

# **Analysis and Design of Steel Shelter – IS800**

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## 1.0 GENERAL

This document covers the Design Calculation of steel Shelter structure.

### 1.1 Scope

This document contains the following

- a) Design Basis comprising of description of the structure, structural analysis methodology
- b) Load Calculation
- c) STAAD Model comprising of framing sketches and input file.
- d) Design of foundation, column, beams and slabs.

### 1.2 Units of Measurement

SI units are followed in the design calculations.

### 1.3 References

The following codes and standards with the latest revisions and drawings have been referred for the structural analysis and design.

#### 1.3.1 Codes and Standards

IS 456	-	Code of Practice for Plain and Reinforced Concrete
SP 16	-	Design Aids for Reinforced Concrete to IS: 456-1978
IS 800	-	Code of Practice for General Construction in Steel
IS 875	-	Code of Practice for Design Loads for Buildings and Structures (other than Earthquake)
Part 1	-	Dead Loads – Unit Weight of Building Material and Stored Materials.
Part 2	-	Imposed Loads
Part 3	-	Wind Loads
Part 5	-	Special Loads and Load Combinations
IS 1893	-	Criteria for Earthquake Resistant Structures
SP:6 (Part-1)	-	Structural Steel Sections

#### 1.4 Computer Program / Software

STAAD.Pro software is used for structural analysis. Microsoft Excel Spreadsheet is used for load calculations and designs.

## 2.0 MATERIALS

### 2.1 CEMENT

Higher grade of Ordinary Portland cement of Grade-43 conforming to IS: 8112 for RCC works

## 2.2 REINFORCEMENT STEEL

Reinforced Steel High Yield Strength TMT deformed steel bars of grade Fe-500 (CRS) bars conforming to IS: 1786 shall be used.

## 2.3 CONCRETE

M35 Grade of concrete shall be used for Beams, Column and foundations

Following materials are generally used in the design of the various components of the structure.

1. All Structural rolled Steel members are of grade Yield strength (Yst) 250 and Columns plates are Yield strength (Yst) 345
2. Grade 8.8 cl HYSD bolts are used for connection and anchor bolts.
3. M30 grade concrete for reinforced concrete of the structure.
4. Fe500 grade reinforcement steel for all diameters.
5. Brick / Block masonry for all walls.

The self-weight of the various elements are computed based on the unit weight of materials as given below:

<b>Materials</b>	<b>Unit weight kN/m<sup>3</sup></b>
Steel	78.50
Plain Cement Concrete	24.00
Reinforced Cement Concrete	25.00
Soil Fill (Landscape)	20.00 (Saturated)

### 3.0 DESIGN DATA AND STRUCTURE DESCRIPTION

The bearing capacity recommendations shall be per the soil investigation report

### 4.0 DESCRIPTION OF STRUCTURES AND DESIGN METHODOLOGY

#### 4.1 Outline of Structure

Potable water steel structure with 2Ton Monorail capacity for the maintenance

Geometry of Structure

Length – 6.0m

Height –6.0m from FFL

#### 4.2 Design Methodology

1. The Super structure of the building comprising of the steel framing system is modeled as a 3D model in STAAD.Pro software.
2. The Dead load, Live load etc. that come on to the structure are calculated manually using spread sheet taking values from the Specification document and corresponding Codal provisions.
3. The calculated loads are then applied on the model and the analysis has been carried out using Staad.Pro
4. Different load combinations with dead load, live load, wind load, seismic load etc as applicable for the super structure and the substructure design considered are furnished.
5. The analysis and design is done in STAAD.Pro software.

### 5.0 BASIC LOADS AND LOAD COMBINATION

The following basic load cases and load combinations are considered.

#### Basic Loads

1	DL1	Dead Load
2	DL2	Equipment Load
3	LL	Live Load
4	CRL	Crane load/Monorail Load
5	WL(+X)	Wind load in positive X direction
6	WL(-X)	Wind load in negative X direction
7	WL(+Z)	Wind load in positive Z direction
8	WL(-Z)	Wind load in negative Z direction
9	SLX	Seismic Load in X direction
10	SLZ	Seismic Load in Z direction

#### Load Combinations

Unfactored load combinations are used for Foundation and base slab stability check like bearing pressure check and Factored load combinations are used for structural design as per limit state design. Hereunder, whenever DL and LL is combined with seismic / wind load,

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Engineering Concepts

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factor of 1.2 is used for all the loads. Whenever DL is combined with seismic/ wind load, factor of 1.5 is used for all the loads. The Load Combinations considered are given below.

Load Combination shall be in accordance with IS 800-2000. Load combinations are as per follows

<b>Load Combinations for Serviceability Limit State</b>				
<b>Load Comb.</b>	<b>DL</b>	<b>LL</b>	<b>WL</b>	<b>EQ.</b>
1	1.0			-
2	1.0	1.0	-	-
3	1.0	0.8	0.8	
4	1.0	0.8		0.8
5	1.0		1.0	
6	1.0			1.0

<b>Load Combinations for Strength Limit State</b>					
<b>Load Comb.</b>	<b>DL</b>	<b>LL</b>	<b>CL (Accompanying)</b>	<b>WL</b>	<b>EQ.</b>
1	1.5				
2	1.5	1.5			
3	1.2	1.2	1.05	0.53	
4	1.2	1.2			0.53
5	1.2	1.2	1.05	1.2	
6	1.2	1.2			1.2
7	1.5			1.5	
8	1.5				1.5

# **Annexure A – Super Structure Primary Loads**

**6.0 DEAD LOAD**

Dead Load includes self-weight of the structural elements and dead load of walkway. Self-weight of the structure is found using self-weight command in STAAD Pro. 20% included for structural connection weight.

**7.0 LIVE LOAD**

Live load on roof =  $75 \text{ kg/m}^2 = 0.75 \text{ kN/m}^2$

**8.0 MONORAIL LOAD**

Monorail loading 2T of lifting capacity =  $20\text{kN}+5\text{kN} = 25\text{kN} \times 1.25 \text{ (impact)} \times 50\% = 31.25\text{kN}$   
Vertical load with impact 10% for Horizontal load

**9.0 WIND LOAD X AND Z DIRECTION**

**Load A.3 Wind Load (WL)**

Wind Load Calculation (IS 875 (PART-3):2015)					
	Basic wind Speed considered ( $V_b$ ) =			39.00	m/sec
	Design Wind Speed is given by $V_z = K_1 \times K_2 \times K_3 \times K_4 \times V_b$				
	Risk Coefficient, $K_1$ (Table:1 IS 875 - PART 3) =			1.06	
Table 2:	Terrain, Height and structure size Coefficient ( $K_2$ - 0m to 10m)=			1.00	
	(for Terrain Category = 2)				
	Topography Factor ( $K_3$ ) =			1.00	
	Importance factor for cyclonic region ( $K_4$ )			1.15	
	Design Wind Speed is given by $V_z$ (0m- 10m) =	$39 \times 1.06 \times 1 \times 1 \times 1.15 =$		47.54	m/sec
	Wind Pressure (0m - 10m) =	$p_z$	$0.6 \times 47.54^2 =$	1.36	$\text{kN/m}^2$
	Tributary area		$17.6 \times 7$	123.20	$\text{m}^2$
	Wind directionality factor	$K_d$		1.00	
CL 7.2.2	Area averaging factor	$k_a$		0.80	
	Combination Factor	$k_c$		1.00	
	Design wind Pressure $P_d =$	$K_d \times K_a \times K_c \times P_z$			
	Design Wind Pressure (0m - 10m) =	$p_d = K_d \times K_a \times K_c \times P_z$	$1.356 \times 1 \times 0.8 \times 1 =$	1.08	$\text{kN/m}^2$
	Ref fig 4B of	Force coefficient for clad building =		1.2	
	IS 875 (p III)	Wind pressure acting on building $= x/1000 =$		1.30	$\text{kN/m}^2$
	Minimum Design Wind pressure design Basis			1.50	$\text{kN/m}^2$

**Building Dimensions:**

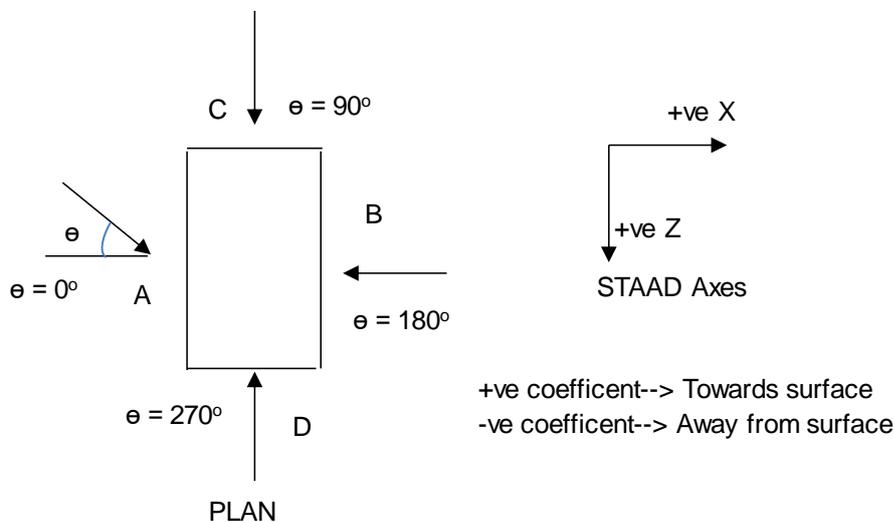
Length of the building	$l$	$=$	<b>6.00</b>	m
Width of the building	$w$	$=$	<b>6.000</b>	m
Height of the building	$h$	$=$	<b>6.000</b>	m
Slope of the roof	1 in 10	$=$	<b>10.0</b>	deg

**Pd from design basis** **1.50** **kN/m<sup>2</sup>**

**Pressure Coefficients:**

Internal pressure coefficient	$C_{pi}$	$=$	<b>0.7</b>	(Cl.6.2.3.1 of IS 875- PART-3)
Opening area			<b>16.0</b>	m <sup>2</sup>
building area			<b>36.0</b>	m <sup>2</sup>
(for % of openings > 20)				
Height to width	$h/w$	$=$	<b>1</b>	( $1/2 < h/w$ )
length to width	$l/w$	$=$	<b>1</b>	( $3/2 < l/w < 4$ )

External Pressure Coefficient:



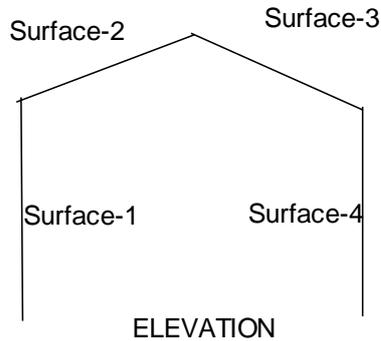
C<sub>pe</sub> for walls:

Table 4 of IS: 875 (III)

External Pressure Coefficient(C<sub>pe</sub>)

$\theta$	A	B	C	D
<b>0</b>	<b>0.70</b>	<b>-0.25</b>	<b>-0.6</b>	<b>-0.6</b>
<b>90</b>	<b>-0.6</b>	<b>-0.6</b>	<b>0.70</b>	<b>-0.25</b>

C<sub>pe</sub> for Roof:



Surface 5 is gable end on side 'C'  
Surface 6 is gable end on side 'D'

For roof slope = 11deg

θ	EF	GH	EG	FH
0	-1.1	-0.60		
90			-0.80	-0.6

Overall pressure coefficient =  $C_{pe} \pm C_{pi}$

Notation

- Vind Transverse (+ve Internal Pressure)
- Vind Transverse (-ve Internal Pressure)
- Vind Longitudinal (+ve Internal Pressure)
- Vind Longitudinal (-ve Internal Pressure)

Load Case	Overall Pressure Coefficient					
	Surface-1	Surface-2	Surface-3	Surface-4	Surface-5	Surface-6
WLx P	0.00	-1.80	-1.30	-0.95	-1.30	-1.30
WLx S	1.40	-0.40	0.10	0.45	0.10	0.10
WLz P	-1.30	-1.50	-1.30	-1.30	0.00	-0.95
WLz S	0.10	-0.10	0.10	0.10	1.40	0.45

Design wind load on surface =  $(C_{pe} \pm C_{pi}) * Pd * \text{tributory width}$  kN/m

**Main Frame**

Frame	Trib, m
Frame-1 and 4	3
Frame-2 to 3	6
Gable end	7
Gable end	3.5

**Gable End**

Col	Trib, m
Col -1	7
Col -2	7

Loads on Frame-1

Load Case	Design Wind Load (kN/m)					
	Surface-1	Surface-2	Surface-3	Surface-4	Surface-5	Surface-6
WLx P	0.00	-8.10	-5.85	-4.28	-6.83	-6.83
WLx S	6.30	-1.80	0.45	2.03	0.53	0.53
WLz P	-5.85	-6.75	-5.85	-5.85	0.00	-4.99
WLz S	0.45	-0.45	0.45	0.45	7.35	2.36

**11.4 Seismic Load (SL)**

**SEISMIC LOAD CALCULATION**

**LOAD CASE 9 & 10 -SEISMIC LOADS**

(Refer : IS 1893, Part 1 & 4)

Seismic Zone Factor (Seismic zone III, per IS 1893-2005, Part IV)	Z	=	<b>0.10</b>	(Table 2, IS-1893 (Part I) : 2002)
Importance factor	I	=	<b>1.50</b>	(Table 2 IS-1893 (Part 4) : 2002)
Damping factor		=	<b>5.0%</b>	( IS-1893 (Part I) : 2002 Cl 7.8.2.1)

**Horizontal Seismic Coefficient for X direction**

Response reduction factor	R	=	<b>5.0</b>	(Table 7, IS-1893 (Part I) : 2002)
	$(Z/2) \times (I/R)$	=	$(0.1/2) \times (1.5/5)$	
		=	<b>0.015</b>	

**Horizontal Seismic Coefficient for Z direction**

Response reduction factor	R	=	<b>5.0</b>	(Table 7, IS-1893 (Part I) : 2002)
	$(Z/2) \times (I/R)$	=	$(0.1/2) \times (1.5/5)$	
		=	<b>0.015</b>	

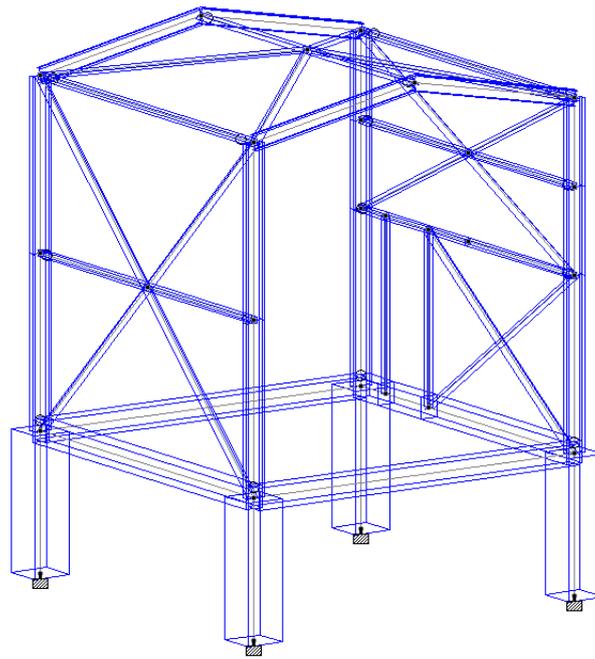
Sa/g values are provided in the table below

**Average Response Acceleration Coefficient**

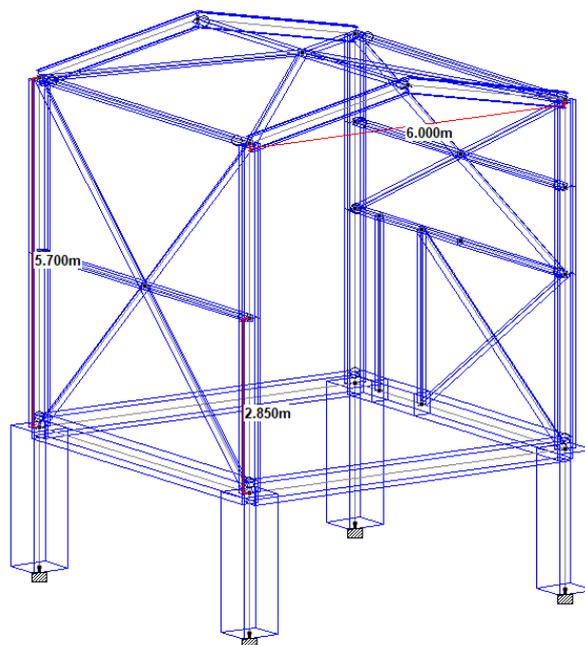
S.No	Time Period (Sec)	Damping Factor (as a % of critical damping)	
		5%	2%
1	<b>0.00</b>	1.000	1.400
2	<b>0.05</b>	1.750	2.450
3	<b>0.10</b>	2.500	3.500
4	<b>0.20</b>	2.500	3.500
5	<b>0.30</b>	2.500	3.500
6	<b>0.40</b>	2.500	3.500
7	<b>0.45</b>	2.500	3.500
8	<b>0.50</b>	2.500	3.500
9	<b>0.55</b>	2.500	3.500
10	<b>0.75</b>	1.813	2.539
11	<b>0.95</b>	1.432	2.004
12	<b>1.15</b>	1.183	1.656
13	<b>1.35</b>	1.007	1.410

S.No	Time Period (Sec)	Damping Factor (as a % of critical damping)	
		5%	2%
14	<b>1.55</b>	0.877	1.228
15	<b>1.75</b>	0.777	1.088
16	<b>1.95</b>	0.697	0.976
17	<b>2.15</b>	0.633	0.886
18	<b>2.35</b>	0.579	0.810
19	<b>2.55</b>	0.533	0.747
20	<b>2.75</b>	0.495	0.692
21	<b>2.95</b>	0.461	0.645
22	<b>3.15</b>	0.432	0.604
23	<b>3.35</b>	0.406	0.568
24	<b>3.55</b>	0.383	0.536
25	<b>3.75</b>	0.363	0.508
26	<b>3.95</b>	0.344	0.482

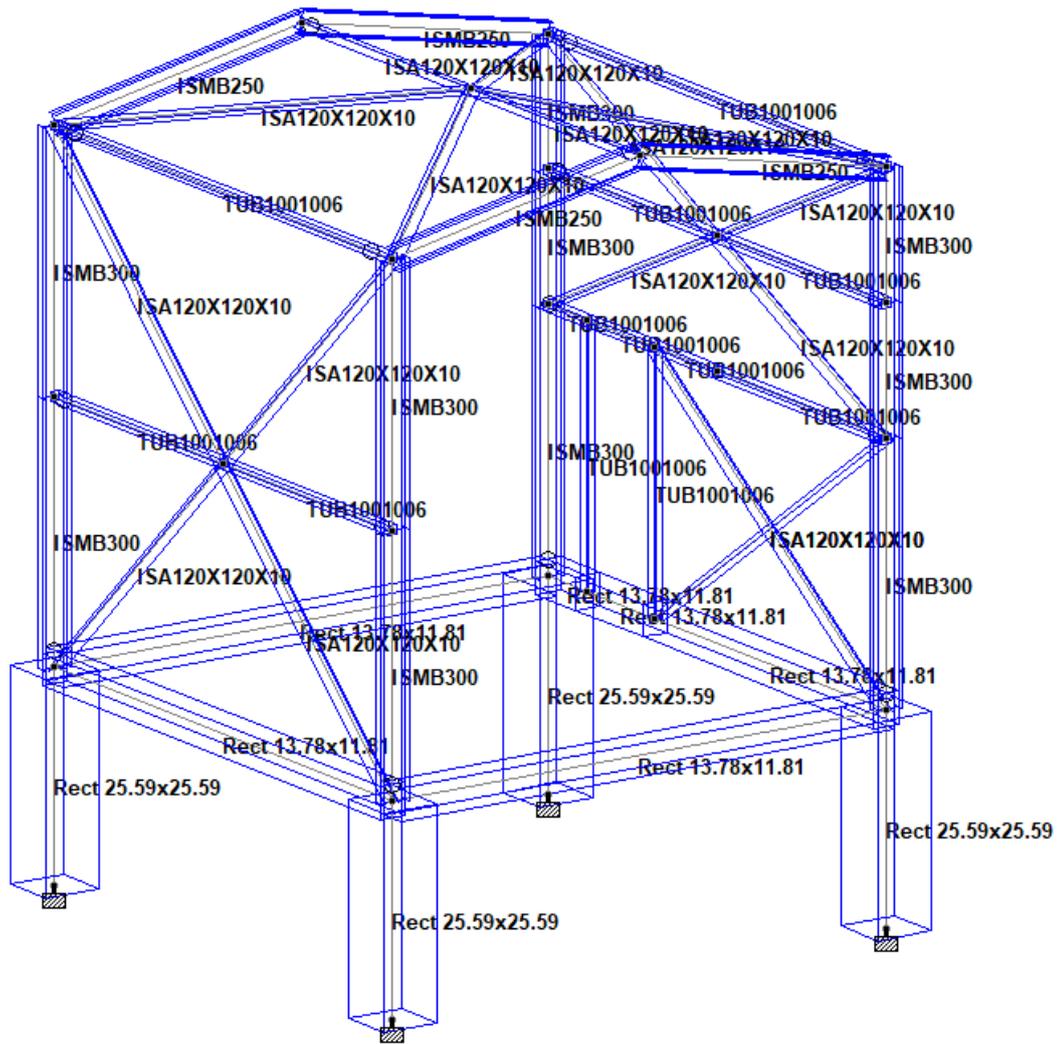
# **Annexure B STAAD Model**



**Staad Iso- Metric view of structure**



**Staad Iso- Metric view of structure with member dimensions**

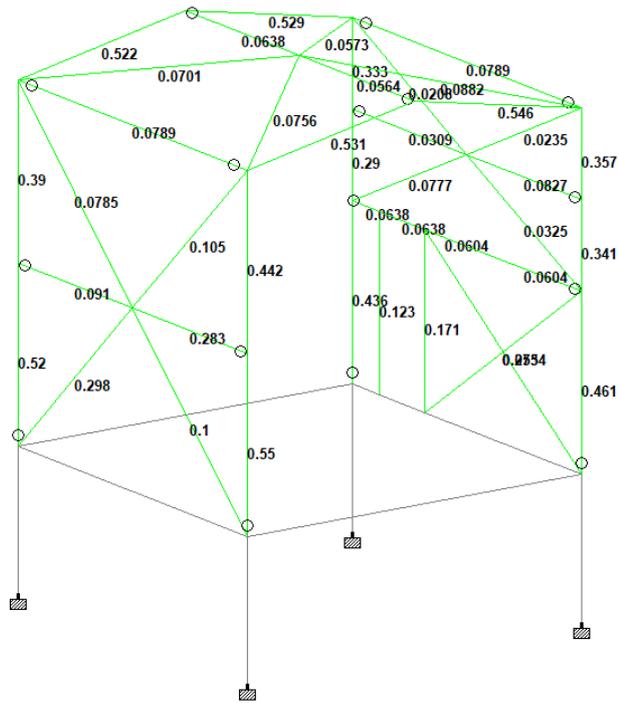


Beam and Column Member Sizes

# Annexure B.1 STAAD INPUT

# **Annexure C**

## **Steel design Unity Ratio and Deflection summary**



Beam and Column Unity Ratio

Beam	Analysis Property	Allowable Ratio	Normalized Ratio (Actual/Allowable)	Clause	L/C
1873	ISMB300	1	0.52	Sec. 9.3.2.2	58
2103	TUB1001006	1	0.079	Sec. 8.2.1.2	50
2535	ISMB250	1	0.522	Sec. 9.3.2.2	52
2727	ISA120X120X10	1	0.064	Sec. 9.3.2.2	52

**For Deflection**

	Node	L/C	Horizontal X mm	Vertical Y mm	Horizontal Z mm	Resultant mm	Rotational rX rad	rY rad	rZ rad
Max X	1066	14 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI -0.5]	10.51	-0.228	-1.158	10.576	0	0	0
Min X	1373	15 1.0DL+0.8LL+0.8CL+0.8W.L (+Z)[CPI +0.5]	-1.418	-0.091	-0.364	1.467	0	0.001	0
Max Y	1379	19 1.0DL+1.0WL(+X)[CPI +0.5]	8.129	1.814	-0.607	8.351	0	0	0
Min Y	1381	12 1.0DL+1.0LL+1.0CL	2.529	-8.434	-0.417	8.815	0	0	0
Max Z	1375	13 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI +0.5]	3.182	-0.07	0.157	3.187	0	0	-0.002
Min Z	1392	21 1.0DL+1.0WL(+Z)[CPI +0.5]	2.225	-0.053	-1.991	2.986	0	0	0
Max rX	1065	22 1.0DL+1.0WL(+Z)[CPI -0.5]	0.312	-0.077	-0.448	0.551	0.004	0.001	-0.001
Min rX	1065	19 1.0DL+1.0WL(+X)[CPI +0.5]	9.604	0.013	0.032	9.604	-0.003	0	-0.001
Max rY	1065	22 1.0DL+1.0WL(+Z)[CPI -0.5]	0.312	-0.077	-0.448	0.551	0.004	0.001	-0.001
Min rY	1065	19 1.0DL+1.0WL(+X)[CPI +0.5]	9.604	0.013	0.032	9.604	-0.003	0	-0.001
Max rZ	1050	12 1.0DL+1.0LL+1.0CL	3.534	-0.23	-1.028	3.688	0	0	0.002
Min rZ	1376	14 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI -0.5]	5.14	-0.141	-1.169	5.273	0	0	-0.003
Max Rst	1380	14 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI -0.5]	9.553	-5.239	-0.539	10.909	0	0	0

Deflection For column allowable –  $H/200 = 28.5\text{mm}$  Hence safe in Column Deflection  
 For LC 12 Vertical deflection for beam =  $8.45\text{mm}$ ;  $L/500 = 12.6\text{mm}$  allowable hence safe in beam vertical deflection

# **Annexure D**

## **Base plate and Anchor bolt design**

## DESIGN OF BASE PLATE & ANCHOR BOLT:

BASEPLATE TYPE: **BP1**

Grid : **0** Node No.: **1383**

### DESIGN DATA :-

STEEL COLUMN SIZE **ISMB300** D = **300 mm**  
T<sub>f</sub> = **8 mm**

### BASE PLATE SIZE

Length L = **500 mm**  
Breadth B = **500 mm**  
Thickness T = **25 mm**

### CONCRETE PEDESTAL SIZE

Length L<sub>p</sub> = **650 mm**  
Breadth B<sub>p</sub> = **650 mm**  
Assume Size of Bolt  $\phi$  = **20 mm**  
Assume Total Nos. of Bolt n = **4 Nos.**  
Edge Dist. for Bolt ed = **50 mm**  
Edge To Centre of Column Flange **104 mm**

### BASE PLATE SIZE BETWEEN STIFFENERS

Distance of bolt center from column flange ef = **50 mm**  
Distance between stiffeners ds = **75 mm**  
Distance between centre of stiffener to edge of base plate = **25 mm**  
No. of stiffeners = **2 nos**

Bolts on both sides of column flange ▼

Centre To Centre of Bolt s = **292 mm**  
Projection of base plate from Column l' = **100 mm**

Concrete grade **35**

Characteristic comp. strength f<sub>ck</sub> = **35 N/mm<sup>2</sup>** *IS:456-2000 Table 2*  
Permissible stress in bending comp.  $\sigma_{cbc}$  = **8.5 N/mm<sup>2</sup>** *IS:456-2000 Table 21*

Permissible bearing stress  $\sigma_b$  = (min of sqrt(A<sub>1</sub>/A<sub>2</sub>), 2) \* 0.25f<sub>ck</sub>  
*IS:456-2000 cl. 34.4*

Area of pedestal, A<sub>1</sub> = L<sub>p</sub> \* B<sub>p</sub> = **422500 mm<sup>2</sup>**  
Area of base plate, A<sub>2</sub> = L \* B = **250000 mm<sup>2</sup>**  
sqrt (A<sub>1</sub>/A<sub>2</sub>) = **1.30**

Permissible bearing stress  $\sigma_b$  = 1.30 x 0.25 x 35  
= **11.38 N/mm<sup>2</sup>**

Permissible bond stress in tension  $\tau_{bd}$  = **0.9 N/mm<sup>2</sup>** *IS:456-2000 Table 21*

m -Modulur Ratio 280/3 $\sigma_{cbc}$  = **10.98** *IS:456-2000, Cl B-2.1.2*

### BOLTS:

Permissible Tensile stress  $\sigma_{tf}$  = **240 N/mm<sup>2</sup>** *Cl.4.6 Grade bolt*

### BASE PLATE:

Yield stress f<sub>y</sub> = **240 N/mm<sup>2</sup>** *(As per Table-3, IS2062:1999)*

**TYPICAL CALCULATION FOR DESIGN OF BASE PLATE & ANCHOR BOLT FOR  $e > L/6$  (TYPE 2)**

BASEPLATE TYPE: BP1

Grid : 0 Node No.: 1383

**LOAD DATA :-**

Unfactored loads from Support Reactions

Vertical Load	$P_c$	=	126.14 kN	(Ref: Annexure-A2)
Moment	$M_x$	=	15.99 kNm	

Load combination LC = 111 1.0DL+1.0SLX+1.0SLYX+1.0TL+1.0DL2

Factor for permissible stress increase for this load combination

Factor for bearing pressure	=	1.00
Factor for bolt tensile strength	=	1.25
Factor for base plate design	=	1.33

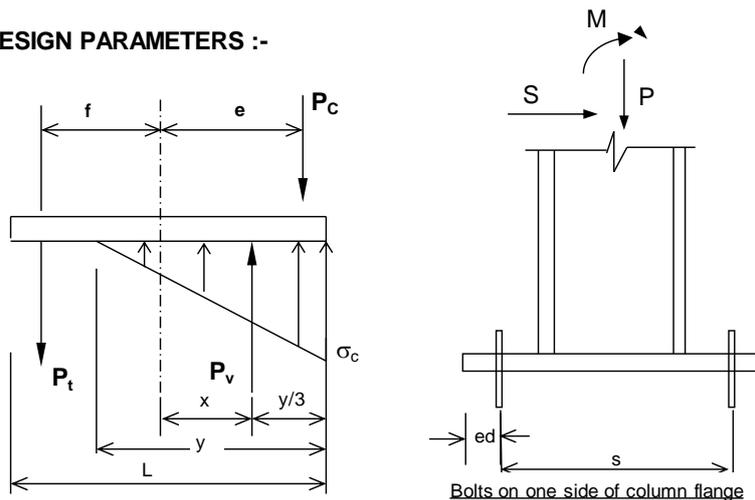
Permissible stresses after including the above factor

Permissible bearing stress	$\sigma_b$	=	11.38 x 1
		=	11.38 N/mm <sup>2</sup>
Permissible bond stress in tension	$\tau_{bd}$	=	0.90 x 1
		=	0.90 N/mm <sup>2</sup>
Permissible Tensile stress in bolts	$\sigma_{tf}$	=	240 x 1.25
		=	300 N/mm <sup>2</sup>

**DESIGN DATA :-**

COLUMN SIZE	ISMB300	D	=	300 mm
BASE PLATE SIZE				
Length	L	=	500 mm	
Breadth	B	=	500 mm	
Thickness	T	=	25 mm	
Assume Size of Bolt	$\phi$	=	20 mm	
Assume Total Nos. of Bolt		=	4 Nos.	
Edge Dist. for Bolt	ed	=	50 mm	
Centre To Centre of Bolt	s	=	292 mm	
Projection of Plate From Column	l'	=	100 mm	
m -Modular Ratio		=	280/3 $\sigma_{cbc}$	
		=	10.98	

**DESIGN PARAMETERS :-**



**Calculation of Tension in Anchor bolts:**

Ref: Structural Steelwork Connections by 'G.W.Owens & B.D.Cheal, Page 170

Calculation of 3 Unknowns  $P_t$ ,  $y$ ,  $\sigma_s$

From Vertical Equilibrium

$$0.5 y \sigma_c B - P_t - P_c = 0.0 \quad \text{————— ①}$$

Moment About Column centroid

$$P_t f + (P_t + P_c) (L/2 - y/3) - P_c * e = 0.0 \quad \text{————— ②}$$

Plane sections condition

$$\frac{\sigma_s / E_s}{\sigma_c / E_c} = \frac{L/2 - y + f}{y} \quad \text{————— ③}$$

Eliminating  $P_t$  &  $\sigma_c$  from these equations

Here  $P_t = \sigma_s A_{ts}$   $A_{ts}$  - Area of Holding down Bolts.

$$y^3 + 3 \left( \frac{e - L/2}{B} \right) y^2 + \frac{6m A_{ts} (f + e)}{B} y - \frac{6m A_{ts} (L/2 + f)(f + e)}{B} = 0$$

	K1	=	3 (e - L/2)
	K2	=	(6m $A_{ts}$ (f + e))/B
	K3	=	(6m $A_{ts}$ (L/2+f)(f+e))/B
		=	K2 *(L/2+f)
	f	=	146 mm
Net area Bolts for tension	$A_{ts}$	=	(3.142 x 20 <sup>2</sup> /4 x 4x 0.8)/2
		=	503 mm <sup>2</sup>
Area of bolts for shear	$A_{ss}$	=	3.14 x 20 <sup>2</sup> /4 x 4x 0.8
		=	1005 mm <sup>2</sup>
Eccentricity	e	=	127 mm

Substituting in the above formula,

K1	=	-369.622
K2	=	18069.28
K3	=	7155433.46

$$f(y) = y^3 + K1 y^2 + K2 y - K3 = 0$$

$$f^1(y) = 3y^2 + 2 K1 y + K2$$

Solving Cubical Equation by Differential Method

Assume y	=	500 mm
$y_1 = y - f(y)/f^1(y)$	=	413.48
$y_2 = y_1 - f(y_1)/f^1(y_1)$	=	378.80
$y_3 = y_2 - f(y_2)/f^1(y_2)$	=	372.83
$y_4 = y_3 - f(y_3)/f^1(y_3)$	=	372.66
$y_5 = y_4 - f(y_4)/f^1(y_4)$	=	372.66
$y_6 = y_5 - f(y_5)/f^1(y_5)$	=	372.66
$y_7 = y_6 - f(y_6)/f^1(y_6)$	=	372.66

Hence  $y = 372.66 \text{ mm}$   $f(y) = 0.00$

**Proceed with this sheet**

Force in anchor bolts  $x = L/2 - y/3 = 125.780 \text{ mm}$

$$P_t = \frac{-P_c (x - e)}{(x + f)}$$

$$P_t = \frac{126 \times 10^3 (126 - 127)}{(126 + 146)}$$

$$P_t = 470 \text{ N}$$

Total compression in concrete  $P_v = P_c + P_t$

$$P_v = 126 \times 10^3 + 470$$

$$P_v = 126605 \text{ N}$$

**CHECK FOR COMP. STRESS IN CONCRETE**

$$\sigma_c = \frac{P_v}{(\frac{1}{2} \times y \times B)} \quad (\text{From eq.1})$$

$$= \frac{126605}{(\frac{1}{2} \times 373 \times 500)}$$

$$= 1.36 \text{ N/mm}^2$$

Compressive stress in concrete due to  $M_z = 6.85 \text{ N/mm}^2$

Total compressive stress in concrete  $8.21 \text{ N/mm}^2$

**CHECK**  $\sigma_c < \sigma_b = 8.21 < 11.38$  **SAFE**

**CHECK FOR TENSILE STRESS IN BOLTS:**

$$f_t = \frac{P_t}{A_{ts}}$$

$$= \frac{470}{503}$$

$$= 1.87 \text{ N/mm}^2$$

Tensile stress in bolts due to  $M_z = 247.05 \text{ N/mm}^2$  (Refer Table)

Total tensile stress in bolts  $= 253.24 \text{ N/mm}^2$

**CHECK**  $f_t < \sigma_f = 253.24 < 300$

**Required Length of Bolt:**

$$l = \frac{T}{(\tau_{bd} \times \pi \times \phi)}$$

$$= \frac{((470/(4/2)) + (247.05 \times \pi) \times 0.25 \times 20^2)}{(0.9 \times \pi) \times 20}$$

$$= 1376.10 \text{ mm}$$

**CHECK FOR BASE PLATE THICKNESS FOR COMPRESSIVE STRESS**

3 sides supported & one end free
▼

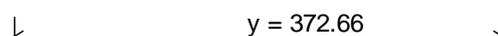
Base Plate is Stiffened by Gusset Plates

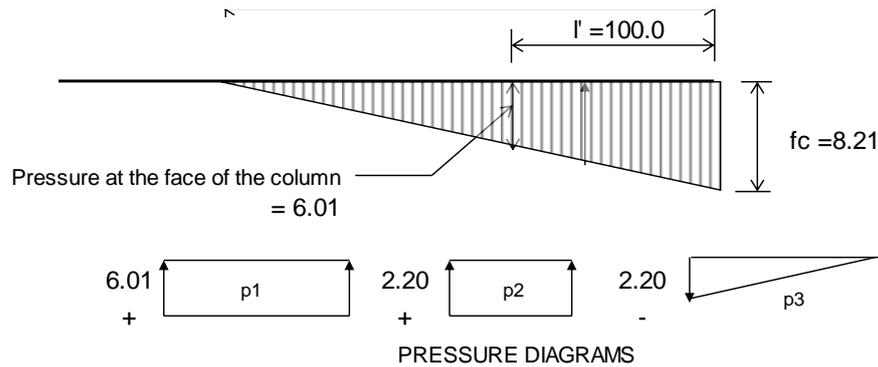
**Case 1 : 3 sides supported; one end free.**

Projection of plate from column,  $l' = 100 \text{ mm}$

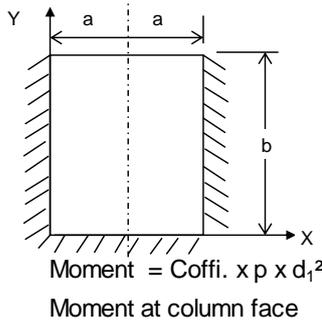
Pressure at face of column  $= \frac{8.21 \times (372.66 - 100.0)}{372.66}$

$$= 6.01 \text{ N/mm}^2$$





Using FIG 1 and FIG 4 of Moody's Chart  
 - three edges supported and one edge free



$a = 37.5 \text{ mm}$   
 $b = l' = 100 \text{ mm}$   
 $a/b = 0.375$

Coeff.	Moment due to			Design Moment
	P1	P2	P3	
Coeff.	0.0476	0.0476	0.0208	
Moment	2859.9	1047.2	457.6	3449.5 (Nmm)

Max design moment at column face = 3449.5 Nmm

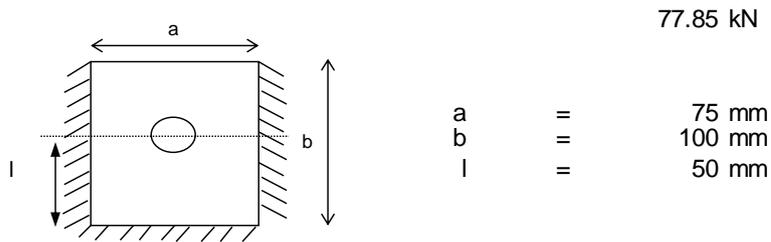
Required base plate thickness (per unit strip)  $t_r = [(6xM) / (185)]^{0.5}$   
 $= \sqrt{(6 \times 3449.5) / (185 \times 1.33)}$   
 $= 9.17 \text{ mm}$

Thickness reqd for compression  $t_{req} = 9.17 \text{ mm}$   
 Flange thickness of column = 8 mm  
 $= \text{Max}(t_{req}, T_{flange})$   
 $= 9.17 \text{ mm}$  **SAFE**

**CHECK FOR BASE PLATE THICKNESS FOR TENSION IN BOLT**

**Case 1 : 3 sides supported; one end free.**

Tensile force / anchor bolt  $F_t = [470 / (2 \times 1000)] + [247.05 \times P1() \times 20^2 / (4 \times 1000)]$



Ref. EQ. 3.68, 'STORAGE STRUCTURES' by Dr.K.RAJAGOPALAN

$$t_{req} = \sqrt{\frac{3.91 Ft}{py \left( \frac{2b}{a} + \frac{a}{2l} - d \left[ \frac{2}{a} + \frac{1}{2l} \right] \right)}}$$

Where,

- Ft = Tensile force per bolt
- a,b,& l = Refer sketch
- py = Design strength
- d = Bolt dia.

$t_{req}$	=	SQRT((3.91*77847.7) / ( 240*(2.67+0.75-20* [0.037] ) )
$t_{req}$	=	21.75
$t_{pro}$	=	25 mm
$t_{req}$	<	$t_{pro}$

Flange thickness of column	=	8 mm
	=	Max ( $t_{req}$ , $T_{flange}$ )
	=	<b>21.75</b> <b>SAFE</b>

# Analysis and Design of Steel Shelter – IS800

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## SUMMARY OF DESIGN OF BASE PLATE - BP1 FOR ALL LOAD COMBINATIONS

Grid: 0

No. of bolts = 4  
Bolt Dia = 20 mm

Joint no	Load case	Factor	Factor	Factor	Shear	Axial	Moment-X	Sl.no	Comp. stress	Tensile																		
1383	111 1.0DL+1.0SLX+1.0SLY+1.0TL+1.0DL2	1	1.25	1.33	41.56	126.14	15.99	500.00	11	6.85	247.05																	
Joint No.	Load	Factor for bearing pressure	Factor for bolt tensile strength	Factor for base plate design	Shear-x (kN)	Shear-z (kN)	Axial (kN)	Moment-x (kNm)	Resultant shear (kN)	Axial (kN)	Moment-x (kNm)	Neutral axis depth (assumed)	Sl.no	Type	Neutral axis depth	Bearing stress on concrete due to P & Mx (N/mm <sup>2</sup> )	Compressive stress in concrete due to Mz (N/mm <sup>2</sup> )	Total compressive stress in concrete (N/mm <sup>2</sup> )	Check	Tensile stress in Bolt due to Mx (or) uplift (N/mm <sup>2</sup> )	Tensile stress in bolt due to Mz (N/mm <sup>2</sup> )	Total tensile stress in bolt (N/mm <sup>2</sup> )	Check	Baseplate thk. reqd. for comp. (mm)	Check	Baseplate thk. reqd. for tension (mm)	Check	
													2		372.66	1.36	6.85	8.21	SAFE	6.19	247.05	253.24	SAFE	9.17	SAFE	21.75	SAFE	
1383	101 1.0DL+1.0LL+1.0TL+1.0DL2	1	1	1	-14.02	17.85	125.20	10.10	22.70	125.20	10.10	500	1	1	-	0.99	0.92	1.91	SAFE	0.00	33.32	33.32	SAFE	5.43	SAFE	0.00	-	
1383	102 1.0DL+1.0LL-1TL+1.0DL2	1	1	1	-14.02	17.85	125.20	10.10	22.70	125.20	10.10	500	2	1	-	0.99	0.92	1.91	SAFE	0.00	33.32	33.32	SAFE	5.43	SAFE	0.00	-	
1383	103 1.0DL+1.0WLLX1+1.0TL+1.0DL2	1	1.25	1	-10.50	16.40	139.27	7.08	19.48	139.27	7.08	500	3	1	-	0.90	1.63	2.53	SAFE	0.00	58.73	58.73	SAFE	6.24	SAFE	0.00	-	
1383	104 1.0DL+1.0WLLZ2+1.0TL+1.0DL2	1	1.25	1	-21.66	17.17	123.64	8.74	27.64	123.64	8.74	500	4	1	-	0.91	6.08	7.00	SAFE	0.00	219.39	219.39	SAFE	10.39	SAFE	0.00	-	
1383	105 1.0DL+1.0WLLZ1+1.0TL+1.0DL2	1	1.25	1	-27.95	18.94	141.32	12.38	33.76	141.32	12.38	500	5	1	-	481.42	1.23	6.19	7.41	SAFE	0.00	223.04	223.04	SAFE	10.70	SAFE	0.00	-
1383	106 1.0DL+1.0WLLZ2+1.0TL+1.0DL2	1	1.25	1	-6.86	27.46	132.89	30.27	28.30	132.89	30.27	500	6	2	-	195.83	3.07	2.24	5.31	SAFE	43.73	80.83	124.56	SAFE	7.98	SAFE	14.38	SAFE
1383	107 1.0DL+1.0WLLX1-1TL+1.0DL2	1	1.25	1	-10.69	26.93	146.80	29.10	28.97	146.80	29.10	500	7	2	-	228.51	2.77	1.40	4.17	SAFE	29.44	50.40	79.84	SAFE	7.21	SAFE	11.41	SAFE
1383	108 1.0DL+1.0WLLZ2-1TL+1.0DL2	1	1.25	1	-13.19	16.69	136.44	7.68	21.27	136.44	7.68	500	8	1	-	0.91	2.39	3.30	SAFE	0.00	86.01	86.01	SAFE	7.14	SAFE	0.00	-	
1383	109 1.0DL+1.0WLLZ1-1TL+1.0DL2	1	1.25	1	-11.55	15.14	132.50	4.42	19.04	132.50	4.42	500	9	1	-	0.74	1.51	2.26	SAFE	0.00	54.60	54.60	SAFE	5.90	SAFE	0.00	-	
1383	110 1.0DL+1.0WLLZ2-1TL+1.0DL2	1	1.25	1	-28.20	18.45	104.04	11.44	33.70	104.04	11.44	500	10	2	-	418.13	1.00	6.73	7.73	SAFE	-1.67	242.49	240.82	SAFE	10.34	SAFE	21.53	SAFE
1383	111 1.0DL+1.0SLX+1.0SLY+1.0TL+1.0DL2	1	1.25	1.33	-36.07	20.66	126.14	15.99	41.56	126.14	15.99	500	11	2	-	372.66	1.36	6.85	8.21	SAFE	6.19	247.05	253.24	SAFE	9.17	SAFE	21.75	SAFE
1383	112 1.0DL-1SLX-1SLY+1.0TL+1.0DL2	1	1.25	1.33	-9.71	31.31	115.60	38.36	32.78	115.60	38.36	500	12	2	-	143.4	4.42	1.92	6.34	SAFE	103.85	69.29	173.14	SAFE	7.15	SAFE	16.22	SAFE
1383	113 1.0DL+1.0SLZ+1.0SLYZ+1.0TL+1.0DL2	1	1.25	1.33	-14.49	30.65	132.99	36.89	33.90	132.99	36.89	500	13	2	-	163.17	4.04	0.87	4.91	SAFE	77.98	31.25	109.23	SAFE	6.46	SAFE	12.50	SAFE
1383	114 1.0DL-1SLZ-1SLYZ+1.0TL+1.0DL2	1	1.25	1.33	-17.61	17.86	120.04	10.12	25.08	120.04	10.12	500	14	1	-	491.12	0.99	2.10	3.09	SAFE	0.00	75.76	75.76	SAFE	5.99	SAFE	0.00	-
1383	115 1.0DL+1.0SLX+1.0SLY-1TL+1.0DL2	1	1.25	1.33	-15.57	15.91	115.11	6.03	22.26	115.11	6.03	500	15	1	-	0.75	1.01	1.76	SAFE	0.00	36.50	36.50	SAFE	4.52	SAFE	0.00	-	
					Maximum values	41.56	146.80	38.36																				
					Minimum values	0.00	0.00	0.00																				

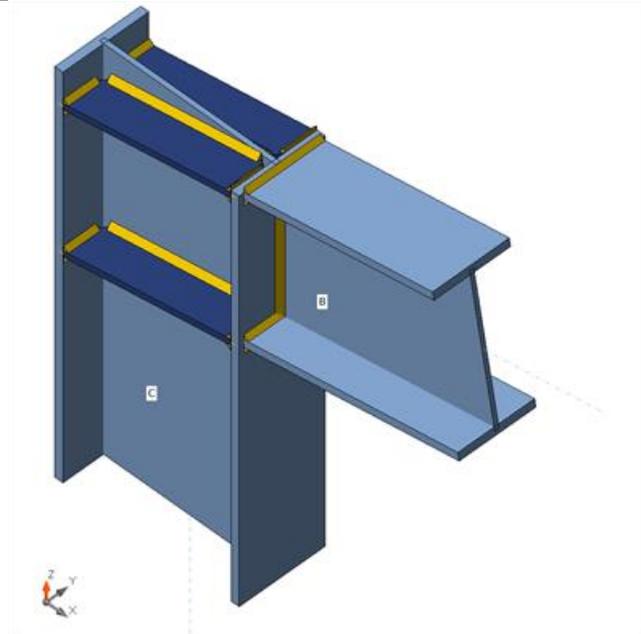
 <b>CMWSSB</b>	<b>150MLD SEA WATER REVERSE OSMOSIS</b>	
	<b>Calculations Report for Lamella Sludge Sump Steel A frame support and Foundation</b>	

# **Annexure E**

## **Beam to Column Moment Connection design**

## Beams and columns

Name	Cross-section	$\beta$ – Direction [°]	$\gamma$ - Pitch [°]	$\alpha$ - Rotation [°]	Offset ex [mm]	Offset ey [mm]	Offset ez [mm]	Forces in
C	1 - ISMB 300	90.0	90.0	90.0	0	0	0	Node
B	2 - ISMB 250	0.0	-10.0	0.0	0	0	0	Node



## Cross-sections

Name	Material
1 - ISMB 300	E 165 (Fe 290)
2 - ISMB 250	E 165 (Fe 290)

## Load effects (equilibrium not required)

Name	Member	N [kN]	Vy [kN]	Vz [kN]	Mx [kNm]	My [kNm]	Mz [kNm]
LE1	B	32.0	0.0	-35.0	0.0	40.0	0.0

## Check

## Summary

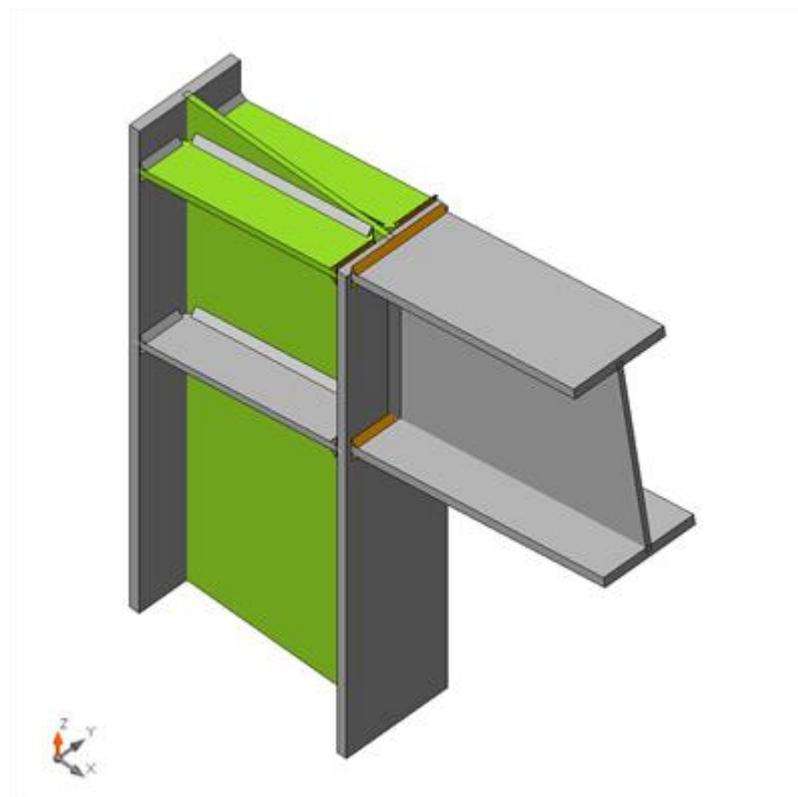
Name	Value	Check status
Analysis	100.0%	OK
Plates	0.0 < 5.0%	OK
Welds	98.0 < 100%	OK

## Plates

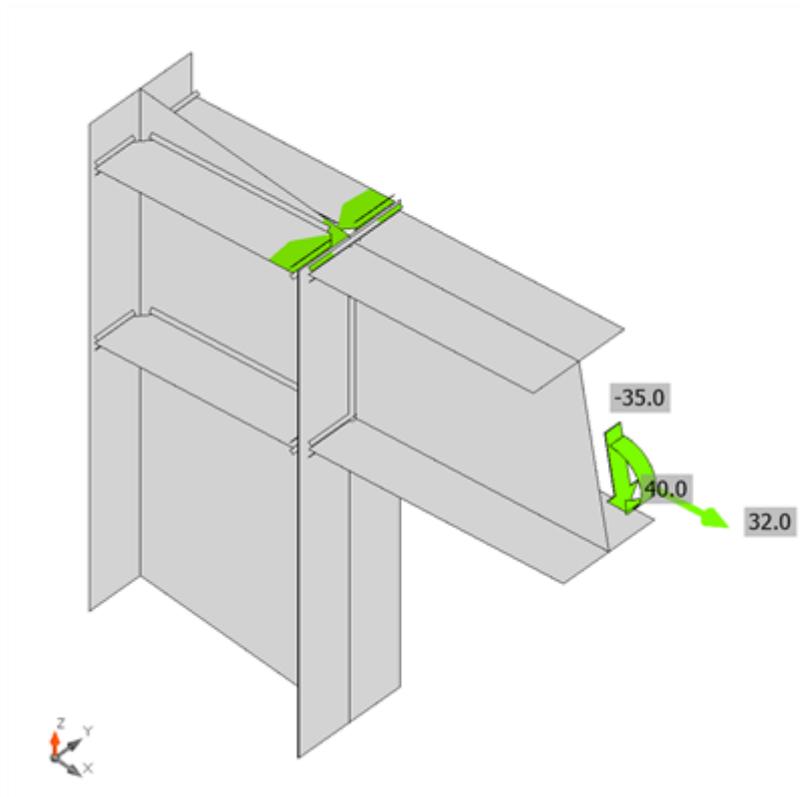
Name	Material	$f_{yd}$ [MPa]	Thickness [mm]	Loads	$\sigma$ [MPa]	$\epsilon_{PI}$ [%]	$\sigma_{CEd}$ [MPa]	Check status
C-bfl 1	E 165 (Fe 290)	150.0	13.1	LE1	92.8	0.0	0.0	OK
C-tfl 1	E 165 (Fe 290)	150.0	13.1	LE1	120.3	0.0	0.0	OK
C-w 1	E 165 (Fe 290)	150.0	7.7	LE1	124.5	0.0	0.0	OK
B-bfl 1	E 165 (Fe 290)	150.0	12.5	LE1	135.6	0.0	0.0	OK
B-tfl 1	E 165 (Fe 290)	150.0	12.5	LE1	113.7	0.0	0.0	OK
B-w 1	E 165 (Fe 290)	150.0	6.9	LE1	101.5	0.0	0.0	OK
STIFF1a	E 165 (Fe 290)	150.0	10.0	LE1	73.8	0.0	0.0	OK
STIFF1b	E 165 (Fe 290)	150.0	10.0	LE1	73.8	0.0	0.0	OK
STIFF1c	E 165 (Fe 290)	150.0	10.0	LE1	117.5	0.0	0.0	OK
STIFF1d	E 165 (Fe 290)	150.0	10.0	LE1	117.4	0.0	0.0	OK

## Symbol explanation

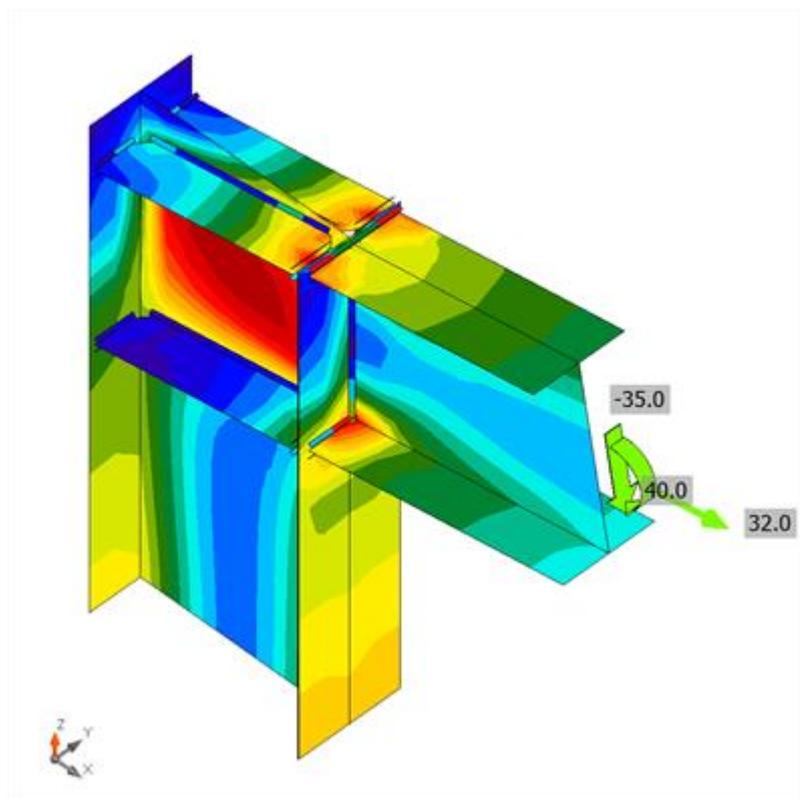
- $\epsilon_{PI}$  Plastic strain
- $\sigma$  Equivalent stress
- $f_{yd}$  Design yield strength
- $\sigma_{CEd}$  Contact stress



Overall check, LE1



Strain check, LE1



Equivalent stress, LE1

## Welds

Item	Edge	Electrode	$t_t$ [mm]	$l_j$ [mm]	$l_{je}$ [mm]	Loads	$f_e$ [MPa]	$f_{wd}$ [MPa]	$U_t$ [%]	Status
C-tfl 1	B-bfl 1	E 165 (Fe 290)	▲6.0▲	125	31	LE1	129.1	133.9	96.4	OK
		E 165 (Fe 290)	▲6.0▲	125	31	LE1	131.3	133.9	98.0	OK
C-tfl 1	B-tfl 1	E 165 (Fe 290)	▲6.0▲	125	31	LE1	131.3	133.9	98.0	OK
		E 165 (Fe 290)	▲6.0▲	125	31	LE1	96.5	133.9	72.0	OK
C-tfl 1	B-w 1	E 165 (Fe 290)	▲6.0▲	241	30	LE1	79.5	133.9	59.4	OK
		E 165 (Fe 290)	▲6.0▲	241	30	LE1	79.5	133.9	59.4	OK
C-bfl 1	STIFF1a	E 165 (Fe 290)	▲6.0▲	52	26	LE1	14.1	133.9	10.5	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	22.4	133.9	16.8	OK
C-w 1	STIFF1a	E 165 (Fe 290)	▲6.0▲	248	35	LE1	28.7	133.9	21.4	OK
		E 165 (Fe 290)	▲6.0▲	248	35	LE1	25.1	133.9	18.7	OK
C-tfl 1	STIFF1a	E 165 (Fe 290)	▲6.0▲	52	26	LE1	79.5	133.9	59.3	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	45.0	133.9	33.6	OK
C-bfl 1	STIFF1b	E 165 (Fe 290)	▲6.0▲	52	26	LE1	22.4	133.9	16.8	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	14.1	133.9	10.5	OK
C-w 1	STIFF1b	E 165 (Fe 290)	▲6.0▲	248	35	LE1	25.1	133.9	18.7	OK
		E 165 (Fe 290)	▲6.0▲	248	35	LE1	28.7	133.9	21.4	OK
C-tfl 1	STIFF1b	E 165 (Fe 290)	▲6.0▲	52	26	LE1	45.0	133.9	33.6	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	79.5	133.9	59.3	OK
C-bfl 1	STIFF1c	E 165 (Fe 290)	▲6.0▲	52	26	LE1	38.1	133.9	28.5	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	41.5	133.9	31.0	OK
C-w 1	STIFF1c	E 165 (Fe 290)	▲6.0▲	248	35	LE1	46.6	133.9	34.8	OK
		E 165 (Fe 290)	▲6.0▲	248	35	LE1	52.6	133.9	39.3	OK
C-tfl 1	STIFF1c	E 165 (Fe 290)	▲6.0▲	52	26	LE1	131.3	133.9	98.0	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	131.3	133.9	98.0	OK
C-bfl 1	STIFF1d	E 165 (Fe 290)	▲6.0▲	52	26	LE1	41.5	133.9	31.0	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	38.1	133.9	28.5	OK
C-w 1	STIFF1d	E 165 (Fe 290)	▲6.0▲	248	35	LE1	52.6	133.9	39.3	OK
		E 165 (Fe 290)	▲6.0▲	248	35	LE1	46.6	133.9	34.8	OK
C-tfl 1	STIFF1d	E 165 (Fe 290)	▲6.0▲	52	26	LE1	131.3	133.9	98.0	OK
		E 165 (Fe 290)	▲6.0▲	52	26	LE1	131.3	133.9	98.0	OK

## Symbol explanation

- $t_t$  Fillet weld throat thickness
- $l_j$  Weld length
- $l_{je}$  Weld element length
- $f_e$  Equivalent stress in the weld
- $f_{wd}$  Design strength of a fillet weld
- $U_t$  Utilization

## Code settings

Item	Value	Unit	Reference
Friction coefficient - concrete	0.45	-	IS 800, Cl. 7.4.1

## Analysis and Design of Steel Shelter – IS800

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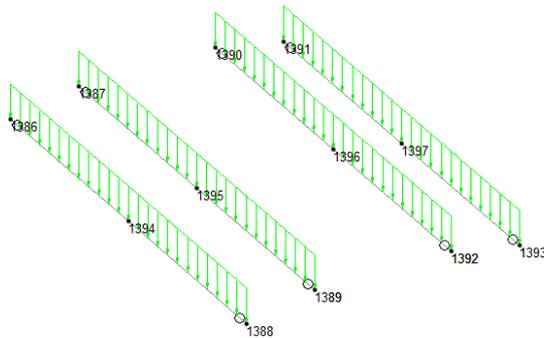
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Friction coefficient in slip-resistance	0.30	-	IS 800, Cl. 10.4.3
Limit plastic strain	0.05	-	
Detailing	No		
Distance between bolts [d]	2.50	-	IS 800, Cl. 10.2.2
Distance between bolts and edge [d]	1.50	-	IS 800, Cl. 10.2.4
Bolt maximum grip length [d]	8.00	-	Limit grip length of bolts as a multiple of bolt diameter - IS 800, Cl. 10.3.3.2
Local deformation check	Yes		
Local deformation limit	0.03	-	CIDECT DG 1, 3 - 1.1
Geometrical nonlinearity (GMNA)	Yes		Analysis with large deformations for hollow section joints
Concrete in compression check	IS800, Cl 7.4		
Braced system (EC stiffness classification)	No		EN1993-1-8 - Cl. 5.2.2.5

**Side Cladding and Roof Purlin Design**

Strength case:



Beam no. = 2751. Section: ISMC150

Length = 3

FC	148.57	FT	484.09
FVZ	177.14	FVY	112.19
MBZ	17.97	MBY	4.37
CMZ	0.9	CMY	0.9

Load	52
Location	3.00001
FX	0.048187 C
MY	0
MZ	-7.751614

Code	Result	Ratio	Critical	KLR
IS800-07	PASS	0.4317188	Sec. 9.3.2.2	137.0921

**Deflection case:**

	Node	L/C	Horizontal X mm	Vertical Y mm	Horizontal Z mm	Resultant mm	Rotational rX rad	rY rad	rZ rad
Max X	1397	14 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI -0.5]	9.805	-11.876	-0.215	15.402	0	0	0.002
Min X	1394	10 1.0DL+1.0LL	-0.439	-13.949	-0.039	13.956	0	0	-0.002
Max Y	1396	19 1.0DL+1.0WL(+X)[CPI +0.5]	8.721	0.681	-0.001	8.748	0	0	0
Min Y	1395	12 1.0DL+1.0LL+1.0CL	2.233	-19.949	0.061	20.073	0	0	-0.003
Max Z	1394	24 1.0DL+1.0EQZ	-0.171	-1.744	0.581	1.846	0	0	-0.001
Min Z	1397	22 1.0DL+1.0WL(+Z)[CPI -0.5]	0.103	-1.922	-1.004	2.171	0	0	0.001
Max rX	1395	18 1.0DL+0.8LL+0.8CL+0.8EQ (+Z)	1.775	-16.427	0.537	16.531	0	0	-0.002
Min rX	1396	22 1.0DL+1.0WL(+Z)[CPI -0.5]	-0.031	-2.579	-0.906	2.733	0	0	0
Max rY	1394	22 1.0DL+1.0WL(+Z)[CPI -0.5]	-0.282	-1.795	-0.572	1.905	0	0	-0.001
Min rY	1397	19 1.0DL+1.0WL(+X)[CPI +0.5]	8.606	0.043	0.004	8.606	0	0	-0.001
Max rZ	1397	12 1.0DL+1.0LL+1.0CL	3.679	-15.763	-0.049	16.186	0	0	0.004
Min rZ	1394	12 1.0DL+1.0LL+1.0CL	1.476	-16.115	0.067	16.183	0	0	-0.004
Max Rst	1395	12 1.0DL+1.0LL+1.0CL	2.233	-19.949	0.061	20.073	0	0	-0.003

$L/180 = 6000 / 180 = 33.33\text{mm}$  Allowable Hence safe;

**Provided ISMC 150 Channel With 1 No Sag Rod.**

# **Annexure F**

## **Foundation and Pedestal Design**

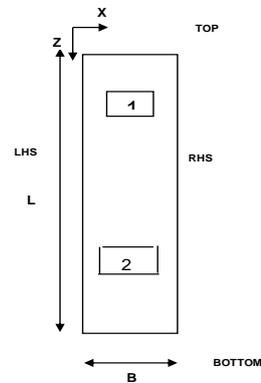
**FOUNDATION – CF1**

**DESIGN OF COMBINED FOOTING - CF1**

Load case 104  
 Load 13 1.0DL+0.8LL+0.8CL+0.8W.L (+X)[CPI +0.5]  
 Net SBC 50 kN/m<sup>2</sup>  
 Factor for inc in BC 1.00  
 Founding Level 1.50 m  
 Pedestal above FGL 0.50 m  
 Staad Jt. No. 1 2

Pedestal Mark	SUM	1	2
X	wrt 1	0.000	0.000
Z	wrt 1	0.000	6.000
P (kN)		-32.87	-21.68
Mz (kNm)		97.70	51.532
Hx (kN)		0.00	
Mx (kNm)		-21.23	-7.93
Hz (kN)		0.00	

DESCRIPTION USED IN SHEET



Depth of Column	1	2
lx	0.65	0.60
lz	0.65	0.60

Depth of foundation from the level of point of application of forces

d <sub>forc</sub>	1	2
	0.0	0.0 m

Depth of foundation below ground level (FGL)

d<sub>f</sub> 1.50 m

Depth of foundation below Natural Ground Level (NGL)

d<sub>fngl</sub> 1.50 m

Unit Weight of soil

γ<sub>s</sub> 18 kN/m<sup>3</sup>

Projections of Footing (from centreline of column)

	1	2
LHS	1.000	
RHS		1.000
TOP	0.850	
BOTTOM		0.850

Cx1 1.000 m

Cx2 1.000 m

Cz1 0.850 m

Cz2 0.850 m

Width of footing

B 2.000 m

Length of footing

L 7.700 m

Thickness of footing

d 0.400 m

**Calculations :**

Col Mark	1	2
xcor	1.000	1.000
zcor	0.850	6.850

Axial Load including weight of Pedestal ( P<sub>conc</sub> = P + P<sub>ped</sub> )

P <sub>conc</sub>	1	2
	-32.87	-21.7
		-11.2

Moment at base of foundation due to Horizontal Forces

(M<sub>xh</sub> = H<sub>y</sub> \* d<sub>forc</sub>) (M<sub>yh</sub> = H<sub>x</sub> \* d<sub>forc</sub>)

M <sub>zh</sub>	1	2
	0	0
	0	0

Moments due to Conc. Moments & Horizontal Forces

(M<sub>yc</sub> = M<sub>y</sub> + M<sub>yh</sub>) (M<sub>xc</sub> = M<sub>x</sub> + M<sub>xh</sub>)

M <sub>zc</sub>	1	2
	97.697	51.532
	-21.23	-7.93
		-13.303

# Analysis and Design of Steel Shelter – IS800

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Gross SBC	$SBC_g = Fbc * SBC_{net} + \gamma_s * d_{rngl}$	=	50 kN/m <sup>2</sup>
Area of foundation ( Provided )	A	L x B	= 15.4 m <sup>2</sup>
Total Axial Load incl wt of pedestal ( $\Sigma P_{conc}$ )	$\Sigma P$		= -33 kN
Load due to soil	$P_{soil}$	$\gamma_s * (d_f - d) * (A - \Sigma(l_x * l_y))$	= 289 kN
Weight of foundation	$F_{base}$	$A * d * 25$	= 154 kN
Total Vertical Load	$P_v$	$\Sigma P + P_{soil} + F_{base} + P_{slab}$	= 411 kN

CG of load system from Top left corner of footing

	X Direction		Z Direction	
Moments due to $P_{conc}$	$\Sigma(P_{conc} * X_{cor})$	-32.87	$\Sigma(P_{conc} * Z_{cor})$	-95.09
External Moments	$\Sigma M_{z_m}$	97.70	$\Sigma M_{x_m}$	-21.23
Moment due to Horizontal Forces	$\Sigma M_{z_h}$	0.00	$\Sigma M_{x_h}$	0.00
Moment due to Soil & Raft	$(P_{soil} + F_{base}) * b/2$	443.43	$(P_{soil} + F_{base}) * l/2$	1707.19
Moment due to grade slab		0.00		0.00
Total Moment	$\Sigma M_z$	508.25	$\Sigma M_x$	1590.87
Horizontal Forces	$\Sigma H_x$	0.00	$\Sigma H_z$	0.00

## Eccentricity ( X - dir )

CG From Left End	$X_{cgcor} = \Sigma M_z / P_v$	=	1.238
Eccentricity along X Dir from CG of Raft	$e_x = X_{cgcor} - b/2$	=	0.238
	$e_x = 0.238 < (b / 6 = 0.33)$		

## Eccentricity ( Z - dir )

CG from Top edge	$Z_{cgcor} = \Sigma M_x / P_v$	=	3.875 Near the Bottom Side
Eccentricity along Z Dir from CG of Raft	$e_z = Z_{cgcor} - l/2$	=	0.025
	$e_z = 0.025 < (l / 6 = 1.283)$		
	$e_x / b$	=	0.119 m
	$e_z / l$	=	0.003 m

Teng's Value	K	= 1.73		
$f_{max}$	=	46 kN/m <sup>2</sup>	<	Gross SBC 50 kN/m <sup>2</sup> <b>Safe</b>
$f_{min}$	=	7 kN/m <sup>2</sup>		

## Design Pressure

Along X - Direction				
LHS	$f_{x_{max}}$	=	45.69 kN/m <sup>2</sup>	
RHS	$f_{x_{min}}$	=	7.63 kN/m <sup>2</sup>	
Contact length	$l_{x_{cont}}$	=	2.000 m	> 60 % (60% = 1.2 m) <b>Safe</b>

Along Z - Direction				
TOP	$f_{z_{max}}$	=	27.18 kN/m <sup>2</sup>	
BOTTOM	$f_{z_{min}}$	=	26.14 kN/m <sup>2</sup>	
Contact length	$l_{z_{cont}}$	=	7.700 m	> 60 % (60% = 4.62 m) <b>Safe</b>

Pressure Along X - Direction				
RHS	$f_{x1}$	=	45.69 kN/m <sup>2</sup>	
LHS	$f_{x2}$	=	7.63 kN/m <sup>2</sup>	

Pressure Along Z - Direction				
BOTTOM	$f_{z1}$	=	26.14 kN/m <sup>2</sup>	
TOP	$f_{z2}$	=	27.18 kN/m <sup>2</sup>	



## Design of Footing - Z direction.( Longitudinal Direction)

### Basic data :

Concrete grade	f <sub>ck</sub> =	35 N/mm <sup>2</sup>
Steel grade	f <sub>y</sub> =	500 N/mm <sup>2</sup>
Load factor	ld=	1.2

### Section Data:

Length of the footing	l =	7700 mm
Breadth of the footing	b =	2000 mm
Depth of the footing	D =	400 mm
Clear cover to reinf.	d' =	50 mm
Dia of bar used	φ =	12 mm

### Load data:

Maximum Bending Moment	M =	29.60 kN-m
Maximum Shear Force (at 'd' dist. from face of pedestal)	V =	18.83 kN

### Reinforcement:

Factored Bending Moment	M <sub>u</sub> =	35.51 kN-m
Eff. depth of footing d = 400 - 50 - 12/2 =		344 mm

$$M_u/bd^2 = 35.51 \times 10^6 / (7700 \times 344^2) = 0.039$$

% of Reinforcement required	p <sub>tr</sub> =	0.009
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Minimum % of steel required	p <sub>min</sub> =	0.12
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$$\therefore p_t = 0.12$$

Area of steel required	A <sub>st</sub> =	960 mm <sup>2</sup>
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Required spacing 12mm dia bars @ 236 mm c/c

Provide 12mm dia bar: 150 mm c/c at Top in Longitudinal Direction

Provided Area of steel	A <sub>stp</sub> =	1508 mm <sup>2</sup>	<b>SAFE</b>
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Provide Minimum Reinforcement at Bottom in Longitudinal Direction

Minimum % of steel required	p <sub>min</sub> =	0.12
-----------------------------	--------------------	------

Area of steel required	A <sub>st</sub> =	960.0 mm <sup>2</sup>
------------------------	-------------------	-----------------------

Dia of bar used	φ =	12 mm
-----------------	-----	-------

Required spacing 12mm dia bars @ 236 mm c/c

Provide 12mm dia bar: 150 mm c/c at Bottom in Longitudinal Director

Provided Area of steel	A <sub>stp</sub> =	1508 mm <sup>2</sup>	<b>SAFE</b>
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### CHECK FOR SHEAR:

Factored shear force	V <sub>u</sub> =	22.60 kN
Nominal Shear stress τ <sub>v</sub> = V <sub>u</sub> /bd		= 23 × 10 <sup>3</sup> / (7700 × 344)
	τ <sub>v</sub> =	0.009 N/mm <sup>2</sup>

Shear strength of concrete for the provided steel % τ <sub>c</sub> =		0.702 N/mm <sup>2</sup>
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CHECK τ <sub>v</sub> < τ <sub>c</sub>	0.009 < 0.702	<b>SAFE</b>
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## Design of Footing - X Direction ( Transverse Direction)

### Basic Data:

Concrete grade	f <sub>ck</sub> =	35 N/mm <sup>2</sup>
Steel grade	f <sub>y</sub> =	500 N/mm <sup>2</sup>
Load factor	ld =	1.2

### Section Data:

Projection of footing from col. face	l =	675 mm
Breadth of the footing	b =	2 mm
Depth of the footing	D =	400 mm
Clear cover to reinf.	d' =	70 mm
Dia of bar used	φ =	12 mm

### Load data:

Maximum pressure	f <sub>max</sub> =	45.69 kN/m <sup>2</sup>
Maximum Bending Moment	M =	10.41 kN-m

### Reinforcement: Consider 1 meter width

Factored Bending Moment	M <sub>u</sub> =	12.49 kN-m
Eff. depth of footing	d = 400 - 70 - 12/2 =	324 mm
$M_u/bd^2$	= $12.49 \times 10^6 / (1000 \times 324^2)$ =	0.119
% of Reinforcement required	p <sub>tr</sub> =	0.03 %
Minimum % of steel required	p <sub>min</sub> =	0.12 %
	∴ p <sub>t</sub> =	0.12 %
Area of steel required	A <sub>st</sub> =	480.0 mm <sup>2</sup> /m
Required spacing	12mm dia bars @ 236 mm c/c	
Provide 12mm dia bar:	150 mm c/c	at Bottom in Transverse Direction

Provided Area of steel                      A<sub>stp</sub> =                      754 mm<sup>2</sup>/m                      **SAFE**

Provide Minimum Reinforcement at Top in Transverse Direction

Minimum % of steel required	p <sub>min</sub> =	0.12 %
Dia of bar used	φ =	12 mm
Area of steel required	A <sub>st</sub> =	480 mm <sup>2</sup> /m
Required spacing	12mm dia bars @ 236 mm c/c	
Provide 12mm dia bar:	150 mm c/c	at Top in Transverse Direction
Provided Area of steel	A <sub>stp</sub> =	754.0 mm <sup>2</sup> /m <b>SAFE</b>

### CHECK FOR ONE WAYSHEAR:

At ' d ' from face of column	d =	324 mm
Shear Force	=	16.04 kN
Factored shear force (V x 1.2)	V <sub>u</sub> =	19.25 kN
Nominal Shear stress τ <sub>v</sub> = V <sub>u</sub> /bd	=	19 × 10 <sup>3</sup> / (324 × 2)
	τ <sub>v</sub> =	0.059 N/mm <sup>2</sup>
Shear strength of concrete for the provided steel %	τ <sub>c</sub> =	0.721 N/mm <sup>2</sup>
CHECK τ <sub>v</sub> < τ <sub>c</sub>	0.059 < 0.721	<b>SAFE</b>



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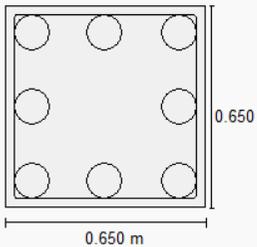
### Pedestal Design:

**Provided 16Nos 16dia reinforcement with 150c/c stirrups.**

NTPL-DSAL-TEC-DE-C-DOC-031\_Portable water Pump Shed - Beam

Geometry Property Loading Shear Bending Deflection Concrete Design

Beam no. = 2733 Design code : IS-456



0.650

0.650 m

Design Load	
Load	58
Location	End 2
Pu(Kns)	133.69
Mz(Kns-Mt)	94.54
My(Kns-Mt)	47.67

Design Parameter	
Fy(Mpa)	500
Fc(Mpa)	35
As Reqd(mm <sup>2</sup> )	808
As (%)	0.381
Bar Size	16
Bar No	8

Print Close

# Analysis and Design of Steel Shelter – IS800

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## Plinth Beam Design:

**Provided 3 Nos 16dia Top and Bottom reinforcement with 150c/c stirrups.**

Portable water Pump Shed with pedsetal - Beam

Geometry Property Loading Shear Bending Deflection Concrete Design

Beam no. = 2738 Design code : IS-456

4#12 @ 314.00 0.00 To 4000.00      4#12 @ 314.00 4000.00 To 6000.00

22 # 8 c/c 135.00      22 # 8 c/c 135.00

3#10 @ 35.00 0.00 To 6000.00

at 0.000      at 3000.000      at 6000.000

Design Load

Mz Kn Met	Dist. Met	Load
25.2	3	58
-47.05	0	60
-55.06	6	57

Design Parameter

Fy(Mpa)	500
Fc(Mpa)	35
Depth(m)	0.3499992736
Width(m)	0.2999993947
Length(m)	5.9999879914

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