DESIGN OF TRUSS FOR 12 METER SPAN AS PER IS 800-1984

SPAN OF TRUSS = 12 M
BAY SPACING = 6.0 M
WIND SPEED = 33 MPERS
SLOPE OF ROOF = 1 IN 3
THETA = 18.44 DEGREE
MATERIAL FOR CONSTRUCTION = SHS/RHS OF TATA STRUCTURA YST-310

DEAD LOAD (REF. IS: 875 PART 1) 1987
S/W OF SHEETING = 60 N/SQMT
S/W OF PURLIN = 58.86 N/MT
*NODAL LOAD = 60X1.5X6.0+100X6.0 = 1140N
*NODAL LOAD = 60X(1.5+0.9123)/2X6.0+100X6.0 = 1035 N
*NODAL LOAD = 60X0.9123X6.0+100X6.0 = 682 N
*NODAL LOAD = 60X0.9123/2X6.0+100X6.0 = 518 N

LIVE LOAD (REF. IS: 875 PART 2) 1987
LIVE LOAD = 750 N/SQMT

AS THE ROOF SLOPE IS GREATER THAN 10°
LIVE LOAD = 750-(20X8.44)
= 581.2 N/SQMT
581.2X2/3 = 388 N/SQMT (WHILE DESIGN OF TRUSS L.L. CAN BE TAKEN AS 2/3 (CL. 4.5.1 IS: 875 PART I))

MINIMUM LL = 400 N/SQMT (TABLE 2, IS: 875 PART II)
*NODAL LOAD = 400X1.5X6.0 = 3600 N
*NODAL LOAD = 400X(1.5+0.9123)/2X6.0 = 2895 N
*NODAL LOAD = 400X0.9123X6.0 = 2190 N
*NODAL LOAD = 400X0.9123/2X6.0 = 1095 N
WIND LOAD  (REF. IS 875: PART 3 1987)

BASIC WIND FORCE CALCULATION

BASIC WIND SPEED ($V_b$) = 33 M/S
DESIGN WIND SPEED ($V_z$) = $K_1 \times K_2 \times K_3 \times V_s$
DESIGNED WIND PRESSURE ($P_z$) = $0.6 \times (V_z)^2$

$K_1 \ (PROBABILITY \ FACTOR) \ \text{ASSUMED AS} = 1.0$
$K_2 \ (TERRAIN \ HEIGHT \ FACTOR) \ \text{ASSUMED AS} = 1.0$
$K_3 \ (TOPOGRAPHY \ FACTOR) \ \text{ASSUMED AS} = 1.0$

DESIGNED WIND SPEED ($V_z$) = $1 \times 1 \times 1 \times 33 = 33 \text{ M/S}$
DESIGNED WIND PRESSURE ($P_z$) = $0.6 \times 33^2$
= 653.4 N/SQM

A) WIND 0+ PRESSURES

EXTERNAL PRESSURE COEFFICIENT ($C_{Pe}$) (REF. IS 875: PART 3 1987 TABLE. No – 5)

BUILDING HEIGHT RATIO = $\frac{1}{2} < h/w < 3/2$
ON WINDWARD SIDE = 0.77
ON LEESIDE SIDE = -0.52

INTERNAL PRESSURE COEFFICIENT ($C_{Pi}$) (REF. IS 875: PART 3 1987)
(ASSUMING NORMAL PERMEABILITY)

$C_{Pi} = \pm 0.2$
TOTAL PRESSURE COEFFICIENT = $C_{Pe} + C_{Pi}$

NODAL LOAD ON WINDWARD SIDE

*NODAL LOAD = 0.97X653.4X6.0X1.5
= 5705 N FX=1805 N FY=5413 N

*NODAL LOAD = 0.97X653.4X6.0X(1.5+0.9123)/2
= 4587 N FX=1451 FY=4352 N

*NODAL LOAD = 0.97X653.4X6.0X0.9123
= 3470 N FX=1098 N FY=3292 N

*NODAL LOAD = 0.97X653.4X6.0X0.9123/2
= 1735 N FX=549 N FY=1646 N

NODAL LOAD ON LEESIDE SIDE

*NODAL LOAD = 0.72X653.4X6.0X1.5
= 4236 N FX=1340 N FY=4019 N

*NODAL LOAD = 0.72X653.4X6.0X(1.5+0.9123)/2
= 3405 N FX=1078 N FY=2331 N

*NODAL LOAD = 0.72X653.4X6.0X0.9123
= 2576 N FX=815 N FY=2444 N

*NODAL LOAD = 0.72X653.4X6.0X0.9123/2
= 1288 N FX=408 N FY=1222 N

(REF. IS 875: PART 3 1987)

(PROBABILTY FACTOR) ASSUMED AS
(TERRAIN HEIGHT FACTOR) ASSUMED AS
(TOPOGRAPHY FACTOR) ASSUMED AS
B) WIND 90+ PRESSURE

CPe ON WINDWARD SIDE = -0.8
CPe ON LEEWARD SIDE = -0.8
CPi = ±0.2
TOTAL PRESSURE COEFFICIENT = CPe + CPi

NODAL LOAD ON WIND WARD/ LEEWARD SIDE
*NODAL LOAD = 1X653.4X6.0X1.5
= 5881 N  FX=1861 N  FY=5580 N
*NODAL LOAD = 1X653.4X6.0X(1.5+0.9123)/2
= 4729 N  FX=1496 N  FY=4487 N
*NODAL LOAD = 1X653.4X6.0X0.9123
= 3577 N  FX=1132 N  FY=3394 N
*NODAL LOAD = 1X653.4X6.0X0.9123/2
= 1789 N  FX=566 N  FY=1697 N

LOAD COMBINATIONS
FOLLOWING LOAD COMBINATIONS ARE CONSIDERED FOR DESIGN OF TRUSSES

1) DL + LL
2) (DL + WL) X 0.75

AS PER IS 800: 1984 CLAUSE 3.9.2 FOR DESIGN OF TRUSS MEMBER, THE STRESSES ARE INCREASED BY 33% WHEN WIND IS CONSIDERED.

SUPPORT CONDITION
WHILE DESIGN OF TRUSS, ONE SUPPORT ASSUMED AS A PINNED AND OTHER ROLLER, SAME SHALL BE ENSURED AT THE SITE.
1. STAAD TRUSS
   INPUT FILE: 12M SPAN BAY 6 SLOPE 1 IN 3 33.STD
2. START JOB INFORMATION
3. ENGINEER DATE 23-JAN-11
4. END JOB INFORMATION
5. INPUT WIDTH 79
6. UNIT METER NEWTON
7. JOINT COORDINATES
   8. 1 0 0 0; 2 6 0 0; 3 6 2 0; 4 4.576897 1.52566 0; 5 3.15395 1.05132 0
   9. 6 1.73093 0.576975 0; 9 3.5044 0 0; 10 1.92325 0 0; 12 12 0 0
   10. 13 7.42303 1.52566 0; 14 8.84605 1.05132 0; 15 10.2691 0.576975 0
   11. 18 8.4956 0 0; 19 10.0767 0 0; 20 7.2478 1 0; 21 0.865465 0.288487 0
   12. 22 11.1346 0.288487 0
8. MEMBER INCIDENCES
   13. 1 4 . 111 0 ; 232 ; 334 ; 445 ; 556 ; 662 1 ; 992 ; 1 01 09 ; 1 261 0 ; 1 359
   14. 14 9 11; 15 11 3; 16 4 11; 18 10 5; 19 5 11; 20 12 19; 21 3 13; 22 13 14
   15. 23 14 15; 24 15 22; 27 18 2; 28 19 18; 30 15 19; 31 14 18; 32 18 20; 33 20 3
   16. 34 13 20; 36 19 14; 37 14 20; 38 21 1; 39 10 21; 40 22 12; 41 19 22
18. DEFINE MATERIAL START
19. ISOTROPIC STEEL
   20. E 2.05E+011
   21. POISSON 0.3
   22. DENSITY 77008.5
   23. ALPHA 1.2E-005
   24. DAMP 0.03
25. END DEFINE MATERIAL
26. MEMBER PROPERTY INDIAN
   27. 2 12 13 16 18 19 30 31 34 36 37 39 41 TABLE ST TUB25252.6
   28. 1 9 10 20 27 28 TABLE ST TUB66333.6
   29. 14 15 32 33 TABLE ST TUB66332.6
   30. 3 TO 6 21 TO 24 38 40 TABLE ST TUB50503.6
31. CONSTANTS
32. BETA 90 MEMB 1 9 10 14 15 20 27 28 32 33
33. MATERIAL STEEL ALL
34. SUPPORTS
35. 1 PINNED
36. 12 FIXED BUT FX FZ MX MY MZ
37. LOAD 1 LOADTYPE DEAD TITLE DL
38. SELFWEIGHT Y -1.05
39. * S/W OF SHEETING 60 N/SQMT
40. * S/W OF PURLIN=58.86 N/MT
41. *NODAL LOAD=60X1.5X4.5+100X4.5=855 N
42. JOINT LOAD
43. 3 TO 5 13 14 FY -855
44. *NODAL LOAD=60X(1.5+0.91)/2X4.5+100X4.5=776 N
45. 6 15 FY -776
46. *NODAL LOAD=60X0.91X4.5+100X4.5=696 N
47. 21 22 FY -696
48. *NODAL LOAD=60X0.91/2X4.5+100X4.5=573 N
49. 1 12 FY -573
50. LOAD 2 LOADTYPE LIVE TITLE LL
51. \(750 - 20 \times 8.44 = 581.2 \text{ N/SQMT}\)
52. \(581.2 \times 2/3 = 388\)
53. \(LL = 400 \text{ N/SQMT}\)
54. JOINT LOAD
55. \(NODAL LOAD = 400 \times 1.5 \times 4.5 = 2700 \text{ N}\)
56. 3 TO 5 13 14 FY -2700
57. \(NODAL LOAD = 400 \times (1.5 + 0.91)/2 \times 4.5 = 2169 \text{ N}\)
58. 6 15 FY -2168
59. \(NODAL LOAD = 400 \times 0.91 \times 4.5 = 1638 \text{ N}\)
60. 1 12 FY -819
61. LOAD 3 LOADTYPE WIND TITLE WIND0+PRE
62. \(WIND SPEED = 33 \text{ M/S}\)
63. \(WIND PRE = 0.6 \times 33^2 = 653.4 \text{ N/SQMT}\)
64. \(CPE ON WND WARD SIDE = 0.77\)
65. \(CPE ON LEWARD SIDE = -0.52\)
66. \(CPE PRESSURE = -0.2\)
67. JOINT LOAD
68. \(NODAL LOAD ON WIND WARD = 0.97 \times 653.4 \times 4.5 \times 1.5 = 4279 \text{ N} FX = 1354 \text{ N} FY = 4059 \text{ N}\)
69. 4 5 FX -1354 FY 4059
70. \(NODAL LOAD ON WIND WARD = 0.97 \times 653.4 \times 4.5 \times (1.5 + 0.91)/2 = 3437 \text{ N} FX = 1087 \text{ N} FY = 3261\)
71. 6 FX -1087 FY 3261
72. \(NODAL LOAD ON WIND WARD = 0.97 \times 653.4 \times 4.5 \times 0.91 = 2596 \text{ N} FX = 821 \text{ N} FY = 2463 \text{ N}\)
73. 21 FX -821 FY 2463
74. \(NODAL LOAD ON WIND WARD = 0.97 \times 653.4 \times 4.5 \times 0.91/2 = 1298 \text{ N} FX = 410 \text{ N} FY = 1231 \text{ N}\)
75. 1 FX -410 FY 1231
76. \(NODAL LOAD ON LEE WARD SIDE = 0.72 \times 653.4 \times 4.5 \times 1.5 = 3176 \text{ N} FX = 1004 \text{ N} FY = 3013 \text{ N}\)
77. 13 14 FX 1004 FY 3013
78. \(NODAL LOAD ON LEE WARD SIDE = 0.72 \times 653.4 \times 4.5 \times (1.5 + 0.91)/2 = 2552 \text{ N} FX = 807 \text{ N} FY = 2422\)
79. 15 FX 807 FY 2422
80. \(NODAL LOAD ON LEE WARD SIDE = 0.72 \times 653.4 \times 4.5 \times 0.91 = 1927 \text{ N} FX = 609 \text{ N} FY = 1829 \text{ N}\)
81. 22 FX 609 FY 1829
82. \(NODAL LOAD ON LEE WARD SIDE = 0.72 \times 653.4 \times 4.5 \times 0.91/2 = 964 \text{ N} FX = 305 \text{ N} FY = 915 \text{ N}\)
83. 12 FX 305 FY 915
84. 3 FX -305 FY 915
85. LOAD 4 LOADTYPE WIND TITLE WIND90+PRE
86. \(CPE ON WND WARD SIDE = 0.8\)
87. \(CPE ON LEWARD SIDE = -0.8\)
88. JOINT LOAD
89. \(NODAL LOAD ON WIND WARD LEEWARD SIDE = 1 \times 653.4 \times 4.5 \times 1.5 = 4410 \text{ N} FX = 1395 \text{ N} FY = 4184\)
90. 4 5 FX -1395 FY 4184
91. \(NODAL LOAD ON WIND WARD LEEWARD SIDE = 1 \times 653.4 \times 4.5 \times (1.5 + 0.91)/2 = 3544 \text{ N} FX = 1120 \text{ N}\)
92. 13 14 FX 1395 FY 4184
93. \(NODAL LOAD ON WIND WARD LEEWARD SIDE = 1 \times 653.4 \times 4.5 \times 0.91 = 2676 \text{ N} FX = 846 \text{ N} FY = 253\)
94. 21 FX -846 FY 2539
95. 22 FX 846 FY 2539
96. \(NODAL LOAD ON WIND WARD LEEWARD SIDE = 1 \times 653.4 \times 4.5 \times 0.91/2 = 1338 \text{ N} FX = 423 \text{ N} FY = 1\)
97. 1 FX -423 FY 1270
98. 12 FX 423 FY 1270
99. 3 FY 4184
100. LOAD 21
101. REPEAT LOAD
102. 1 1.333
103. LOAD 22
104. REPEAT LOAD
105. 2 1.333
PROBLEM STATISTICS

NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 18/33/2

SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER

ORIGINAL/FINAL BAND-WIDTH = 16/4/10 DOF
TOTAL PRIMARY LOAD CASES = 8, TOTAL DEGREES OF FREEDOM = 33
SIZE OF STIFFNESS MATRIX = 1 DOUBLE KILO-WORDS
REQRD/AVAIL. DISK SPACE = 12.1/111196.4 MB

123. LOAD LIST 21 TO 24
124. PRINT SUPPORT REACTION

SUPPORT REACTIONS - UNIT NEWT METE STRUCTURE TYPE = TRUSS

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<th>FORCE-Y</th>
<th>FORCE-Z</th>
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************************** END OF LATEST ANALYSIS RESULT **************************

125. LOAD LIST 5 TO 7
126. UNIT MMS NEWTON
127. PARAMETER 1
128. CODE INDIAN
129. KY 0.85 ALL
130. KZ 0.85 ALL
131. FYLD 310 ALL
132. LZ 3200 MEMB 14 15 32 33
133. LZ 3500 MEMB 1 10 20 28
134. LZ 2500 MEMB 9 27
135. CHECK CODE ALL
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ALL UNITS ARE - NEWT MMS (UNLESS OTHERWISE NOTED)

STAAD.Pro CODE CHECKING - (IS-800)
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*********************************************************************** END OF TABULATED RESULT OF DESIGN *******************************
### STEEL TAKE-OFF

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<th>PROFILE</th>
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<th>WEIGHT (KG)</th>
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TOTAL = 171.168

-------------------

### MEMBER TABLE 12 M SPAN SLOPE 1 IN 3 BAY 6.0

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TOTAL = 171.168

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### REACTION

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118
**DIMENSIONS OF SHED**
- Span = 5 M
- HT of Shed = 5 M
- Bay Spacing = 4.5 M
- Wind Speed = 33 MPERS
- Slope = 1 IN 3
- Theta = 18.44 DEGREE
- Type of Sheeting = A.C Sheets
- Material for Construction = SHS/RHS of Tata Structura YST-310 Grade

**FOLLOWING LOADS ARE CONSIDERED WHILE DESIGN OF PORTALS**

**Dead Load (As per IS-875 Part I)**
- S/W of Sheeting = 170 N/SQMT
- S/W of Purlin = 100 N/MT
- Nodal Load = 170X1.32X4.5 + 100X4.5 = 1460

**Live Load (As per IS-875 Part II)**
- 750-20X8.44 = 581.2 N/SQMT
- 581.2X2/3 = 388 N/SQMT (While designing of Trusses LL can be taken as 2/3, (CL4.5.1 IS-875 Part I)

**Minimum LL**
- = 400 N/SQMT (Table 2 IS-875, Part II)

**Nodal Load**
- = 400X1.32X4.5 = 2376 N

**Wind Load (As per IS-875 Part III)**

**Basic Wind Force Calculation**
- Wind Speed ($V_w$) = 33 m/s
- Design Wind Speed ($V_z$) = $K_1 \times K_2 \times K_3 \times V_w$
- Design Wind Pressure ($P_z$) = 0.6$V_z^2$
- $K_1$ (Probability Factor) = Assumed as = 1
- $K_2$ (Terrain Height Factor) = Assumed as = 1
- $K_3$ (Topography Factor) = Assumed as = 1
- Design Wind Speed ($V_z$) = $1 \times 1 \times 33 = 33$ m/s
- Design Wind Pressure ($P_z$) = 0.6$33^2$
- = 653.4 N/SQMT
A) WIND 0

EXTERNAL PRESSURE COEFFICIENT (Cpe)  *(REF. IS 875: PART 3 1987 TABLE, No – 5)*

\[
\begin{align*}
h/W & = 5/5 = 1 \\
L/W & = 25/5 = 5 \\
Cpe ON WIND WARD SIDE & = -0.763 \\
Cpe ON LEEWARD SIDE & = -0.52 \\
LOADING ON WINDWARD SIDE & = 653.4 \times 0.763 \times 1.32 \times 4.5 = 2962 \text{ N/M} \\
LOADING ON LEEWARD SIDE & = 653.4 \times 0.52 \times 1.32 \times 4.5 = 2019 \text{ N/M} \\
\end{align*}
\]

WIND LOAD ON CLADDING

\[
\begin{align*}
\text{HT OF CLADDING 3.0 M FROM BOTTOM} \\
Cpe ON WINDWARD SIDE & = -0.7 \\
Cpe ON LEEWARD SIDE & = -0.3 \\
LOADING ON WINDWARD SIDE & = 653.4 \times 0.7 \times 4.5 = 2059 \text{ N/M} \\
LOADING ON LEEWARD SIDE & = 653.4 \times 0.3 \times 4.5 = 883 \text{ N/M} \\
\end{align*}
\]

C) SUCTION

\[
\begin{align*}
\text{Cpi ON WINDWARD & LEEWARD SIDE} & = -0.2 \\
\text{LOAD ON WINDWARD & LEEWARD SIDE} & = 0.2 \times 653.4 \times 1.32 \times 4.5 = 776 \text{ N/M} \\
\text{WIND LOAD ON CLADDING} & = \text{HT OF CLADDING 3.0 M FROM BOTTOM} \\
\text{Cpi ON WIND WARD SIDE} & = -0.2 \\
\text{Cpi ON LEEWARD SIDE} & = -0.2 \\
\text{LOADING ON WINDWARD & LEEWARD} & = 653.4 \times 0.2 \times 4.5 = 588 \text{ N/M} \\
\end{align*}
\]

D) PRESSURE

\[
\begin{align*}
\text{Cpi ON WINDWARD & LEEWARD SIDE} & = 0.2 \\
\text{LOAD ON WINDWARD & LEEWARD SIDE} & = 0.2 \times 653.4 \times 1.32 \times 4.5 = 776 \text{ N/M} \\
\text{WIND LOAD ON CLADDING} & = \text{HT OF CLADDING 3.0 M FROM BOTTOM} \\
\text{Cpi ON WIND WARD SIDE} & = 0.2 \\
\text{Cpi ON LEEWARD SIDE} & = 0.2 \\
\text{LOADING ON WINDWARD & LEEWARD SIDE} & = 653.4 \times 0.2 \times 4.5 = 588 \text{ N/M} \\
\end{align*}
\]

LOAD COMBINATIONS

FOLLOWING LOAD COMBINATIONS ARE CONSIDERED FOR DESIGN OF TRUSSES

A) DL+LL
B) DL X 0.75 + WL0 X 0.75 + SUCTION X 0.75
C) DL X 0.75 + WL0 X 0.75 + PRESSURE X 0.75
D) DL X 0.75 + WL90 X 0.75 + SUCTION X 0.75
E) DL X 0.75 + WL90 X 0.75 + PRESSURE X 0.75

FOR DESIGN OF PORTAL MEMBERS, THE STRESSES ARE INCREASED BY 33% WHEN WIND IS CONSIDERED. SO COMBINATION IS APPLIED WITH FACTOR OF 0.75 (AS PER IS-800 CL 3.9.2)

SUPPORTS CONDITIONS:

*WHILE DESIGN OF PORTALS, SUPPORTS ARE CONSIDERED AS FIXED AND SAME SHALL BE ENSURED AT SITE.*
1. STAAD SPACE
   INPUT FILE: PORTAL 5M HT5 4.5 33 1 IN 3.STD

2. START JOB INFORMATION
3. ENGINEER DATE 21-JUL-10
4. JOB COMMENT PORTAL SPAN 5 M
5. JOB COMMENT BAY 4.5M
6. JOB COMMENT WIND SPEED 33M/S
7. JOB COMMENT SLOPE 1 IN 3
8. JOB COMMENT THETA 18.44
9. END JOB INFORMATION
10. INPUT WIDTH 79
11. UNIT METER NEWTON
12. JOINT COORDINATES
   1 0 0 0 ; 2 5 0 0 ; 3 0 5 0 ; 4 5 5 0 ; 5 2.5 5.84 0

14. MEMBER INCIDENCES
15. 1 1 3 ; 2 2 4 ; 3 3 5 ; 4 5 4
16. DEFINE MATERIAL START
17. ISOTROPIC STEEL
18. E 2.05E+011
19. POISSON 0.3
20. DENSITY 77008.5
21. ALPHA 1.2E-005
22. DAMP 0.03
23. END DEFINE MATERIAL
24. MEMBER PROPERTY INDIAN
25. 1 2 TABLE ST TUB145824.8
26. 3 4 TABLE ST TUB122613.6
27. CONSTANTS
28. MATERIAL STEEL ALL
29. SUPPORTS
30. 1 2 FIXED
31. LOAD 1 LOADTYPE DEAD TITLE DL
32. * S/W OF SHEETING 170 N/SQMT
33. * S/W OF PURLIN=100 N/MT
34. *NODAL LOAD=170X1.32X4.5+100X4.5=1460 N
35. SELFWEIGHT Y -1.05 LIST 1 TO 4
36. MEMBER LOAD
37. 3 4 CON GY -1460
38. JOINT LOAD
39. 3 4 FY -955
40. 5 FY -1460
41. LOAD 2 LOADTYPE LIVE TITLE LL
42. *750-20X8.44=581.2N/SQMT
43. *581.2X2/3=388
44. *LL=400 N/SQMT
45. *NODAL LOAD =400X1.32X4.5=2376 N
46. MEMBER LOAD
47. 3 4 CON GY -2376
48. JOINT LOAD
49. 5 FY -2376
50. 3 4 FY -1188
51. LOAD 3 LOADTYPE WIND TITLE WIND 0
52. *WIND SPEED 33 M/S
53. *WIND PRE = .6X33^2=653.4
54. *CPE ON WIND WARD SIDE=-0.763
55. *CPE ON LEEWARD SIDE=-0.52
56. *LOAD ON WINDWARD SIDE=653.4X0.763X1.32X4.5=2962 N
57. *LOAD ON LEEWARD SIDE =653.4X0.52X1.32X4.5=2019 N
58. MEMBER LOAD
59. 3 CON Y 2962
60. 4 CON Y 2019
61. JOINT LOAD
62. 3 5 FX -469 FY 1405
63. 4 5 FX 320 FY 958
64. *CPE FOR CLADDING
65. *ON WIND WARD =-0.7 ON LEEWARD -0.3
66. *LOAD ON CLADDING ON WINDWARD SIDE
67. *7X653.4X4.5=2059
68. *0.3X653.4X4.5=883
69. MEMBER LOAD
70. 1 UNI GX 2059 3 5
71. 2 UNI GX 883 3 5
72. LOAD 4 LOADTYPE WIND TITLE WIND 90
73. *CPE ON WIND WARD AND LEEWARD SIDE=-0.8
74. *LOAD =0.8X653.4X1.32X4.5=3105
75. MEMBER LOAD
76. 3 4 CON Y 3105
77. JOINT LOAD
78. 3 FX -492 FY 1473
79. 4 FX 492 FY 1473
80. 5 FY 2945
81. *CPE FOR CLADDING
82. *ON WIND WARD =-0.5 ON LEEWARD -0.5
83. *LOAD=0.5X653.4X4.5=1470
84. MEMBER LOAD
85. 1 UNI GX -1470 3 5
86. 2 UNI GX 1470 3 5
87. LOAD 5 LOADTYPE WIND TITLE SUCTION
88. *CPE =0.2
89. *LOAD=0.2X653.4X1.32X4.5=776
90. MEMBER LOAD
91. 3 4 CON Y -776
92. JOINT LOAD
93. 3 FX 123 FY -369
94. 4 FX -123 FY -369
95. 5 FY -737
96. *CPE =0.2
97. *LOAD=0.2X653.4X4.5=588
98. MEMBER LOAD
99. 1 UNI GX 588 3 5
100. 2 UNI GX-588 3 5
101. LOAD 6 LOADTYPE WIND TITLE PRE
102. MEMBER LOAD
103. 3 4 CON Y 776
104. JOINT LOAD
105. 3 FX -123 FY 369
106. 4 FX 123 FY 369
107. 5 FY 737
108. *CPE =0.2
109. *LOAD=0.2X653.4X4.5=588
110. MEMBER LOAD
111. 1 UNI GX -588 3 5
112. 2 UNI GX 588 3 5
113. LOAD COMB 7 COMBINATION LOAD CASE 7
114. 1.0 2 1.0
115. LOAD COMB 8 COMBINATION LOAD CASE 8
116. 1.0 75 3 0.75 5 0.75
117. LOAD COMB 9 COMBINATION LOAD CASE 9
118. 1.0 75 3 0.75 6 0.75
119. LOAD COMB 10 COMBINATION LOAD CASE 10
120. 1.0 75 4 0.75 5 0.75
121. LOAD COMB 11 COMBINATION LOAD CASE 11
122. 1.0 75 4 0.75 6 0.75
123. PERFORM ANALYSIS
PROBLEM STATISTICS
NUMBER OF JOINTS/MEMBER+ELEMENTS/SUPPORTS = 5/ 4/ 2
SOLVER USED IS THE OUT-OF-CORE BASIC SOLVER
ORIGINAL/FINAL BAND-WIDTH = 2/ 2/ 18 DOF
TOTAL PRIMARY LOAD CASES = 6, TOTAL DEGREES OF FREEDOM = 18
SIZE OF STIFFNESS MATRIX = 1 DOUBLE KILO-WORDS
REQRD/AVAIL. DISK SPACE = 12.0/ 107609.8 MB

124. LOAD LIST 1 TO 6
125. PRINT SUPPORT REACTION ALL

SUPPORT REACTIONS -UNIT NEWT METE STRUCTURE TYPE = SPACE

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<tr>
<th>JOINT</th>
<th>LOAD</th>
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<th>FORCE-Y</th>
<th>FORCE-Z</th>
<th>MOM-X</th>
<th>MOM-Y</th>
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<td>-180.79</td>
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*********************************** END OF LATEST ANALYSIS RESULT ***********************************

126. LOAD LIST 7 TO 11
127. UNIT MMS NEWTON
128. PARAMETER 1
129. CODE INDIAN
130. BEAM 1 ALL
131. FYLD 310 ALL
132. LY 1320 MEMB 3 4
133. UNL 1320 MEMB 3 4
134. CHECK CODE ALL

STAAD.Pro CODE CHECKING - (IS-800)
ALL UNITS ARE - NEWT MMS (UNLESS OTHERWISE NOTED)

<table>
<thead>
<tr>
<th>MEMBER</th>
<th>TABLE</th>
<th>RESULT/ FX</th>
<th>CRITICAL COND/ MY</th>
<th>RATIO/ MZ</th>
<th>LOADING/ LOCATION</th>
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</thead>
<tbody>
<tr>
<td>1 ST</td>
<td>TUB145824.8</td>
<td>PASS 443.77 T</td>
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<td>IS-7.1.1(A)</td>
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*********************************** END OF TABULATED RESULT OF DESIGN ***********************************
### MEMBER TABLE FOR 5M SPAN SLOPE 1 IN 3 BAY 4.5

<table>
<thead>
<tr>
<th>WIND 33</th>
<th>WIND 39</th>
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<tbody>
<tr>
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<td><strong>SECTION</strong></td>
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<tr>
<td>HT 5M</td>
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------------------------------- END OF DATA FROM INTERNAL STORAGE -------------------------------

139. FINISH

------------------------------- END OF THE STAAD.Pro RUN -------------------------------

### STEEL TAKE-OFF

<table>
<thead>
<tr>
<th>PROFILE</th>
<th>LENGTH(ME)</th>
<th>WEIGHT(KG)</th>
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<tbody>
<tr>
<td>ST TUB145824.8</td>
<td>10.00</td>
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<td>ST TUB122613.6</td>
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<td><strong>TOTAL</strong> =</td>
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135. PARAMETER 3
136. CODE INDIAN
137. UNIT METER KG
138. STEEL TAKE OFF ALL
### WIND 44

<table>
<thead>
<tr>
<th>MEM NO</th>
<th>SECTION</th>
<th>UNIT WT</th>
<th>LENGTH</th>
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</tr>
<tr>
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</tr>
<tr>
<td>HT 7.5M</td>
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<td>250X250X6</td>
<td>45.24</td>
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<td></td>
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### WIND 47

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</thead>
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<td>22.26</td>
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<td>122X61X4.5</td>
<td>11.88</td>
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<tr>
<td>HT 7.5M</td>
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### Reaction

- **DL**: 705.98, 1048.97, -7025.54, -1999.47, 241.36, 521.73, 733.92, -18244.39, -18205.95, 5743.45, -1828.95, 652.73, 773.92, -1828.95, 652.73
- **LL**: 1048.97, 705.98, 733.92, 705.98, 733.92, 705.98, 733.92, 705.98, 733.92, 705.98, 733.92, 705.98, 733.92
- **WL0**: -7025.54, -1999.47, -18244.39, -18205.95, 5743.45, -1828.95, 652.73, -1828.95, 652.73
- **WL90**: 241.36, 521.73, 733.92, 18244.39, 18205.95, 652.73, 773.92, 1828.95
- **SUCT**: -241.36, 521.73, 733.92, -18244.39, -18205.95, 652.73, 773.92, 1828.95
- **PRESSURE**: 241.36, 521.73, 733.92, -18244.39, -18205.95, 652.73, 773.92, 1828.95

### Wind 50

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<td>145X82X4.8</td>
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### Wind 55

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### Reaction

- **DL**: 732.28, 1047.16, -9067.94, -257.12, 311.25, 521.73, 733.92, -18482.33, -2349.8, 5148.9, 521.73, 733.92, -18482.33, -2349.8, 5148.9
- **LL**: 1047.16, 732.28, -257.12, -9067.94, 311.25, 733.92, 732.28, 733.92, 732.28, 733.92, 732.28, 733.92, 732.28, 733.92
- **WL0**: -257.12, -18482.33, -9067.94, -2349.8, 5148.9, -257.12, -9067.94, -2349.8, 5148.9
- **WL90**: 311.25, 732.28, -18482.33, -257.12, 5148.9, 311.25, 732.28, -18482.33, -257.12
- **SUCT**: -311.25, 732.28, -18482.33, -257.12, 5148.9, -311.25, 732.28, -18482.33, -257.12
- **PRESSURE**: 311.25, -311.25, 732.28, -18482.33, -257.12, -311.25, 732.28, -18482.33, -257.12
DESIGN OF PURLIN FOR 310 GRADE STEEL

BASIC WIND SPEED \( (V_b) \) = 33 M/S

FACTORS
\( K_1 \) PROBABILITY FACTOR = 1
\( K_2 \) TERRAIN HEIGHT FACTOR = 1
\( K_3 \) TOPOGRAPHY FACTOR = 1

For calculating ext. pressure co-eff. Ref. Table 16, is 875-(Part-III)-1987

Height, \( H \) = 10.5 M
Length, \( L \) = 4.5 M
Width, \( W \) = 20 M

\( L/W = 0.225 \)
\( H/L = 2.333 = 2.5 \)
\( H/W = 0.525 \)

DESIGNED WIND PRESSURE \( (P_z) \) = 0.6 \( V_z \) = 653.4 KG/SQMT

SPACING OF PURLIN = 1.5 M
BAY SPACING = 6 M
SLOPE OF ROOF = 1 IN 3
THETA = 18.44

\( \cos 18.44 = 0.95 \)
\( \sin 18.44 = 0.32 \)

LOAD CALCULATION

DEAD LOAD
WT OF SHEETING = 60 N/SQMT
WT OF SHEETING ON PURLIN = 90 N/M
SELF WEIGHT OF PURLIN = 96.75 N/M

TOTAL DEAD LOAD ON PURLIN = 186.75 N/M

LIVE LOAD

\( = 750-20 \times 8.44 \)
\( = 581.11 \) N/SQMT

TOTAL LIVE LOAD ON PURLIN = 871.67 N/M

WIND LOAD

WIND CPE EXTERNAL = -0.77 WIND WARD SIDE
\( \) = -0.52 LEEWARD SIDE
PRESSURE CPE INTERNAL = -0.2
TOTAL CPE = -0.97

TOTAL WIND LOAD ON PURLIN = 950.697 N/M

MOMENT = \( W \times L^2/10 \)
<table>
<thead>
<tr>
<th>MOMENT</th>
<th>DL</th>
<th>LL</th>
<th>WL</th>
<th>TOTAL DL+LL</th>
<th>TOTAL DL-WL</th>
<th>MAX</th>
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</thead>
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<td></td>
<td>NMM</td>
<td>NMM</td>
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<td>NMM</td>
<td>NMM</td>
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<td>212595.48</td>
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<td>212595.4775</td>
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\[ Z_{xx} \text{ REQ} = 17.667 \text{ CM}^3 \]
\[ Z_{yy} \text{ REQ} = 5.889 \text{ CM}^3 \]

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<th>PROVIDE</th>
<th>SECTION</th>
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<th>61</th>
<th>3.6</th>
<th>RATIO</th>
<th>L/C</th>
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<td>IXX</td>
<td>IYY</td>
<td>ZXX</td>
<td>ZYY</td>
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<td>232.61</td>
<td>78.83</td>
<td>38.13</td>
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</table>

### CHECK FOR DEFLECTION

DEFLECTION = \( \frac{0.5x5wxl'}{(384xEI)} \)

ACTUAL DEFLECTION:
- DL+LL = 19.20 MM
- DL+WL = 13.86 MM

PERMISSIBLE = \( L/200 = 30.00 \text{ MM} \)

FOR DL+LL

\[
\frac{\text{FCBX}}{\text{FCBY}} = \frac{M_{xx}/Z_{xx}}{M_{yy}/Z_{yy}} = \frac{94.80}{46.62} = 0.69
\]

LESSTHAN 1 OK

FOR DL-WL

\[
\frac{\text{FCBX}}{\text{FCBY}} = \frac{M_{xx}/Z_{xx}}{M_{yy}/Z_{yy}} = \frac{73.03}{8.23} = 0.40
\]

LESSTHAN 1.33 OK
DESIGN OF BUILT-UP PURLIN FOR 310 GRADE STEEL

BASIC WIND SPEED = 55 M/S

FACTORs K1 PROBABILITY FACTOR = 1
K2 TERRAIN HEIGHT FACTOR = 1
K3 TOPOGRAPHY FACTOR = 1

For calculating ext. pressure co-eff. Ref. Table 16, is 875-(Part-III)-1987

Height, H = 10.5 M
Length, L = 45 M
Width, W = 20 M

L/W = 2.25
H/L = 0.23333 = 0.25
H/W = 0.525

WIND PRESSURE = K1 x K2 x K3 x 0.6 x WIND SPEED²
= 1815 KG/SQMT

SPACING OF PURLIN = 1.5 M
BAY SPACING = 12 M
SLOPE OF ROOF = 1 IN 10
THETA = 5.713

COS 5.71349 = 0.995
SIN 5.71349 = 0.100

LOAD CALCULATION

DEAD LOAD
WT OF SHEETING = 60 N/SQMT
WT OF SHEETING ON PURLIN = 90 N/M
SELF WEIGHT OF PURLIN = 120.24 N/M
TOTAL DEAD LOAD ON PURLIN = 210.24 N/M

LIVE LOAD
LOAD = 750 N/SQMT
TOTAL LIVE LOAD ON PURLIN = 1125 N/M

WIND LOAD
WIND CPe EXTERNAL = -0.77 WIND WARD SIDE
= -0.52 LEEWARD SIDE
PRESSURE CPI INTERNAL = -0.2
TOTAL CPE = -0.97
TOTAL WIND LOAD ON PURLIN = 2640.825 N/M
MOMENT = WxL²/10
### Table 1: Moment and Load Distribution

<table>
<thead>
<tr>
<th>MOMENT</th>
<th>DL</th>
<th>LL</th>
<th>WL</th>
<th>TOTAL DL+LL</th>
<th>TOTAL DL-WL</th>
<th>MAX</th>
<th>UNIT</th>
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<td>301238.4036</td>
<td>1913198.652</td>
<td>NMM</td>
</tr>
</tbody>
</table>

**ZXX REQ** = 171.141231 CM^3  
**ZYY REQ** = 9.35092205 CM^3

### Deflection Calculations

**Deflection** = \(\frac{0.5 \times 5 \times w \times l^4}{(384 \times E \times I)}\)  
**Deflection Check**

**Deflection DL+LL** = 16.99 MM OK  
**Deflection DL+WL** = 30.93 MM OK  

**Permissible** \(\frac{L}{200}\) = 60.00 MM

### Section Properties

#### Top Member

<table>
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<th>SECTION</th>
<th>96</th>
<th>48</th>
<th>4</th>
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#### Bottom Member

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<td>27.13</td>
<td>10.16</td>
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<tr>
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</table>

### Built-Up Section Properties

**C/C of Top & Bot Member** = 450 MM  
**AX TOTAL** = 13.92 CM^2  
**Self Weight** = 120.24 N/MT  
**CG from Cen of Top Mem** = 11.16819 CM  
**CG from Cen of Bot Mem** = 33.83181 CM  
**IX of Built Up Section** = 5303.857 CM^4  
**IY of Built Up Section** = 127.71 CM^4  
**ZXX of Built Up From Top** = 390.9038 CM^3  
**ZXX of Built Up From Bot** = 151.1854 CM^3  
**ZYY of Built Up Section** = 26.60532 CM^3
DESIGN OF GIRT

LOAD CALCULATION
SPAN OF GIRT = 6.0 m
MAX. SPACING OF GIRT = 1.7 m
C.G.I. SHEETING WEIGHT = 60 N/m²
WT. ON GIRT. = 102 N/m
GIRT SELF WEIGHT = 100 N/m  .......... (assumed)
TOTAL LOAD = 202 N/m

VERTICAL BENDING MOMENT

\[ \text{M}_{yy} = \frac{wL^2}{10} = \frac{202 \times 6^2}{10} = 727.2 \text{ Nm} \]

HORIZONTAL BENDING MOMENT DUE TO WIND

\[ \frac{h}{w} = \frac{10.5}{12} = 0.875 \quad \text{Cpe} = 0.7 \]
\[ \frac{L}{w} = \frac{45}{12} = 3.75 \quad \text{Cpi} = 0.2 \]

PRESSURE COEFFICIENT = \( CP = \text{Cpe} + \text{Cpi} \)

WIND SPEED (\( V_b \)) = 33 m/s
DESIGN WIND SPEED (\( V_z \)) = 1X1X1X33 = 33 m/s
DESIGN WIND PRESSURE = 0.6 \( V_z^2 \)
\[ = 0.6 \times 33^2 = 653.4 \text{ m/s} \]

WIND LOAD ON WINDWORD WALL
\[ = 0.9 \times 653.4 \times 1.7 \]
\[ = 999.70 \]

HORIZONTAL BENDING MOMENT

\[ \text{M}_{xx} = \frac{wL^2}{10} = \frac{999.7 \times 6^2}{10} = 3598.9272 \text{ Nm} \]

REQUIRED SECTIONAL MODULUS

\[ \frac{Z}{\text{req}} = \frac{M}{0.66f_y} = \frac{3598.93}{0.66 \times 310} \]
\[ = 17.59006452 \]
TRY RHS 80 X 40 X 4.8 @ 10.01 KG/m

\[ f_{bc} = \frac{359892.72}{18.3} + \frac{72720}{12.02} = 257.16 \text{ Mpa} \]

\[ f_{bc} < 204.6 \times 1.33 = 272.118 \text{ Mpa} \]

HENCE OK

DESIGN WITH CONVENSIONAL SECTION
REFER BIS, Sp38, 1987 RECOMMENDED SECTION

ISMC 125@12.7 KG/m WITH ONE 12 φ SAG ROD @ 0.9 KG/m AT CENTER
TOTAL WEIGHT (INCLUDING SAG ROD) = 619.2 \times 12.7 + 225 = 8088.8 Kg
DESIGN OF COLUMN

THE COLUMN IS DESIGN AS PER PROPPED CANTILEVER FIXED AT BASE

CALCULATION OF LOADS

a) DEAD LOAD

WT. OF C.G.I. SIDE WALL = 60 X 10.5 X 6 = 3780 N
WT. OF GIRT (8 NO’s) = 100 X 6 X 8 = 4800 N
SELF WT. OF COLUMN = 650 X 10.5 = 6825 N ....(assumed)@650 Kg/m

TOTAL = 15405 N

REACTION FROM ROOF TRUSS DUE TO DEAD LOAD = 6750 N
TOTAL DEAD LOAD ON COLUMN = 22155 N

b) LIVE LOAD

REACTION FROM ROOF TRUSS DUE TO LIVE LOAD = 15165 N

MAX. COMPRESSIVE FORCE ON COLUMN CAP = (6750 + 15165 )
(D.L. +L.L.) = 21915 N
MAX. COMPRESSIVE FORCE ON COLUMN BASE = (22155 + 15165)
(D.L. +L.L.) = 37320 N

c) WIND LOAD

REACTION FROM ROOF

1. REACTION DUE WIND PERPENDICULAR TO RIDGE (0° WIND )
HORIZONTAL FORCE = 1963 N  VERTICAL UPLIFT = 21165 N

2. REACTION DUE TO WIND PARELLEL TO RIDGE (90° WIND )
VERTICAL UPLIFT = 23504 N
MAX. UPLIFT AT COLUMN CAP = (23504 -6750) = 16754 N
MAX. UPLIFT AT COLUMN BASE = (16754 -15405) = 1349 N

WIND FORCE REFER IS 875 PART 3
h/w = 0.875  l/w = 3.75
WIND PERPENDICULAR TO RIDGE (REF. FIG.)

a) PRESSURE

Cpe at A = 0.7
Cpe at B = 0.3
Cpe at C = 0.7
Cpe at D = 0.7
Cpi = 0.2
PRESSURE COEFFICIENT

CP AT FACE A = (0.7 - 0.2) = 0.5
CP AT FACE B = (0.3 + 0.2) = 0.5
CP AT FACE C = (0.7 + 0.2) = 0.9
CP AT FACE D = (0.7 + 0.2) = 0.9

}\quad \text{CP} = \text{Cpe} + \text{Cpi}

b) SUCTION

PRESSURE COEFFICIENT

CP AT FACE A = (0.7 + 0.2) = 0.9
CP AT FACE B = (0.3 - 0.2) = 0.1
CP AT FACE C = (0.7 - 0.2) = 0.5
CP AT FACE D = (0.7 - 0.2) = 0.5

WIND LOAD ON COLUMN

a) PRESSURE

= 0.5 \times 653.4 \times 6.0 = 1960.2 \text{ N/m} \quad \text{ON FACE A \& B}

b) SUCTION

= 0.9 \times 653.4 \times 6.0 = 3529 \text{ N/m} \quad \text{ON FACE A}

= 0.1 \times 653.4 \times 6.0 = 393 \text{ N/m} \quad \text{ON FACE B}

WIND PARALLEL TO RIDGE (REF. FIG.)

a) PRESSURE

Cpe at A = 0.5
Cpe at B = 0.5
Cpe at C = 0.7
Cpe at D = 0.1
Cpi = 0.2

PRESSURE COEFFICIENT

CP AT FACE A = (0.5 + 0.2) = 0.7
CP AT FACE B = (0.5 + 0.2) = 0.7
CP AT FACE C = (0.7 + 0.2) = 0.9
CP AT FACE D = (0.1 - 0.2) = -0.1
b) SUCTION

PRESSURE COEFFICIENT
CP AT FACE A = (0.5 - 0.2) = 0.3
CP AT FACE B = (0.5 - 0.2) = 0.3
CP AT FACE C = (0.7 - 0.2) = 0.5
CP AT FACE D = (0.1 + 0.2) = 0.3

WIND LOAD ON COLUMN
a) PRESSURE
= 0.9 X 653.4X 6.0 = 3529 N/m ................. ON FACE C
= 0.1 X 653.4X 6.0 = 393 N/m ................. ON FACE D

b) SUCTION
= 0.5 X 653.4X 6.0 = 1961 N/m ................. ON FACE C
= 0.3 X 653.4X 6.0 = 1177 N/m ................. ON FACE D

MOMENT AT BASE
WORST WIND CASE FOR MAX. MOMENT AT THE BASE IS AS SHOWN

TO DETERMINE FORCE IN THE TIE, CONSIDER THE FRAME GIVEN BELOW
NET HORIZONTAL FORCE AT TIE LEVEL DUE TO WIND ON ROOF
\[ = (0.77-0.52) \times \sin 18.43^\circ \times 6.32 \times 3921.4 \]
\[ = 1959 \text{ N} \]

DEFLECTION OF COLUMN AB
\[ = \frac{3529 \times 10^5}{8E} + \frac{T \times 10^5}{3E} - \frac{1959 \times 10^5}{3E} \]

DEFLECTION OF COLUMN CD
\[ = \frac{393 \times 10^5}{8E} - \frac{T \times 10^5}{3E} \]

EQUIVATING DEFLECTION OF AB AND CD
\[ = \frac{3529 \times 10^5}{8E} + \frac{T \times 10^5}{3E} - \frac{T \times 10^5}{3E} \]
\[ = \frac{393 \times 10^5}{8E} - \frac{T \times 10^5}{3E} \]

\[ T = -5195 \text{ N} \] (COMPRESSION)

MOMENT AT BASE OF AB
\[ = \frac{(3529 \times 10^5 \times 2)}{2} + (1959 \times 10^5 ) - (5195 \times 10^5) \]
\[ = 160.5575 \times 10^3 \text{ Nm} \]

SHEAR FORCE AT BASE AB
\[ = (3529 \times 10^5 - 1959 - 5195) \]
\[ = 29901 \text{ N} \]

MOMENT AT BASE OF CD
\[ = \frac{(393 \times 10^5 \times 2)}{2} + (5195 \times 10^5) \]
\[ = 76.211 \times 10^3 \text{ Nm} \]

SHEAR FORCE AT BASE CD
\[ = (393 \times 10^5 + 5195) \]
\[ = 9322 \text{ N} \]

DESIGN FORCES ON COLUMN

<table>
<thead>
<tr>
<th>DESIGN FORCES / LOCATION</th>
<th>TENSION (KN)</th>
<th>COMPRESSION (KN)</th>
<th>SHEAR (KN)</th>
<th>MOMENT (Knm)</th>
</tr>
</thead>
<tbody>
<tr>
<td>COLUMN CAP</td>
<td>16.754</td>
<td>21.915</td>
<td>1.968</td>
<td>-</td>
</tr>
<tr>
<td>COLUMN BASE</td>
<td>1.349</td>
<td>37.32</td>
<td>29.901</td>
<td>160.55</td>
</tr>
</tbody>
</table>
COMPOUND COLUMN PROPERTIES

\[ A = 4 \times 11.75 = 47 \text{ cm}^2 \]
\[ I_{xx} = 4 \times (111.04 + 47 \times (52/2)^2) = 127532.16 \]
\[ I_{yy} = 4 \times (111.04 + 47 \times (27/2)^2) = 9009.91 \]
\[ R_{xx} = 52.09 \text{ cm} \]
\[ R_{yy} = 13.85 \text{ cm} \]
\[ \lambda = 1.5 \times 1050 / 52.09 = 30.236 \]
\[ \lambda = 0.85 \times 1050 / 13.85 = 64.44 \]

DESIGN OF COLUMN SECTION

1. ALLOWABLE AXIAL COMPRRESSIVE STRESS
   \[ (\sigma_{ac \text{ permissible}}) = 136.34 \text{ Mpa (REF. IS800:1984 TABLE 5.1)} \]
2. COMPRESSIVE FORCE = 37320 N
3. TENSILE FORCE = 1349 N
4. SHEAR FORCE = 29901 N
5. BENDING MOMENT = 160550 Nm

**COMPRESSIVE FORCE PER LEG**
- \[ \frac{37320}{4} = 9330 \text{ N} \]
- \[ \frac{160550}{2 \times 0.52} = 154375 \text{ N} \]
- \[ 9330 + 154375 = 163705 \text{ N} \]
- \[ \text{HENCE MAX. COMPRESSIVE FORCE PER LEG} (\sigma_{ac \text{ actual}}) \]
  \[ \frac{163705}{1175} = 139.3234 \text{ Mpa} \]
  \[ 1.33 \times 136.34 > 139.3234 \]
  \[ (\sigma_{ac \text{ actual}}) \]

**CHECK FOR SIMULTANEOUS ACTION OF BENDING MOMENT AND AXIAL TENSION**

MAX. TENSILE FORCE PER LEG DUE TO UPLIFT
- \[ \frac{1349}{4} = 337.25 \text{ Mpa} \]

**COMPRESSIVE FORCE PER LEG DUE TO BENDING MOMENT**
- \[ \frac{160550}{2 \times 0.52} = 154375 \text{ N} \]

**TOTAL COMPRESSIVE FORCE**
- \[ 337.25 + 154712.3 = 154712.3 \text{ N} \]
- \[ \text{HENCE MAX. COMPRESSIVE FORCE PER LEG} (\sigma_{at \text{ actual}}) \]
  \[ \frac{154712.25}{1175} = 131.67 \text{ Mpa} \]
  \[ 1.33 \times 186 > 131.67 \]
  \[ (\sigma_{at \text{ actual}}) \]

**CHECK FOR MAXIMUM DEFLECTION AT EAVES LEVEL**

\[ \text{DEFLECTION AT TOP} = \frac{0.393 \times 1050}{8 \times 2.1 \times 10^6 \times 32142.84} - \frac{5.195 \times 1050}{3 \times 2.1 \times 10^6 \times 32142.84} \]
\[ \text{= 0.915 cm} \]

ALLOWABLE DEFLECTION = SPAN / 325
\[ = \frac{1050}{325} = 3.231 \text{ cm} \]

\[ \text{ALLOWABLE DEFLECTION} > \text{ACTUAL DEFLECTION} \text{ HENCE OK} \]
DESIGN OF COLUMN LACING PARALLEL TO MINOR AXIS (Y-Y AXIS)

USING SINGLE LACING MAKING AN ANGLE OF 48.490 WITH THE AXIS OF COLUMN
THE SPACING BETWEEN THE CONNECTION IS 900 mm (REF. FIG: A)

L/r RATIO FOR THE MAIN MEMBER

\[
L/r = \frac{0.85 \times 90}{3.07} = 24.919 < 50
\]

\[
< 0.7 \times 59.35
\]

HENCE OK

MIN. RATIO FOR COMPOUNDED COLUMN=59.35

TRANSVERSE SHEAR FORCE

\[
= 2.5 \% \text{ AXIAL FORCE} + \text{CALCULATED SHEAR FORCE}
\]

\[
= 0.025 \times 37320 + 29901 = 30834 \text{ N}
\]

FORCE IN EACH LACING MEMBER

\[
= \frac{30834}{2 \times \cos(90 - 48.49)} = 20587.841
\]

TRY SHS 38X38X2.6 @ 2.75 kg /m

AREA

\[
= 3.51 \text{ cm}^2
\]

\[
I_{xx}=I_{yy} = 7.14 \text{ cm}^4
\]

\[
R_{xx}=R_{yy} = 1.43 \text{ cm}
\]

\[
L_{eff} = \frac{0.7 \times 450}{\cos 48.49} = 47.53 \text{ cm}
\]

\[
R_{min} = \frac{47.53}{1.43} = 33.238
\]

ALLOWABLE COMPRESSIVE STRESS = 174.78 Mpa (REF. IS800:1984 TABLE 5.1)

MAX. COMPRESSIVE FORCE

\[
= \frac{20587.841}{351} = 58.655 \text{ Mpa}
\]

\[
< \text{ALLOWABLE COMPRESSIVE STRESS (174.78 Mpa)}
\]

HENCE OK
DESIGN OF COLUMN BATTENING PARALLEL TO MAJOR AXIS (X-X AXIS)

TRANSVERSE SHEAR FORCE (V) = 2.5% OF (DIRECT AXIAL LOAD /2 + 2X AXIAL LOAD PER LEG DUE TO BENDING MOMENT)

= 0.025 X (37320/2 + 2 X 154375)
= 8185.3 N

SHEAR FORCE ON EACH BATTEN MEMBER

= V. C / S = 8185.3 X 450 / 270
= 13642.083 N

BENDING MOMENT ON EACH BATTEN MEMBER

= V.C / 2 = 8185.3 X 0.45 /2
= 1841.681 N

TRY SHS 60X60X2.6 @ 4.55 kg /m

AREA = 5.8 cm²
Zxx=Zyy = 10.44 cm³

MAXIMUM AVERAGE SHEAR STRESS = \(\frac{13642.083 \times 2}{580}\)
= 47.042 Mpa
< 140 MPa (PERMISSIBLE SHEAR STRESS)

HENCE OK

MAXIMUM BENDING STRESS = \(\frac{1841.681 \times 10^3}{10440}\)
= 176.406 Mpa
< 205 MPa (PERMISSIBLE SHEAR STRESS)

HENCE OK

TOTAL WEIGHT OF EACH BUILT UP COLUMN MEMBER
1 WEIGHT OF MAIN MEMBER = 4 X 9.22 X 10.5 = 387.24 Kg
2 WEIGHT OF LACING AND BATTENS

= (2X24X0.668+2X2X0.52)X2.75+(2X24X.23X4.55)
= 144.128 Kg

DESIGN WITH CONVENSIONAL SECTIONS

REF. BIS SP38: 1987
DESIGN SECTION FOR COLUMN : ISMB550 @ 86.9 Kg/m

NOTE: CONSIDERING WIND LOAD AS PER IS 875:1987 AS CALCULATED ABOVE THAT REQUIRED CONVENSIONAL SECTION SHOULD BE ISMB550 @ 103.7 Kg/m TO MAKE ADEQUATE TO TAKE CARE OF INCREASED DESIGNED LOAD.

TOTAL WEIGHT WEIGHT OF ONE COLUMN = 103.7 X 10.5 = 1088.85 Kg
DESIGN OF BRACING

COLUMN HAVE BEEN DESIGNED AS A TIED CANTILEVERED TO RESIST WIND FORCE NORMAL TO THE RIDGE CONSEQUENTLY THE TIE BRACING IN THE L-DIRECTION IS DESIGNED NOMINALLY TO MINIMIZE THE DIFFERENCIAL DEFLECTION OF VARIOUS FRAMES AND TO PROVIDE MORE RIGIDITY.

THE BRACING AT THE LEVEL AT TWO END BAY IS DESIGNED TO TRANSFER WIND LOAD ON THE BUILDING DUE TO WIND PARELLEL TO RIDGE.
DESIGN OF TIE LEVE BRACING

LENGTH OF BRACING MEMBER = $\sqrt{(6^2 + 3.5^2)} = 6.946$ m

SINCE THESE ARE TENSION MEMBER

\[
R_{xx} \text{ REQ.} = \frac{694.6}{350} = 1.985
\]

\[
R_{yy} \text{ REQ.} = \frac{694.6}{2 \times 350} = 0.992
\]

PROVIDE RHS 66X33X2.6 @ 3.69 Kg/m

\[
R_{xx} = 2.31 \text{ cm}
\]

\[
R_{yy} = 1.34 \text{ cm}
\]

TOTAL WEIGHT WITH RHS = $4 \times 5 \times 6.946 \times 3.69 = 512.61$ Kg

DESIGN WITH CONVENSIONAL BRACING

REF. BIS SP38 : 1987

RECOMMENDED SECTION ISA 65X65X6 @ 5.8Kg/m

\[
(R_{xx} = R_{yy} = 1.98 \text{ cm}, R_{vu} = 2.5 \text{ cm}, R_{vv} = 1.26 \text{ cm})
\]

TOTAL WEIGHT = $4 \times 5 \times 6.946 \times 5.8 = 805.74$ Kg

DESIGN OF TIE RUNNERS

TIE RUNNER IS DESIGN ON THE BASIS OF $l/r$ RATIO

LENGTH OF TIE RUNNER = 6 m

\[
R_{min} = \frac{600}{350} = 1.714
\]

PROVIDES SHS 50X50X2.6 @ 3.74 Kg/m

\[
R_{min} = 1.92 \text{ cm}
\]

TOTAL WEIGHT WITH SHS = $3 \times 42 \times 3.74 = 471.24$ Kg

DESIGN WITH CONVENSIONAL BRACING

REF. BIS SP38 : 1987

RECOMMENDED SECTION ISA 90X90X6 @ 8.2Kg/m

\[
(R_{xx} = R_{yy} = 1.75 \text{ cm})
\]

TOTAL WEIGHT = $3 \times 42 \times 8.2 = 1033.2$ Kg
DESIGN OF GABLE WIND COLUMN

WIND LOAD

HEIGHT OF THE COLUMN = 10.5 m
WIND FORCE ON COLUMN = 0.9 X 653.4 X (3.5+2.5)/2
= 1765 N/m
BENDING MOMENT \( M_{\text{max}} \) = 1765 X 10.5 \( ^2 \)/8
= 24323.91 Nm
SHEAR FORCE AT BASE = 5/8 X 1765 X 10.5
= 11582.8 N

DEAD LOAD

SELF WEIGHT @500 N/m = 500 X10.5 =1050 N
WEIGHT OF SIDE GIRT @100N/m = (3.5 X 7/2 +5 X 4/2) X100
= 2225 N
WEIGHT OF C.G.I. SHEETING = (3.5 X10.5/2 +5 X 10.5/2 ) X60
= 2677.5 N
TOTAL = 5952.5 N

\[ A = 4 \times 5.25 = 21 \text{ cm}^2 \]
\[ I_{xx} = 4(18.99+21(40/2)^2) \]
= 33675.96 cm\(^4\)
\[ I_{yy} = 4(18.99+21(18/2)^2) \]
= 3624.96 cm\(^4\)
\[ R_{xx} = \sqrt{I_{xx}}/A \]
= 40.045 cm
\[ R_{yy} = \sqrt{I_{yy}}/A \]
= 13.14 cm
\[ \lambda = I_{xx}/ R_{xx} = 26.2211 \]
\[ \lambda = I_{yy}/ R_{yy} = 79.909 \]

ALLOWABLE AXIAL COMPRESSIVE STRESS

\( (\sigma_{\text{ac permissible}}) = 179.578 \text{ Mpa} \) (REF. IS800:1984 TABLE 5.1)

COMPRESSIVE FORCE PER LEG DUE TO D.L. = \( \frac{5952.5}{4} \) = 1488.125 Mpa

COMPRESSIVE FORCE PER LEG DUE TO MOMENT
= \( \frac{24323.91}{2 \times 0.4} \) = 30404.88281 Mpa

TOTAL COMPRESSIVE FORCE PER LEG = 1488.125+30404.88 = 31893.00781 Mpa

MAX. COMPRESSIVE STRESS PER LEG DUE TO DEAD LOAD & BENDING MOMENT
= \( \frac{31893.01}{525} \) = 60.75 Mpa

1.33 X 179.578 > 60.749
\( (\sigma_{\text{ac permissible}}) > (\sigma_{\text{ac actual}}) \) 
HENCE SAFE
DESIGN OF COMPOUND COLUMN LACING

CONSIDER SINGLE LACING MEMBER AT 45° WITH THE VERTICAL AS SHOWN
UNSUPPORTED LENGTH = 900 mm
LENGTH OF LACING MEMBER = 450 / cos 45 = 637

MAX. SHEAR FORCE
= 2.5% OF AXIAL LOAD + CALCULATED SHEAR
= 0.025 x 5952.5 + 11582.5
= 11731.62 N

MAX. AXIAL LOAD ON LACING MEMBER = 11731.62 / cos 45 = 16591.01 N

TRY 80X40X2.6 @ 4.55 KG/m

Rxx = 2.84 cm  A = 5.8 cm²
Ryy = 1.65 cm
L_{ef} = 0.7 x 637 = 446
L_{ef}/R_y = 446/1.65 = 27.03

ALLOWABLE AXIAL COMPRRESSIVE STRESS
(\sigma_{ac\,permissible}) = 179.485 Mpa (REF. IS800:1984 TABLE 5.1)

MAX. COMPRESSIVE FORCE = 16591.01 / 580 = 28.605 Mpa

< ALLOWABLE COMPRESSIVE STRESS (179.485 Mpa)

HENCE OK
DESIGN OF BRACING FOR WIND PERPENDICULAR TO RIDGE

WIND DRAG ON ROOF AND WALL

\( \frac{d}{h} = 4 \)

\( d = 42.0 \text{ m}, \ h = 10.5 \text{ m}, \ b = 12 \text{ m} \)

As \( \frac{d}{h} = 4 \), wind drag on wall and roof should not be considered.

TIE LEVEL BRACING AT GABLE END

Windword side bracing (Refer Fig. Below)

Wind force at node (2) & (3)

Reaction from gable end column

\[ \frac{(3.5 + 5)}{2} \times 653.4 \times 0.9 = 2500 \text{ N} \]

Wind force at node (1) & (4)

Assuming extra projection 250 mm, \( \text{Leff} = 3.5 + 0.23 = 3.73 \text{ m} \)

Reaction from gable end column

\[ \frac{3.73 \times 653.4 \times 0.9}{2} = 1097 \text{ N} \]

Length of bracing member

\[ \sqrt{(6^2 + 3.5^2)} = 6.946 \text{ m} \]

Maximum tension in bracing

\[ \frac{2500 \times 6.946}{6} = 2895 \text{ N} \]

Net effective area required

\[ \frac{2895}{205 \times 1.33 \times 100} = 0.106 \text{ cm}^2 \]

Length of bracing member

\[ \sqrt{(6^2 + 5^2)} = 7.81 \text{ m} \]

Net effective area required

\[ \frac{1097}{205 \times 1.33 \times 100} = 0.04 \text{ cm}^2 \]
SINCE THESE ARE TENSION MEMBER \((L/r) = 350\)

\[
R_{xx} \text{ REQ.} = \frac{781}{350} = 2.231
\]

\[
R_{yy} \text{ REQ.} = \frac{781}{2 \times 350} = 1.116
\]

PROVIDE RHS 66X33X2.6 @ 3.69 Kg/m

\[
R_{xx} = 2.31 \text{ cm} \\
R_{yy} = 1.34 \text{ cm}
\]

TOTAL WEIGHT WITH RHS = \(2 \times (4 \times 6.946 + 2 \times 7.81) \times 3.69 = 320.32 \text{ Kg}\)

DESIGN WITH CONVENSIONAL BRACING SECTION

REF. BIS SP38 : 1987

RECOMMENDED SECTION ISA 70X70X6 @ 6.3Kg/m

TOTAL WEIGHT = \(2 \times (4 \times 6.946 + 2 \times 7.81) \times 6.3 = 547 \text{ Kg}\)

RAFT BRACING

RAFT BRACING IS PROVIDED IN THE END PAIR OF TRUSSES TO TAKE CARE OF ERRECTION LOAD.

EXTREME TWO SETS PURLIN ARE TO BE CONNECTED WITH BRACING MEMBER.

LENGTH OF BRACING MEMBER = \(\sqrt{(6^2 + 6.32^2)} = 8.714 \text{ m}\)

SINCE THESE ARE TENSION MEMBER \((L/r) = 350\)

\[
R_{xx} \text{ REQ.} = \frac{871.4}{350} = 2.490
\]

\[
R_{yy} \text{ REQ.} = \frac{871.4}{2 \times 350} = 1.245
\]

PROVIDE RHS 80X40X2.6 @ 4.55 Kg/m

\[
R_{xx} = 2.31 \text{ cm} \\
R_{yy} = 1.34 \text{ cm}
\]

TOTAL WEIGHT WITH RHS = \((2 \times 4 \times 8.714 \times 4.55) = 317.19 \text{ Kg}\)

DESIGN WITH CONVENSIONAL BRACING SECTION

REF. BIS SP38 : 1987

RECOMMENDED SECTION ISA 80X80X6 @ 7.3Kg/m

TOTAL WEIGHT = \((2 \times 4 \times 8.714 \times 7.3) = 509 \text{ Kg}\)

WIND BRACING IN BAY (D) - (E)

(REFER FIG. BELOW)
DESIGN OF EAVES BEAM AND VERTICAL BRACING

EAVES BEAM
FORCE FROM TIE LEVEL BRACING = 2500 + 1097 = 3597 N

\[ R_{xx} = \frac{600}{250} = 2.4 \text{ cm} \]

TRY SECTION 72 X72X 3.2 @ 6.71 Kg/m
\[ A = 8.54 \text{ cm}^2, \ R_{\text{min}} = 2.79 \text{ cm} \]
MAXIMUM AXIAL STRESS = \[ \frac{3597}{854} \] = 4.21 Mpa
\[ \frac{L}{R} = \frac{600}{2.79} = 215.054 \]
ALLOWABLE COMPRESSIVE STRESS = 24.75 Mpa REF. IS800:1984 TABLE 5.1) > MAXIMUM AXIAL STRESS

HENCE OK

TOTAL WEIGHT WITH SHS = (2 X 42 X 6.71) 563.64 Kg

DESIGN WITH CONVENSIONAL BRACING SECTION
REF . BIS SP38 : 1987
RECOMMENDED SECTION ISMB250 @ 37.3Kg/m
TOTAL WEIGHT = (2 X 42 X 37.3) 3133.2 Kg

BRACING IN VERTICAL PLANE BETWEEN (D) & (E) COLUMN
LENGTH OF BRACING MEMBER = \( \sqrt{(6^2 + 5.25^2)} \) = 7.97m

SINCE THESE ARE TENSION MEMBER \( (L/r) = 350 \)
\[ R_{xx} \text{ REQ.} = \frac{797}{350} = 2.277 \]
\[ R_{yy} \text{ REQ.} = \frac{797}{2X350} = 1.139 \]
MAXIMUM TENSION IN BRACING = \[ \frac{3597 \times 7.97}{6} \] = 4778N

PROVIDE RHS 80X40X2.6 @ 4.55 Kg/m
\[ R_{xx} = 2.84 \text{ cm} \ A = 5.8 \text{ cm}^2 \]
\[ R_{yy} = 1.65 \text{ cm} \]
MAX. TENSILE STRESS = \[ \frac{4778}{580} \] = 8.24 Mpa
\[ < \frac{1.33 \times 205}{\text{Mpa}} \]
HENCE OK

TOTAL WEIGHT WITH RHS = (2(4 X 7.97 X 4.55) = 290Kg

DESIGN WITH CONVENSIONAL BRACING SECTION
REF . BIS SP38 : 1987
RECOMMENDED SECTION ISA70X70X6 @ 6.3Kg/m
TOTAL WEIGHT = (2(4 X 7.97 X 6.3) = 401.69Kg

THE FOUNDATION OF THESE COLUMNS TO BE CHECKED FOR ADDITIONAL AXIAL FORCE TUE TO BRACING FORCE
DESIGN OF GABLE END BRACING

SINCE COLUMN ARE DESIGNED AS PROPPED CANTILEVER TO RESIST WIND FORCE, THE GABLE END BRACING AS SHOWN BELOW ARE DESIGNED NOMINALLY FOR OVERALL STIFFNESS OF THE STRUCTURE.

LENGTH OF BRACING MEMBER  = \sqrt{(3.5^2 + 5.25^2)}  =  6.31 m

SINCE THESE ARE TENSION MEMBER  \( L/r \) = 350

\[ R_{xx} \text{ REQUIREMENT} = \frac{631}{350} = 1.803 \]

\[ R_{yy} \text{ REQUIREMENT} = \frac{631}{2 \times 350} = 0.901 \]

PROVIDE RHS 66X33X2.6 @ 3.69 Kg/m

\[ R_{xx} = 2.31 \text{ cm} \quad A = 4.7 \text{ cm}^2 \]
\[ R_{yy} = 1.34 \text{ cm} \]

TOTAL WEIGHT WITH RHS = \( 2(8 \times 6.31 \times 3.69) = 372.54 \text{ Kg} \)

DESIGN WITH CONVENTIONAL BRACING SECTION

REF : BIS SP38 : 1987

RECOMMENDED SECTION ISA55X55X6 @ 4.9Kg/m

TOTAL WEIGHT = \( 2(8 \times 6.31 \times 4.9) = 495 \text{ Kg} \)

NOTE :

DETAIL DESIGN OF DIFFERENT CONNECTION LIKE COLUMN BASE PLATE, ROOF TRUSS GUSSET PLATE, SUPPORT ETC. ARE NOT WORKED OUT HERE TO LIMIT THE SIZE OF THIS HANDBOOK WHICH MAY BE DONE FOLLOWING ANY STANDARD PROCEDURE BASED ON CORRESPONDING REACTION VALUE. HOWEVER FOR CALCULATION OF TOTAL WEIGHT, CONNECTING PLATE WEIGHTS HAVE BEEN CONSIDERED AS PER BIS PROVISION FOR CONVENTIONAL DESIGN & AS PER PRACTICAL EXPERIENCE FOR RHS / SHS DESIGN.