BASEPLATE DESIGN TO IS 800 (LSD)

DESIGN INPUT (Ref. Table-A1 & Table-A2):

Table-A1 (Governing load combinations (LC) in local axes of b/p (Ref. fig. on sheet-3):

Node	LC	F_x (kN)	F_y (kN)	F_z (kN)	M_x (kNm)	M_y (kNm)	M_z (kNm)	Stress Inc. (%)	Governing Criteria
Factored reactions obtained from structure analysis									
78	2101	255.88	106.3	-11.3	0.79	-0.59	-488.76	0	Max. bolt dia.
78	2101	255.88	106.3	-11.3	0.79	-0.59	-488.76	0	Max. base press.
Reactions adjusted for stress increase and/or shear key presence									
78	2101	255.88	26.58	-2.82	0.79	-0.59	-488.76	0	Max. bolt dia.
78	2101	255.88	26.58	-2.82	0.79	-0.59	-488.76	0	Max. base press.
78	2101	255.88	106.3	11.28	0.79	-0.59	-488.76	0	Shear key size

Table-A2 (Material properties):

(i) Anchor bolt (Grade 4.6): fyb = 240 MPa fub = 400 MPa	(ii) Baseplate: fyp = 345 MPa fup = 500 MPa	(iii) Stiffener: fys = 345 MPa fus = 500 MPa
(iv) Shear key: (Profile: '+') fy = 345 MPa fu = 500 MPa	(v) Fillet weld: Min size = 6 mm fw = 480 MPa	(vi) Concrete: fcu = 25 MPa fa,per = 15 MPa

ANCHOR BOLT DESIGN (for 25% shear)

Assume 30 mm dia bolts & 25 mm thk b/p

[a] Shear:

<u>(i) Shear force per bolt (Vsb):</u>

$F_y = 26.58/16 = 1.66$ kN; $F_z = 2.82/16 = 0.18$ kN

=> Resultant, $R = \sqrt{(F_y^2 + F_z^2)} = 1.67$ kN

Hence, $V_{sb} = R = 1.67 = 1.67$ kN

<u>(ii) Shear capacity of bolt (Vdsb):</u>

$V_{nsb} = f_{ub}(n_n.A_{nb} + n_s.A_{sb})/\sqrt{3}$, where

fub = ult. tensile strength of bolt = 400 MPa

nn = no. of shear planes with threads = 1

ns = no. of shear planes w/o threads = 0

$A_{sb} = 706.9\ mm^2$; $A_{nb} = 561\ mm^2$

=> $V_{nsb} = f_{ub}(n_n.A_{nb} + n_s.A_{sb})/\sqrt{3} = 129.56$ kN

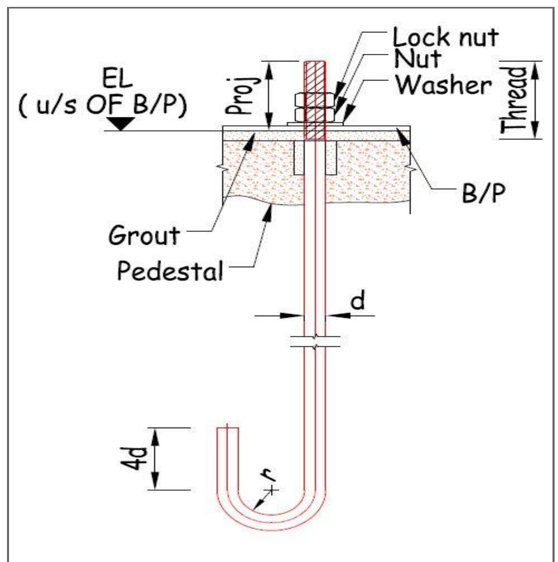
Hence, $V_{dsb} = V_{nsb}/1.25 = 129.56/1.25 = 103.65$ kN

<u>(iii) Bearing capacity of bolt (Vdpb) [Cl. 10.3.4]:</u>

End dist., e = 45 mm; Pitch, p = 180 mm

Hole dia., do = 38 mm [Table 14-2; AISC]

$e/3d_o = 45/(3 \times 38) = 0.39$; $p/3d_o - 0.25 = 180/(3 \times 38) - 0.25 = 1.33$



ANCHOR BOLT

Ratio of ult. tensile stress of bolt to that of plate, fub/ fup = 400/ 500 = 0.8
=> kb = Min (e/ 3do, p/ 3do - 0.25, fub/ fup, 1.0) = Min (0.39, 1.33, 0.8, 1) = 0.39
=> Nominal bearing strength, Vnpb = 2.5 kb.d.t.fup = 2.5 x 0.39 x 30 x 25 x 500 = 365.6 kN
Hence, Vdpb = Vnpb/ 1.25 = 365.6/ 1.25 = 292.5 kN

(iv) Check for shear [Cl. 10.3.2]:

Design strength of bolt, Vdb = Min (Vdsb, Vdpb) = Min (103.65, 292.5) = 103.65 kN
Since, Vsb < Vdb (= 103.65 kN), Hence OK

[b] Tension:

(i) Tension capacity of the bolt (Tdb) [Cl. 10.3.5]:

Nominal tensile capacity of bolt, Tnb, is given by,
Tnb = 0.90 fub.An < fyb.Asb (1.25/ 1.10)
Where, fub = Ult. ten. stress of bolt; An = Net shear area of bolt; fyb = yield stress of bolt
Hence, Tnb = Min {0.90 x 400 x 561, 240 x 706.9 x (1.25/ 1.10)} = 192.79 kN
=> Tdb = Tnb/ 1.25 = 192.79/ 1.25 = 154.23 kN

(ii) Factored tension per bolt (Tb):

a. Tension due to axial force (Tba):
Factored axial tension, Fy = 0 kN
Tba = Fy/ n = 0/ 16 = 0 kN
b. Tension due to bending (Tbz & Tby):
Factored moment, Mz = -488.76 kNm
Equating moment of areas of 'T' & 'C', we get,
Tdb.n.(L/ 2 + D/ 2 - Tf/ 2 - Ybar,z) = (1/ 2).fa.Ybar,z.B.(2/ 3).Ybar,z
[n = eff. bolts = 8; L = b/p length = 850 mm; D = col. depth = 600 mm; Tf = flg. thk. = 14 mm]
fa = allow. conc. stress = Min (0.6 fck, fa,per) = Min (0.6 x 25, 15) = 15 MPa [Cl. 7.4.1]
Solving, Ybar,z = 364.7 mm => Lever arm, LAz = (L/ 2 + D/ 2 - Tf/ 2 - Ybar,z/ 3) = 593.4 mm
=> Tbz = Mz/ (n.LAz) = 488.76 x 1000/(8 x 593.4) = 102.96 kN
Similarly, Tby = 0.34 kN
c. Tension due to prying action [Cl. 10.4.7]:
(Note: Prying force ignored, being too small for a b/p bearing against concrete/grout)
Hence, Tb = Tba + Tbz + Tby = 0 + 102.96 + 0.34 = 103.3 kN

(iii) Check for tension [Cl. 10.3.5]:

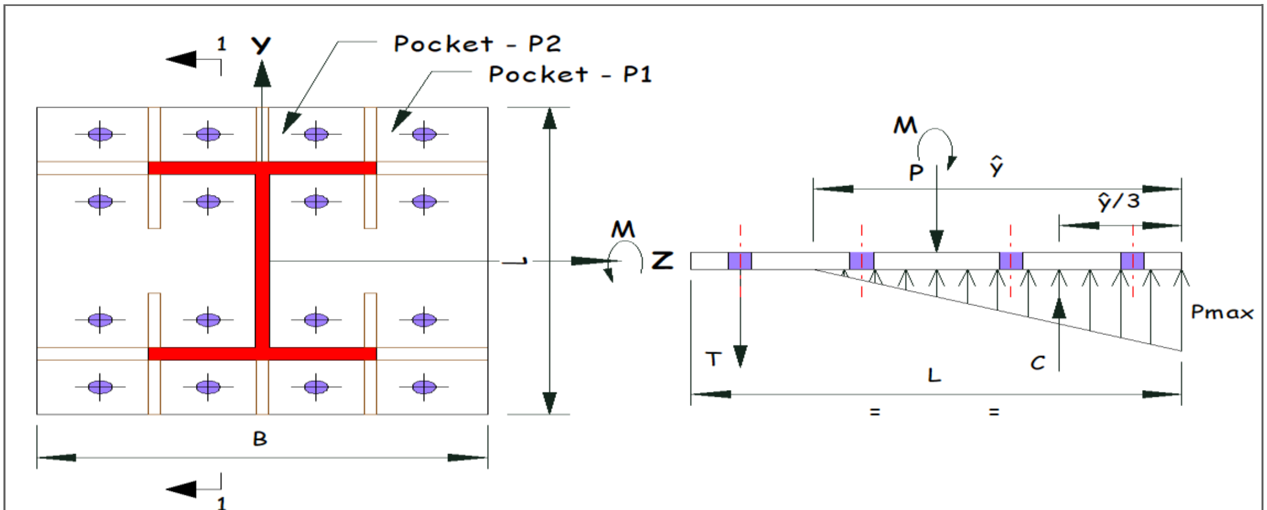
Since, Tb < Tdb (= 154.23 kN), Hence OK

[c] Combined shear and tension [Cl. 10.3.6]:

Vsb/ Vdb = 1.67/ 103.65 = 0.016
Tb/ Tdb = 103.3/ 154.23 = 0.67
Hence, $(Vsb/ Vdb)^2 + (Tb/ Tdb)^2 = 0.016^2 + 0.67^2 = 0 + 0.449 = 0.449 < 1$, Hence OK

Provide 16-M30 anchor bolts

BASEPLATE DESIGN



B/P PLAN

SECTION 1-1
(STRESS DIAGRAM)

[a] Factored loads (Ref. Table-P1):

F_x = 255.9 kN (C)

M_z = -488.8 kNm; M_y = -0.6 kNm

Table-P1 (Base press. cals.): [Corresp bolt dia. = 30 mm; Tdb = 154.23 kN]

Direct		Z-axis bending				Y-axis bending				Tot. (Mpa)	Result
F_x (kN)	f_{ca} (MPa)	M_z (kNm)	$\dot{y}_z$ (mm)	LA_z (mm)	$f_{cb,z}$ (MPa)	M_y (kNm)	$\dot{y}_y$ (mm)	LA_y (mm)	$f_{cb,y}$ (MPa)	$f_c=(f_{ca}+f_{cb,z}+f_{cb,y})$	
255.9	0.46	-488.8	364.7	593.4	6.95	-0.6	207.7	435.8	0.02	7.43	< 15.0 (OK)

[b] Sectional properties:

a = 150 mm; b = 125 mm; $c^2 = (a^2 - 0.3\, b^2)$ => c = 133.5 mm

Eff. b/p area, Aeff = 552500 mm^2

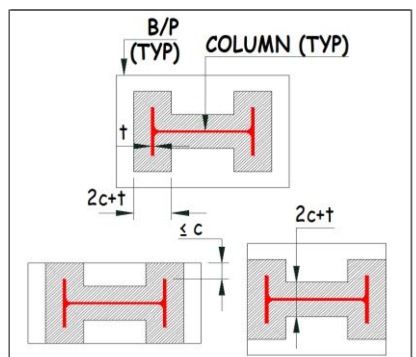
[b/p area for direct comp., 100 Aeff/ (L.B) = 100%]

$t_{req} = \sqrt{(2.5\, f_{ca}.c^2.\gamma_{mo}/ f_y)}$

$= \sqrt{(2.5 \times 0.46 \times 133.5^2 \times 1.1/ 345)}$ = 8.1 mm

t_{prov} = 25 mm > Max [t_{req} (= 8.1 mm), t_f (= 14 mm)]... Hence OK

Sectional modulus, $Z_{prov} = 1.t^2/ 6 = 1 \times 25^2/ 6$ = 104.2 mm^3/ mm



EFF. AREA FOR I - COLUMN
(STIFF. NOT SHOWN FOR CLARITY)

[c] Stresses under B/P:

(i) Due to axial compression (f_{ca}):

f_{ca} = Fx/ Aeff = 1000 x 255.9/ 552500 = 0.46 MPa

(ii) Due to moments M_z ($f_{cb,z}$) & M_y ($f_{cb,y}$):

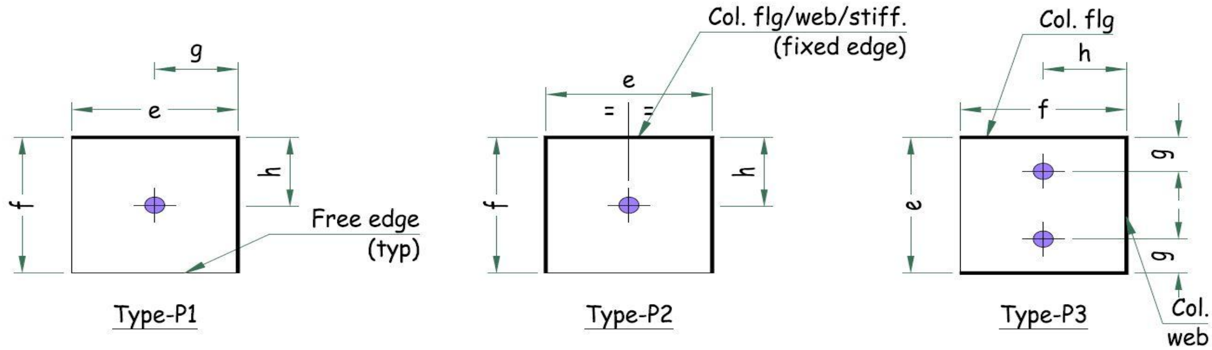
NA depth ($\dot{y}$) and lever arm are obtained by equating BM to MR.

$|M_z| = \{(1/2).f_{cb,z}.\dot{y}_z.B\}.LA_z \Rightarrow f_{cb,z} = |M_z|/ \{(1/2).\dot{y}_z.B.LA_z\}$

$|M_y| = \{(1/2).f_{cb,y}.\dot{y}_y.L\}.LA_y \Rightarrow f_{cb,y} = |M_y|/ \{(1/2).\dot{y}_y.L.LA_y\}$

Note: Calculations for f_{ca}, $f_{cb,z}$ & $f_{cb,y}$ along with results are tabulated in Table-P1

[c] Flexural stresses in B/P:



BOLT POCKETS

Table-P2 (BM: B/P in comp.): [[1]For pinned b/p in direct comp, 't' is calculated as per Cl. 7.4.3.1]

Pocket type	Dims. (mm)		Design Press.	Max. BM coeff.		Max. BM (Nmm/mm)		Max. BM for panel (Nmm/mm)
	e	f	pmax (MPa)	Sagging (α)	Hogging (β)	Sagging (α.pmax.f²)	Hogging (β.pmax.f²)	
P1	145	125	7.43	0.2949	0.0324	34236	3761	34236
P2	168	125	7.43	0.1496	0.069	17368	8010	17368
P3	-	-	-	-	-	-	-	-

Table-P3 (BM: A/B in tension):

Pocket type	Dims. (mm)				A/B	Max. BM				Max. BM for panel (Nmm/mm)
	e	f	g	h	Tb (kN)	Msag (kNm)	beff (mm)	Mhog (kNm)	beff (mm)	
P1	145	125	90	80	103.3	0	0	4.375	125	35000
P2	168	125	84	80	103.3	2.169	99	2.169	137	21912
P3	-	-	-	-	-	-	-	-	-	-

Table-P4 (Check for flexure):

M (B/P in comp.); N, mm			M (A/B in tens.); N, mm			t (mm)	Sect. Mod (mm³/mm)			Result
Pocket	Mmax	beff	Pocket	Mmax	beff		Zreq,gov	</>	Zprov	
P1	34236	1	P1	35000	1	25	101.4	<	104.2	OK

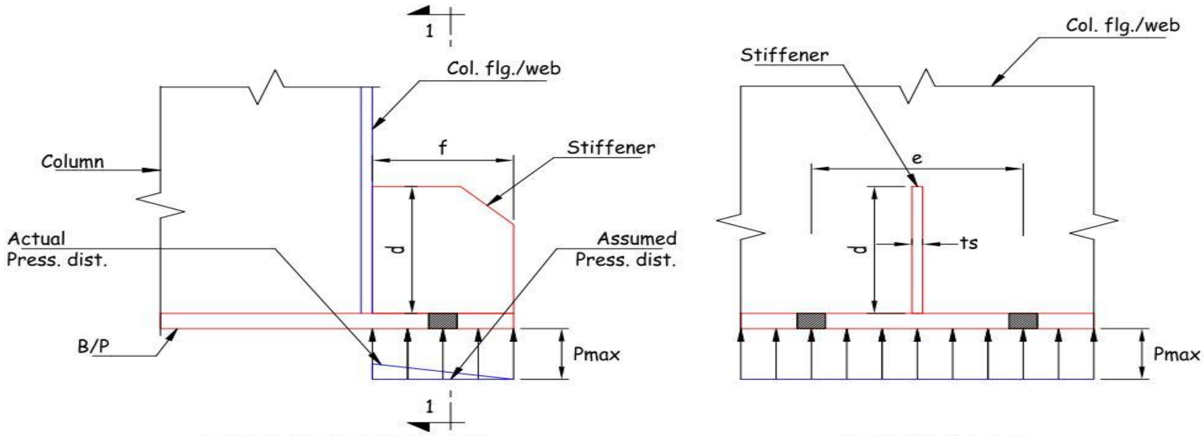
Calculations & design assumptions:

a. Calculations for B/P thickness have been tabulated in Tables P2, P3 & P4.
b. For 'A/B in tension' case, eff. B/P widths for hogging & sagging moments are calculated from cl. 24.3.2 of IS 456:2000. Where B/P panel is subjected to a sagging momment, width of bolt hole is deducted from 'effective' width to get the 'net effective' width.
c. In Table-P4, $Z_{req,gov}$ corresponds to the BM that governs the B/P design.

Provide 25 mm thk. baseplate

epicenter Consulting Engineers Pvt. Ltd.	Client : EPICENTER CONSULTING ENGINEERS					Element: Baseplate
	Project:	1002	Doc. No.:	1002-CAL-ST-10		Location/ A-1
	Rev.	Ppd. by	Date	Chd. by	Date	
Project: WAREHOUSE	2					Designation: BP1
Structure: PR-05A	1					
Type: PEB	0	-	20-01-2026	-	20-01-2026	Sht. 4 of 8

STIFFENER DESIGN



TYP.B/P STIFFENER　　SECTION 1-1

[a] Dimensions (Ref. above figure):

Dimension 'e' is as per the baseplate plan.

Dimension 'f' is calculated assuming a load dispersion angle of 45° from column-stiffener connection.

Assumed values of 'd' & 't_s' for each stiffener type are as per Table-S1.

Shear area, section modulus & section class for stiffeners are as per Table-S1.

For stiffeners, fy = 345 Mpa

=> Yield stress ratio, e1 = √(250/ fy) = 0.85

Table-S1 (Stiffener: Dimensions, section properties & section class):

Stiff. type	Dimensions				Sectional properties		Sectional class		
	e (mm)	f (mm)	d (mm)	t_s (mm)	A_v (cm^2)	Z_z (cm^3)	e_1 √(250/f_y)	d ($t_s.e_1$)	Class
S1	215	125	150	16.0	24.00	60.00	0.85	11.03	<18.9 [Class3-SC]
S2	-	-	-	-	-	-	-	-	-
S3	275	150	150	16.0	24.00	60.00	0.85	11.03	<18.9 [Class3-SC]

[b] Loads (Ref. above figure):

(Stiffener resembles the stem of an inverted 'T' beam, and is restrained laterally by col flg/ web).

Base pressures at stiffener ends are calculated by triangle similarity for the contact length.

Design press. '$p_{,des}$' for each stiffener type is assumed as the max. press. anywhere on its length.

Rectangular press. distribution is assumed for stiffener design....a conservative assumption.

UDL on the stiffener, w = $p_{,des}$ x e

All stiffeners are designed as cantilevers for a UDL of 'w' kN/m over their entire length.

[c] Check for shear [Cl. 8.4]:

Shear force, V = w.f, where w is the UDL on the stiffener

Shear area, A_v = d.t_s

=> Vp = Av.fy/ √3

	Client :	EPICENTER CONSULTING ENGINEERS				Element: Baseplate
	Project:	1002	Doc. No.:	1002-CAL-ST-10		Location/ A-1
	Rev.	Ppd. by	Date	Chd. by	Date	
Project: WAREHOUSE	2					Designation:
Structure: PR-05A	1					BP1
Type: PEB	0	-	20-01-2026	-	20-01-2026	Sht. 5 of 8

Design shear strength $V_d = V_p / 1.10$
Values for V_u & V_d are tabulated in Table-S2

Table-S2 (Stiffener: Loads, shear & moments):

Stiff. type	Loads				Shear (kN)			Moment (kNm)			Result
	Base Pressure (MPa)			w (kN/m)	Actual V_u	</>	60% Cap. $0.6V_d$	Actual M_u	</>	Capacity M_d	
	$p_{,max}$	$p_{,min}$	$p_{,des}$								
S1	7.42	5.03	7.42	1595	199.4	<	260.8	12.46	<	18.82	OK
S2	-	-	-	-	-	-	-	-	-	-	-
S3	5.05	5.03	5.05	1389	208.4	<	260.8	15.63	<	18.82	OK

[d] Check for flexure [Cl. 8.2]:
Design moment, $M_u = w.f^2 / 2$
Since, $d/t_s < 67\, e1$ & $V_u < 0.6V_d$ & stiffener is a cantilever [Cl. 8.2.1.2]
=> Design bending strength, $M_d = 1.5\, Z_z.f_y / 1.10$
Values for M_u & M_d are tabulated in Table-S2

Table-S3 (Stiffener weld: Size, strength & allowable stresses):

Stiff. type	Weld size (mm)			Design strength				Stresses (MPa)			Result
	Size s (mm)	Throat t_t (mm)	Length l_w(mm)	fu (MPa)	γ_{mw}	f_{wn} (MPa)	f_{wd} (MPa)	f_a	q	f_e	
S1	12	8.4	300	480	1.25	277.1	221.7	0	79.1	137	fe < fwd, OK
S2	-	-	-	-	-	-	-	-	-	-	-
S3	-	-	-	-	-	-	-	-	-	-	FS butt weld

[e] Fastener design [Cl. 10.5]:
Assume 2 lines of fillet welds for all stiffeners
Length of fillet weld, $l_w = 2 \times$ depth of stiffener = 2d
<u>(i) Design strength (f_{wd}) [Cl. 10.5.7.1.1]:</u>
Throat thickness, $t_t = 0.7 \times s$, where s = size of fillet weld
$f_{wn} = f_u / \sqrt{3}$ => $f_{wd} = f_{wn} / \gamma_{mw}$
<u>(ii) Normal stress (f_a) [Cl. 10.5.9]:</u>
Normal stress, $f_a = M_u / Z_z$
<u>(iii) Shear stress (q) [Cl. 10.5.9]:</u>
=> Shear stress, $q = V_u / (t_t.l_w)$
<u>(iv) Combination of stresses [Cl. 10.5.10]:</u>
Equivalent stress, $f_e = \sqrt{(f_a^2 + 3q^2)}$

Note: Weld dimensions, design strength & stresses are tabulated in Table-S3

Stiffener-S1: 150x16 thk. connected with 12 mm fillet weld
Stiffener-S3: 150x16 thk. connected with full strength butt weld

SHEAR KEY DESIGN

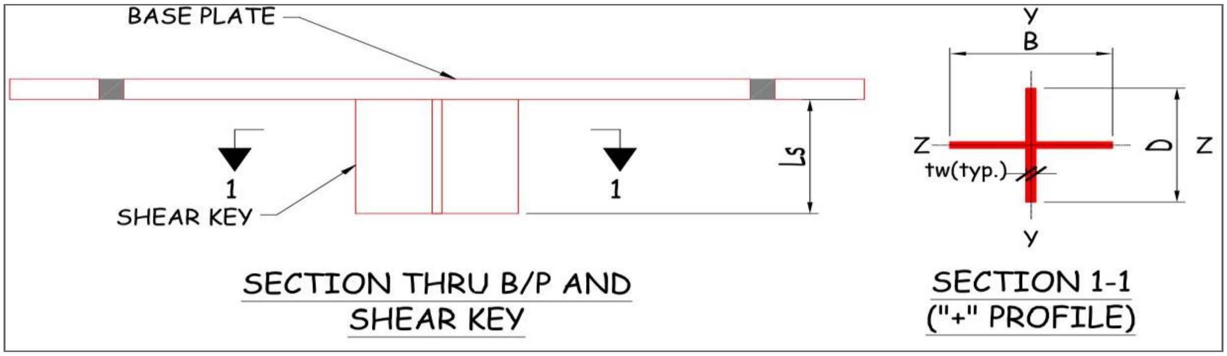


Table-K1 (Shear key: Dimensions & sectional properties):

Type	Section	Dimensions				Properties		
		D (mm)	B (mm)	Tf (mm)	tw (mm)	A (cm^2)	Zz (cm^3)	Zy (cm^3)
+	200x200x12	200.0	200.0	12.0	12.0	46.56	80.27	80.27

Table-K2 (Shear key: Shear in Y-dirn):

Length Ls (mm)	Shear (u/s of b/p)				BM (u/s of b/p)				
	$A_{vy} = D.t_w$ (cm^2)	F_y (kN)	</>	$0.6V_{dy}$ (kN)	τ_{by} (MPa)	M_z (kNm)	</>	M_{dz} (kNm)	Result
200	24.00	106.3	<	260.8	2.66	10.63	<	25.18	OK

Table-K3 (Shear key: Shear in Z-dirn):

Length Ls (mm)	Shear (u/s of b/p)				BM (u/s of b/p)				
	$A_{vz} = 2.B.T_f$ (cm^2)	F_z (kN)	</>	$0.6V_{dz}$ (kN)	τ_{bz} (MPa)	M_y (kNm)	</>	M_{dy} (kNm)	Result
200	24.00	11.3	<	260.8	0.28	1.13	<	25.18	OK

[a] Shear u/s of b/p (Ref. Tables K2 & K3):

$V_{py} = A_{vy}.f_y/\sqrt{3}$ => Design shear strength, $V_{dy} = V_{py}/1.10$

$V_{pz} = A_{vz}.f_y/\sqrt{3}$ => Design shear strength, $V_{dz} = V_{pz}/1.10$

[b] Bearing stress on conc. (Ref. Tables K2 & K3):

$\tau_{by} = F_y/(L_s.B)$; $\tau_{bz} = F_z/(L_s.D)$

[c] Flexure (Ref. Tables K2 & K3):

$M_z = \tau_{by}.B.L_s^2/2$; $M_{dz} = Z_z.f_y/1.1$

$M_y = \tau_{bz}.D.L_s^2/2$; $M_{dy} = Z_y.f_y/1.1$

Use 200 long 200x200x12 SK butt welded to B/P

BILL OF QUANTITIES

Table-Q1 (Baseplate):

Sr. No.	Element	Dimensions L (mm)	B (mm)	T (mm)	BP nos.	Weight Wt./BP (kg)	Total wt. (kg)
1	Baseplate	850	650	25	12	108.43	1,301.14

Table-Q2 (Anchor bolts):

Sr. No.	Element	Dia (mm)	Type	Nos. Nos./BP	BP nos.	Tot. nos.	Weight Unit wt.	Wt./BP (kg)	Total wt. (kg)
1	Anchor bolts	30	B	16	12	192	6.95	111.20	1,333.80

Table-Q3 (Stiffeners):

Sr. No.	Element	Nos.	Dimensions L (mm)	H (mm)	ts (mm)	BP nos.	Weight Wt./BP (kg)	Total wt. (kg)
1	S1	10	125	150	16	12	23.55	282.60
2	S2	-	-	-	-	-	-	-
3	S3	4	150	150	16	12	11.30	135.60
4	S4	-	-	-	-	-	-	-

Table-Q4 (Shear key):

Sr. No.	Type	Member/ profile	Dimensions L (mm)	B (mm)	D (mm)	T (mm)	t (mm)	BP nos.	Weight Wt./BP (kg)	Total wt. (kg)
1	+	200x200x12	200	200	200	12.00	12.00	12	7.31	87.72

Table-Q5 (Summary):

Sr. No.	Element	Per BP Nos.	Weight (kg)	Total Nos.	Weight (kg)	(%)	Remarks
1	Baseplate	1	108.43	12	1,301.14	41.43	
2	Anchor bolts	16	111.20	192	1,333.80	42.47	
3	Stiffeners	14	34.85	168	418.20	13.31	
4	Shear key	1	7.31	12	87.72	2.80	
			Total -->		3140.86	100	

MIN. PEDESTAL DIMS.

850 mm (|| to col. flange); 1100 mm (|| to col. web)