ignoring slenderness.

CRITICAL LIMIT STATE . . .  Bolt rupture strength (prying)
UTILIZATION RATIO  . . . .     94%
STRENGTH RATIO . . . . . .    1.063   Pass
```

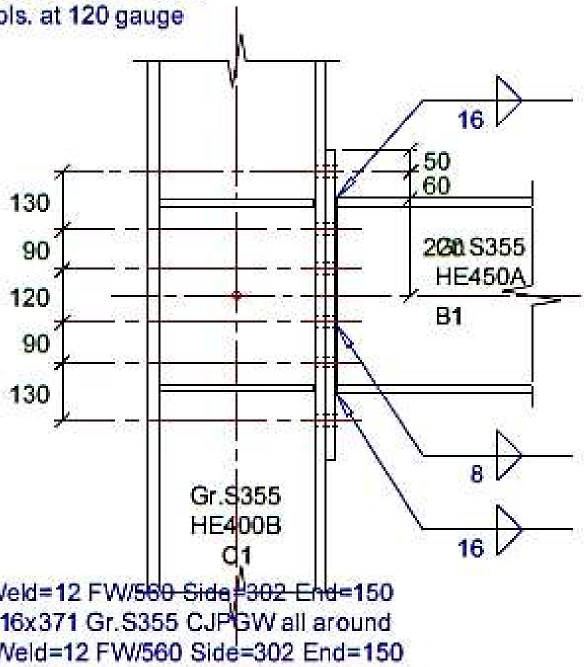



```
Geometry Check:
*** WARNING -- Cleat top edge distance > 2 x dia.
1 warning.

LIMCON V3.63.1.11 (0)                                                10-DEC-15
                                                                       16:16:34
 Connection: COON.3GRID(A-2)
       Type: Fin Plate
    Country: UK
      Units: SI metric
Design code: ANSI/AISC 360 (ASD)

Beam:  Mark=B1  Section=IPE450   Grade=S355   Span=9.0 m
     d =    450 mm   Root rad. =     21 mm    Fyf =  355 N/mm2
     b =    190 mm        Area =   9880       Fyw =  355 N/mm2
    tf =   14.6 mm          Sx = 1500000       Fu =  490 N/mm2
    tw =    9.4 mm          Zx = 1702000
 .Section shear strength . . . . .   599.5 kN                  A360 (G2-1)
 .Section tension strength . . . .  2100.2 kN                  A360-(D2-1)
 .Section compression strength . .  2027.8 kN                  A360 E7
 Top cope depth  . . . . . . . . .      55 mm
 Top cope length . . . . . . . . .      50 mm
 Bottom cope depth . . . . . . . .      55 mm
 Bottom cope length  . . . . . . .      50 mm

Plate:
 250x180x10  Gr./Fy/Fu=S355/35/490N/mm2

Bolts:
 6 x M20 10.9/N in 2 columns.
 Bolt pitch, sp  . . . . . . . . .      80 mm
 Support to 1st bolt column, sg1 .      55 mm
 Bolt gauge, sg2 . . . . . . . . .      70 mm

Weld:
 10 FW/560N/mm2 to support.

Support:  Mark=C1  Section=HE400A   Grade=S275
     d =    390 mm   Root rad. =     27 mm    Fyf =  275 N/mm2
     b =    300 mm        Area = 1.5900E+04   Fyw =  275 N/mm2
    tf =   19.0 mm          Sx = 2311000       Fu =  430 N/mm2
```

```
    tw =   11.0 mm          Zx = 2562000
  .Section compression strength . .  2618.3 kN                                        A360 E7
  Connection to column web.
  Unspecified support condition.

 BILL OF MATERIALS
 Plates:
  1 no. - 250x180x10 Grade=S355 . . . . . . .    3.5 kg
 Bolts:
  6 no. - M20 10.9/N x 60 long  . . . . . . .    2.7 kg
 Welds:
  500 mm - FW 10 fu=560N/mm2  . . . . . . . .    0.2 kg

 MINIMUM SHEAR FORCE CHECK
  Input design actions are not automatically increased if they are less than the specified
  minimum actions. Minimum actions may be set in any load case. This check warns if the
  shear force is less than the specified minimum (40 kN) for all load cases.
  * Shear force exceeds specified minimum in at least one load case.

 INPUT DESIGN ACTIONS
  Shear, V  . . . . . . . . . . . .    87.3 kN
  Axial, P  . . . . . . . . . . . .     0.0 kN

 Bolt group eccentricity  . . . . .    90.0 mm
 Max. eccentricity moment . . . . .    7.86 kN.m

 BEAM TABLE SHEAR
  For uniform load on simply supported beam:
   Section moment strength  . . . .   361.8 kN.m                                      Note 1
   Span . . . . . . . . . . . . . .  9.0 m
   Max. total uniform load  . . . .   321.6 kN
   Shear  . . . . . . . . . . . . .   160.8 kN

 Using AISC SCM 14th model...

 GEOMETRY CHECKS
 CHECK 1 - Detailing Requirements:
  Ref. 6: Steel Construction Manual - 14th Edition - AISC - 2011 (SCM14)
  == This is an EXTENDED CONFIGURATION connection. ==
  Plate fillet weld leg . . . . . .     10.0          ≥     6.2              Yes    SCM14 p.10-102
  Plate length  . . . . . . . . . .      250          ≥     189              Yes    SCM14 p.10-106
  Plate thickness . . . . . . . . .     10.0          ≥     6.3              Yes    Informative
  C' (pure moment)  . . . . . . . .     402 mm                                      SCM14 p.7-19
  Mmax  . . . . . . . . . . . . . .    64.2 kN.m                                    SCM14 p.10-104
  Max. ductile plate thickness  . .    17.4 mm                                      SCM14 p.10-104
  Plate thickness . . . . . . . . .     10.0          ≤    17.4              Yes

 ASD STRENGTH CHECKS
                                                  Strength ratio
                                                  Required strength
                                                  ASD strength                       Reference

 CHECK 2 - Weld:
  Weld length (each side) . . . . .     250 mm
  Weld design moment  . . . . . . .    7.86 kN.m
  Weld strength (elastic) . . . . . . . . . .   249.5  ≥  V    =   87.3  2.86  Pass  Informative
   NOTE: Weld leg not less than 5/8 of plate thickness » check not required.

 CHECK 3 - Bolts:
  Bolt group: 6 x M20 10.9/N
  Bolt group design eccentricity  .    90.0 mm
  Bolt group design moment  . . . .    7.86 kN.m
  Single bolt shear strength  . . .    71.8 kN                                      A360-T:J3.2
  Zb  . . . . . . . . . . . . . . .   2.929
  Zev . . . . . . . . . . . . . . .   0.635
  Zeh . . . . . . . . . . . . . . .   0.763
  Bolt group shear strength . . . . . . . . .   210.3  ≥  V    =   87.3  2.41  Pass
  Plate bearing strength  . . . . . . . . . .   344.5  ≥  V    =   87.3  3.95  Pass  A360 (J3-6a)
  Web bearing strength  . . . . . . . . . . .   323.8  ≥  V    =   87.3  3.71  Pass  A360 (J3-6a)
  Plate vert. tearing (top) . . . . . . . . .   381.1  ≥  V    =   87.3  4.37  Pass  A360 (J3-6a)
  Web vert. tearing (int.)  . . . . . . . . .   611.2  ≥  V    =   87.3  7.00  Pass  A360 (J3-6a)
  Web vert. tearing (top) . . . . . . . . . .   779.8  ≥  V    =   87.3  8.93  Pass  A360 (J3-6a)

 CHECK 4 - Plate:
  Plate shear yield strength  . . . . . . . .   355.0  ≥  V    =   87.3  4.07  Pass  A360 (J4-3)
  Plate shear rupture strength  . . . . . . .   261.7  ≥  V    =   87.3  3.00  Pass  A360 (J4-4)
  Flexural yield strength . . . . . . . . . .   369.1  ≥  V    =   87.3  4.23  Pass  A360 (F11-1)
  Flexural rupture strength . . . . . . . . .   316.9  ≥  V    =   87.3  3.63  Pass  SCM13 II.A-19
  Flexural strength reduced for shear . . . .   364.8  ≥  V    =   87.3  4.18  Pass  SCM14 p.10-104
  Block shear strength  . . . . . . . . . . .   322.2  ≥  V    =   87.3  3.69  Pass  A360-(J4-5)
```

```
CHECK 5 - Beam Shear:
 Double-coped web shear yield strength . . .   453.8  ≥  V     =   87.3  5.20  Pass  A360 (J4-3)
 Double-coped web shear rupture strength . .   370.3  ≥  V     =   87.3  4.24  Pass  A360 (J4-4)

CHECK 6 - Beam Web Block Shear:
 Coped web block shear strength  . . . . . .   324.7  ≥  V     =   87.3  3.72  Pass  A360-(J4-5)

CHECK 7 - Coped Beam Bending:
 Coped section moment strength . . . . . . .    57.7 kN.m
 Coped web moment strength . . . . . . . . .   825.0  ≥  V     =   87.3  9.45  Pass
  Estimated max. cope length . . .      450 mm

CHECK 8 - Beam Rotation:
 Rotation for UDL (rad.) . . . . .    0.009
 Rotation for contact (rad.) . . .    0.087
 Contact rotation strength . . . . . . . . .   869.3  ≥  V     =   87.3  9.96  Pass
 Max. bolt shear deformation . . .      0.8 mm
 Max. recommended deformation  . .      4.8 mm
 Strength at bolt deformation limit  . . . .   545.2  ≥  V     =   87.3  6.25  Pass  Informative

CHECK 9 - Coped Beam Buckling:
  NOTE: Coped beam must be restrained against lateral torsional buckling.
 Buckling check to AISC SCM 13th p.9-7 (Cheng, Yura, Johnston)...
  Critical stress, Fcr . . . . . .  5229 N/mm2
  Critical stress, Fcr ≤ Fy  . . .   355 N/mm2
  Coped web elastic modulus  . . .  181106 mm3
  Double-coped web buckling strength . . . .   550.0  ≥  V     =   87.3  6.30  Pass
  Estimated max. cope length . . .      450 mm

CHECK 10 - Support Member:
 Depth of shear surface  . . . . .      250 mm
 Column web local shear strength . . . . . .   605.0  ≥  V     =   87.3  6.93  Pass  P212 p.167
 Max. plate thickness (punching) .     13.3 mm
 Plate thickness . . . . . . . . .      10.0         ≤    13.3              Yes
 Informative checks for double-sided connection...
  Local shear strength for beam both sides .   605.0  ≥  2V    -  174.6  3.47  Pass

NOTES:
  1. Flexural yield capacity ignoring section slenderness.

CRITICAL LIMIT STATE . . .  Bolt group shear strength
UTILIZATION RATIO  . . . .     42%
STRENGTH RATIO . . . . . .   2.409    Pass
```

CONNECTION: conn2.grid(3-B) - Bolted Moment End Plate
End plate: 313x25x700 Gr.S355
Flange weld: 16 FW/560 - Web weld: 8 FW/560
Bolts: 12 x M24 FH/B/N in 2 cols. at 120 gauge




Stiffeners:
Top: 140x16x352 Gr.S355 - Weld=12 FW/560 Side=302 End=150
Web doubler both sides: 316x16x371 Gr.S355 CJPGW all around
Btm.: 140x16x352 Gr.S355 - Weld=12 FW/560 Side=302 End=150

Plates: 97.2 kg
Bolts: 10.0 kg
Welds: 9.2 kg

```
LIMCON V3.63.1.11 (0)                                                  10-DEC-15
                                                                        16:19:15
 Connection: conn2.grid(3-B)
       Type: Bolted Moment End Plate
             A - End plate square to beam
    Country: UK
      Units: SI metric
Design code: ANSI/AISC 360 (ASD)

Beam 1:  Mark=B1  Section=HE450A   Grade=S355   Angle= 0.00°
     d =    440 mm   Root rad. =    27 mm    Fyf =  355 N/mm2
     b =    300 mm        Area = 1.7800E+04  Fyw =  355 N/mm2
    tf =   21.0 mm          Sx = 2896000      Fu =  490 N/mm2
    tw =   11.5 mm          Zx = 3216000
 .Section moment strength  . . . .   683.6 kN.m                   Note 1
 .Section shear strength . . . . .   717.1 kN                     A360 (G2-1)
 .Section tension strength . . . .  3783.8 kN                     A360-(D2-1)
 .Section compression strength . .  3783.8 kN                     A360 E7

Column:  Mark=C1  Section=HE400B   Grade=S355
     d =    400 mm   Root rad. =    27 mm    Fyf =  355 N/mm2
     b =    300 mm        Area = 1.9800E+04  Fyw =  355 N/mm2
    tf =   24.0 mm          Sx = 2884000      Fu =  490 N/mm2
    tw =   13.5 mm          Zx = 3232000
 .Section moment strength  . . . .   687.0 kN.m                   Note 1
 .Section shear strength . . . . .   765.3 kN                     A360 (G2-1)
 .Section compression strength . .  4209.0 kN                     A360 E7

End plate:
 700x313x25  Gr./Fy/Fu=S355/35/490N/mm2
 Beam to end plate angle . . . . .   90.00°

End plate welds:
 16 FW/560N/mm2 flanges.
 8 FW/560N/mm2 web.

Bolts:
 6 x M24 FH/B/N top flange, 120 gauge.
 6 x M24 FH/B/N btm. flange, 120 gauge.

Stiffeners:  Gr./Fy/Fu=S355/35/490N/mm2   Welds fu=560N/mm2
```

```
 2/140 x16 top, 12 FW 302 at midpoint and across ends.
 2/140 x16 btm., 12 FW 302 at midpoint and across ends.

Web doubler plates:
 371x316x16  Gr./Fy/Fu=S355/35/490N/mm2
 No. web doubler plates  . . . . .       2
 Weld CJPGW/560N/mm2 all around.

BILL OF MATERIALS
Plates:
 1 no. - 700x313x25 Grade=S355 . . . . . . .   43.0 kg
 4 no. - 352x140x16 Grade=S355 . . . . . . .   24.8 kg
 2 no. - 371x316x16 Grade=S355 . . . . . . .   29.4 kg
 Total mass of plates  . . . . . . . . . . .   97.2 kg
Bolts:
 12 no. - M24 FH/B/N x 90 long . . . . . . .   10.0 kg
Welds:
 1200 mm - FW 16 fu=560N/mm2 . . . . . . . .    1.2 kg
 796 mm - FW 8 fu=560N/mm2 . . . . . . . . .    0.2 kg
 4816 mm - FW 12 fu=560N/mm2 . . . . . . . .    2.7 kg
 1484 mm - CJPGW 16x16x85% fu=560N/mm2 . . .    2.5 kg
 1264 mm - CJPGW 16x16x100% fu=560N/mm2  . .    2.5 kg
 Total mass of welds . . . . . . . . . . . .    9.2 kg

MINIMUM ACTION CHECK
(Minima are based on section capacity, not member capacity.)
Specified minimum design actions:
 Bending       50% of Ms/Ω (  683.6)  =  341.8 kN.m
 Shear          0% of Vs/Ω (  718.5)  =    0.0 kN
                                          40.0 kN
 Tension        0% of Ns/Ω ( 3783.8)  =    0.0 kN
 Compression    0% of Nc/Ω ( 3783.8)  =    0.0 kN
 NOTE: Input design actions are not automatically increased if they are less than
       the specified minimum actions. Minimum actions may be set in any load case.
       This check warns if any design action is less than the specified minimum
       for all load cases.

INPUT DESIGN ACTIONS
 Beam 1:  Moment, M  . . . . . . .   387.5 kN.m
          Shear, V . . . . . . . .   229.4 kN
          Axial, P . . . . . . . .     0.0 kN
 Column:  Shear, Vc  . . . . . . .     0.0 kN
          Compression, Pc  . . . .     0.0 kN

SECTION ANALYSIS RESULT
Simplified analysis:
 Beam 1...              Pft  =   924.8t   Pfc  =   924.8c
                        Pwt  =     0.0    Pwc  =     0.0
                        Mw   =    0.00
                        Vw   =   229.4
Elastic analysis:
 Beam 1...              Pft  =   837.1t   Pfc  =   837.1c
                        Pwt  =     0.0    Pwc  =     0.0
                        Mw   =    36.7
                        Vw   =   229.4
Plastic analysis:
 Beam 1...              Pft  =   803.3t   Pfc  =   803.3c
                        Pwt  =     0.0    Pwc  =     0.0
                        Mw   =    50.9
                        Vw   =   229.4

NOTE: Simplified analysis results used.

BOLT ARRANGEMENT
 6-Bolt 2/4 Extended End Plate
 Connection checked for tension at top flange.

Using AISC/ASI model...

 Ref. 18: Steel Design Guide 16 (SDG16)
          Flush and Extended Multiple Row Moment End Plate Connections
          T.M. Murray & W.L. Shoemaker - AISC - 2002
 Ref. 19: Steel Design Guide 4 (SDG4)
          Extended End Plate Moment Connections - Seismic and Wind Applications
          T.M. Murray & E.A. Sumner - AISC - 2004
 Ref. 43: Design Guide 12 - Bolted End Plate to Column Moment Connections (DG12)
          T.J. Hogan & N. van der Kreek - ASI - 2009

GEOMETRY CHECKS
CHECK 1 - Detailing Requirements:
 Bolt UTS  . . . . . . . . . . . .     1000          ≥     800                Yes
```

```
CHECK 10/16 - Unstiffened Column Flange Bending at Beam Tension Flange:
  NOTE: This capacity is required for checking stiffeners.
 Using method from AISC SDG 4 (Ref.19)...
 End plate design moment, Meq  . .   387.5 kN.m
 Yield line parameter, Yc  . . . .    4060 mm
 Col. flange no prying thickness .      23 mm                                SDG4 (3.20)
 Column flange strength  . . . . .   497.1 kN.m
 Equivalent flange force, Rn/Ω . .  1186.4 kN                                SDG4 (3.22)
 End plate design moment, Meq  . .   387.5 kN.m
 No prying strength, Mmin  . . . .   437.2 kN.m
 Unstiffened col. flange strength, Mcf/Ω . .   497.1  ≥  Meq   =  387.5  1.28  Pass  Informative
 » Tension flange stiffeners may not be required.

CHECK 11 - Unstiffened Column Web Yielding at Beam Tension Flange:
  NOTE: This capacity is required for checking stiffeners.
 Unstiffened col. web yield strength, Rn/Ω .  1306.8  ≥  Pft   =  924.8  1.41  Pass  Informative
 » Tension flange stiffeners may not be required.

CHECK 12 - Unstiffened Column Web Yielding at Beam Compression Flange:
  NOTE: This capacity is required for checking stiffeners.
 Unstiffened col. web yield strength, Rn/Ω .  1306.8  ≥  Pfc   =  924.8  1.41  Pass  Informative
 » Compression flange stiffeners may not be required.

CHECK 13 - Unstiffened Column Web Crippling at Beam Compression Flange:
  NOTE: This check not required with compression flange stiffeners.
 Column web crippling strength, Rn/Ω . . . .  1085.8  ≥  Pfc   =  924.8  1.17  Pass  Informative
 » Compression flange stiffeners may not be required.

CHECK 14 - Unstiffened Column Web Buckling at Beam Compression Flange:
  NOTE: This check not required with compression flange stiffeners.
 Column web buckling strength, Rn/Ω  . . . .   999.8  ≥  Pfc   =  924.8  1.08  Pass  Informative
 » Compression flange stiffeners may not be required.

CHECK 15/21 - Unstiffened Column Web Panel in Shear:
 Web doubler thickness . . . . . .      16           ≥      14             Yes
 Total web doubler thickness . . .      32 mm
 Column nominal axial capacity, Py   11573 kN                                Note 4
 Ratio Pu/Py . . . . . . . . . . .    0.00
 Column web panel shear  . . . . .   924.8 kN
 Column web panel shear strength, Rv/Ω . . .  2321.3  ≥  Vp    =  924.8  2.51  Pass  SDG13 (2.2-1)

CHECK 22 - Transverse Stiffeners at Beam Tension Flange:
 Stiffener width . . . . . . . . .     140           ≥     143             Yes
                                       140           ≤     143             Yes
 Stiffener effective width . . . .     125 mm
 Stiffener thickness . . . . . . .    16.0           ≥    10.5             Yes
 Column flange strength, Rft/Ω . .  1186.4 kN
 Column web yield strength, Rwt/Ω   1304.1 kN
 » Unstiffened column strength . .  1186.4 kN
 Flange tension  . . . . . . . . .   924.8 kN
 » Check of stiffeners not required.

CHECK 22A - Stiffened Column Flange Bending at Beam Tension Flange:
 Using method from AISC SDG 4 (Ref.19)...
 Yield line parameter, Ycs . . . .    6073 mm
 Col. flange no prying thickness .      18 mm                                SDG4 (3.20)
 Column flange strength  . . . . .   743.6 kN.m
 End plate design moment, Meq  . .   387.5 kN.m
 Stiffened col. flange strength, Mcts/Ω  . .   743.6  ≥  Meq   =  387.5  1.92  Pass
 No prying strength, Mmin  . . . .   437.2 kN.m
 Is col. flange strong enough for no prying?  1117.6  ≥   657.2            Yes
 » Bolt prying may not occur.

CHECK 23 - Transverse Stiffeners at Beam Compression Flange:
 Stiffener width . . . . . . . . .     140           ≥     143             Yes
                                       140           ≤     143             Yes
 Stiffener thickness . . . . . . .    16.0           ≥    10.5             Yes
 Stiffener side weld . . . . . . .     302           ≥     302             Yes
 Column web yield strength . . . .  1304.1 kN
 Column web crippling strength . .  1083.6 kN
 Column web buckling strength  . .   999.8 kN
 » Unstiffened column strength . .   999.8 kN
 Flange compression  . . . . . . .   924.8 kN
 » Check of stiffeners not required.

CHECK 24 - Diagonal Shear Stiffeners:
 No diagonal shear stiffeners.

NOTES:
  1. Flexural yield capacity ignoring section slenderness.
```

```
  4. Compression yield capacity of section ignoring slenderness.

CRITICAL LIMIT STATE . . .  Bolt rupture strength (no prying)
UTILIZATION RATIO  . . . .     98%
STRENGTH RATIO . . . . . .   1.019   Pass
```

# References

**Lecture Notes:**

**Design of Steel Structures,** Dr. Andi Asiz, spring 2015

**Reinforced Concrete Design,** Eng. Danish Ahmed, spring 2015

**Intro. To Geotechnical Engineering,** Dr. Alaa salman, fall 2015

**Books/Articles:**


Wang, C., & Salmon, C. (1985). *Reinforced concrete design* (4th ed.). New York: Harper & Row.

*Building code requirements for structural concrete (ACI 318-08) and commentary*. (2008). Farmington Hills, Mich.: American Concrete Institute.

كود البناء السعودي. (n.d.). Retrieved December 26, 2015, from http://www.sbc.gov.sa/

*Steel construction manual* (13th ed.). (2005). Chicago, Ill.: American Institute of Steel Construction.

*Steel construction, a manual for architects, engineers and fabricators of buildings and other steel structures.* (5th ed.). (1947). New York, N.Y.: American Institute of Steel Construction.