Date: 05-02-2024

DESIGN OF STEEL MEMBERS

FOR

PROPOSED EXTENSION AT MAIN STREET, ARVAGH, CO. CAVAN

FOR

JOHN & LORNA GUILFOYLE

Submitted By

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Revision: R1

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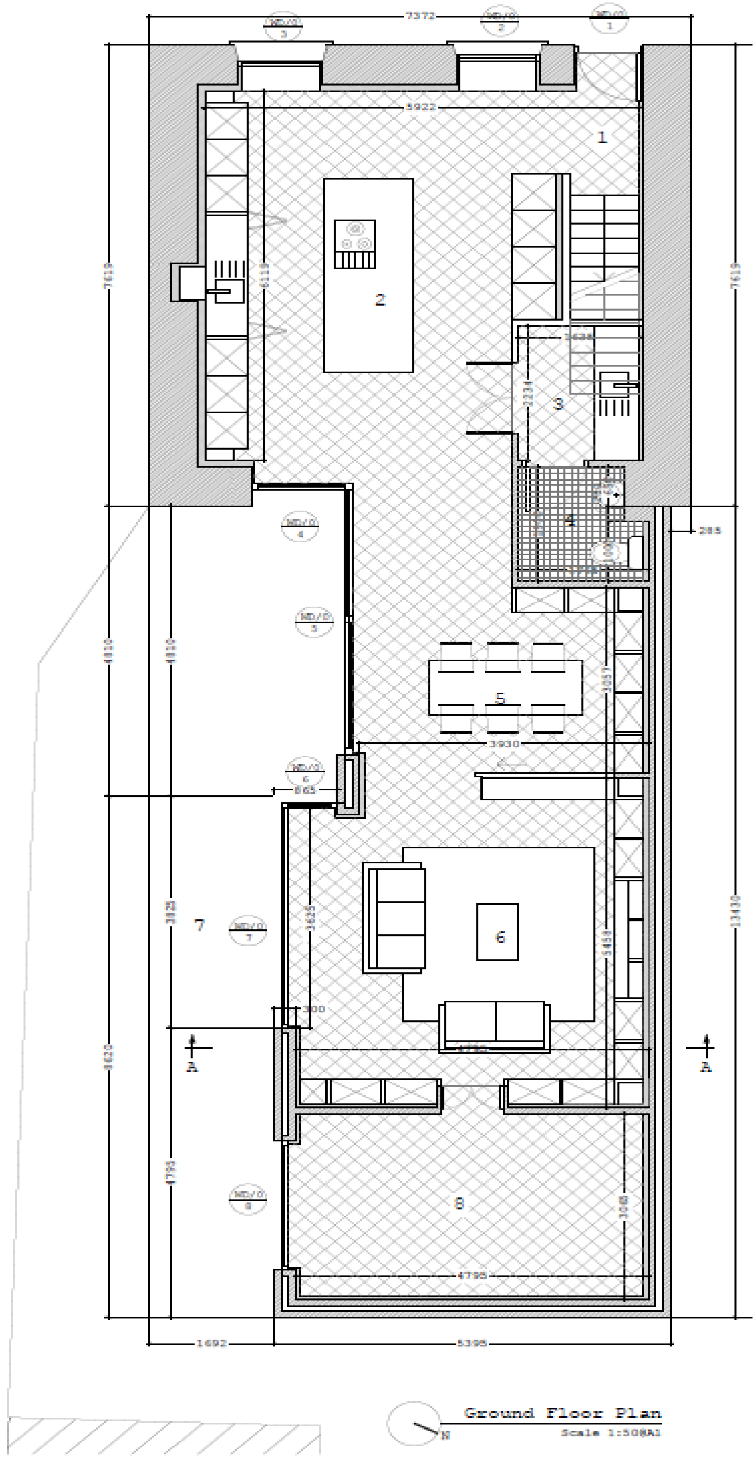
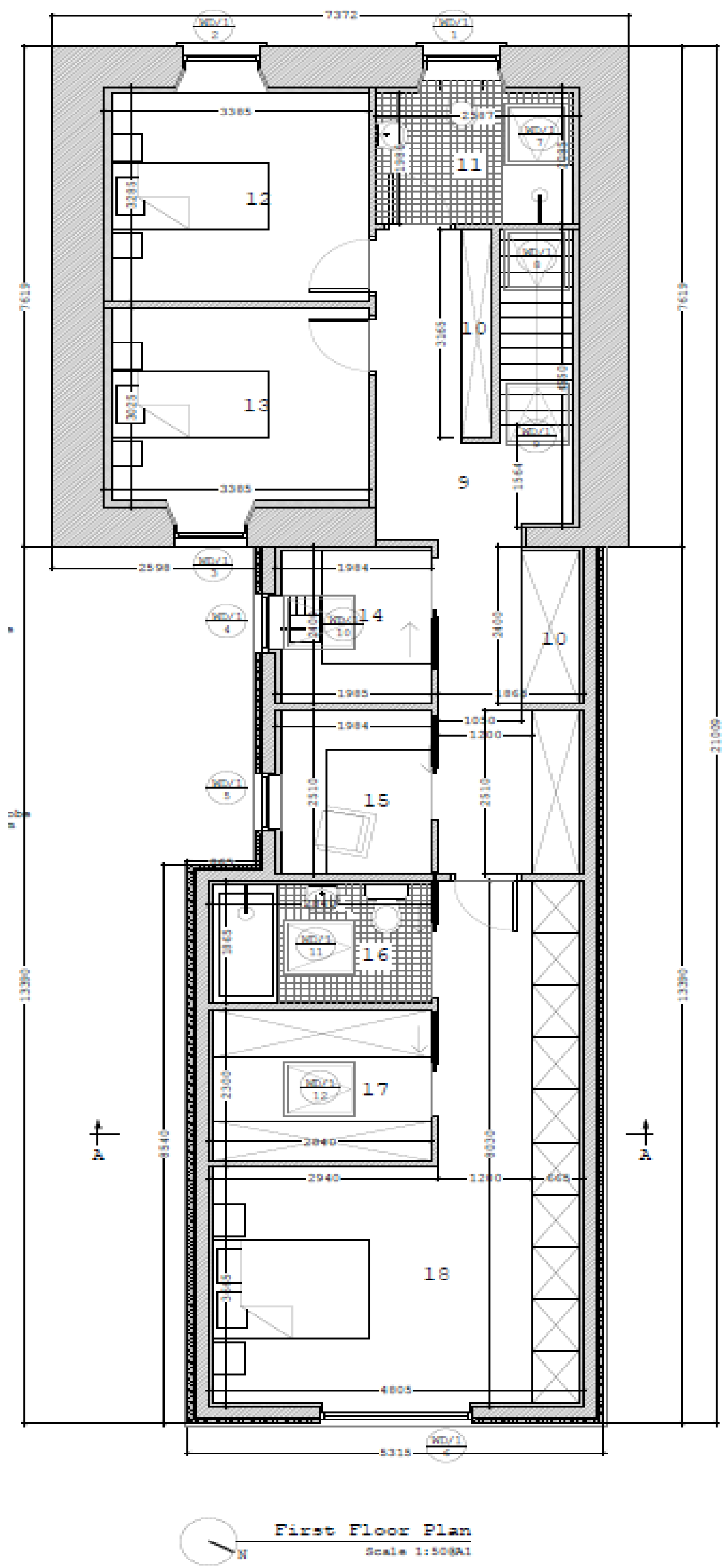
1. INTRODUCTION

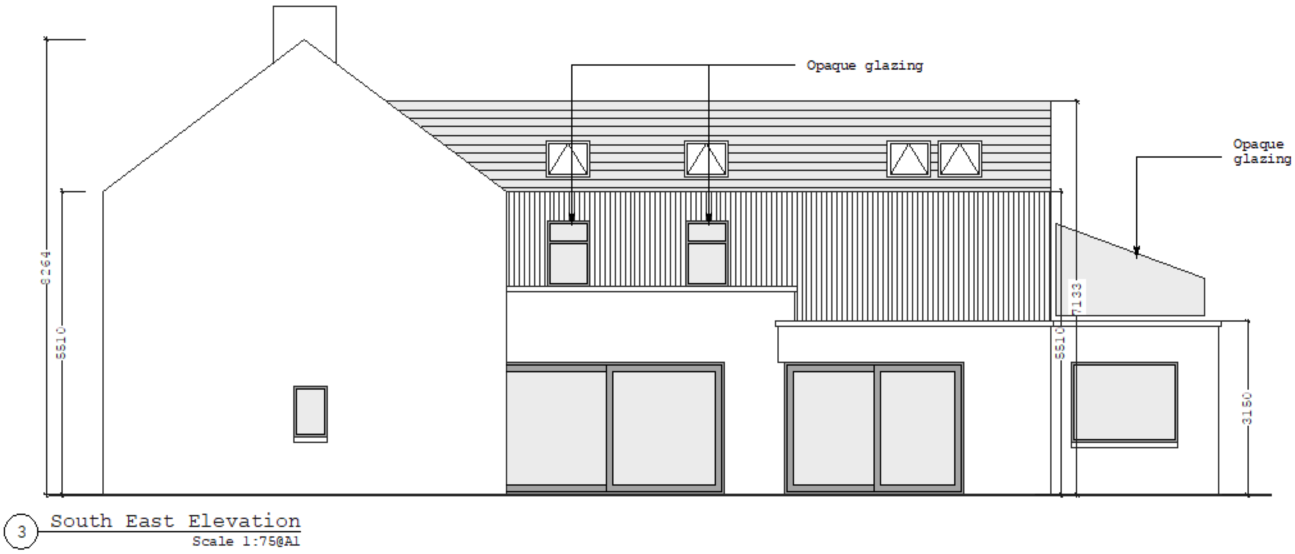
This document forms the Engineering Design Basis for Analysis and Design of Structural steel Members for proposed extension at main street, Arvagh, co. cavan.

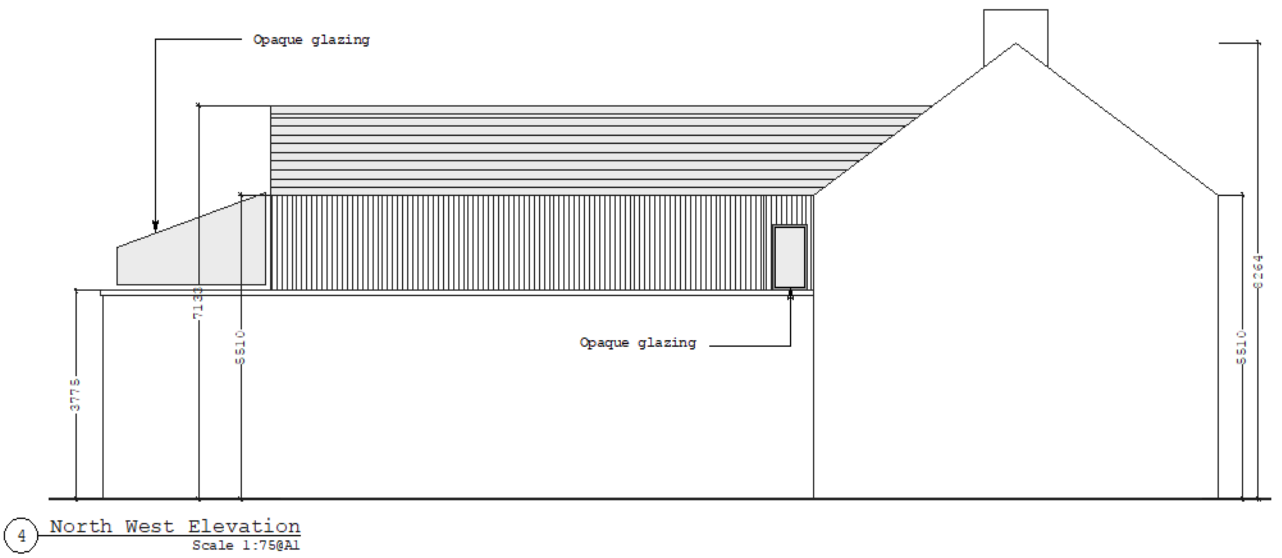


Proposed Ground Floor plan

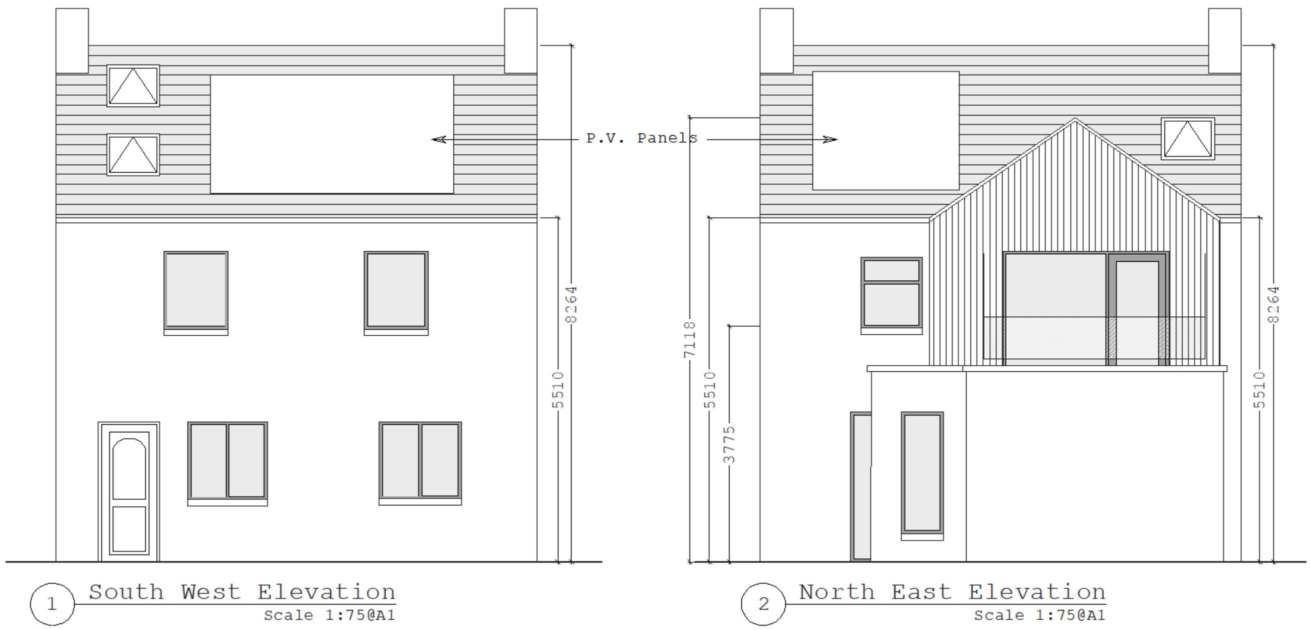


Proposed First Floor plan





Proposed front and rear elevations



Proposed both side elevations

2. MATERIAL PROPERTIES

Material Specifications

Section Type	Hot rolled
Material grade	S355
Design Strength	355 N/mm^2
Density	78.33 kN/m^3
Modulus of elasticity "Es"	205.0kN/mm^2
Shear modulus "Gs"	78846.0 MPa
Poisson's ratio "μ"	0.3
Co-efficient of linear expansion	$12*10^{-6}$ / °C
Density of Kingspan insulation	0.45 kN/m^3

3. CODES CONSIDERED

Following codes are referred for analysis and design of proposed structure.

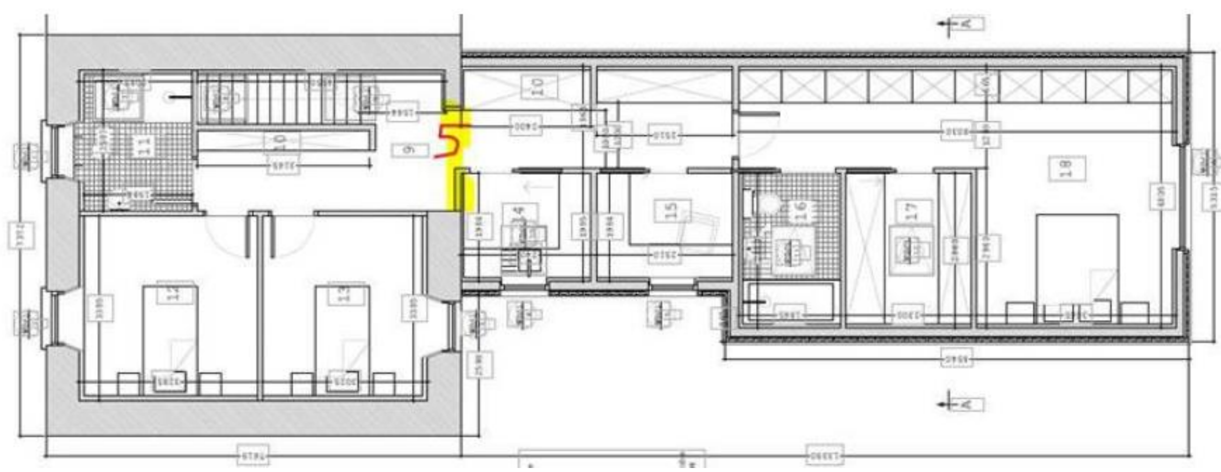
- BS EN 1990 Eurocode Basis of Structural Design
- EN 1993-1-1 (2005) Eurocode 3: Design of steel structures - Part 1-1: General rules and rules for buildings
- EN 1991-1-1 (2002) Eurocode 1: Actions on structures - Part 1-1: General actions - Densities, self-weight, imposed loads for buildings

4. DESIGN ASSUMPTIONS

- All members both end support conditions considered as pinned support either supported on main steel member or on load bearing wall and for column considered as fixed,.

5. STRUCTURAL SYSTEM AND MEMBER LOCATION

Beam arrangement at ground floor slab



Beam arrangement atroofr slab

5.1. LOADING DATA

5.2. Design Loads:

Building Design Loads will be in accordance with the more stringent of either the following criteria or as set forth by governing local and national codes. Structural design will be coordinated with architectural, mechanical and electrical drawings to ensure all loads impacting structural elements are adequately supported.

Load consideration/Assumptions:

Self-weight of 150mm Hollow core slab =	3.75kN/m^2
Weight considered for 75mm Screed (22kN/m2 x 0.075m)=	1.65kN/m^2
Weight of High Density Kingspan Insulation (0.45kN/m^2 x0.15)	0.0675kN/m
Wall load (consider 200mm thick wall), (0.2 x2.6 x20)	10.4kN/m
Internal partition wall load on slab per sq.m slab area =	1 kN/m2
Live load on Roof (As per Category H of BS EN 1991-1-1:2002)=	0.5 kN/m^2
Live load on floor (As per Category A of BS EN 1991-1-1:2002)=	2kN/m^2
Live load on stair (As per Category H of BS EN 1991-1-1:2002)=	3kN/m^2
Weight considered for Roofing material =	0.35 kN/m^2
Weight considered for Roof finishes =	0.2 kN/m^2

5.3. Load Combinations:

Design Load Combinations as per EN 1993-1-1 (2005) Eurocode 3:

- 1.35DL+1.50LL

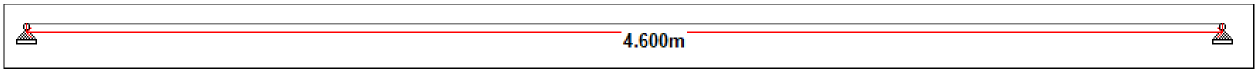
Serviceability Load Combination as per EN 1993-1-1 (2005) Eurocode 3:

- DL+LL

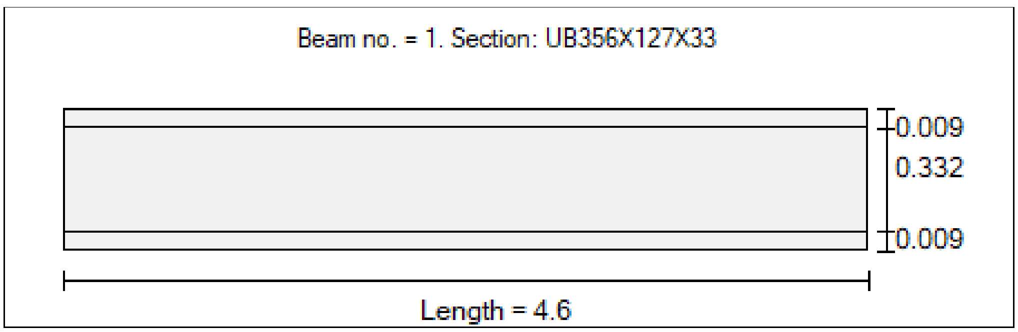
6. ANALYSIS AND DESIGN OF BEAM B2:

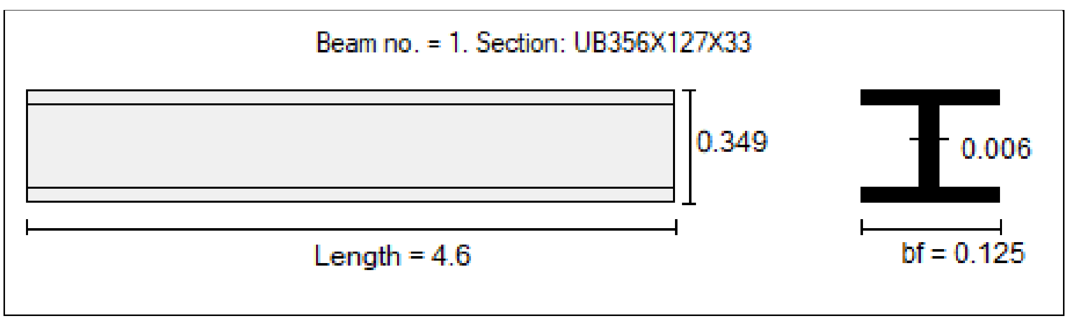
Analysis and design of steel Beam B2 performed in STAAD Pro software according to EN 1993-1-1 (2005) Eurocode 3:

Refer below image showing view of beam B2 modeled in STAAD Pro:



6.1. Member property:





6.2. Load calculation of Beam B2

Calculation of Dead and Live load on steel beam,

Dead load

1. Self-weight of Slab of hollow core slab,

   = (0.15 x 25kN/m^2 + 0.075 x 22kN/m^2 + 0.15 x 0.45kN/m^2)

   = 5.5kN/m^2

   Load on Beam B2

   = (5.5 x 4.0)/2

   = 11kN/m^2

2. 200mm thick Wall load,

   = (0.2 x 2.6 x 20 kN/m^2)

   = 10kN/m

3. DL of roof slab,

   = (Self-weight of pitched roof + Natural slate finish to roof)

   = (0.35 + 0.2)

   = 0.55kN/m^2

   Load on Beam B2

   = (0.55 x 4.0)/2

   = 1.1kN/m

Hence, Total Dead load on Beam B2 = 11kN/m + 10kN/m + 1.1kN/m

= 22.1kN/m

Live Load

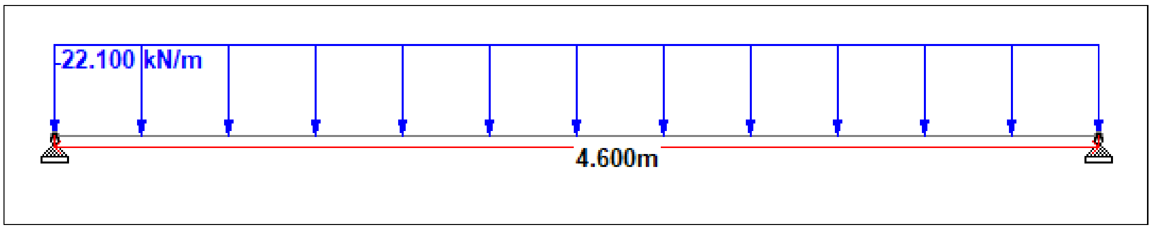
1. LL of GF slab = $2kN/m^2$

LL on Beam B2 = (2 x 4.0)/2 = 4.0kN/m

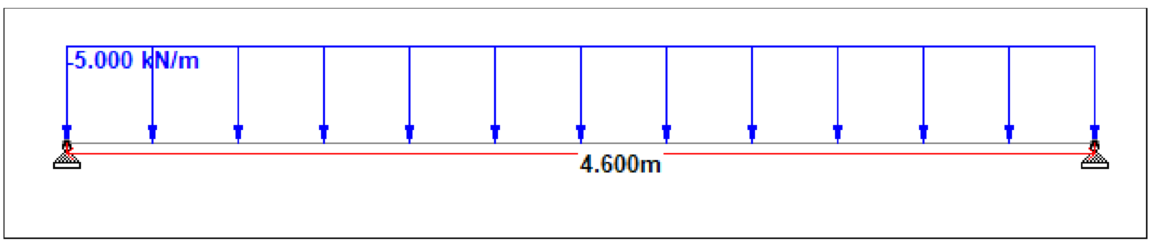
2. LL of Roof = $0.5kN/m^2$

LL on Beam B2 = (0.5 x 4.0)/2 = 1kN/m

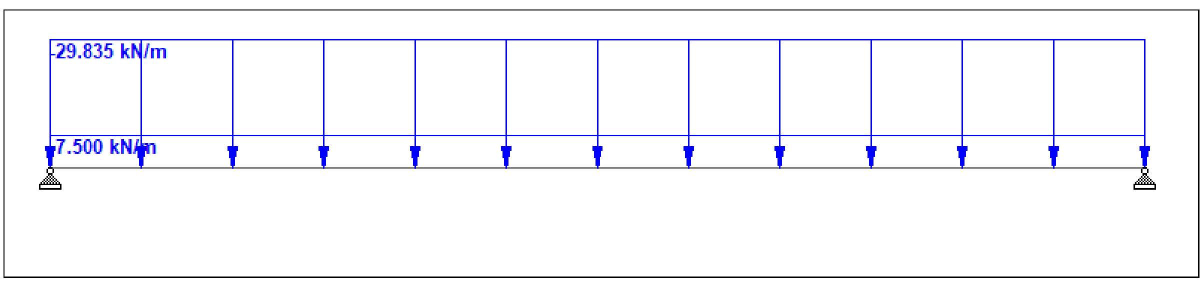
Hence, Total LL on Beam B2 = 4.0kN/m + 1.0kN/m = 5.0kN/m



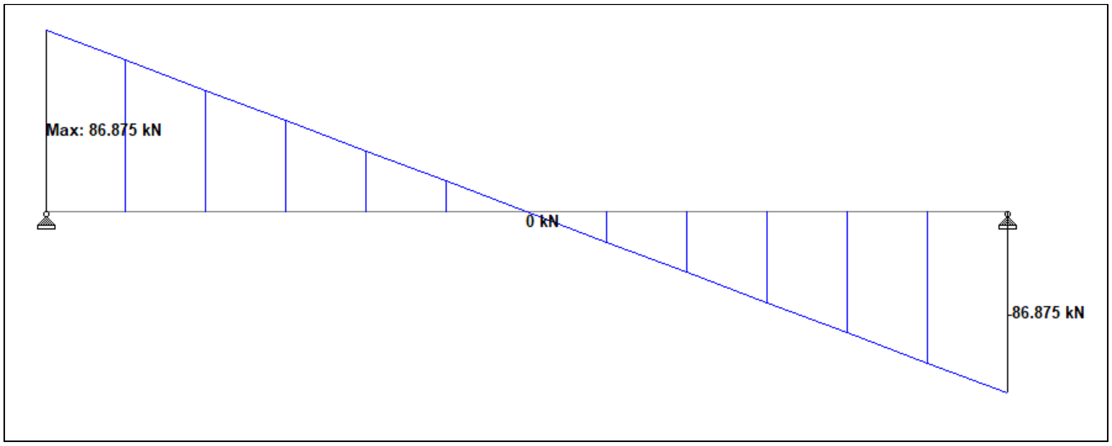
Dead load on beam B2



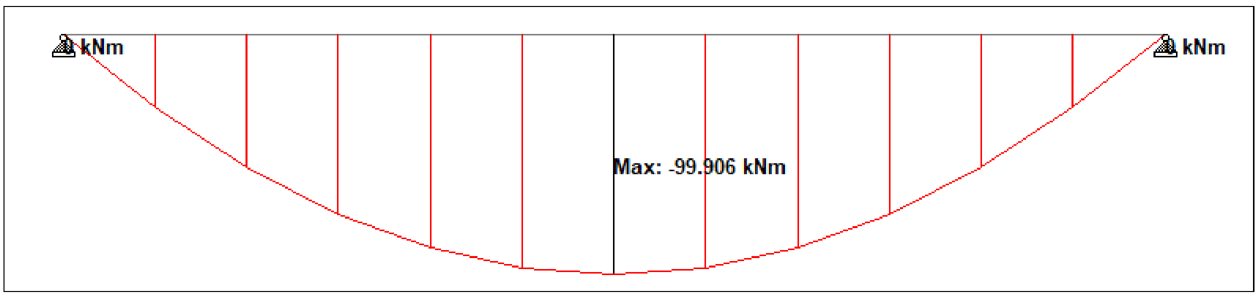
Live load on beam B2



Loading Diagram (1.35DL+1.5LL)

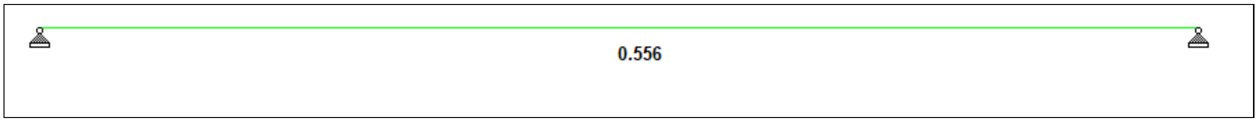


Shear force diagram (1.35DL+1.5LL)



Bending Moment Diagram (1.35DL+1.5LL)

6.3. Utilization ratio check:

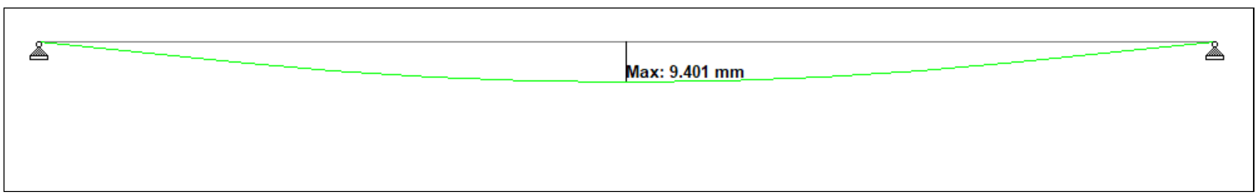


Utility ratio of Beam B2

Above Figure shows member utilization ratio. Failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors. It can be seen from image that all members are green. Hence, all members have passed in design.

6.4. Deflection check

Below image shows displacement diagram of member having maximum vertical deflection for serviceability load combinations.



Deflection diagram of beam B2 (DL+LL)

Maximum vertical displacement of beam in Y direction= 9.401 mm

Permissible vertical deflection = Span / 360 = 4600 /360 = 12.77 mm

Actual maximum vertical deflection of beam = 9.401 mm < 12.77 mm (Hence, OK)

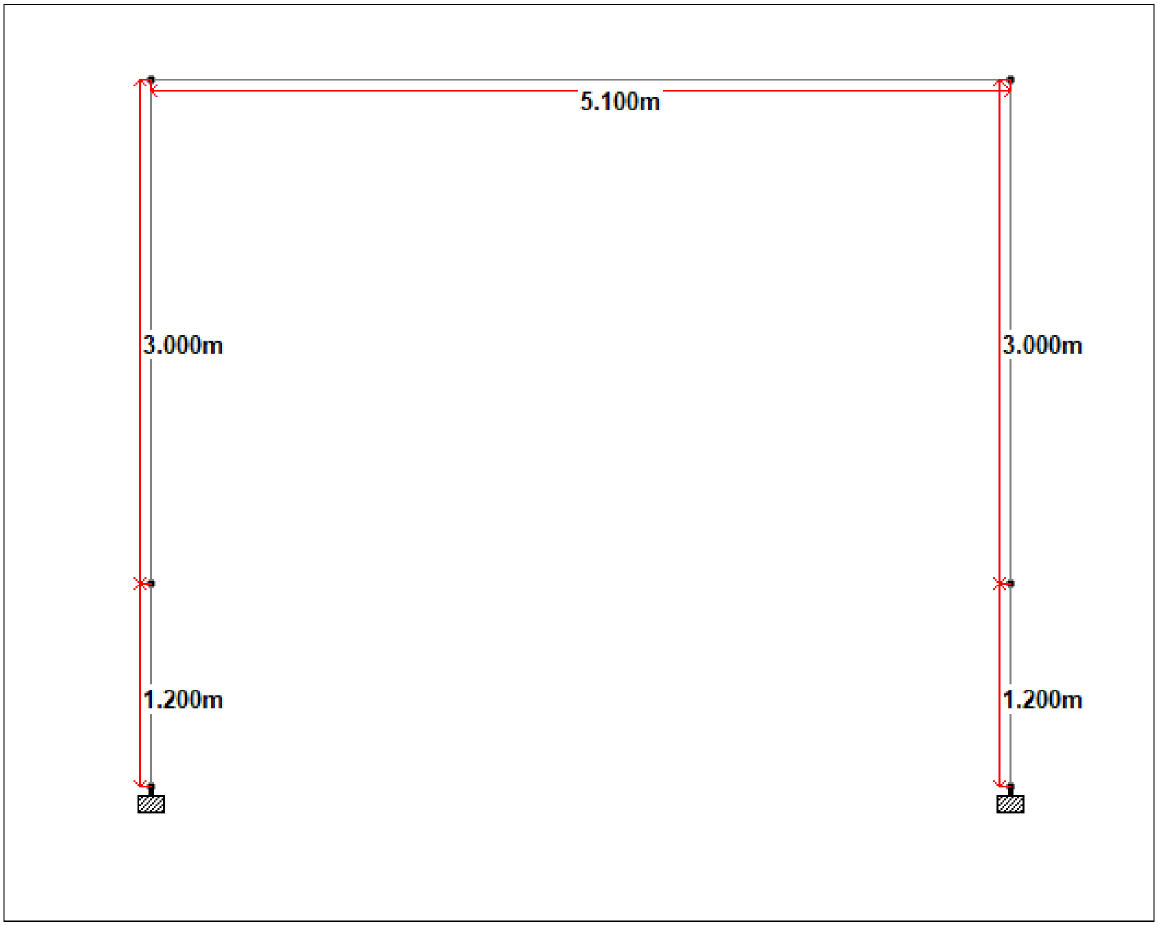
6.5. STAAD design output results

STAAD output results for BEAM B2 (4.6m):

```
    1 ST  UB356X127X33 (BRITISH SECTIONS)
                    PASS     EC-6.3.2 LTB       0.556         3
                 0.00            0.00        -99.91        2.30
=======================================================================
  MATERIAL DATA
     Grade of steel            =  S 355
     Modulus of elasticity     =  205 kN/mm2
     Design Strength  (py)     =  355  N/mm2

  SECTION PROPERTIES (units - cm)
     Member Length =     460.00
     Gross Area =   42.10          Net Area =   42.10

                                     z-axis          y-axis
     Moment of inertia         :     8250.001        280.000
     Plastic modulus           :      543.000         70.200
     Elastic modulus           :      472.779         44.657
     Shear Area                :       19.186         23.026
     Radius of gyration        :       13.999          2.579
     Effective Length          :      460.000        460.000

  DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
     Section Class             :   CLASS 1
     Squash Load               :   1494.55
     Axial force/Squash load   :     0.000
     GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                     z-axis          y-axis
     Slenderness ratio (KL/r) :         32.9          178.4
     Compression Capacity     :       1411.9          236.7
     Tension Capacity         :       1485.3         1485.3
     Moment Capacity          :        192.8           24.9
     Reduced Moment Capacity  :        192.8           24.9
     Shear Capacity           :        393.2          471.9

  BUCKLING CALCULATIONS  (units - kN,m)
     Lateral Torsional Buckling Moment       MB =  179.8
     co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 1.000
     Elastic Critical Moment for LTB,              Mcr   =   713.6
     Critical Load For Torsional Buckling,         NcrT  =  8446.5
     Critical Load For Torsional-Flexural Buckling, NcrTF =  8446.5

  SHEAR BUCKLING CALCULATIONS [EN1993-1-5] (units - kN,m)
     Stiffeners not provided.
     Design Resistance for Shear Buckling,         VbRd  =   430.5
```

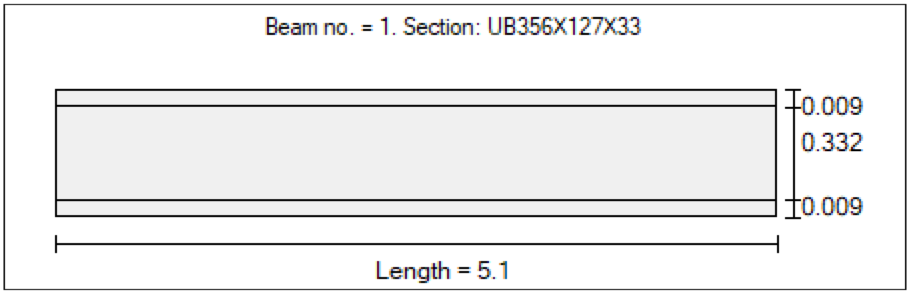
7. ANALYSIS AND DESIGN OF BEAM B1:

Refer below image showing view of beam B1 modeled in STAAD Pro:

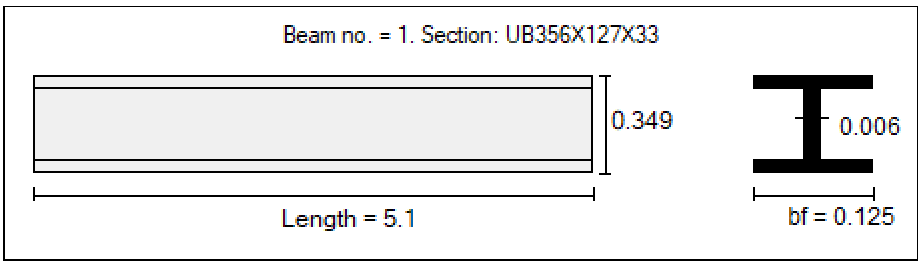


Beam B1 Dimensions

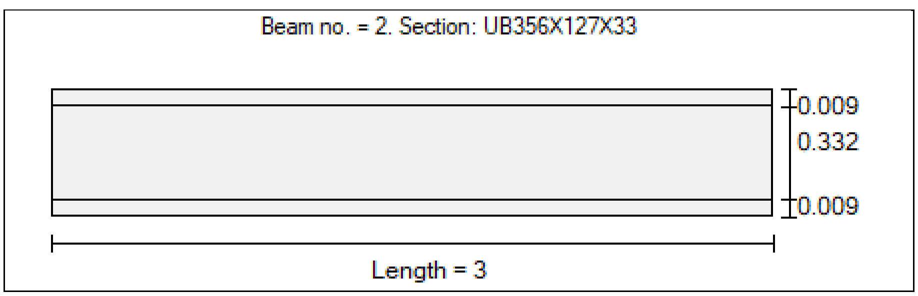
7.1. Member Property:



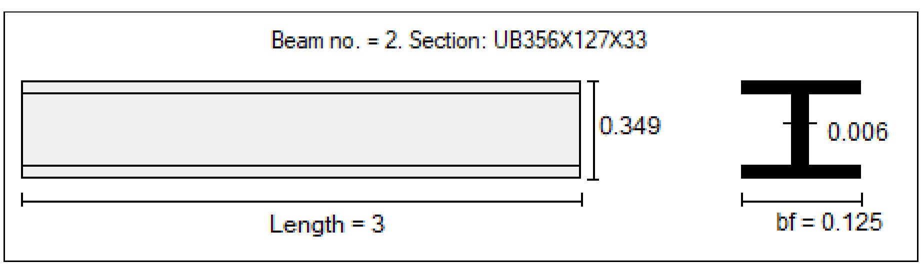
Beam B1 property



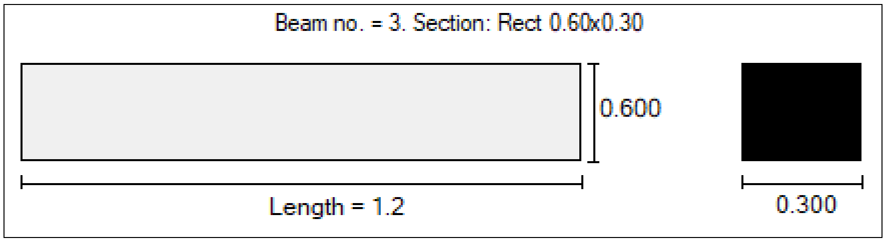
Beam B1 property



Column property



Column property



Pedestal property

7.2. Loading on Beam B1:

Calculation Dead and Live load on steel beam,

Dead load

200mm thick Wall load,

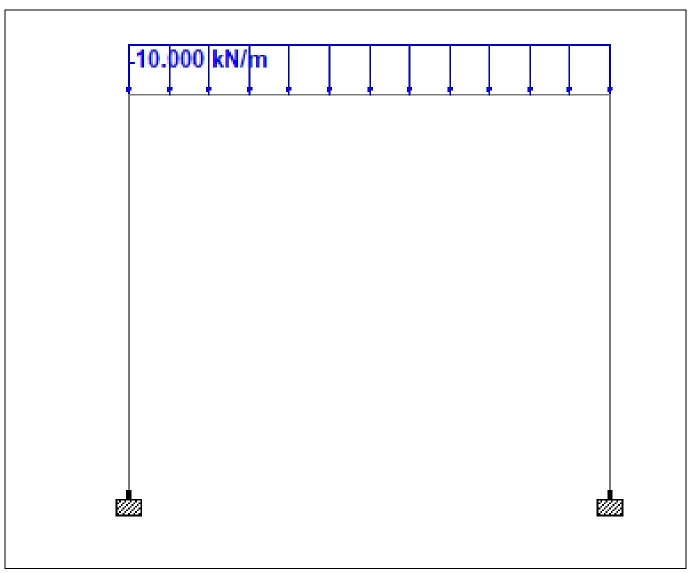
= (0.2 x 2.6 x 20 kN/m^2)

= 10kN/m

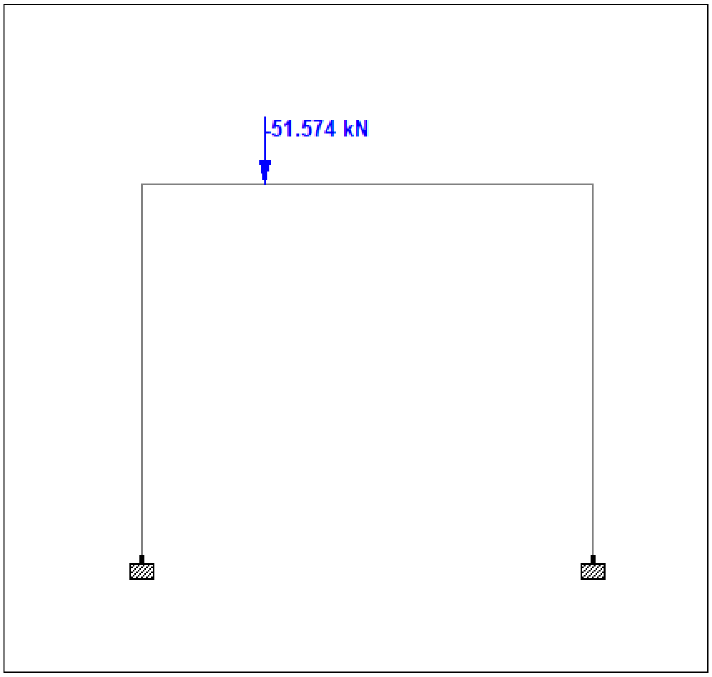
- DL from Beam B2 = 51.574 kN

Live Load

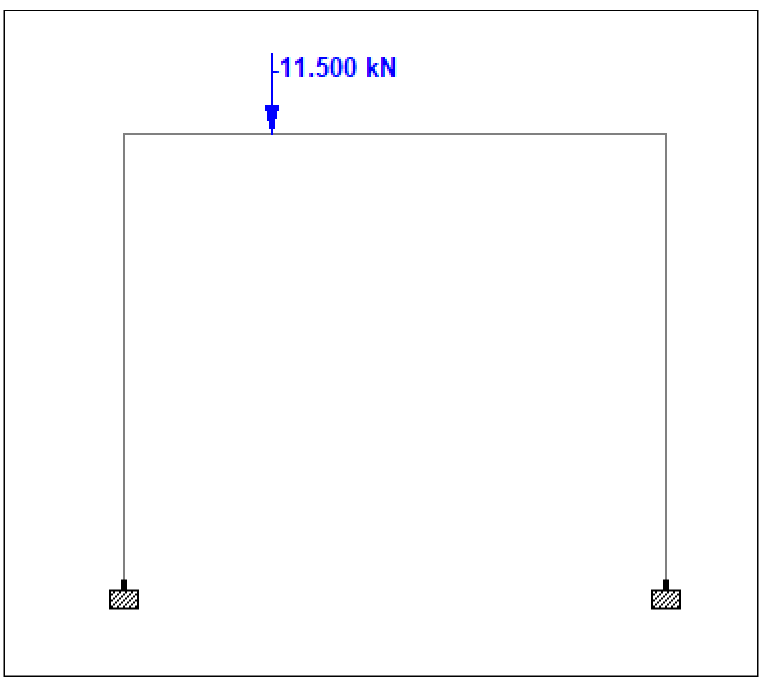
- LL from Beam B2 = 11.5 kN



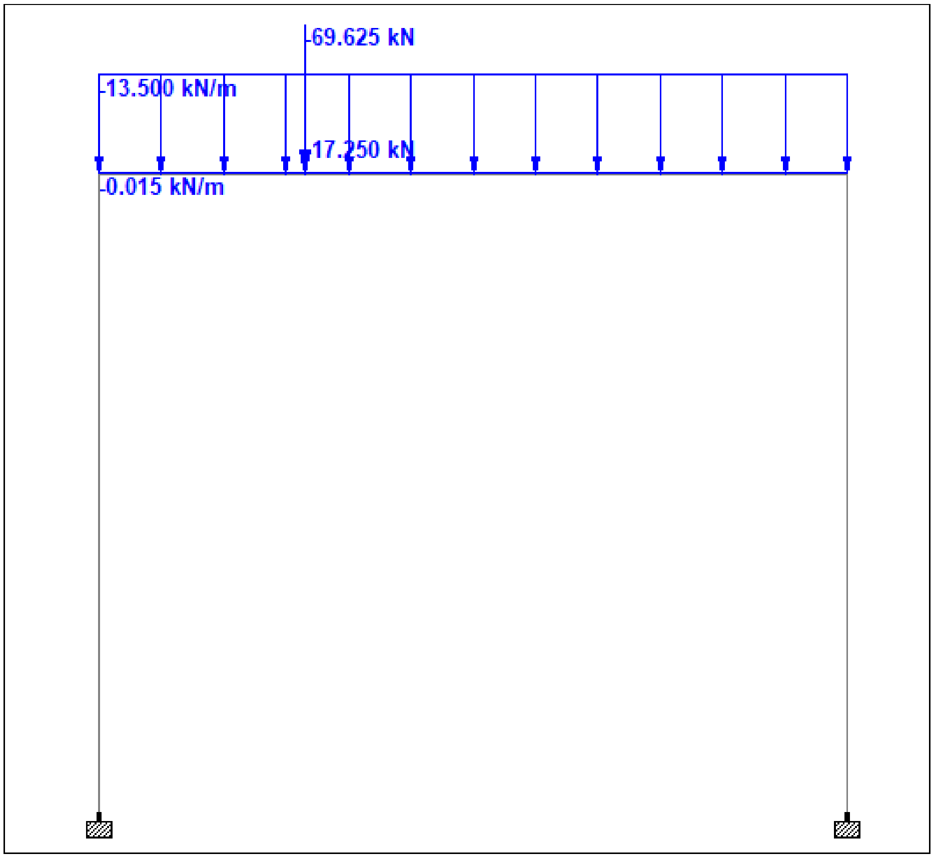
Dead load on beam B1



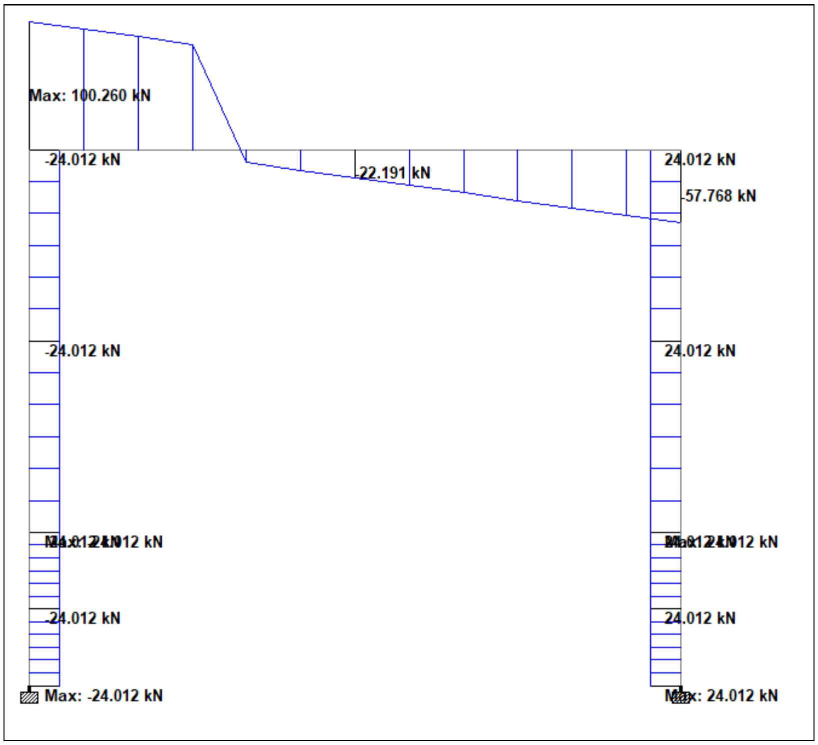
Dead load on beam B1



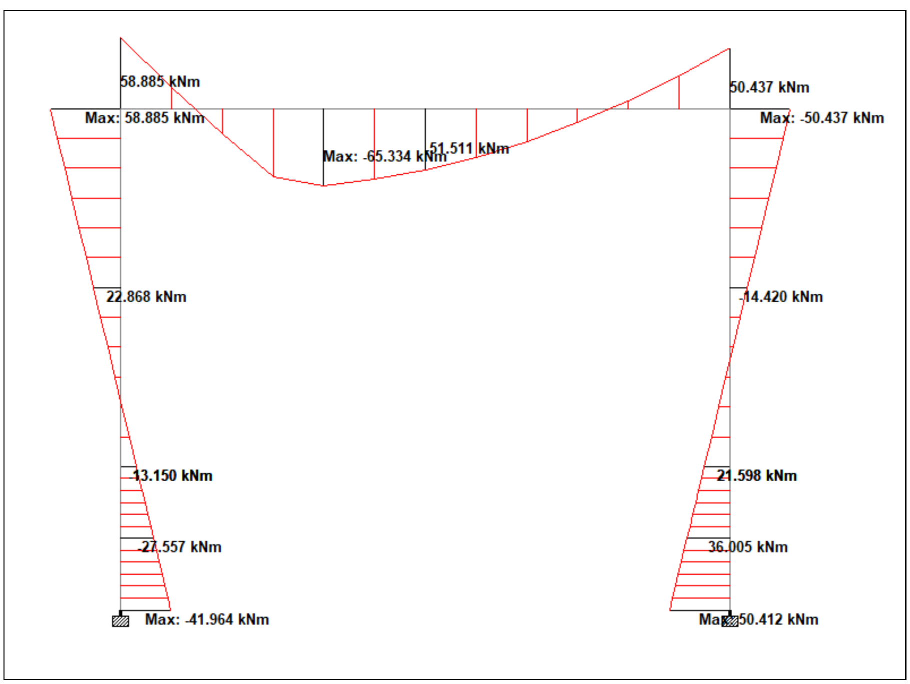
Live load on beam B1



Loading Diagram (1.35DL+1.5LL)

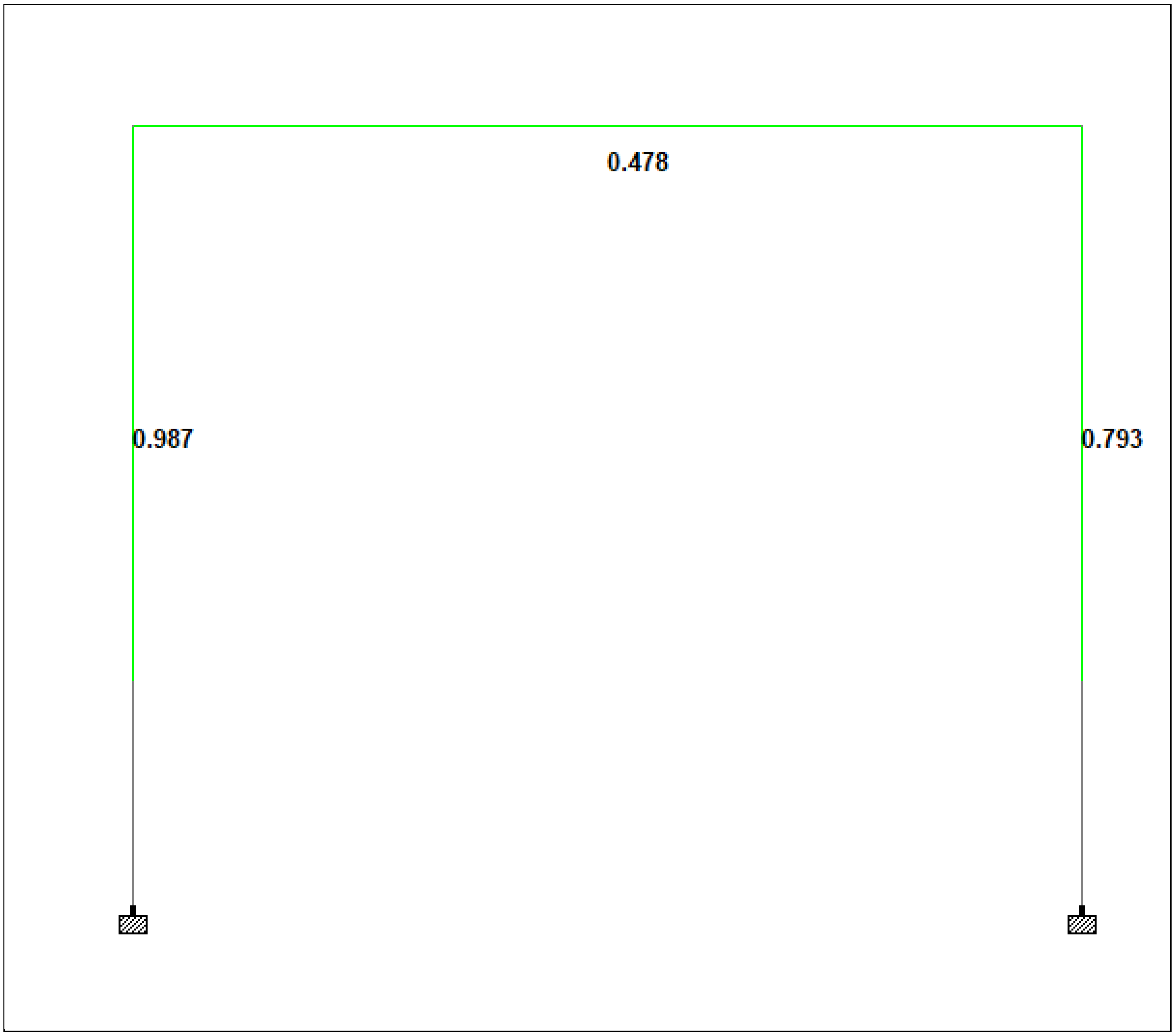


Shear force diagram (1.35DL+1.5LL)



Bending Moment Diagram (1.35DL+1.5LL)

7.3. Utilization ratio check

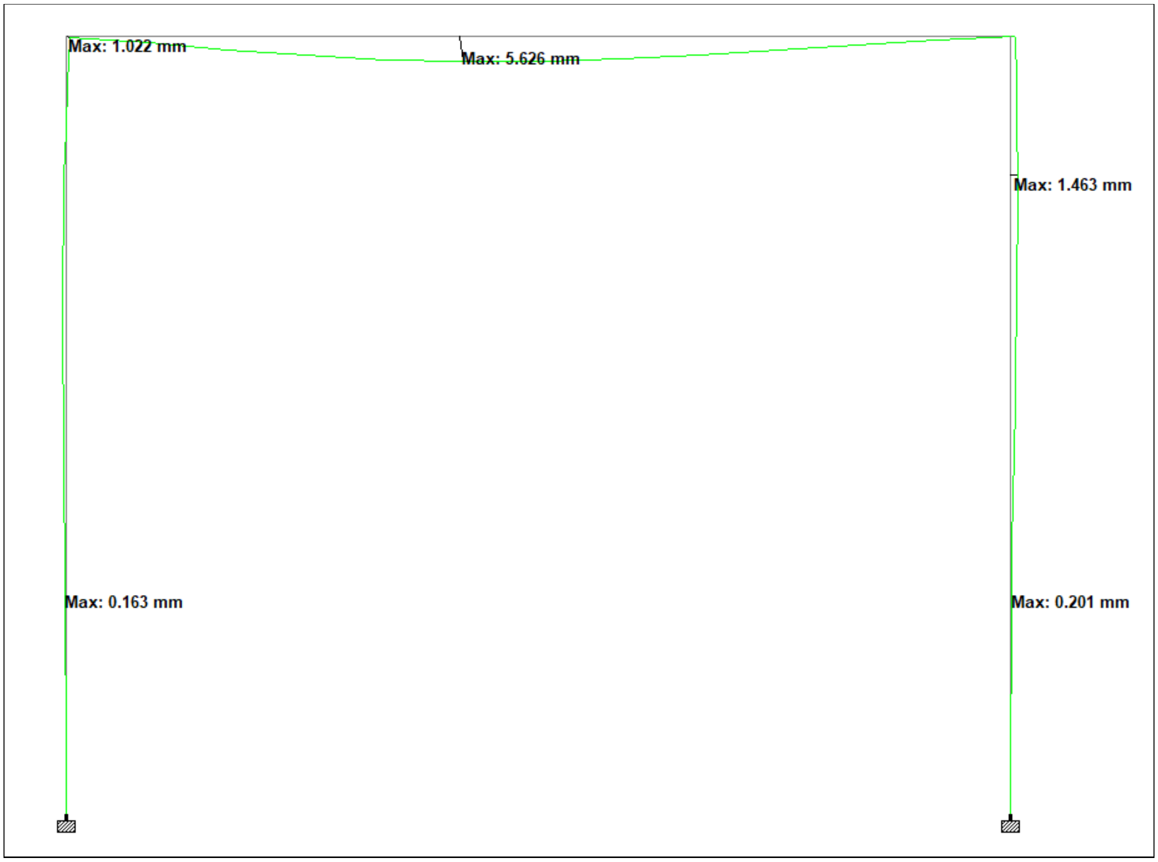


Utilization ratio of Beam B1

Above Figure shows member utilization ratio. Failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors. It can be seen from image that all members are green. Hence, all members have passed in design.

7.4. Deflection check

Below figure shows displacement diagram of member having maximum vertical deflection for serviceability load combinations.



Deflection diagram of beam B1 (DL+LL)

Maximum vertical displacement of beam in Y direction= 5.626 mm

Permissible vertical deflection = Span / 360 = 5100/360 = 14.167 mm

Actual maximum vertical deflection of beam = 5.626 mm < 14.167 mm (Hence, OK)

Maximum displacement of Column in X direction= 1.463 mm

Permissible deflection = Height / 300 = 3000/300 = 10 mm

Actual maximum deflection of beam = 1.463 mm < 10 mm (Hence, OK)

7.5. STAAD design output results

STAAD output results for BEAM B1:

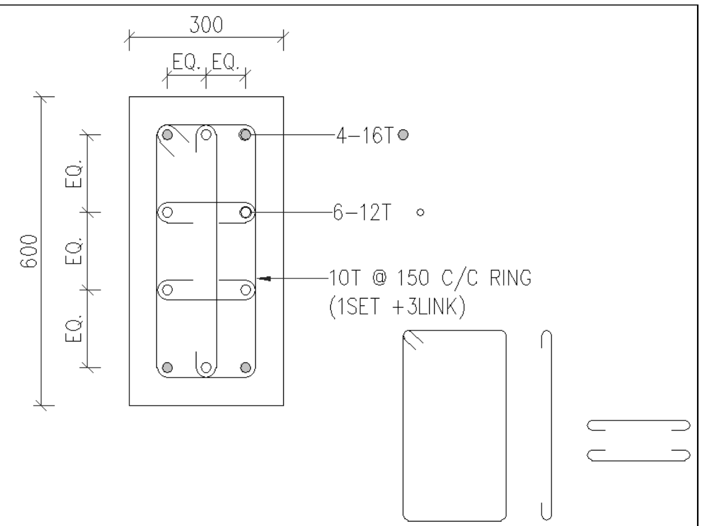
```
    1 ST  UB356X127X33 (BRITISH SECTIONS)
                        PASS     EC-6.3.3-662       0.478         3
                    24.01 C         0.00        -65.33        1.70
======================================================================
 MATERIAL DATA
    Grade of steel           =  S 355
    Modulus of elasticity    =  205 kN/mm2
    Design Strength  (py)    =  355  N/mm2

 SECTION PROPERTIES (units - cm)
    Member Length =    510.00
    Gross Area =   42.10         Net Area =   42.10

                                   z-axis          y-axis
    Moment of inertia        :     8250.001        280.000
    Plastic modulus          :      543.000         70.200
    Elastic modulus          :      472.779         44.657
    Shear Area               :       19.186         23.026
    Radius of gyration       :       13.999          2.579
    Effective Length         :      510.000        510.000

 DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
    Section Class            :   CLASS 1
    Squash Load              :   1494.55
    Axial force/Squash load  :     0.016
    GM0 :  1.00          GM1 :  1.00         GM2 :  1.25

                                   z-axis          y-axis
    Slenderness ratio (KL/r) :         36.4          197.8
    Compression Capacity     :       1391.9          195.8
    Tension Capacity         :       1485.3         1485.3
    Moment Capacity          :        192.8           24.9
    Reduced Moment Capacity  :        192.8           24.9
    Shear Capacity           :        393.2          471.9

 BUCKLING CALCULATIONS (units - kN,m)
    Lateral Torsional Buckling Moment       MB =  179.8
    co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 1.000
    Elastic Critical Moment for LTB,              Mcr     =   713.6
    Critical Load For Torsional Buckling,         NcrT    =  8446.5
    Critical Load For Torsional-Flexural Buckling, NcrTF =  8446.5

 SHEAR BUCKLING CALCULATIONS [EN1993-1-5] (units - kN,m)
    Stiffeners not provided.
    Design Resistance for Shear Buckling,         VbRd    =   430.5
```

STAAD output results for Column & pedestal:

```
     2 ST  UB356X127X33 (BRITISH SECTIONS)
                          PASS     EC-6.3.3-662       0.987         3
                     100.26 C          0.00         -58.89        3.00
=======================================================================
 MATERIAL DATA
    Grade of steel            =  S 235
    Modulus of elasticity     =  205 kN/mm2
    Design Strength  (py)     =  235  N/mm2

 SECTION PROPERTIES (units - cm)
    Member Length =    300.00
    Gross Area =   42.10          Net Area =   42.10

                                   z-axis          y-axis
    Moment of inertia         :     8250.001        280.000
    Plastic modulus           :      543.000         70.200
    Elastic modulus           :      472.779         44.657
    Shear Area                :       19.186         23.026
    Radius of gyration        :       13.999          2.579
    Effective Length          :      300.000        300.000

 DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
    Section Class             :   CLASS 1
    Squash Load               :    989.35
    Axial force/Squash load   :     0.101
    GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                   z-axis          y-axis
    Slenderness ratio (KL/r) :        21.4          116.3
    Compression Capacity      :       983.2          452.6
    Tension Capacity          :       989.4          989.4
    Moment Capacity           :       127.6           16.5
    Reduced Moment Capacity   :       127.6           16.5
    Shear Capacity            :       260.3          312.4

 BUCKLING CALCULATIONS (units - kN,m)
    Lateral Torsional Buckling Moment       MB =   70.4
    co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 3.000
    Elastic Critical Moment for LTB,             Mcr    =    96.4
    Critical Load For Torsional Buckling,        NcrT   =  1242.6
    Critical Load For Torsional-Flexural Buckling, NcrTF =  1242.6
```

```
        C O L U M N   N O.      3  D E S I G N   R E S U L T S

      M20                      Fe500 (Main)              Fe415 (Sec.)

LENGTH:  1200.0 mm   CROSS SECTION:  300.0 mm X  600.0 mm  COVER: 40.0 mm

** GUIDING LOAD CASE:    3 END JOINT:     4  SHORT COLUMN


REQD. STEEL AREA    :      173.53 Sq.mm.
REQD. CONCRETE AREA:    21691.32 Sq.mm.
MAIN REINFORCEMENT : Provide   8 - 12 dia. (0.50%,    904.78 Sq.mm.)
                     (Equally distributed)
TIE REINFORCEMENT   : Provide  8 mm dia. rectangular ties @ 190 mm c/c

SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)
----------------------------------------------------------
Puz :   1683.51   Muz1 :     49.43   Muy1 :     24.17

INTERACTION RATIO: 0.94 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)
----------------------------------------------------------
WORST LOAD CASE:     3
END JOINT:      4 Puz :   1951.15   Muz :    127.55   Muy :     57.42   IR: 0.37
============================================================================
```

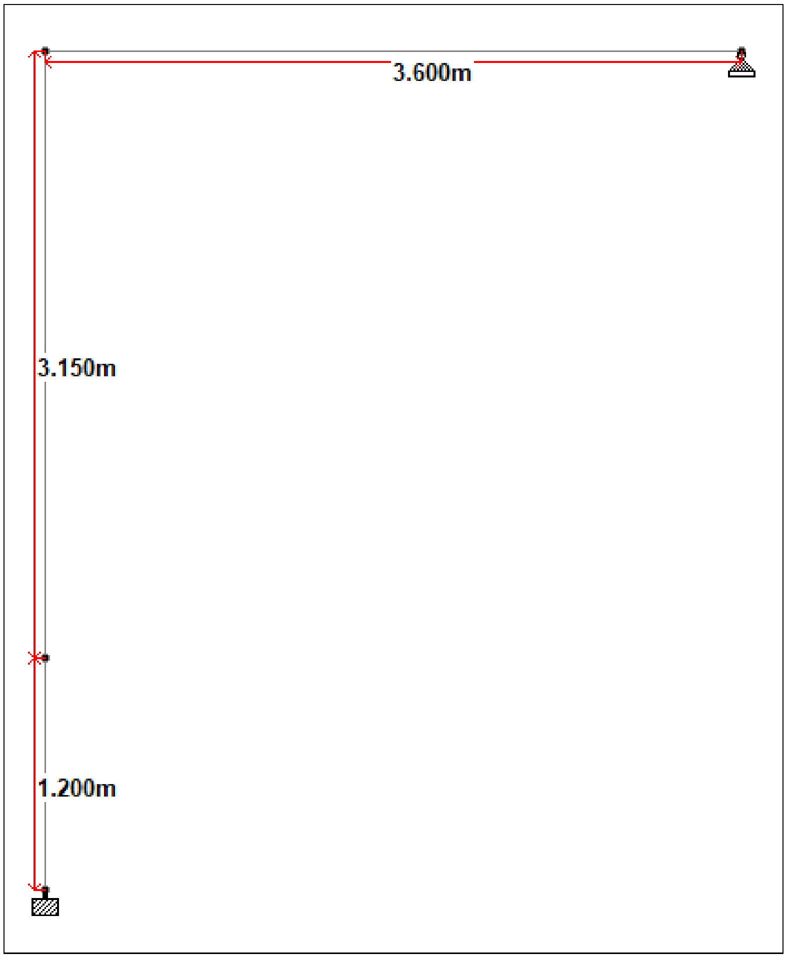
- **Hence, provide 4nos 16dia + 6nos 12dia rebar with 10T @ 150c/c stirrups.**



PEDESTAL REINFORCEMENT DETAIL
P1 (300x600)
SCALE 1:10

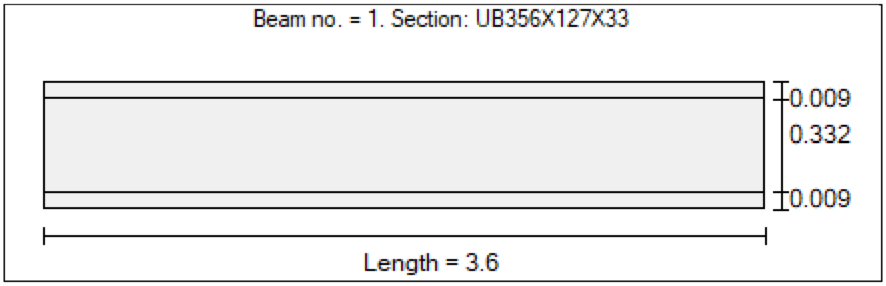
8. ANALYSIS AND DESIGN OF BEAM B3:

Refer below image showing view of beam B3 modeled in STAAD Pro:

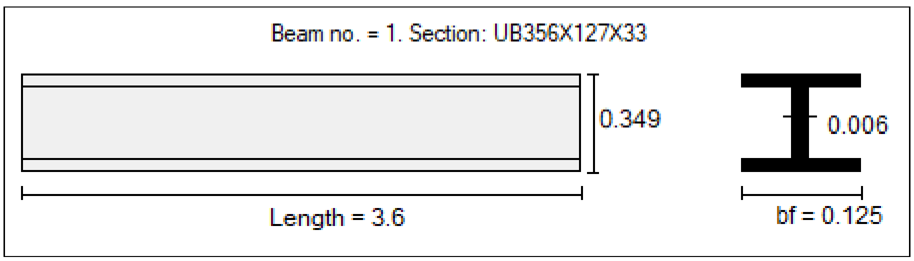


Beam B3 Dimensions

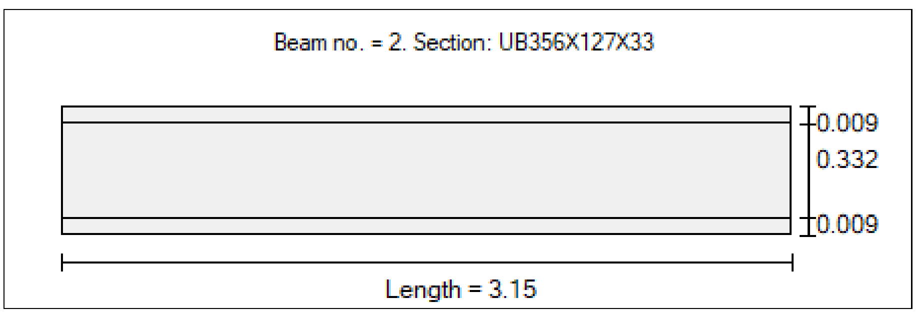
8.1. Member Property:



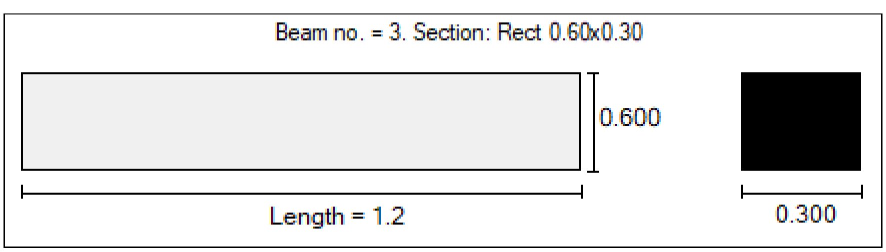
Beam B3 Property



Beam B3 Property



Column Property



Pedestal Property

8.2. Loading on Beam B3

Calculation Dead and Live load on steel beam,

Dead load

1. Self-weight of Slab of hollow core slab,

   = (0.15 x 25kN/m^2 + 0.075 x 22kN/m^2 + 0.15 x 0.45kN/m^2)

   = 5.5kN/m^2

   Load on Beam B3

   = (5.5 x 4.8)/2

   = 13.2kN/m^2

2. 200mm thick Wall load,

   = (0.2 x 2.6 x 20 kN/m^2)

   = 10kN/m

3. DL of roof slab,

   = (Self-weight of pitched roof + Natural slate finish to roof)

   = (0.35 + 0.2)

   = 0.55kN/m^2

   Load on Beam B3

   = (0.55 x 4.8)/2

   = 1.32kN/m

Hence, Total Dead load on Beam B3 = 13.2kN/m + 10kN/m + 1.32kN/m

= 24.52kN/m

Live Load

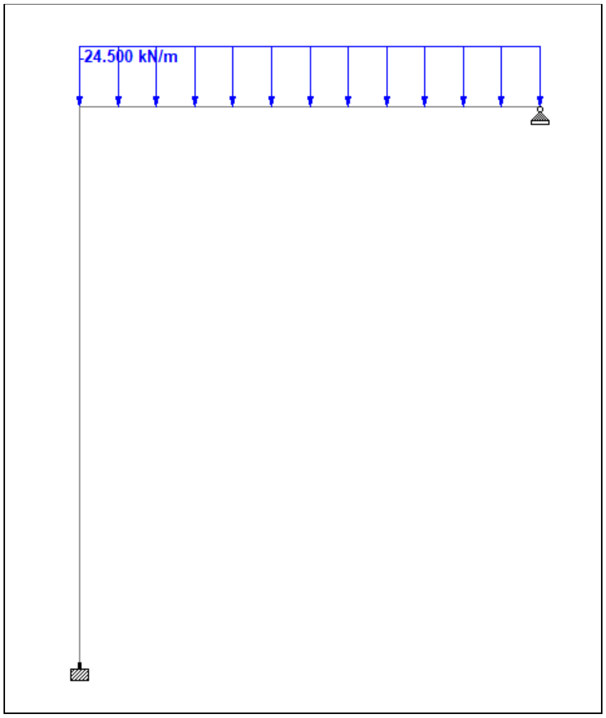
1. LL of GF slab = 2kN/m^2

LL on Beam B3 = (2 x 4.8)/2 = 4.8kN/m

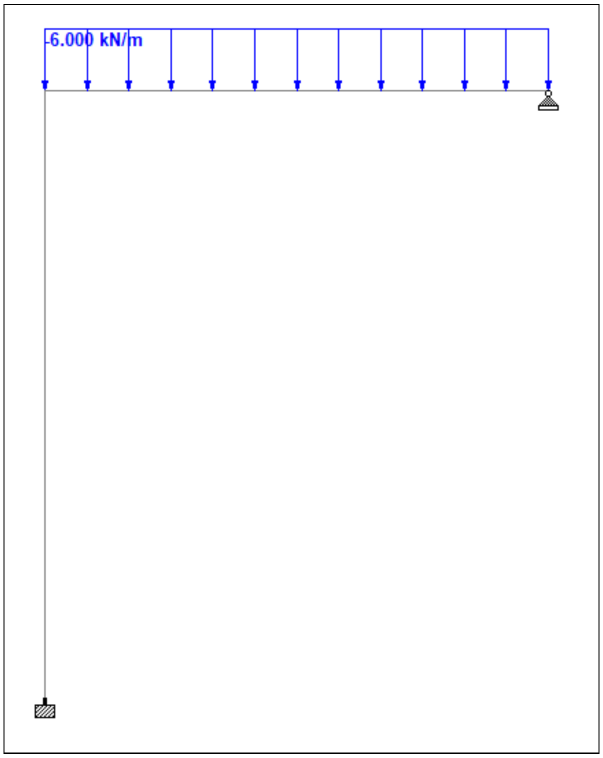
2. LL of Roof = 0.5kN/m^2

LL on Beam B3 = (0.5 x 4.8)/2 = 1.2kN/m

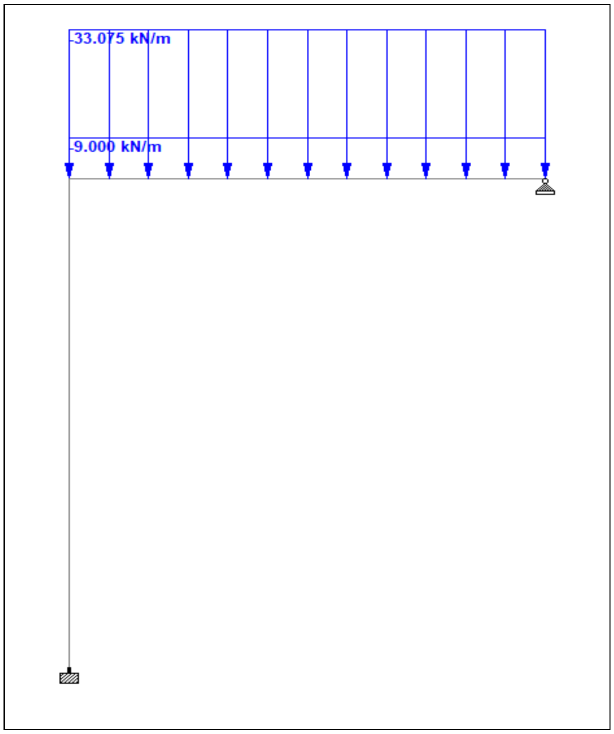
Hence, Total LL on Beam B3 = 4.8kN/m + 1.2kN/m = 6.0kN/m



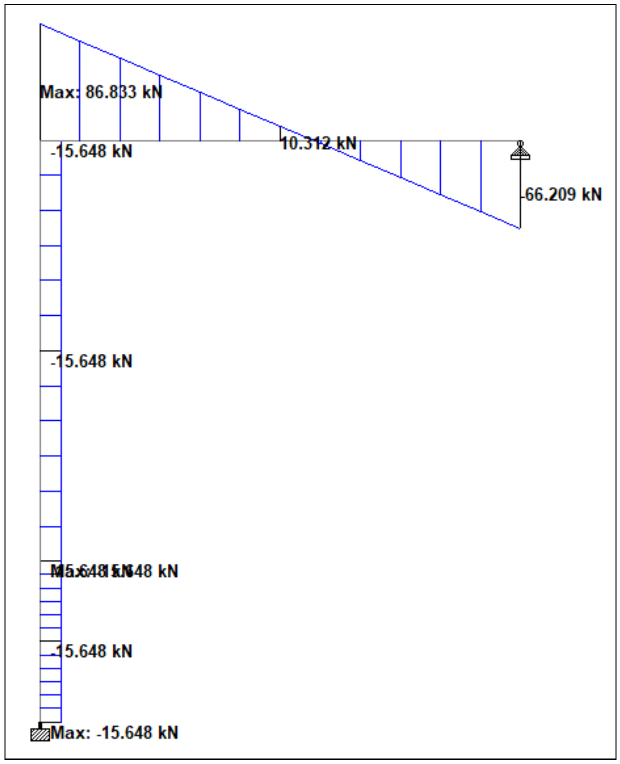
Dead load on beam B3



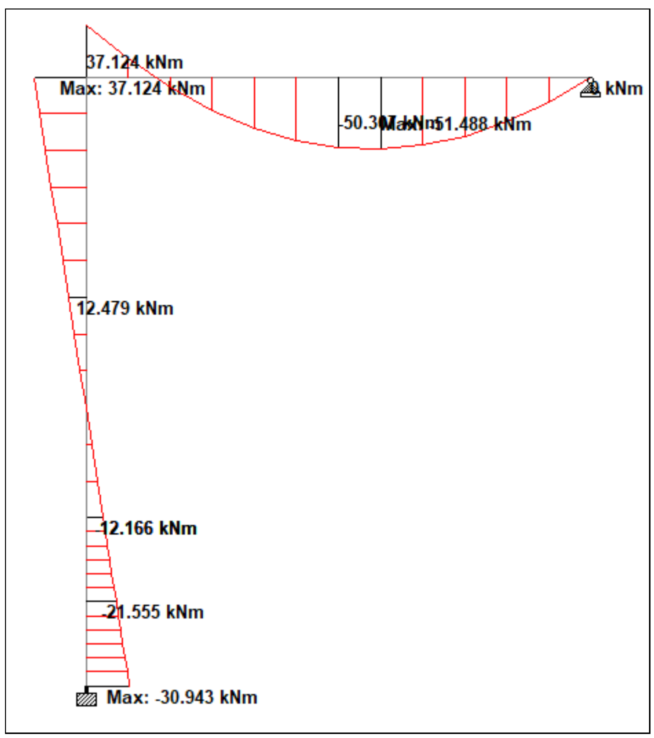
Live load on beam B3



Loading Diagram (1.35DL+1.5LL)

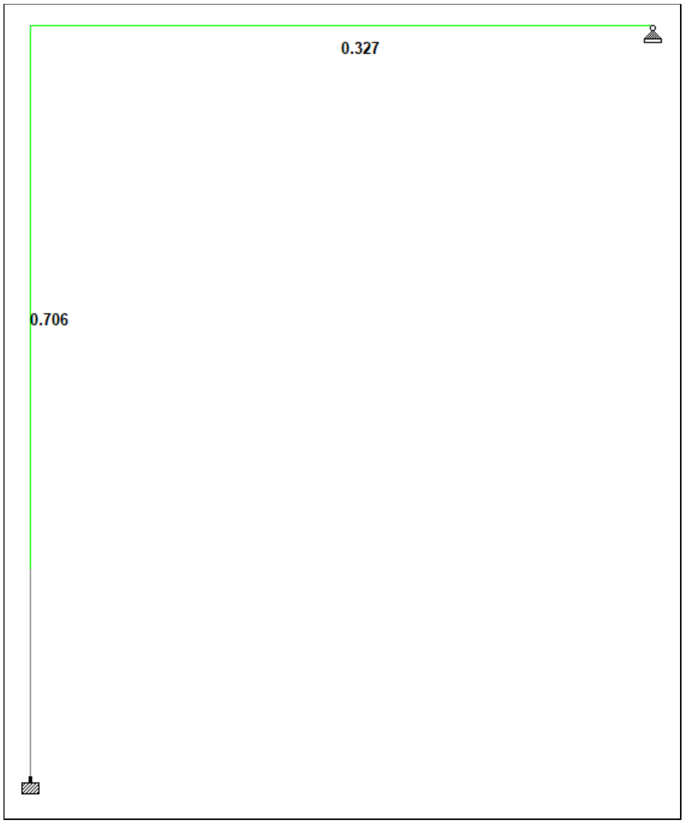


Shear force diagram (1.35DL+1.5LL)



Bending Moment Diagram (1.35DL+1.5LL)

8.3. Utilization ratio check:

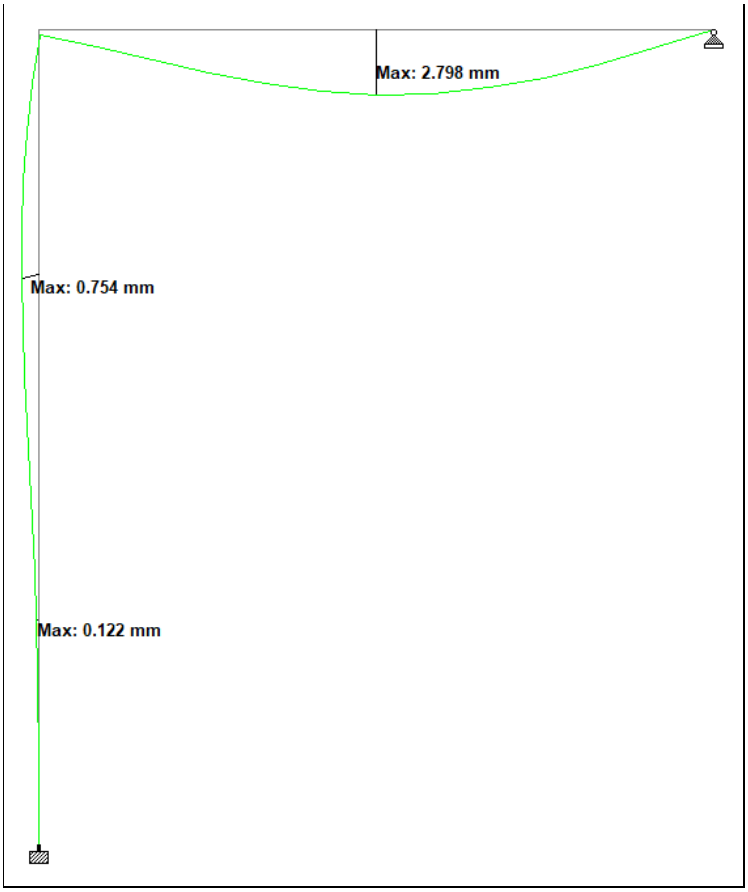


Utilization ratio of Beam B3

Above shows that failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors. It can be seen from image that all members are green. Hence, all members have passed in design.

8.4. Deflection check

Below image shows displacement diagram of member having maximum vertical deflection for serviceability load combinations.



Deflection diagram of beam B3 (DL+LL)

Maximum vertical displacement of beam in Y direction= 2.798 mm

Permissible vertical deflection = Span / 360 = 3600/360 = 10 mm

Actual maximum vertical deflection of beam = 2.798 mm <10 mm (Hence, OK)

Maximum displacement of Column in X direction= 0.754 mm

Permissible deflection = Height / 300 = 3150/300 = 10.5 mm

Actual maximum deflection of Column = 0.754 mm <10.5 mm (Hence, OK)

8.5. STAAD design output results

STAAD output results for BEAM B3:

```
    1 ST  UB356X127X33 (BRITISH SECTIONS)
                        PASS     EC-6.3.3-662       0.327         3
                    15.65 C         0.00         -51.49        2.10
=======================================================================
 MATERIAL DATA
    Grade of steel           =  S 355
    Modulus of elasticity    =  205 kN/mm2
    Design Strength  (py)    =  355  N/mm2

 SECTION PROPERTIES (units - cm)
    Member Length =    360.00
    Gross Area =   42.10          Net Area =   42.10

                                   z-axis          y-axis
    Moment of inertia        :     8250.001        280.000
    Plastic modulus          :      543.000         70.200
    Elastic modulus          :      472.779         44.657
    Shear Area               :       19.186         23.026
    Radius of gyration       :       13.999          2.579
    Effective Length         :      360.000        360.000

 DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
    Section Class            :   CLASS 1
    Squash Load              :   1494.55
    Axial force/Squash load  :     0.010
    GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                   z-axis          y-axis
    Slenderness ratio (KL/r) :        25.7          139.6
    Compression Capacity     :      1447.9          367.0
    Tension Capacity         :      1485.3         1485.3
    Moment Capacity          :       192.8           24.9
    Reduced Moment Capacity  :       192.8           24.9
    Shear Capacity           :       393.2          471.9

 BUCKLING CALCULATIONS (units - kN,m)
    Lateral Torsional Buckling Moment       MB =  179.8
    co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 1.000
    Elastic Critical Moment for LTB,              Mcr    =   713.6
    Critical Load For Torsional Buckling,         NcrT   =  8446.5
    Critical Load For Torsional-Flexural Buckling, NcrTF =  8446.5

 SHEAR BUCKLING CALCULATIONS [EN1993-1-5] (units - kN,m)
    Stiffeners not provided.
    Design Resistance for Shear Buckling,         VbRd   =   430.5
```

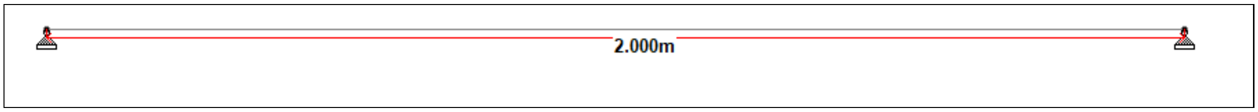
STAAD output results for Column & pedestal:

```
     2 ST  UB356X127X33 (BRITISH SECTIONS)
                    PASS     EC-6.3.3-662       0.971         3
                91.90 C          0.00         -56.20        3.15
=======================================================================
  MATERIAL DATA
     Grade of steel            =  S 235
     Modulus of elasticity     =  205 kN/mm2
     Design Strength  (py)     =  235  N/mm2

  SECTION PROPERTIES (units - cm)
     Member Length =    315.00
     Gross Area =   42.10          Net Area =   42.10

                                     z-axis          y-axis
     Moment of inertia         :     8250.001        280.000
     Plastic modulus           :      543.000         70.200
     Elastic modulus           :      472.779         44.657
     Shear Area                :       19.186         23.026
     Radius of gyration        :       13.999          2.579
     Effective Length          :      315.000        315.000

  DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
     Section Class             :   CLASS 1
     Squash Load               :    989.35
     Axial force/Squash load   :     0.093
     GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                     z-axis          y-axis
     Slenderness ratio (KL/r) :        22.5          122.1
     Compression Capacity     :       980.7          422.0
     Tension Capacity         :       989.4          989.4
     Moment Capacity          :       127.6           16.5
     Reduced Moment Capacity  :       127.6           16.5
     Shear Capacity           :       260.3          312.4

  BUCKLING CALCULATIONS (units - kN,m)
     Lateral Torsional Buckling Moment       MB =    67.2
     co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 3.150
     Elastic Critical Moment for LTB,              Mcr    =    89.2
     Critical Load For Torsional Buckling,         NcrT   =  1158.9
     Critical Load For Torsional-Flexural Buckling, NcrTF =  1158.9
```

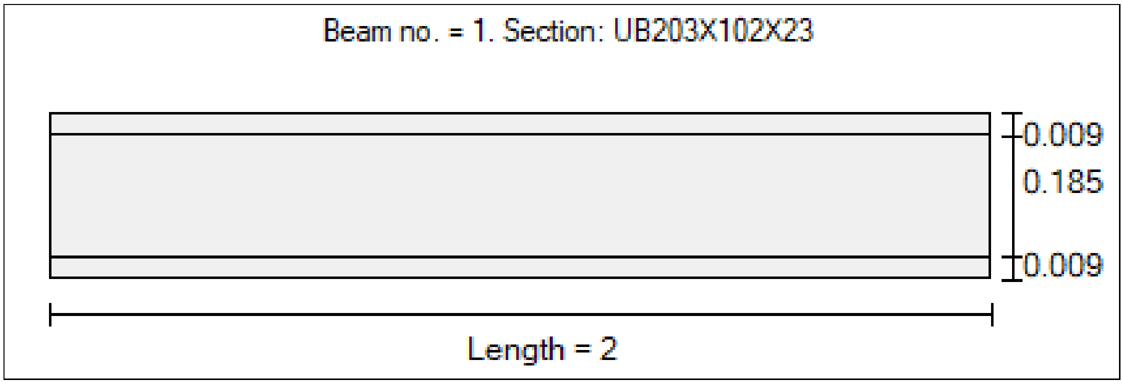
```
        C O L U M N   N O.     3  D E S I G N   R E S U L T S

      M20                    Fe500 (Main)                Fe415 (Sec.)

LENGTH:  1200.0 mm   CROSS SECTION:  300.0 mm X  600.0 mm  COVER: 40.0 mm

** GUIDING LOAD CASE:    3 END JOINT:     4  SHORT COLUMN


REQD. STEEL AREA   :       173.53 Sq.mm.
REQD. CONCRETE AREA:     21691.32 Sq.mm.
MAIN REINFORCEMENT : Provide   8 - 12 dia. (0.50%,    904.78 Sq.mm.)
                     (Equally distributed)
TIE REINFORCEMENT  : Provide  8 mm dia. rectangular ties @ 190 mm c/c

SECTION CAPACITY BASED ON REINFORCEMENT REQUIRED (KNS-MET)
----------------------------------------------------------
Puz :   1683.51   Muz1 :     49.43   Muy1 :     24.17

INTERACTION RATIO: 0.94 (as per Cl. 39.6, IS456:2000)

SECTION CAPACITY BASED ON REINFORCEMENT PROVIDED (KNS-MET)
----------------------------------------------------------
WORST LOAD CASE:     3
END JOINT:     4 Puz :   1951.15   Muz :    127.55   Muy :     57.42   IR: 0.37
============================================================================
```

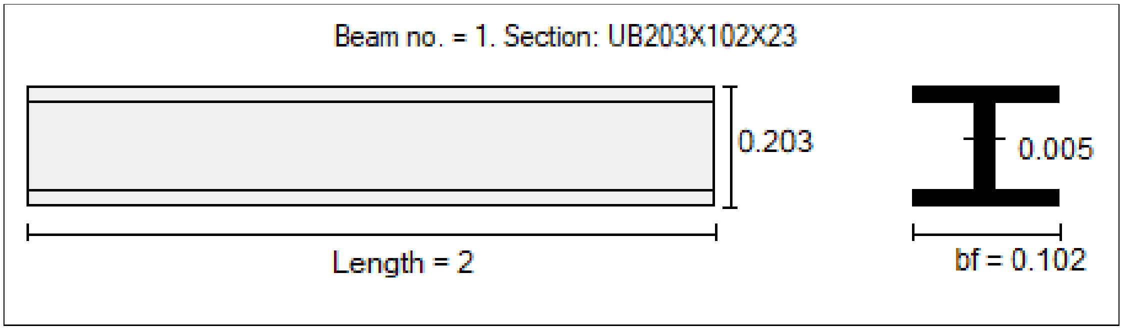
9. ANALYSIS AND DESIGN OF BEAM B4:

Refer below image showing view of beam B4 modeled in STAAD Pro:



9.1. Member Properties:





9.2. Load on Beam B4

Calculation Dead and Live load on steel beam,

Dead load

1. Self-weight of Slab of hollow core slab,

   = (0.15 x 25kN/m^2 + 0.075 x 22kN/m^2 + 0.15 x 0.45kN/m^2)

   = 5.5kN/m^2

   Load on Beam B4

   = (5.5 x 4.8)/2

   = 13.2kN/m^2

2. 200mm thick Wall load,

   = (0.2 x 2.6 x 20 kN/m^2)

   = 10kN/m

3. DL of roof slab,

   = (Self-weight of pitched roof + Natural slate finish to roof)

   = (0.35 + 0.2)

   = 0.55kN/m^2

   Load on Beam B4

   = (0.55 x 4.8)/2

   = 1.32kN/m

Hence, Total Dead load on Beam B4 = 13.2kN/m + 10kN/m + 1.32kN/m

= 24.52kN/m

Live Load

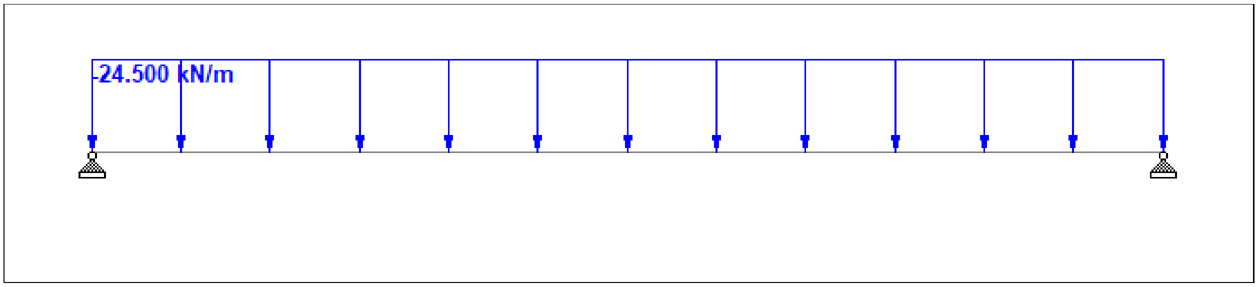
1. LL of GF slab = $2kN/m^2$

LL on Beam B4 = (2 x 4.8)/2 = 4.8kN/m

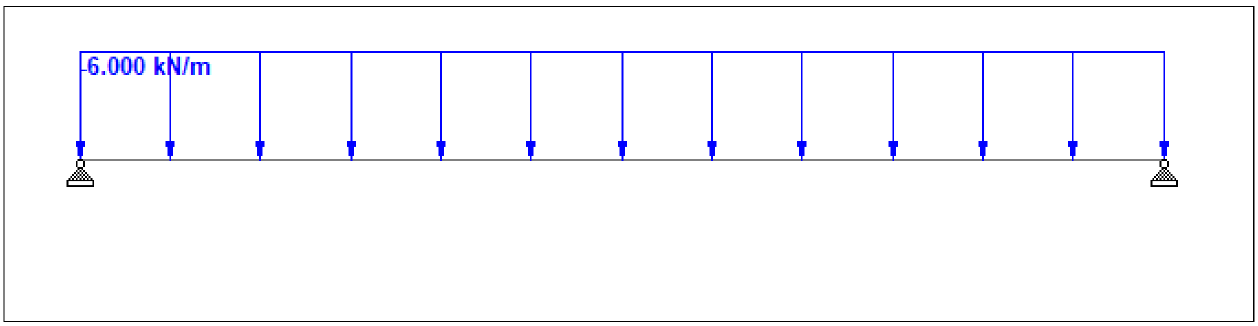
2. LL of Roof = $0.5kN/m^2$

LL on Beam B4 = (0.5 x 4.8)/2 = 1.2kN/m

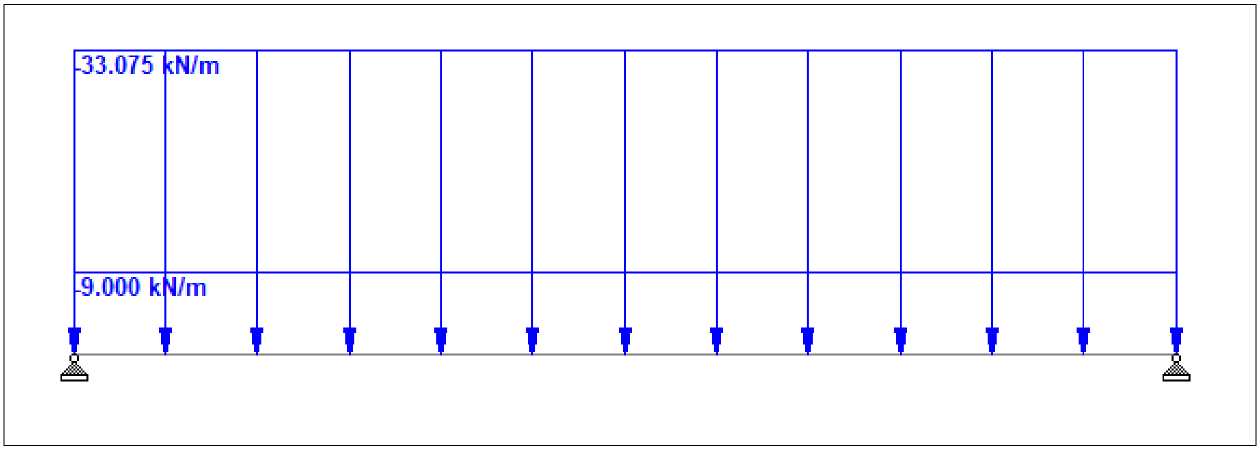
Hence, Total LL on Beam B4 = 4.8kN/m + 1.2kN/m = 6.0kN/m



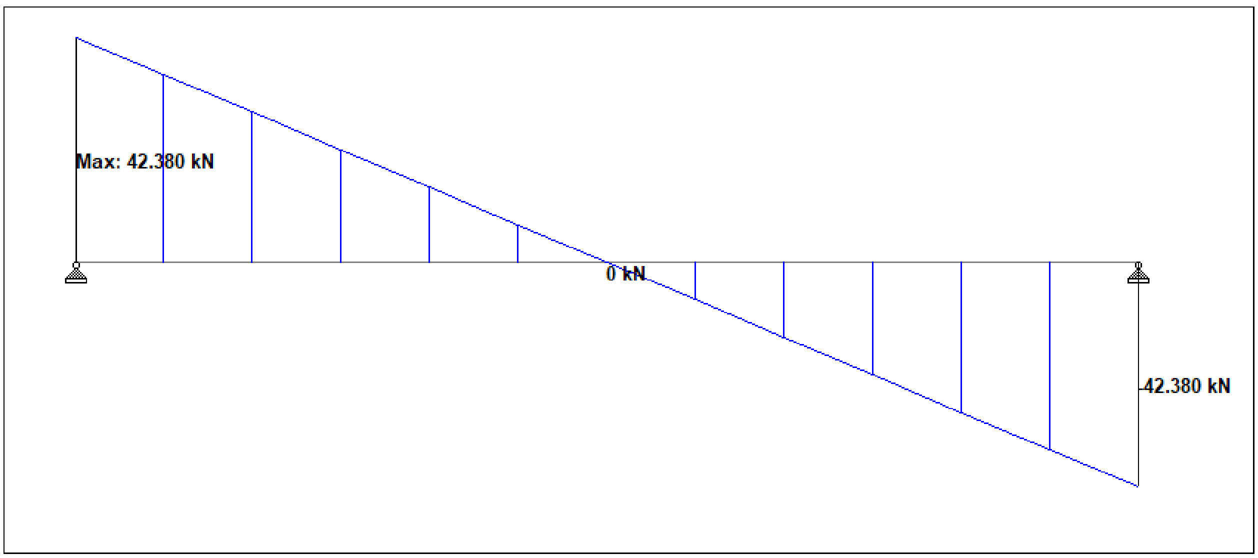
Dead load on beam B4



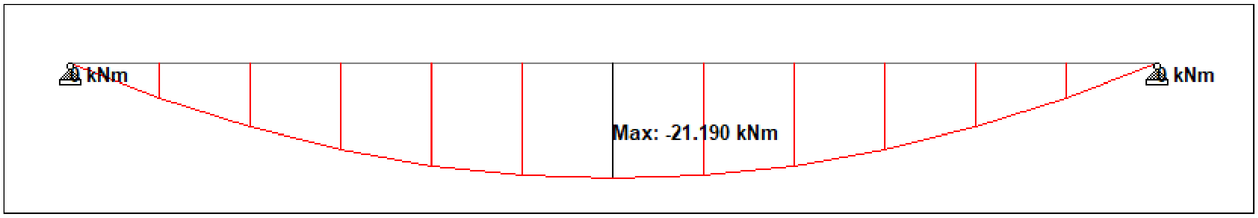
Live load on beam B4



Loading Diagram (1.35DL+1.5LL)

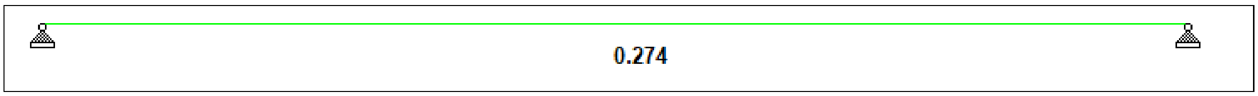


Shear force diagram (1.35DL+1.5LL)



Bending Moment Diagram (1.35DL+1.5LL)

9.3. Utilization ratio check:

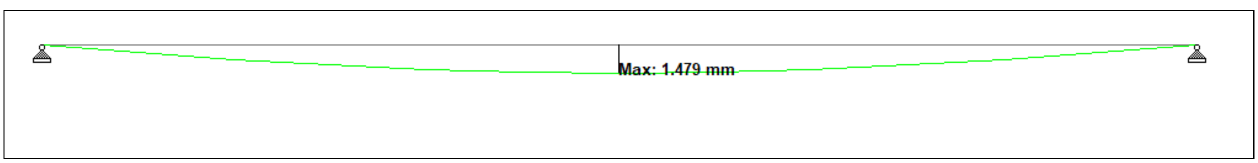


Utilization ratio of Beam B4

Above shows that failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors. It can be seen from image that all members are green. Hence, all members have passed in design.

9.4. Deflection check:

Below image shows displacement diagram of member having maximum vertical deflection for serviceability load combinations.



Deflection diagram of beam B4 (DL+LL)

Maximum vertical displacement of beam in Y direction= 1.479 mm

Permissible vertical deflection = Span / 360 = 2000/360 = 5.55 mm

Actual maximum vertical deflection of beam = 1.479 mm <5.55 mm (Hence, OK)

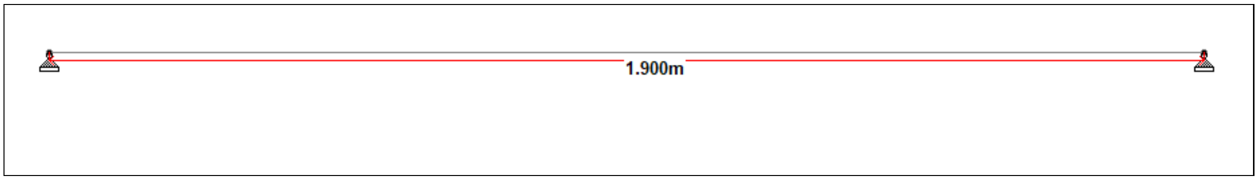
9.5. STAAD design output results

STAAD output results for BEAM B4:

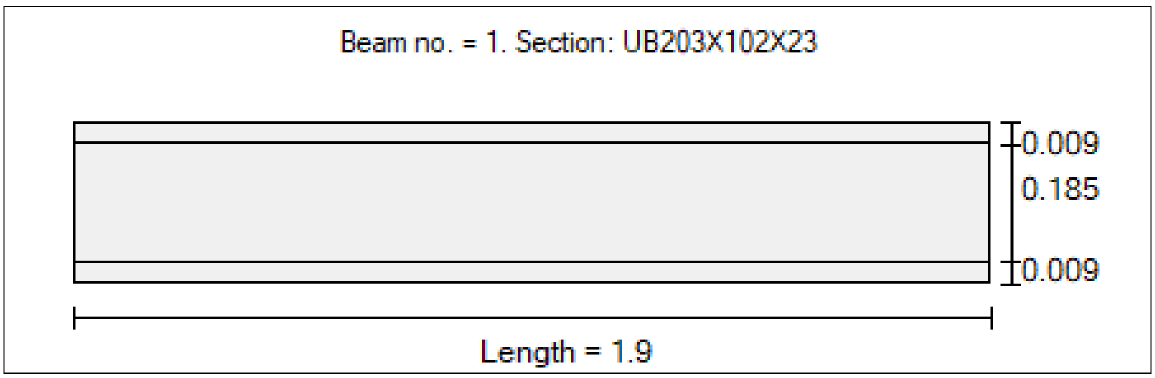
```
     1 ST  UB203X102X23 (BRITISH SECTIONS)
                          PASS     EC-6.3.2 LTB       0.274         3
                       0.00           0.00         -21.19        1.00
=======================================================================
  MATERIAL DATA
     Grade of steel           =  S 355
     Modulus of elasticity    =  205 kN/mm2
     Design Strength  (py)    =  355  N/mm2

  SECTION PROPERTIES (units - cm)
     Member Length =    200.00
     Gross Area =   29.40          Net Area =   29.40

                                    z-axis          y-axis
     Moment of inertia        :     2100.000        164.000
     Plastic modulus          :      234.000         49.700
     Elastic modulus          :      206.693         32.220
     Shear Area               :       17.041         12.381
     Radius of gyration       :        8.452          2.362
     Effective Length         :      200.000        200.000

  DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
     Section Class            :   CLASS 1
     Squash Load              :   1043.70
     Axial force/Squash load  :     0.000
     GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                    z-axis          y-axis
     Slenderness ratio (KL/r) :        23.7            84.7
     Compression Capacity     :      1017.8           553.4
     Tension Capacity         :      1037.2          1037.2
     Moment Capacity          :        83.1            17.6
     Reduced Moment Capacity  :        83.1            17.6
     Shear Capacity           :       349.3           253.8

  BUCKLING CALCULATIONS (units - kN,m)
     Lateral Torsional Buckling Moment       MB =   77.3
     co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 1.000
     Elastic Critical Moment for LTB,               Mcr    =   257.1
     Critical Load For Torsional Buckling,          NcrT   =  4768.8
     Critical Load For Torsional-Flexural Buckling, NcrTF =  4768.8
```

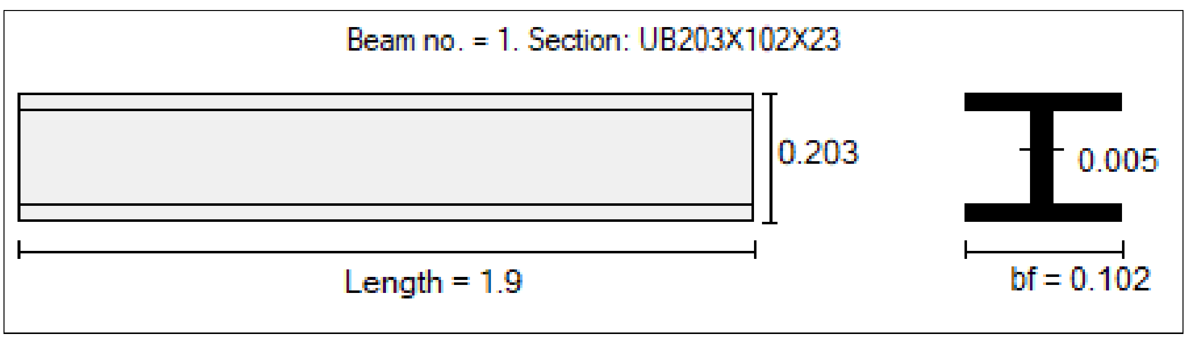
10. ANALYSIS AND DESIGN OF BEAM B5:

Refer below image showing view of beam B5 modeled in STAAD Pro:



10.1. Member Properties:





10.2. Load on Beam B5

Calculation Dead and Live load on steel beam,

Dead load

DL of roof slab,

= (Self-weight of pitched roof + Natural slate finish to roof)

= (0.35 + 0.2)

= 0.55kN/m^2

Load on Beam B5

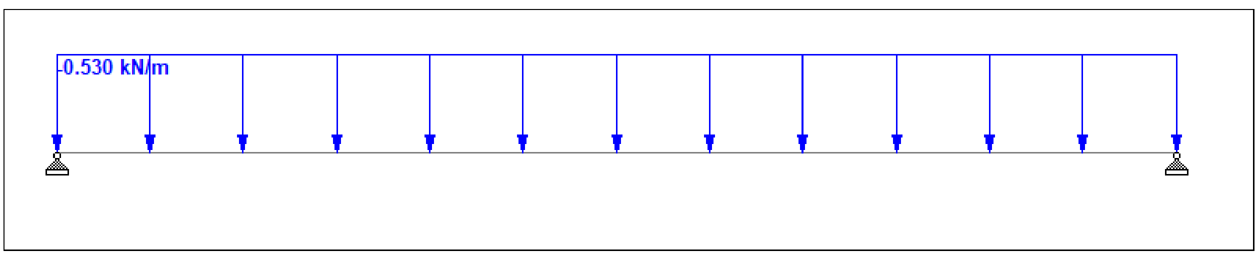
= (0.55 x 1.9)/2

= 0.53kN/m

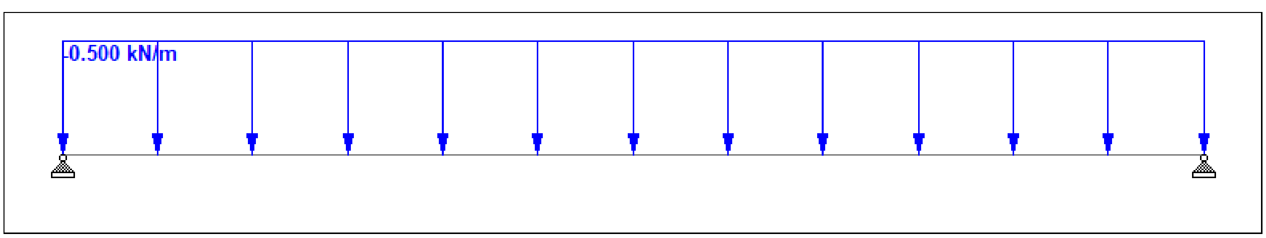
Live Load

LL of Roof = 0.5kN/m^2

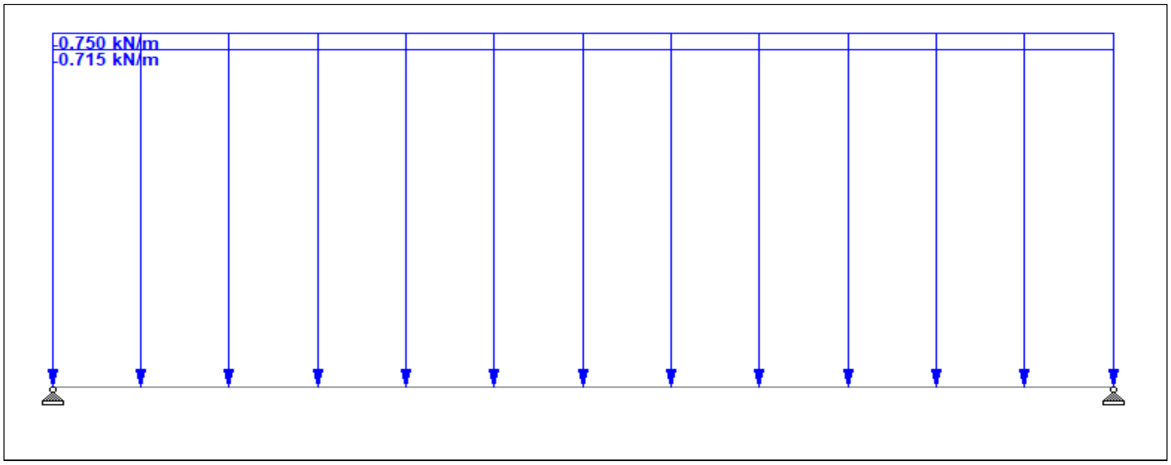
LL on Beam B4 = (0.5 x 1.9)/2 = 0.5kN/m



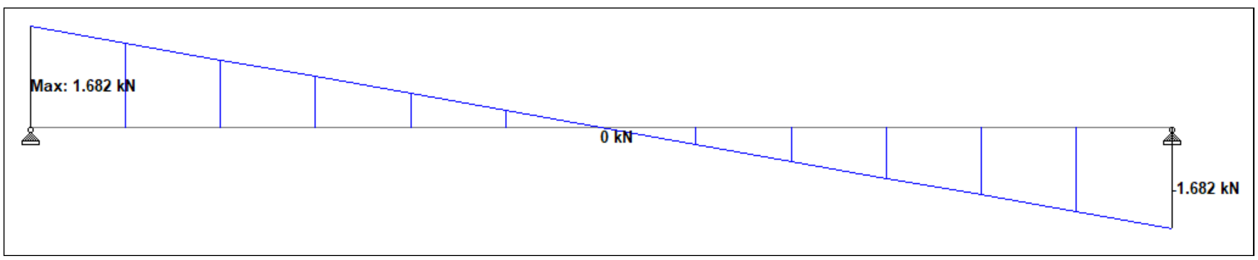
Dead load on beam B5



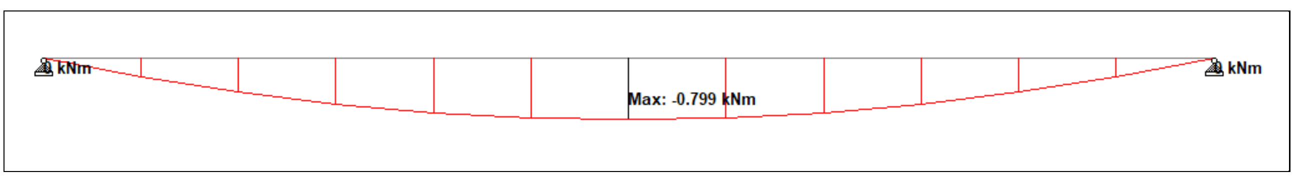
Live load on beam B5



Loading Diagram (1.35DL+1.5LL)

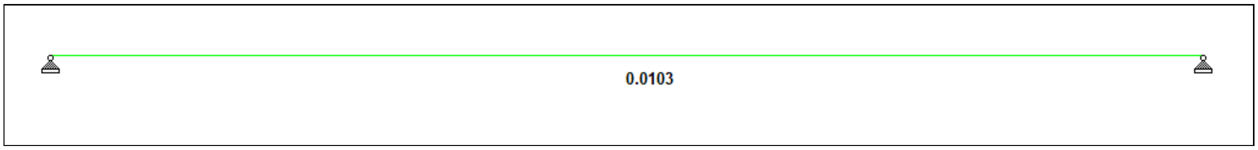


Shear force diagram (1.35DL+1.5LL)



Bending Moment Diagram (1.35DL+1.5LL)

10.3. Utilization ratio check:

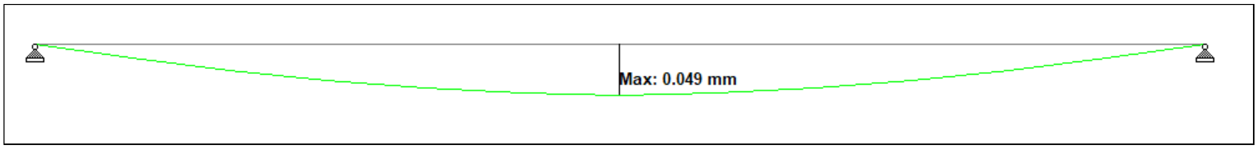


Utilization ratio of Beam B5

Above shows that failed members (i.e., members having utility ratio more than 1) will be highlighted with red colors. It can be seen from image that all members are green. Hence, all members have passed in design.

10.4. Deflection check:

Below image shows displacement diagram of member having maximum vertical deflection for serviceability load combinations.



Deflection diagram of beam B5 (DL+LL)

Maximum vertical displacement of beam in Y direction= 0.049 mm

Permissible vertical deflection = Span / 360 = 1900/360 = 5.277 mm

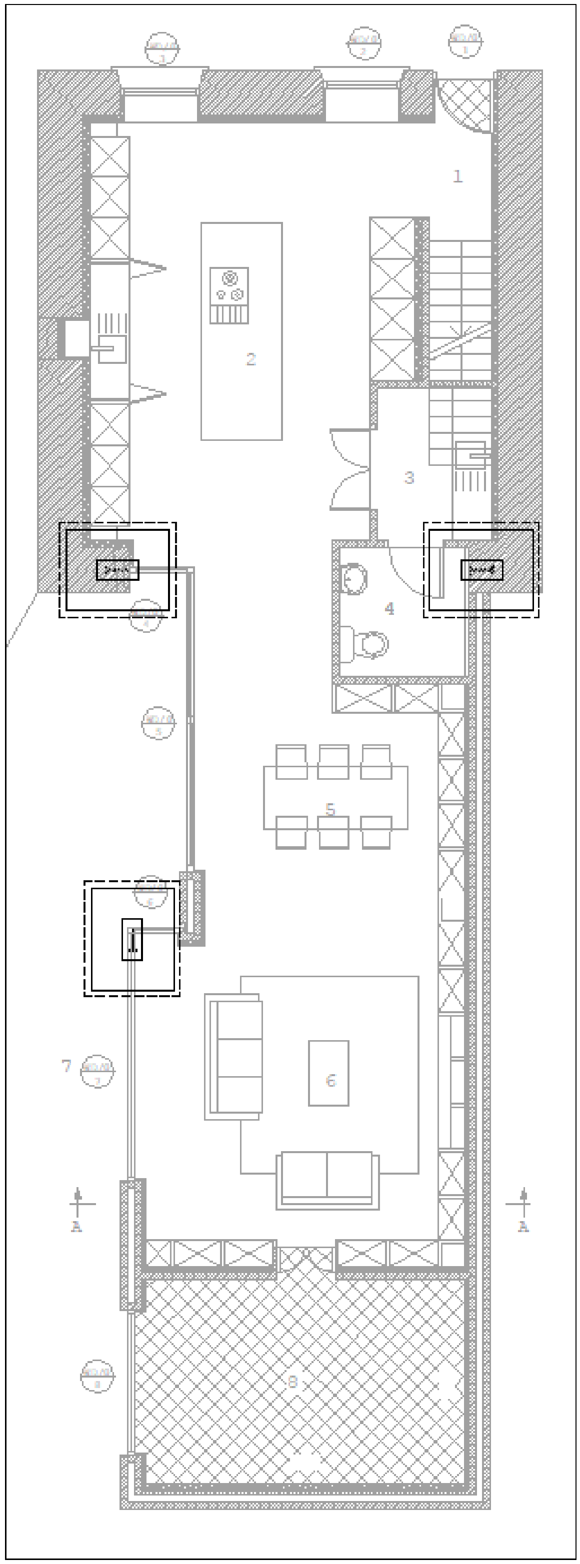
Actual maximum vertical deflection of beam = 0.049 mm <5.277 mm (Hence, OK)

10.5. STAAD design output results

STAAD output results for BEAM B5:

```
     1 ST  UB203X102X23 (BRITISH SECTIONS)
                      PASS     EC-6.3.2 LTB       0.010         3
                   0.00            0.00          -0.80        0.95
=======================================================================
  MATERIAL DATA
     Grade of steel            =  S 355
     Modulus of elasticity     =  205 kN/mm2
     Design Strength  (py)     =  355  N/mm2

  SECTION PROPERTIES (units - cm)
     Member Length =    190.00
     Gross Area =   29.40          Net Area =   29.40

                                  z-axis          y-axis
     Moment of inertia        :     2100.000        164.000
     Plastic modulus          :      234.000         49.700
     Elastic modulus          :      206.693         32.220
     Shear Area               :       17.041         12.381
     Radius of gyration       :        8.452          2.362
     Effective Length         :      190.000        190.000

  DESIGN DATA (units - kN,m)   EUROCODE NO.3 /2005
     Section Class            :   CLASS 1
     Squash Load              :   1043.70
     Axial force/Squash load  :     0.000
     GM0 :  1.00          GM1 :  1.00          GM2 :  1.25

                                  z-axis          y-axis
     Slenderness ratio (KL/r) :        22.5          80.4
     Compression Capacity     :      1021.6         588.4
     Tension Capacity         :      1037.2        1037.2
     Moment Capacity          :        83.1          17.6
     Reduced Moment Capacity  :        83.1          17.6
     Shear Capacity           :       349.3         253.8

  BUCKLING CALCULATIONS (units - kN,m)
     Lateral Torsional Buckling Moment       MB =   77.3
     co-efficients C1 & K : C1 =1.132 K =1.0, Effective Length= 1.000
     Elastic Critical Moment for LTB,              Mcr   =   257.1
     Critical Load For Torsional Buckling,         NcrT  =  4768.8
     Critical Load For Torsional-Flexural Buckling, NcrTF =  4768.8
```

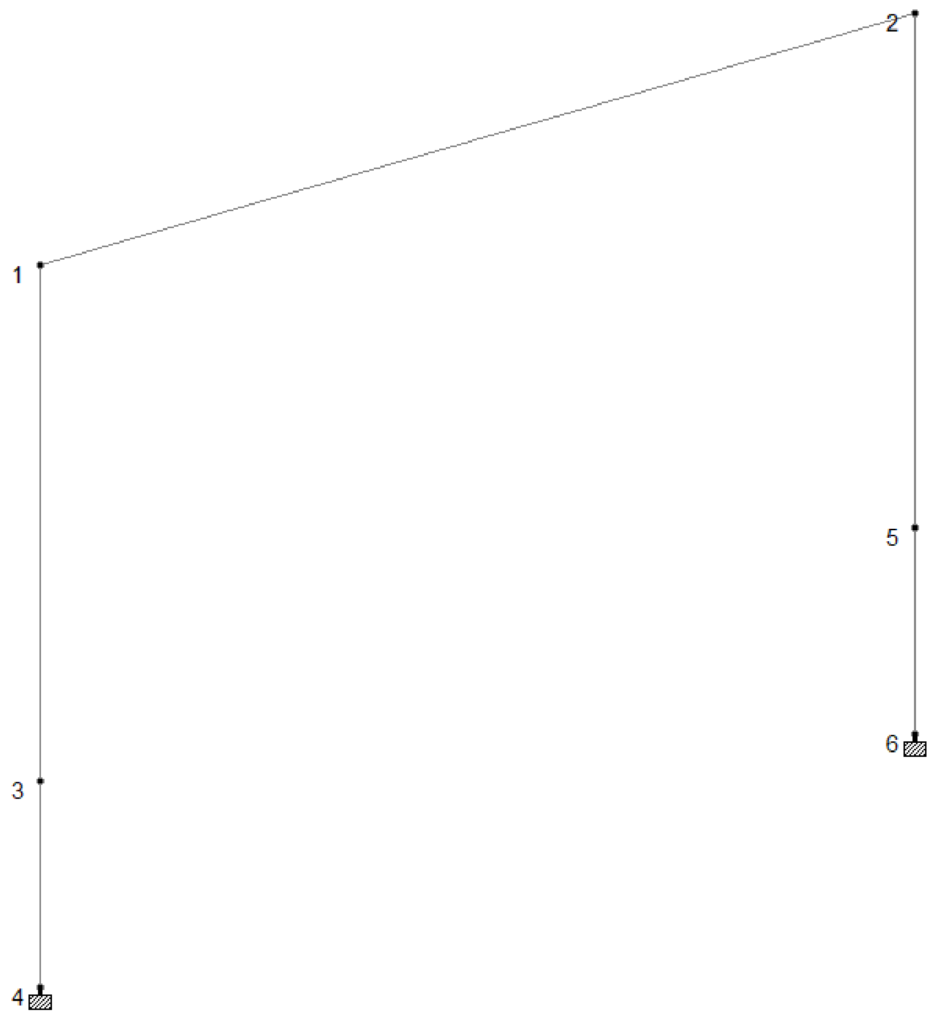
11. Footing Design



Footing layout

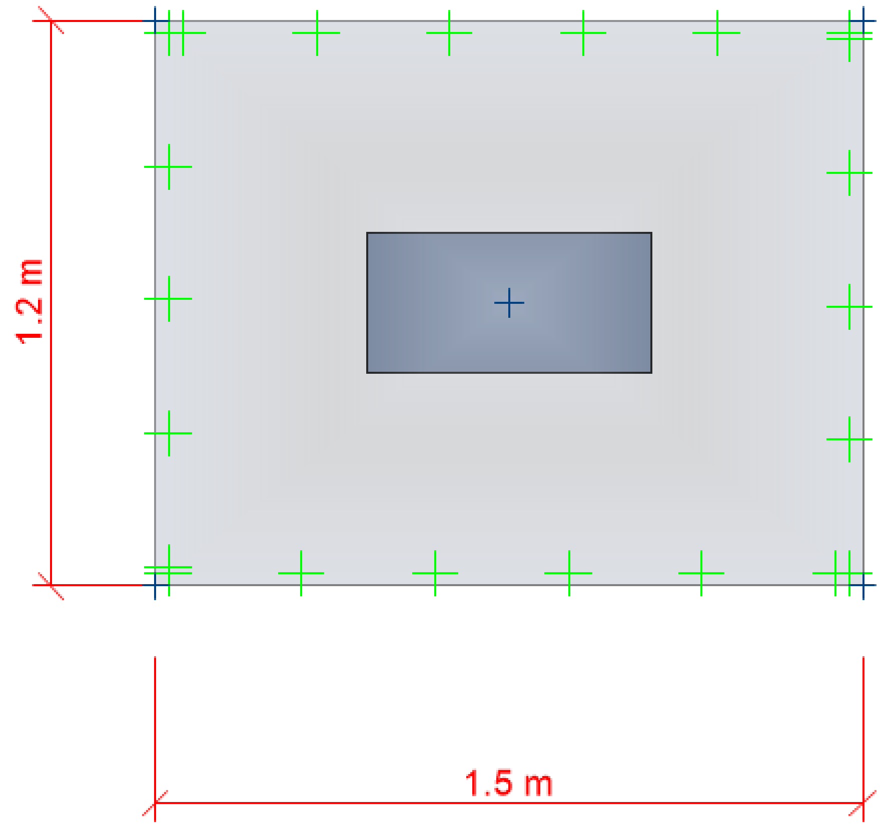
Footing is design for conservative reaction of column over supported beam B1.

- Support reaction



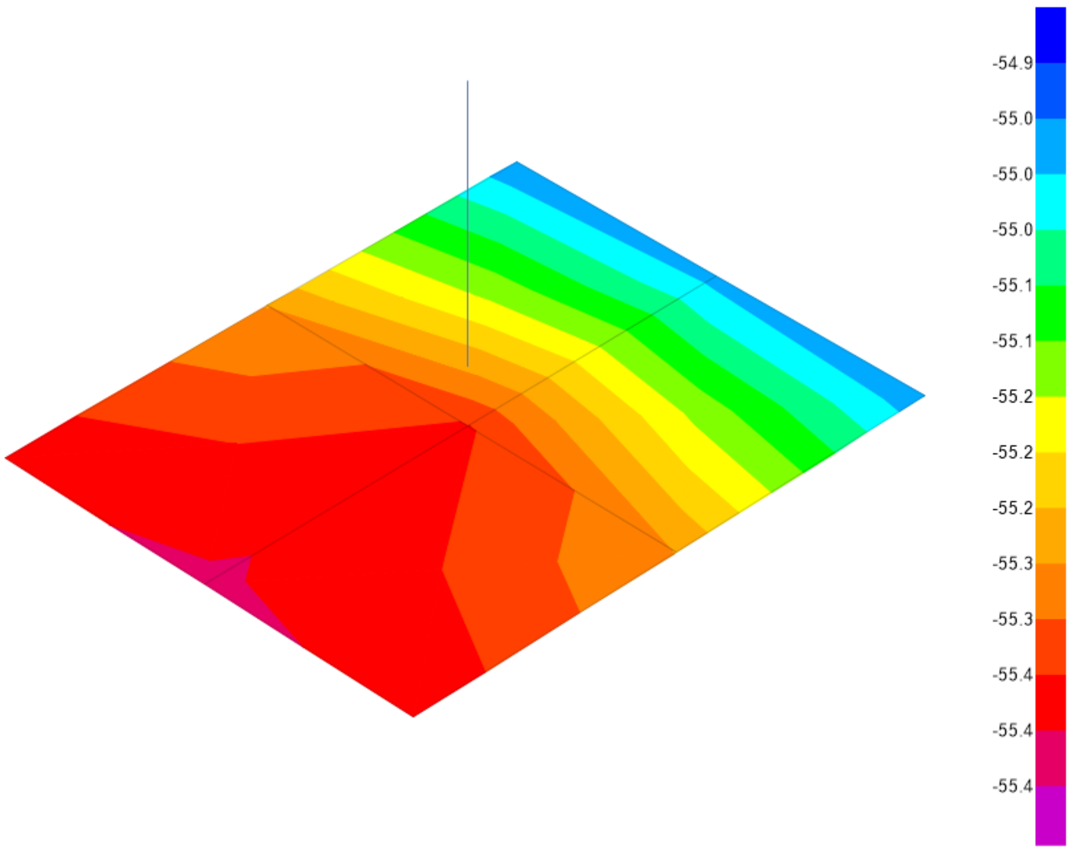
Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
4	4	17.576	81.351	0	0	0	-30.742
6	4	-17.576	50.501	0	0	0	36.876

11.1. Footing model in SAFE



Plan view of footing F1

11.2. Pressure check for F1

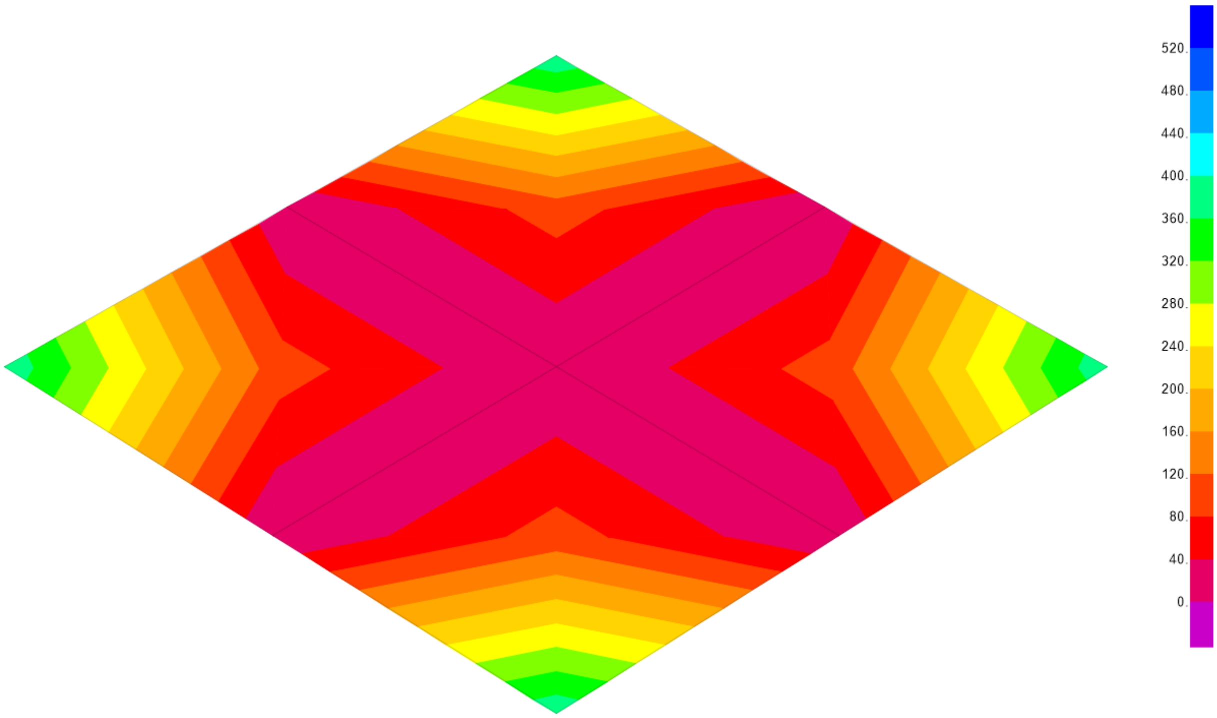


SBC considered = 150 kN/m^2

Max base pressure = 55 kN/m^2

Max Pressure < SBC….. Hence, OK

11.3. Top & Bottom Reinforcement for F1

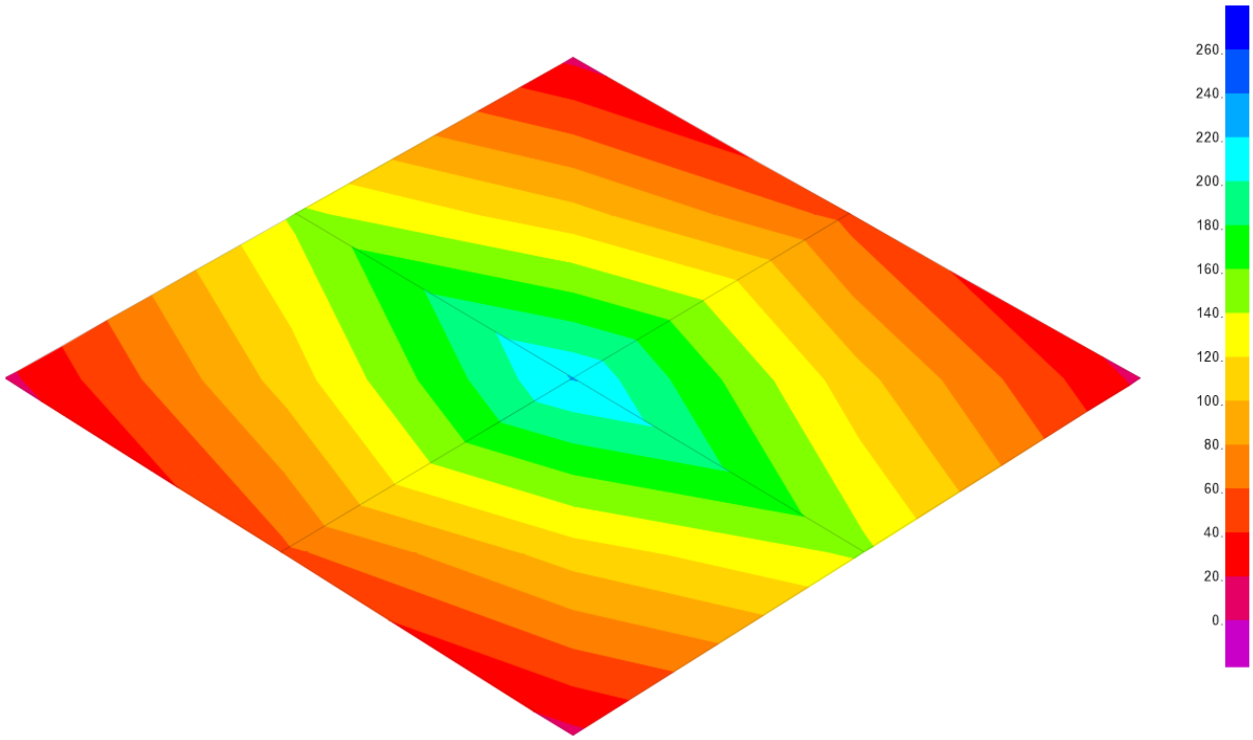


Top R/F for F1

Required area of top reinforcement = 80 mm^2/m

Provided area of top reinforcement = 402 mm^2/m

Required R/F < Provided R/F….. Hence, OK



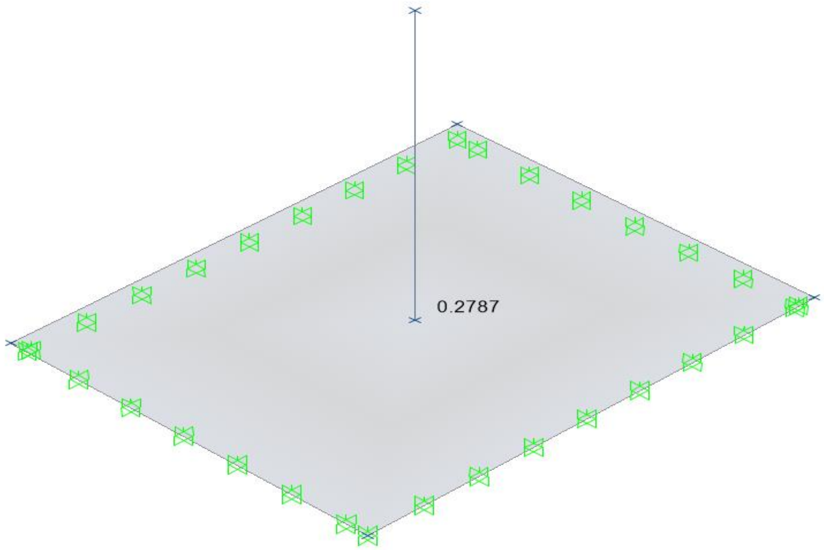
Bottom R/F for F1

Required area of bottom reinforcement = 390 mm^2/m

Provided area of bottom reinforcement = 402 mm^2/m

Required R/F < Provided R/F….. Hence, OK

11.4. Punching Check



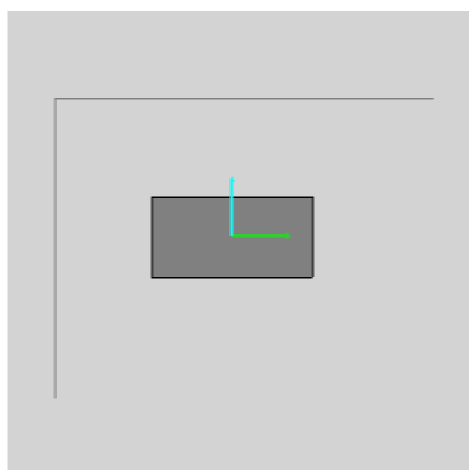
BS 8110-1997 Punching Shear Check & Design

Geometric Properties

Combination = 3.1.35DL+1.5LL
Point Label = 1
Column Shape = Rectangular
Column Location = Corner
Global X-Coordinate = 0 m
Global Y-Coordinate = 0 m

Column Punching Check

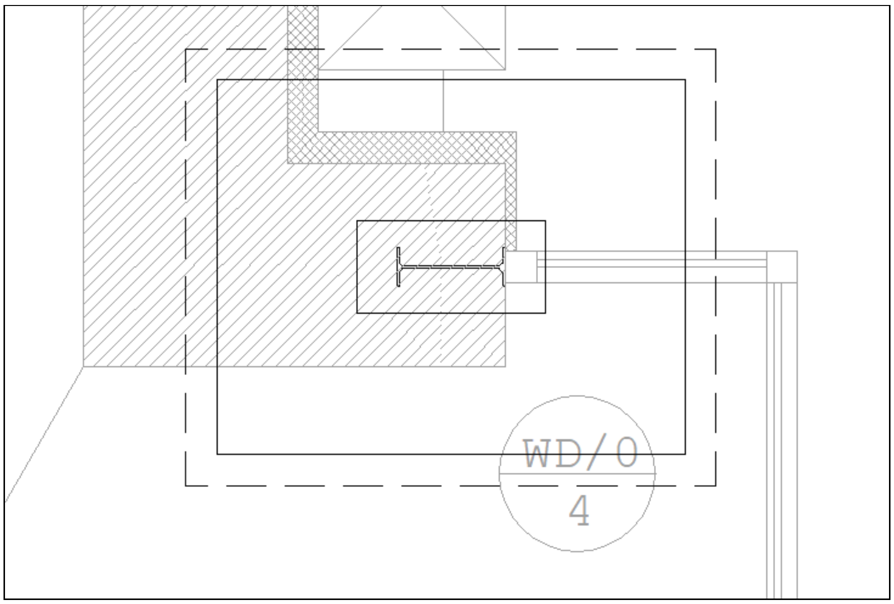
Avg. Eff. Slab Thickness = 240 mm
Eff. Punching Perimeter = 2520.2 mm
Cover = 60 mm
Conc. Comp. Strength = 25 N/mm2
Reinforcement Ratio = 0.0000
Section Width x-22 = 1410.1 mm
Section Width x-33 = 1110.1 mm
Shear Force = -45.694 kN
Moment Mu2 = -1.7295 kN-m
Moment Mu3= -5.3223 kN-m
Max Design Shear Stress = 0.106323 N/mm2
Conc. Shear Stress Capacity = 0.381545 N/mm2
Punching Shear Ratio = 0.28



Column Punching Perimeter

11.5. Provided foundation details

F1 footing in drawing:



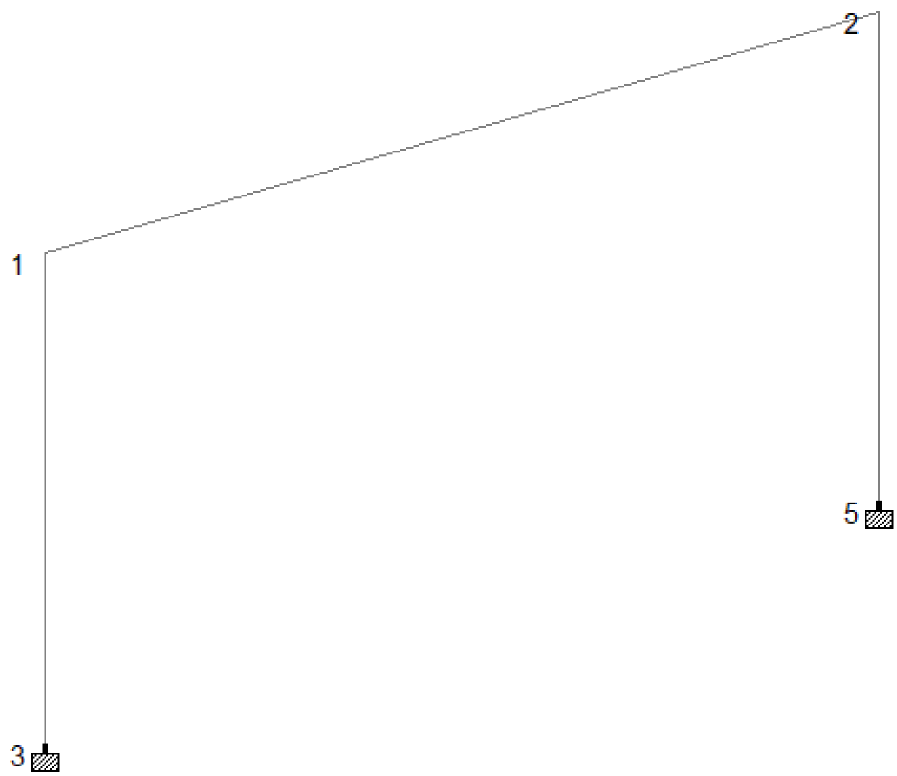
Foundation schedule:

SCHEDULE OF FOOTING (PAD FOOTING)									
NO. FOOTING	FOOTING GRADE	FOOTING SIZE R.C.C.		FOOTING DEPTH	BOTTOM REINFORCEMENT		TOP REINFORCEMENT		PEDESTAL SIZE
		B	L	D	ALONG SHORT SPAN	ALONG LONG SPAN	ALONG SHORT SPAN	ALONG LONG SPAN	
F1	C25	1200	1500	300	10T@150 C/C	10T@150 C/C	10T@150 C/C	10T@150 C/C	300 x 600

12. Connection design

12.1. Base plate design for Column C1 – UB 356x127x33

- Support reaction for base plate design



Node	L/C	Force-X kN	Force-Y kN	Force-Z kN	Moment-X kNm	Moment-Y kNm	Moment-Z kNm
3	3	26.932	101.649	0	0	0	-20.367
5	3	-26.932	58.999	0	0	0	29.22

Node	L/C	FX	FY	FZ	MX	MY	MZ
3	1.35DL+1.5LL	26.932	-59	0	0	0	29.22
5	1.35DL+1.5LL	26.932	-101.649	0	0	0	20.367

Bolts in each row		3		
Lever arm		450	mm	
Pull/Push		45.260	kN	
Tensile force per bolt		22.630		
Tensile for due to uplift		16.94150	kN	
Total tensile force in bolt		39.57	kN	
Plate size		250	w	mm
		550	L	mm
Thickness		16		mm
Conc grade		25	N/mm^2	
Conc bearing check	P/Area of base plate			
Conc bearing check		-0.43	N/mm^2	
Permissible stress in conc		0.45 fck		
σ in concrete		11.25	N/mm^2	Hence ok
Thickness of Plate due to Bolt Tension				
Anchor Bolt dia		16	mm	
Grade of Bolt		8.8		
Tensile capacity of Bolt		90.4	kN	
Shear capacity of Bolt		33.7	kN	
Combine Bolt check				

<u>Actual Tension</u> + <u>Actual Shear</u> < 1

Allowable Tension Allowable Shear

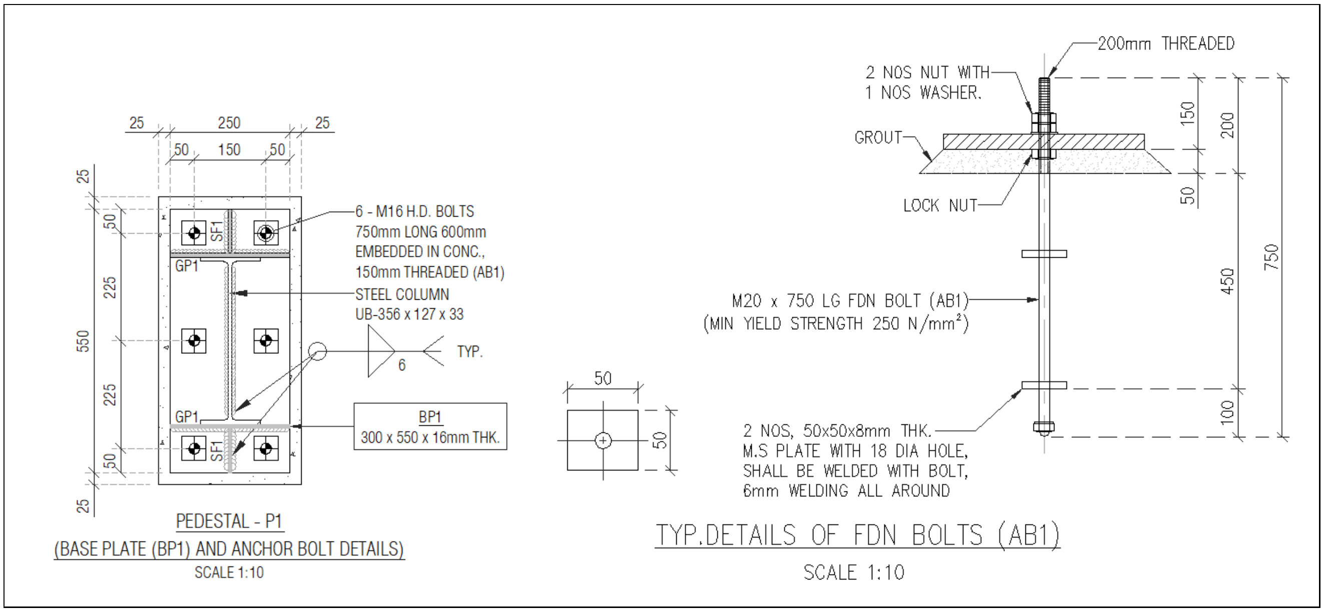
((39.5715/90.4)+(26.932/3/33.7) = 0.70 < 1 Hence ok

Total per Bolt force(tensile)	39.5715	kN		
L	150	mm		
Moment in Plate due to tensile force	5.94	kN.m		
M/Z = σ	139.1185547	N/mm^2		
Allowable σ	0.6 *355	N/mm^2		
Allowable σ	213	N/mm^2	Hence ok	

Check for Embedment depth of Anchor Bolts

Bond stress for concrete fck	25	=	1	
Required length of bolt =	Tension in bolt / (π x bolt dia)x(bond stress)	:		
			39.5715x 1000 /(π x16x1)	
		=	787.64928 mm	

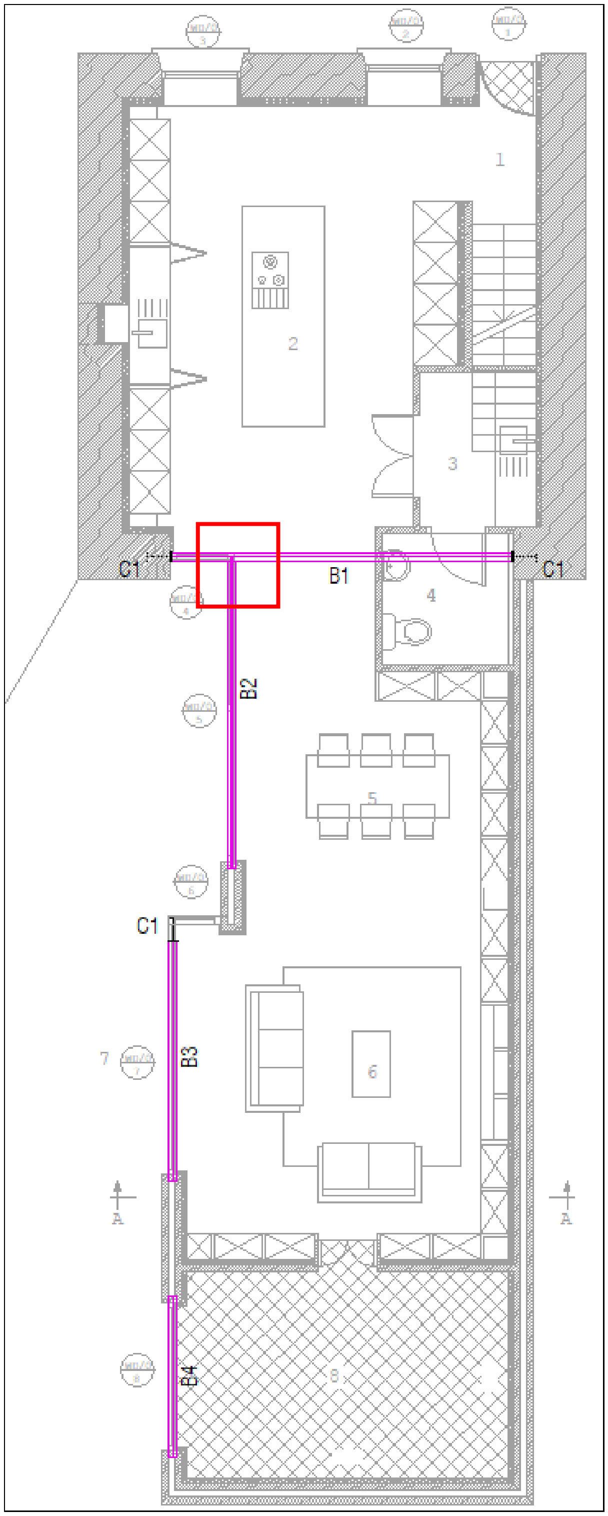
Using Plates for Anchorage in Anchor bolts at bottom				
Size of plate	50	mm		
	50	mm		
thickness of anchorage plate	8	mm		
Perimeter of anchorage plate	200	mm		
Required length of bolt =	39.5715x 1000 /200x1)			
	197.86	mm		
Using Anchor Bolt embed depth	600	mm		
Grout thickness	50	mm		
Projection from conc pedestal	90.8	~	150	mm
Total Anchor Bolt length	750	mm		



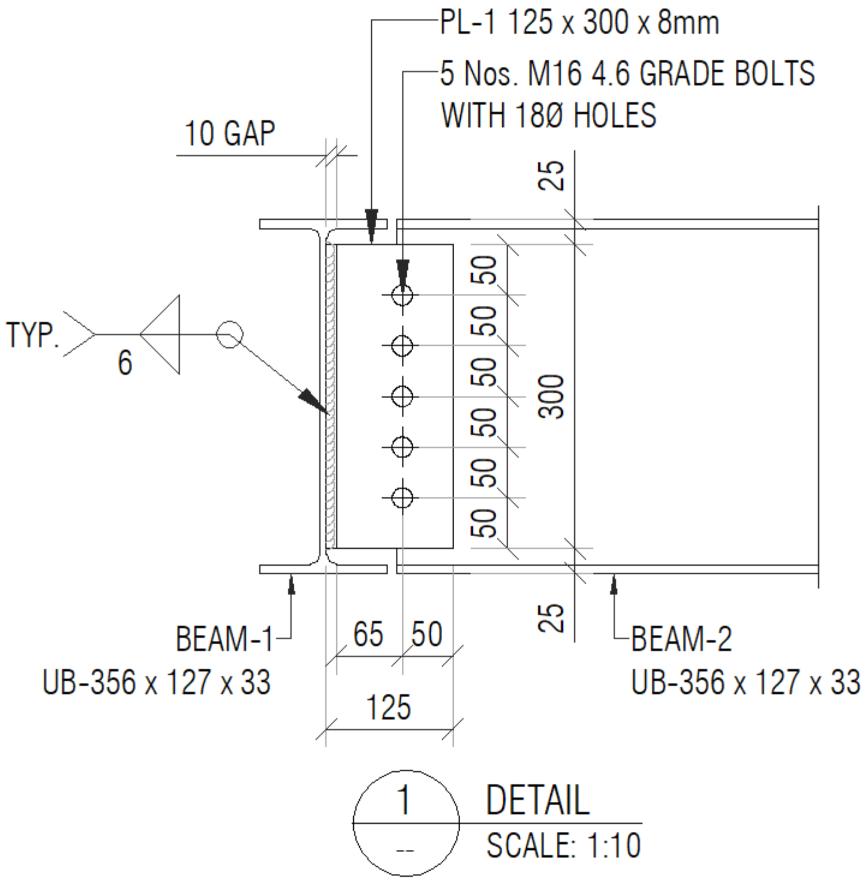
6 - M16 H.D. BOLTS
750mm LONG 600mm
EMBEDDED IN CONC.,
150mm THREADED (AB1)
STEEL COLUMN
UB-356 x 127 x 33
TYP.
BP1
300 x 550 x 16mm THK.
GP1
SF1
PEDESTAL - P1
(BASE PLATE (BP1) AND ANCHOR BOLT DETAILS)
SCALE 1:10
200mm THREADED
2 NOS NUT WITH
1 NOS WASHER.
GROUT
LOCK NUT
M20 x 750 LG FDN BOLT (AB1)
(MIN YIELD STRENGTH 250 N/mm²)
2 NOS, 50x50x8mm THK.
M.S PLATE WITH 18 DIA HOLE,
SHALL BE WELDED WITH BOLT,
6mm WELDING ALL AROUND
TYP.DETAILS OF FDN BOLTS (AB1)
SCALE 1:10

12.2. Beam B2 to Beam B1 Shear Connection

Refer below image showing beam B2 to B1 shear junction.

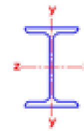


Ground floor slab layout



- Refer below image showing support reactions of Beam B2:

Typ. B2 member connection



Steel section used			UB 356X127X33	
Ultimate strength of steel	f_u	=	490	N/mm^2
yield strength of steel	f_y	=	355	N/mm^2
modulus of elasticity of steel	E	=	200000	N/mm^2
Depth of Section	D	=	349	mm
Width of Section	B_f	=	125	mm
Thick. of Web	t_w	=	9	mm
Thick. of Flange	t_f	=	6	mm
Notch Dimension	n	=	0	mm

Taking actual max shear force V_d = 63.075 kN

Factored shear capacity of beam = 1.4 X Shear capacity of beam

= 89 kN

Shear force	0
Axial force	0
Resultant force	0

Design of bolts

Grade of bolt = 4.6 f_{ub} 400 N/mm^2

Bolt diameter = 16 mm net area A_{nb} = 156.8 mm^2

Shear Capacity per Bolt (Vdsf) = $\mu_f x\ n_e\ x\ K_h\ x\ F_{o/\gamma\ mf}$

where, μ_f Coefficient of friction(slip factor) as specified in table 20 = 0.48

n_e nos. of effective interfaces in frictional resistance to slip 1

K_h 1 (taking fastners in clearance holes)

F_o minimum bolt tension(proof load) at installation given by

$A_{nb}\ x\ f_o$ where, A_{nb} = net area of bolt at threads

f_o = proof stress (0.7 f_{ub})

γ_{mf} = 1.1

∴ V_{dsf} = 19.16 kN

Bearing capacity of bolt 'V_{dpb}'

V_{dpb}	=	V_{npb} / γ_{mb}		d_o	18.00 mm	taking
	=	$2.5\ K_b.d.t.f_u / \gamma_{mb}$		e	30.00 mm	35 mm
K_b = Smaller of =				pitch, p	40.00 mm	45 mm
		$e / 3_{do}$,	=	0.56		hori bolt row dist
		$p / 3_{do}$-0.25,	=	0.49		0 mm
		f_{ub} / f_u	=	0.82		
				1		

∴ k_b = 0.49

∴ V_{dpb} = 69.25 kN

∴ Nos of Bolts required = (Factored shear capacity of beam / shear capacity per bolt)

= 4.6 say 5.0 nos.

using6 -M16 Dia. 4.6 grade bolts.

providing 1 Rows 5.0 Nos. each

∴ Connecting plate size

L= 489 mm

W= 195 mm

thickness= 8 mm

Block shear check

T_{db} = $[A_{vg} \times f_y / (\sqrt{3} \times 1.1) + (0.9 \times f_u A_{tn})/1.25]$ λ_{m0} 1.1

or λ_{m1} 1.25

T_{db} = $[0.9X f_u x A_{vn} / (\sqrt{3} \times 1.25) + f_y \times A_{tg} / 1.1]$

strenght of web	=	355 N/mm^2
Plate or web thickness	=	8 mm
Adding extra plate	=	0 mm
	=	8 mm
Net length of shear face	=	134.00 mm
Net length of tension face	=	8.00 mm

A_{vg} = 1720 mm^2

A_{vn} = 1072 mm^2

A_{tg} = 280 mm^2

A_{tn} = 64 mm^2

∴ T_{db} = 343.06 kN

Or T_{db} = 308.72 kN

value of T_{db} = 308.72 kN > 89 HENCE OK

∴

Using extra plate on face of web 8 mm

Now, plate or web thickness 8 mm

strenght of web	=	355 N/mm^2
Net length of shear face	=	218.00 mm
Net length of tension face	=	43.00 mm

A_{vg} = 1520 mm^2

A_{vn} = 1744 mm^2

A_{tg} = 240 mm^2

A_{tn} = 344 mm^2

T_{db} = 404.58 kN

Or T_{db} = 432.69 kN

value of T_{db} = 404.58 kN > 89 HENCE OK

Shear check for Connecting End plate

End Plate size = 489.0 mm 8 mm

Shear capacity of plate (V_d) $= \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = \frac{P}{t_t \times l_w}$

Where, f_{yw} = Yield strength of web = 355 Mpa

A_v = Shear area = 489 X 8 = 3912 mm2

Υ m0 = Partial safety factor against shear failure = 1.10

∴ V_d = 728.91 kN >=89 kN HENCE OK

∴ **Using End Plate489x195x8mm thick**

Weld check for Connecting End plate to Beam

Weld size = 6 mm

weld size is less than web thick, Hence ok

Design strength of fillet weld (f_{wd}) $= \frac{f_u}{\sqrt{3} \times \gamma_{mw}} = \frac{P}{t_t \times l_w}$

Where, f_u = Smaller of the ultimate stress of the weld or of the parent metal = 490 Mpa

Υ mw = Partial safety factor = 1.25

t_t = Effective throat thickness of weld = 0.7 X 6 = 4.2 mm

l_w = Effective length of weld

Length of weld required (l_w) = (P ×√3 ×[γ]_mw)/(t_t×f_u)

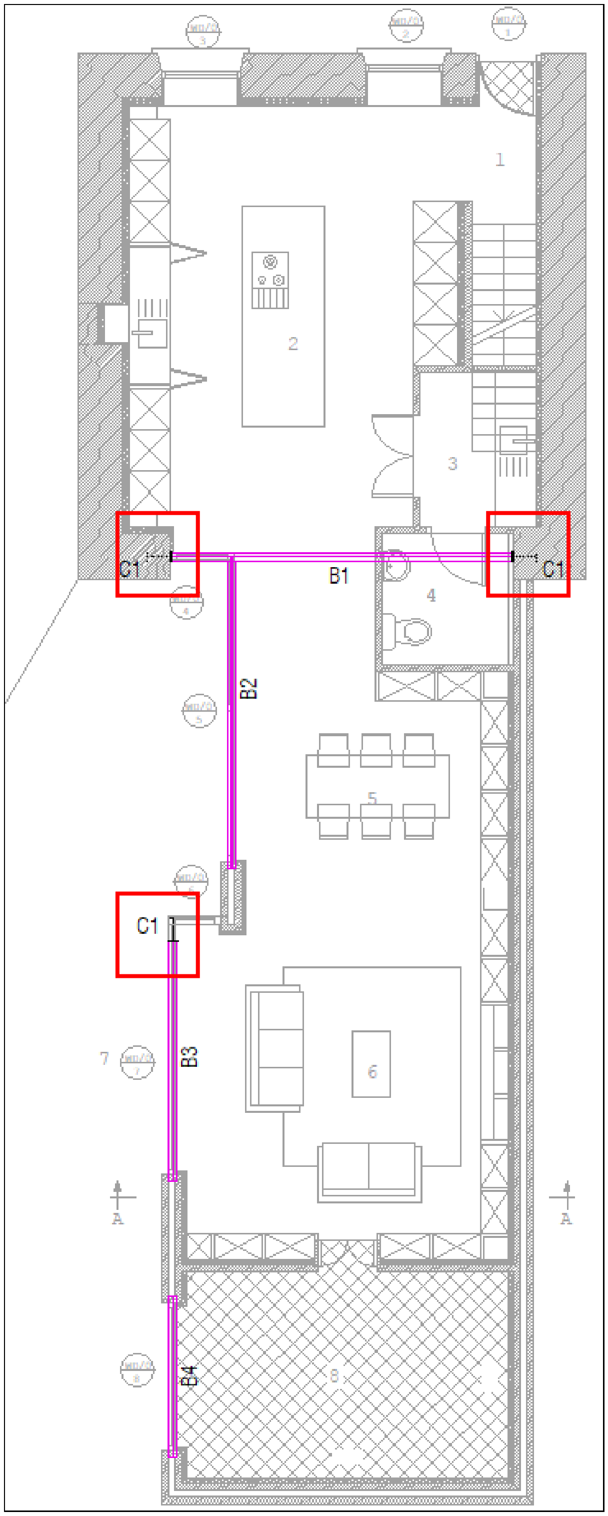
= 94 mm

Length of weld provided = 948 mm HENCE OK

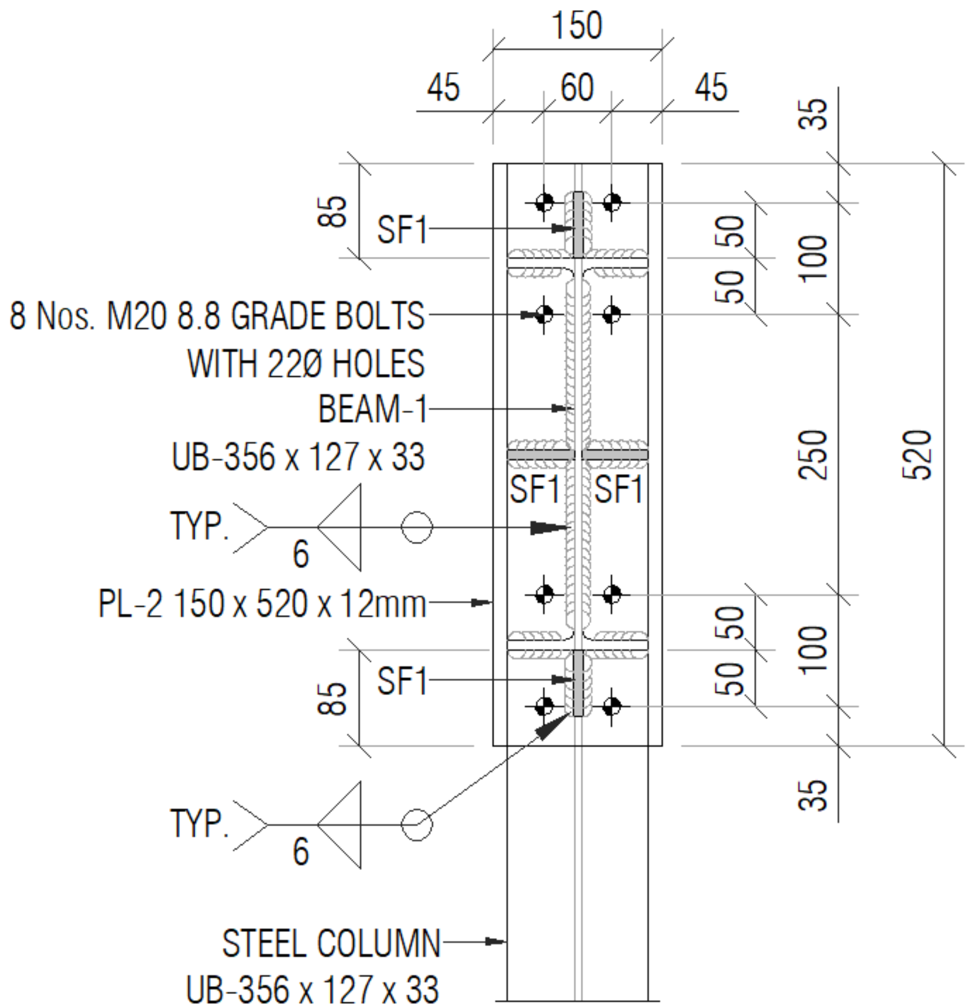
(Considering weld on both sides of fin plate)

12.3. Beam B1 & B3 to Column C1 Moment Connection

Refer below image showing beam column Moment junction.



Ground floor slab layout



2
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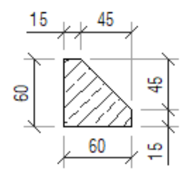
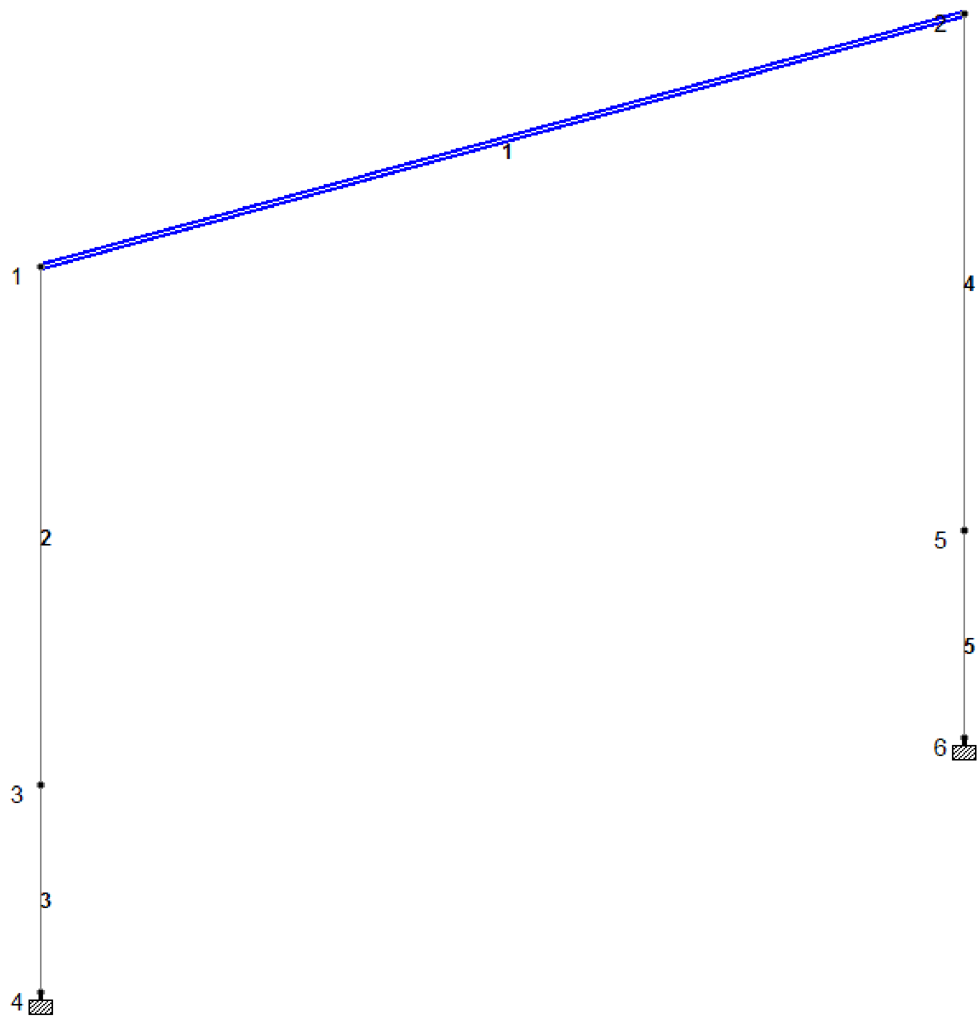
DETAIL
SCALE: 1:10



ENLARGED DETAIL FOR
STIFFENER PLATE- SF1-8mm THK.
SCALE 1:5

- Refer below image showing Beam B1 number and node number and beam End forces:



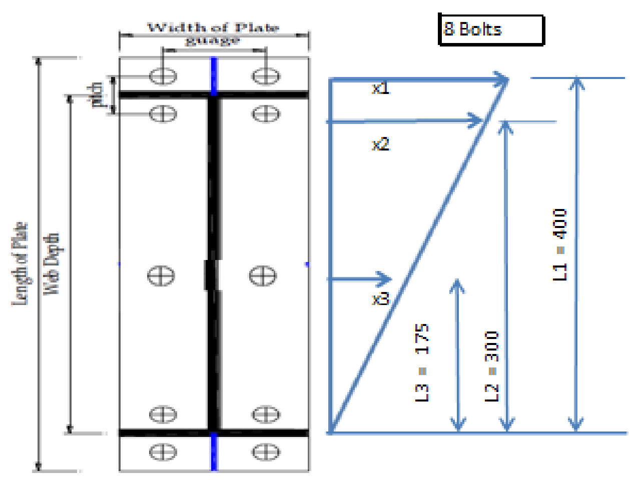
Beam	L/C	Node	Axial Force kN	Shear-Y kN	Shear-Z kN	Torsion kNm	Moment-Y kNm	Moment-Z kNm
1	3	1	24.078	100.261	0	0	0	58.913
		2	-24.078	57.767	0	0	0	-50.458

Design Sheet Beam to Column

Project Connection ID Grid Location		Date	05-02-2024

Design of Moment connection

Type:



Inputs

Bending Moment(M)	Kn.m	58.92
Shear Force (Fx)	Kn	100.3
Axial Force(Fy)	Kn	24.1
Web Depth	mm	350
Flange Width	mm	125
Bolts Dia	mm	20
Nos. of Bolts(n)	-	8
Grade of Bolts	-	8.8
Fu of bolts	Mpa	800
Plate thickness(t)	mm	12
Pitch(p)	mm	100
Guage(g)	mm	60
Material (fy) yield stress	Mpa	355
Depth of Web(mm)	:	350
Plate Length(mm)	:	520
Plate Width(mm)	:	150
Edge dist(v)mm	:	35
Edge dist(h)mm	:	45

Distance of extreme tension fiber(L) : 400 mm

Total Tension in each extreme bolts(2Te) or x1=x2(due to moment M) : Max moment/Eff Lever arm → [$x1 = M*(L1/ \Sigma Li^2)$]

: 58.92/0.4

: 147.3 Kn

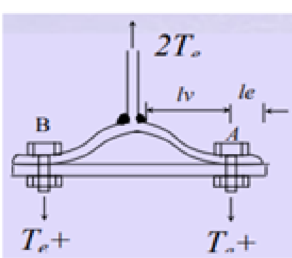
Check for Bolt size

Forces in the Bolts

Max. Tension in each Bolt (Te) = 2Te/4 : 36.83 Kn

Tension due to Axial Force (Fy/n) : 3.02 Kn

Check for prying forces:-



S= Assume Fillet weld thickness(mm)	6	
Total tensile force taken to calculate Q = Te due to moment + Tension due to axial	39.85	Kn
lv= (Bolt center to Toe of weld) (mm)	24	
le= (Bolt center to Edge distance of plate) (mm)	45	

$$Q = \frac{l_v}{2\,l_e}\left[T_e - \frac{\beta \gamma f_o b_e t^4}{27 l_e l_v^2}\right]$$

Where,

β = 2 for non pre-tensioned bolt and 1 for pre-tensioned bolt

γ = 1.5

b_e = effective width of flange per pair of bolts

f_o = proof stress in consistent units

t = thickness of the end plate

Prying Force in Bolt (Q) = 9.83 Kn

fo(Proof stress) = 0.7 x fu = 0.56

fu = 800 N/mm^2

	Actual (Kn)	Allowable (Kn)	
Total Tension (Te+Q+Fy/n) :	49.69	97.4	Hence ok
Shear in each Bolt (Fx/n) :	20.90	58.44	Hence ok

Check for Thickness of Plate

Maximum bending moment in the plate (Ma) : (P x g/2) /6

: (49.69 x 60/2) /6

: 0.25 Knm

Thickness of plate required (t) : $\sqrt{4x\ Ma\ /\ f\ x\ p}$

: 6.95 < 12 Hence ok

Combined Tension & Shear Check(based on considering bolts which induced shear & tension both)

$\frac{\text{Actual Tension}}{\text{Allowable Tension}} + \frac{\text{Actual Shear}}{\text{Allowable Shear}} < 1$

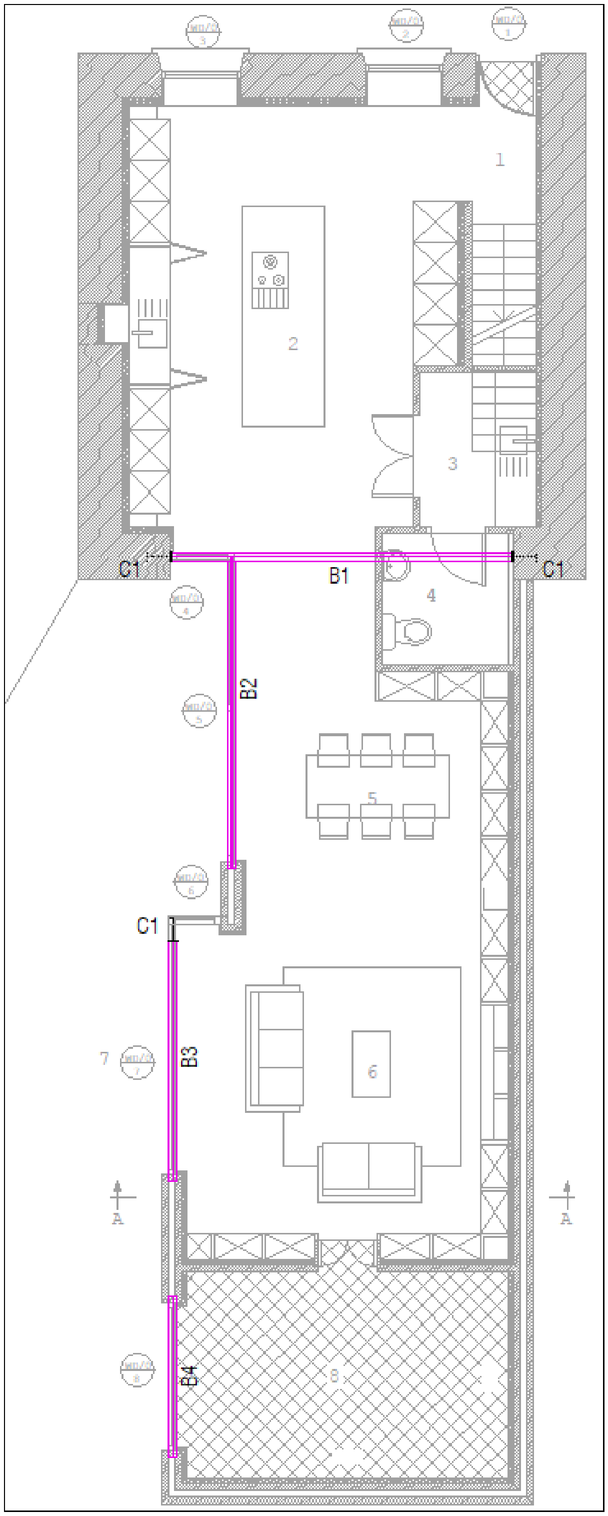
((49.69/97.4)+(20.9/58.44) = 0.87 < 1 Hence ok

Check for Weld Design

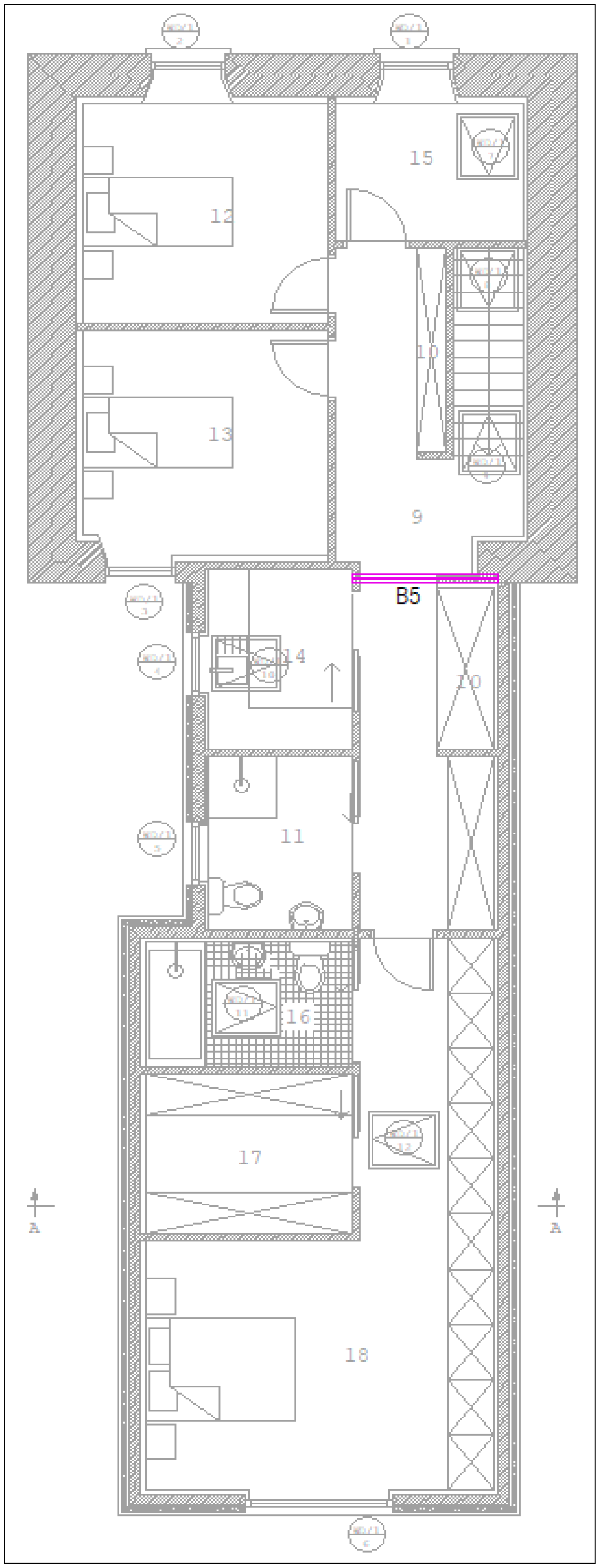
For E70XX Electrode(Fu)	41	Kn/cm^2	
Alowable stress in weld (Fw)= 0.3 x Fu	12.30	Kn/cm^2	
Length of Weld at Web(Lww)=2(dw-20)	660	mm	
Length of Weld at Flanges (Lwf)=2(bf1+bf2)	500	mm	
*Weld capacity in shear=(0.707*S*Fw*Lww)	344.37	Kn	Hence ok
*Weld capacity in shear=(0.707*S*Fw*Lwf)	260.89	Kn	Hence ok

12.4. Conclusion:

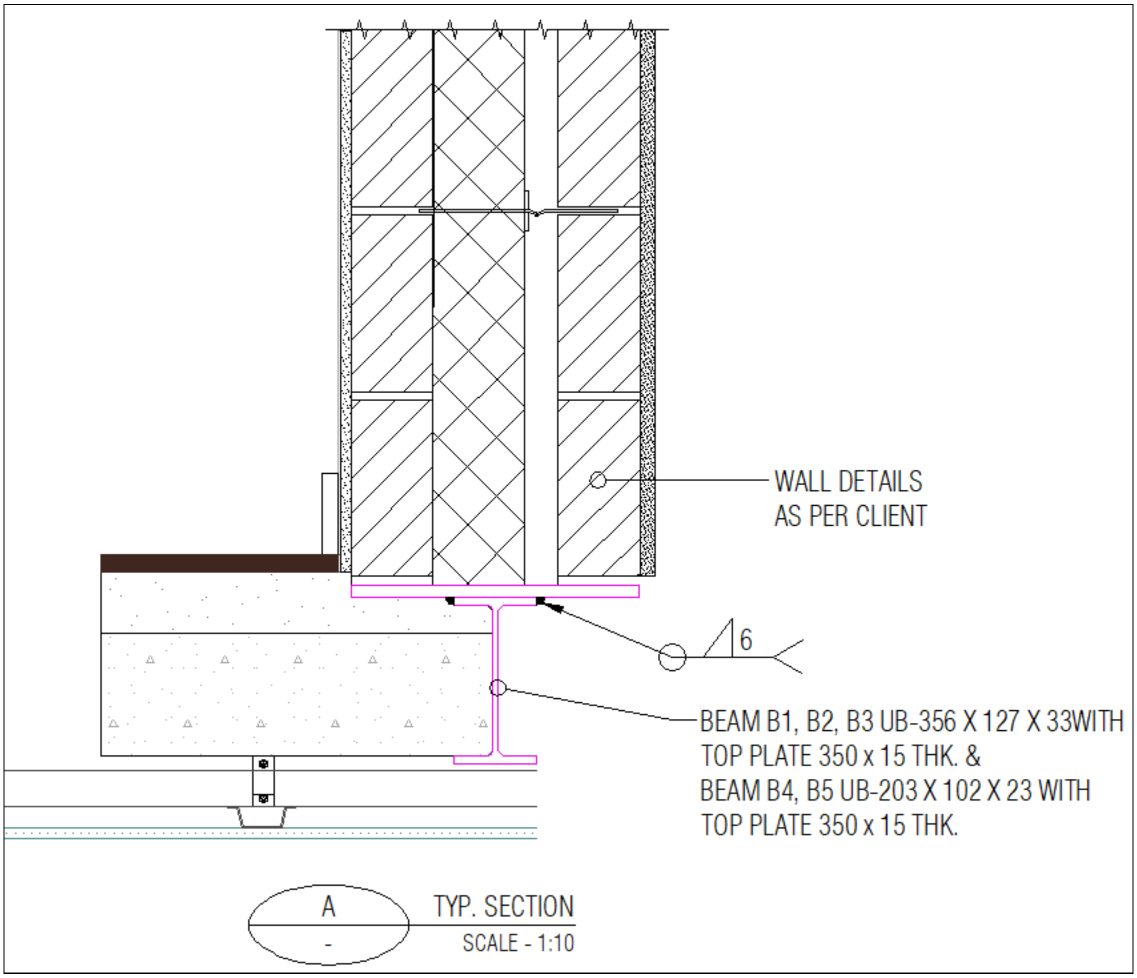
Refer below image showing beam layout and member sizes for proposed structure.



Ground floor slab layout



First floor slab layout



MEMBER MARK	MEMBER SIZE		GRADE N/mm^2
C1		COLUMN UB-356 x 127 x 33	S355
B1, B2, B3		BEAM UB-356 X 127 X 33 WITH TOP PLATE 350 x 15 THK.	S355
B4, B5		BEAM UB-203 X 102 X 23 WITH TOP PLATE 350 x 15 THK.	S355