BASEPLATE DESIGN (LSM-IS 800:2007)

[A] DESIGN INPUT (Ref. Sheet-1, Table-A1 & Table-A2):

Table-A1 (Governing loads; Z = major bending axis; Y = minor bending axis; X = column axis]:

Node	L/C	Fy (kN)	Fz (kN)	Fx (kN)	Mz (kNm)	My (kNm)	Mx (kNm)	Stress Inc. (Y/N)	Governing Criteria
Unf. reactions from structure analysis									
1401	0	-60.2	0	56.83	383.1	0	0	N	Max. bolt dia.
1401	0	-60.2	0	56.83	383.1	0	0	N	Max. base press.
Unf. reactions modified for stress incr. (0.75 x load) and shear key presence									
1401	0	-15.1	0	56.83	383.1	0	0	N	Max. bolt dia.
1401	0	-15.1	0	56.83	383.1	0	0	N	Max. base press.
1401	0	73.6	0	-1.97	360.4	0	0	N	Shear key size

Table-A2 (Material properties):

(i) Anchor bolt (Grade 4.6):	(ii) Baseplate:	(iii) Stiffener:
fyb = 240 MPa	fyp = 345 MPa	fys = 345 MPa
fub = 400 MPa	fup = 500 MPa	fus = 500 MPa
(iv) Shear key:	(v) Fillet weld:	(vi) Concrete:
Shear key: Yes	Min size = 6 mm	fcu = 35 MPa
Profile: 'H'	fw = 400 MPa	fa,per = 10 MPa

[B] ANCHOR BOLT DESIGN (with shear key)

Assume 36 mm dia bolts & 32 mm thk b/p

[a] Shear [Cl. 10.3.2]:

(i) Factored shear force per bolt (Vsb):

Fy = 15.06/ 8 = 1.88 kN; Fz = 0/ 8 = 0 kN

=> Resultant, R = $(Fy^2 + Fz^2)$ ^0.5 = 1.88 kN

Hence, Vsb = 1.5 x R = 1.5 x 1.88 = 2.82 kN

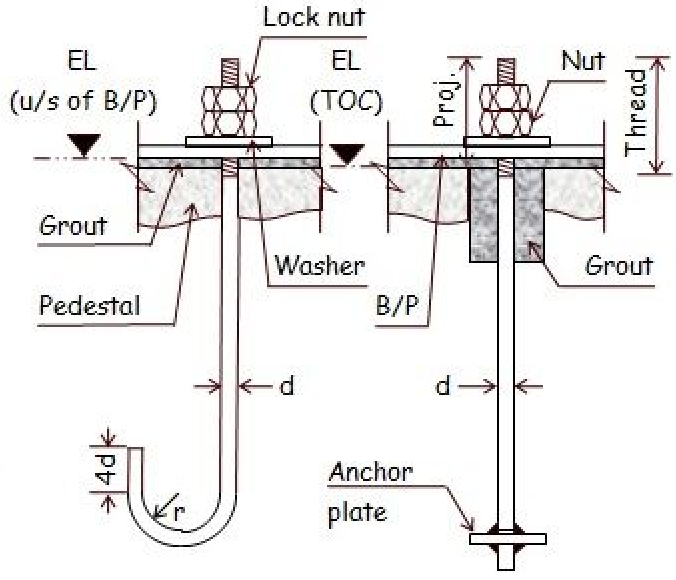
(ii) Shear capacity of bolt (Vdsb) [Cl. 10.3.3]:

Vnsb = fub (nn.Anb + ns.Asb)/ $\sqrt{3}$, 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
- Asb = 1017.9 mm^2; Anb = 817 mm^2

=> Vnsb = fub (nn.Anb + ns.Asb)/ 3^0.5 = 188.68 kN

Hence, Vdsb = Vnsb/ 1.25 = 188.68/ 1.25 = 150.94 kN



Type-A Type-B

ANCHOR BOLT

(iii) Bearing capacity of bolt (Vdpb) [Cl. 10.3.4]:

End dist. (e) = 75 mm; Pitch (p) = 950 mm; Hole dia. (do) = 39 mm

e/3do = {75/ (3 x 39)} = 0.64; p/3do - 0.25 = {950/ (3 x 39) - 0.25} = 7.87

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 (0.64, 7.87, 0.8, 1) = 0.64
=> Nominal bearing strength, Vnpb = 2.5 kb.d.t.fup = 2.5 x 0.64 x 36 x 32 x 500 = 921.6 kN
Hence, Vdpb = Vnpb/ 1.25 = 921.6/ 1.25 = 737.3 kN

(iv) Check for shear [Cl. 10.3.2]:
Design strength of bolt, Vdb = Min (Vdsb, Vdpb) = Min (150.94, 737.3) = 150.94 kN
Since, Vsb < Vdb (= 150.94 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 817, 240 x 1017.9 x (1.25/ 1.10)} = 277.61 kN
=> Tdb = Tnb/ 1.25 = 277.61/ 1.25 = 222.09 kN

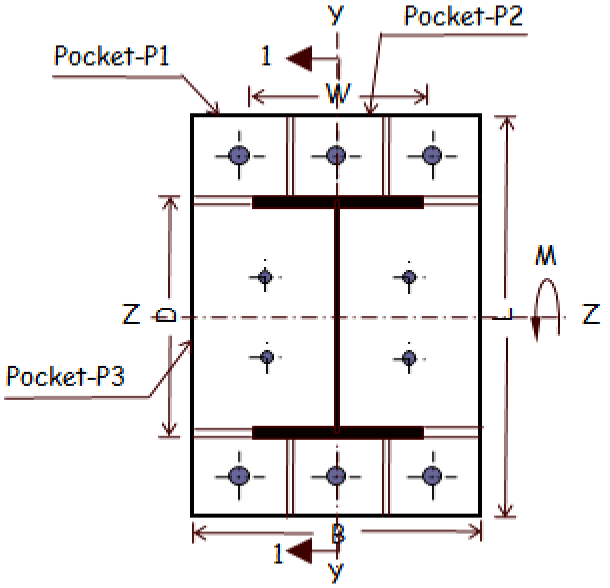
(ii) Factored tension per bolt (Tb):
a. Tension due to axial force (Tba):
Factored axial tension, Fuy = 1.5 Fy = 0 kN
Tba = Fuy/ n = 0/ 8 = 0 kN
b. Tension due to bending (Tbz & Tby):
Factored moment, Muz = 1.5 x Mz = 574.65 kNm
Equating moment of areas of 'T' & 'C', we get,
Tdb.n.(L-e-Ybar,z) = (1/2).fa.Ybar,z.B.(2/3).Ybar,z
where, n = no. of eff. bolts = 4; L = b/p length = 1100 mm; e = edge distance = 75 mm;
fa = allow. conc. stress = Min (0.6 fck, fa,per) = Min (0.6 x 35, 10) = 10 MPa [Cl. 7.4.1]
Solving, Ybar,z = 398.7 mm => Lever arm, LAz = (L - e - Ybar, z/3) = 892.1 mm
=> Tbz = Muz/ (n.LAz) = 574.65 x 1000/(4 x 892.1) = 161.04 kN
Similarly, Tby = 0 kN
c. Tension due to prying action [Cl. 10.4.7]:
(Note: Prying force ignored, being too small for a baseplate)
Hence, Tb = Tba + Tbz + Tby = 0 + 161.04 + 0 = 161.04 kN

(iii) Check for tension [Cl. 10.3.5]:
Since, Tb < Tdb (= 222.09 kN), Hence OK

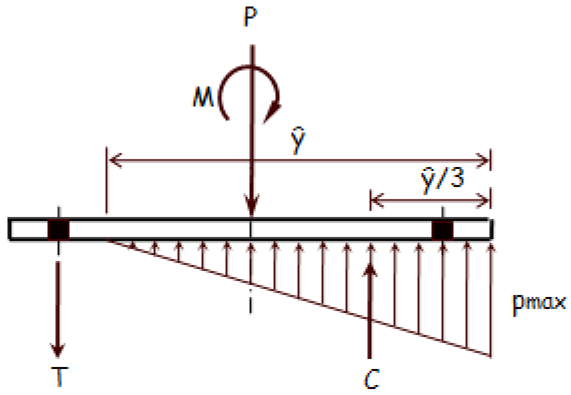
[c] Combined shear and tension [Cl. 10.3.6]:
Vsb/Vdb = 2.82/ 150.94 = 0.019
Tb/Tdb = 161.04 / 222.09 = 0.725
Hence, $(Vsb/Vdb)^2 + (Tb/Tdb)^2$ = 0 + 0.526 = 0.526 < 1.0, Hence OK

Provide 8-M36 Anchor Bolts (Type-B)

[C] BASEPLATE DESIGN [Cl. 7.4]



TYP. B/P SHOWING BOLT POCKETS



SECTION 1-1 (STRESS DIAGRAM)

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

$F_{ux} = 1.5\ F_x = 1.5 \times 56.83 = 85.3$ kN

$M_{uz} = 1.5\ M_z = 1.5 \times 383.1 = 574.7$ kNm

$M_{uy} = 1.5\ M_y = 1.5 \times 0 = 0$ kNm

Table-P1 (Base press. cals.): [Corresp bolt dia. = 36 mm; Tdb = 222.09 kN]

Direct		Z-axis bending				Y-axis bending				Tot. (Mpa)	Result
Fux	fca	Muz	ẏz	LAz	fcb,z	Muy	ẏy	LAy	fcb,y	fc=(fca+	
(kN)	(MPa)	(kNm)	(mm)	(mm)	(MPa)	(kNm)	(mm)	(mm)	(MPa)	fcb,z+fcb,y)	
85.3	0.07	574.7	398.7	892.1	3.08	0.0	-	-	0.00	3.15	< 10.0 (OK)

[b] Sectional properties (Assume 32 thk. b/p):

$A = L.B = 1100 \times 1050/\ 100 = 11550$ sq.cm; $Z = b.t^2/\ 6 = 1 \times 32^2/\ 6 = 170.7\ mm^3/mm$

[c] Stresses under B/P:

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

$f_{ca} = F_{ux}/\ A$

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

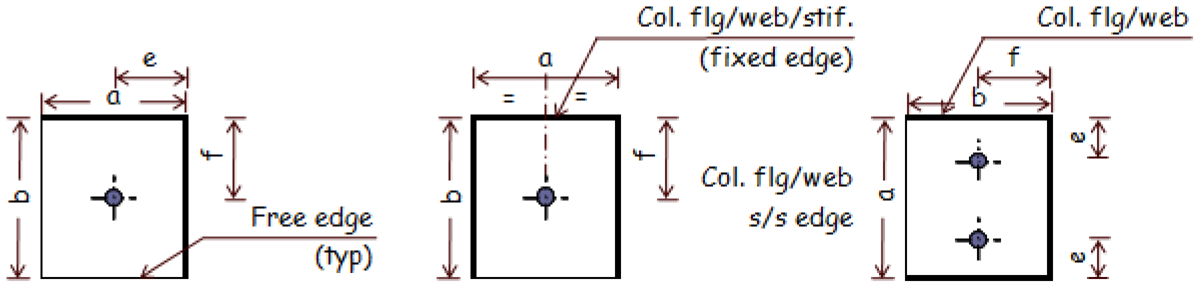
NA depth ($\dot{y}$) and lever arm (LA) have been obtained by equating BM to MR.

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

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

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:



Type-P1 **Type-P2** **Type-P3**

BOLT POCKETS

Table-P2 (BM: B/P in compression):

Pocket type	Dims. (mm)		Design Press.	Max. BM coeff.		Max. BM (Nmm/mm)		Max. BM
	a	b	pmax (MPa)	Sagging (α)	Hogging (β)	Sagging ($\alpha.p_{max}.b^2$)	Hogging ($\beta.p_{max}.b^2$)	for panel (Nmm/mm)
P1	225	175	3.15	0.2949	0.0324	28449	3126	28449
P2	300	175	3.15	0.2142	0.0893	20664	8615	20664
P3	-	-	-	-	-	-	-	-

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

Pocket type	Dims. (mm)				A/B	Max. BM				Max. BM
	a	b	e	f	$T_{b,fac}$ (kN)	$M_{u,sag}$ (kNm)	beff (mm)	$M_{u,hog}$ (kNm)	beff (mm)	for panel (Nmm/mm)
P1	225	175	150	100	161	0	0	9.662	175	55214
P2	300	175	150	100	161	6.039	160	6.039	199	37744
P3	-	-	-	-	-	-	-	-	-	-

Table-P4 (Check for flexure):

Mu (B/P in comp.); N, mm			Mu (A/B in tens.); N, mm			t	Sect. Mod (mm³/mm)			Result
Pocket	$M_{u,max}$	beff	Pocket	$M_{u,max}$	beff	(mm)	$Z_{req,gov}$	</>	Z_{prov}	
P1	28449	1	P1	55214	1	32	160.0	<	170.7	OK

Calculations & design assumptions:

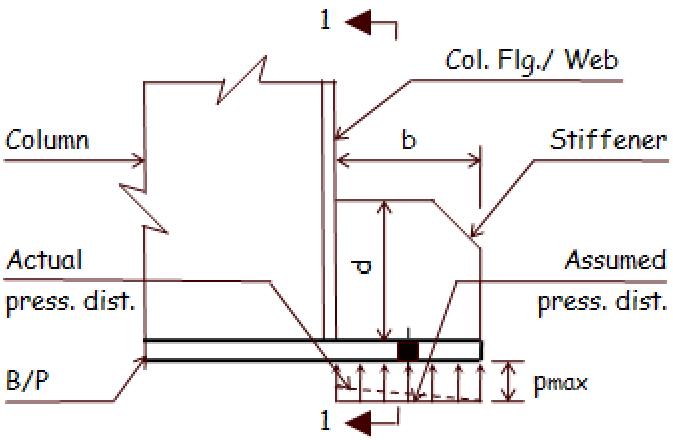
a. Calculations for B/P thickness have been tabulated in Tables P2, P3 & P4.

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 mommemt, width of bolt hole is deducted from 'effective' width to get the 'net effective' width.

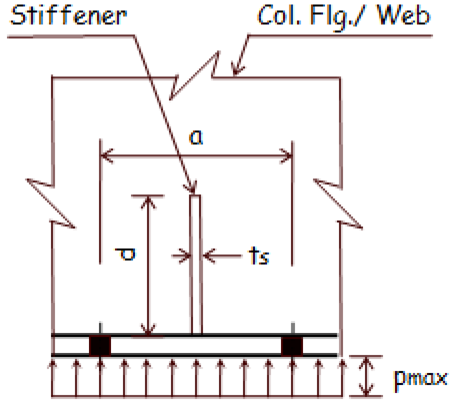
c. In Table-P4, $Z_{req,gov}$ corresponds to the BM that governs the B/P design.

Provide 32 mm thk. baseplate

[D] STIFFENER DESIGN



TYP. B/P STIFFENER



SECTION 1-1

[a] Dimensions (Ref. above figure):

Dimension 'a' is as per the baseplate plan.

Dimension 'b' is calculated assuming a load dispersion angle of 45^0 from column-stiffener connection.

Assumed values of 'd' & 'ts' 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)^0.5 = 0.85

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

Stiff. type	Dimensions				Sectional properties		Sectional class		
	a (mm)	b (mm)	d (mm)	ts (mm)	Av (cm^2)	Zz (cm^3)	e_1 $\sqrt{(250/fy)}$	$\frac{d}{(ts.e_1)}$	Class
S1	325	175	225	16.0	36.00	135.00	0.85	16.54	<18.9 [Class3-SC]
S2	-	-	-	-	-	-	-	-	-
S3	500	325	225	16.0	36.00	135.00	0.85	16.54	<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.

Factored UDL on the stiffener, w = p,des x a

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

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

Factored shear force, Vu = w.b, where w is the factored UDL on the stiffener

Shear area, Av = d.ts

=> Vp = Av.fy/√3

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 (fact.)				Shear (kN)			Moment (kNm)			Result
	Base Pressure (MPa)			w	Actual	Vs	60% Cap.	Actual	Vs	Capacity	
	p,max	p,min	p,des	(kN/m)	V_u		$0.6V_d$	M_u		M_d	
S1	3.15	1.8	3.15	1024	179.2	<	391.1	15.68	<	63.51	OK
S2	-	-	-	-	-	-	-	-	-	-	-
S3	1.8	1.8	1.8	900	292.5	<	391.1	47.53	<	63.51	OK

[d] Check for flexure [Cl. 8.2]:
Design moment, $M_u = w.b^2 / 2$
Since, $d/t_s < 67\ \varepsilon 1$ & $V_u < 0.6V_d$ & 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 tt (mm)	Length lw(mm)	fu (MPa)	γ_{mw}	fwn (MPa)	fwd (MPa)	fa	q	fe	
S1	8	5.6	450	400	1.25	230.9	184.7	116.1	71.1	169.2	fe < fwd, OK
S2	-	-	-	-	-	-	-	-	-	-	-
S3	-	-	-	-	-	-	-	-	-	-	Butt weld

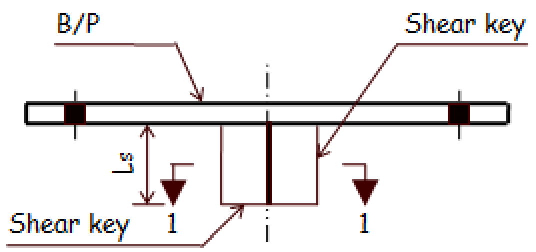
[e] Fastener design [Cl. 10.5]:
Assume 2 lines of fillet welds for all stiffeners
Length of fillet weld, $l_w = 2 \times$ depth of stiffener = 2d
(i) Design strength (fwd) [Cl. 10.5.7.1.1]:
Throat thickness, $t_t = 0.7 \times s$, where s = size of fillet weld
$f_{wn} = f_u / \sqrt{3}$ => $f_{wd} = f_{wn}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

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

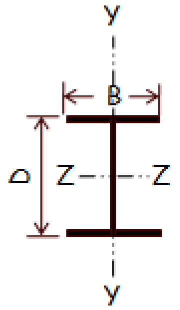
Stiffener-S1: 225x16 thk. connected with 8 mm fillet weld

Stiffener-S3: 225x16 thk. connected with full depth butt weld

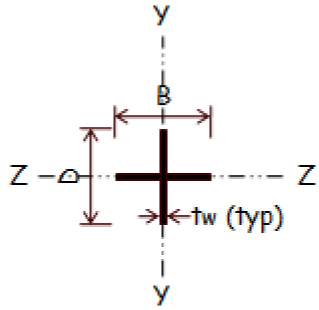
[E] SHEAR KEY DESIGN



SECTION THRU B/P
AND SHEAR KEY



SECTION 1-1
("H" PROFILE)



SECTION 1-1
("+" PROFILE)

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

Profile	Section	Dimensions				Properties		
		D (mm)	B (mm)	Tf (mm)	tw (mm)	A (cm^2)	Zz (cm^3)	Zy (cm^3)
H	UC152X152X30	157.6	152.9	9.4	6.5	38.30	248.00	112.00

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

Length	Shear (u/s of b/p)					BM (u/s of b/p)					
Ls (mm)	Avy=D.tw (cm^2)	Fy (kN)	Fuy (kN)	Vs	0.6Vdy (kN)	τby (MPa)	Mz (kNm)	Muz (kNm)	</>	Mdz (kNm)	Result
150	10.24	73.6	110.4	<	111.3	3.21	5.52	8.28	<	116.67	OK

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

Length	Shear (u/s of b/p)					BM (u/s of b/p)					
Ls (mm)	Avz=2.B.Tf /B.tw(cm^2)	Fz (kN)	Fuz (kN)	Vs	0.6Vdz (kN)	τbz (MPa)	My (kNm)	Muy (kNm)	</>	Mdy (kNm)	Result
150	28.75	0.0	0.0	<	312.3	0.00	0.00	0.00	<	52.69	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):

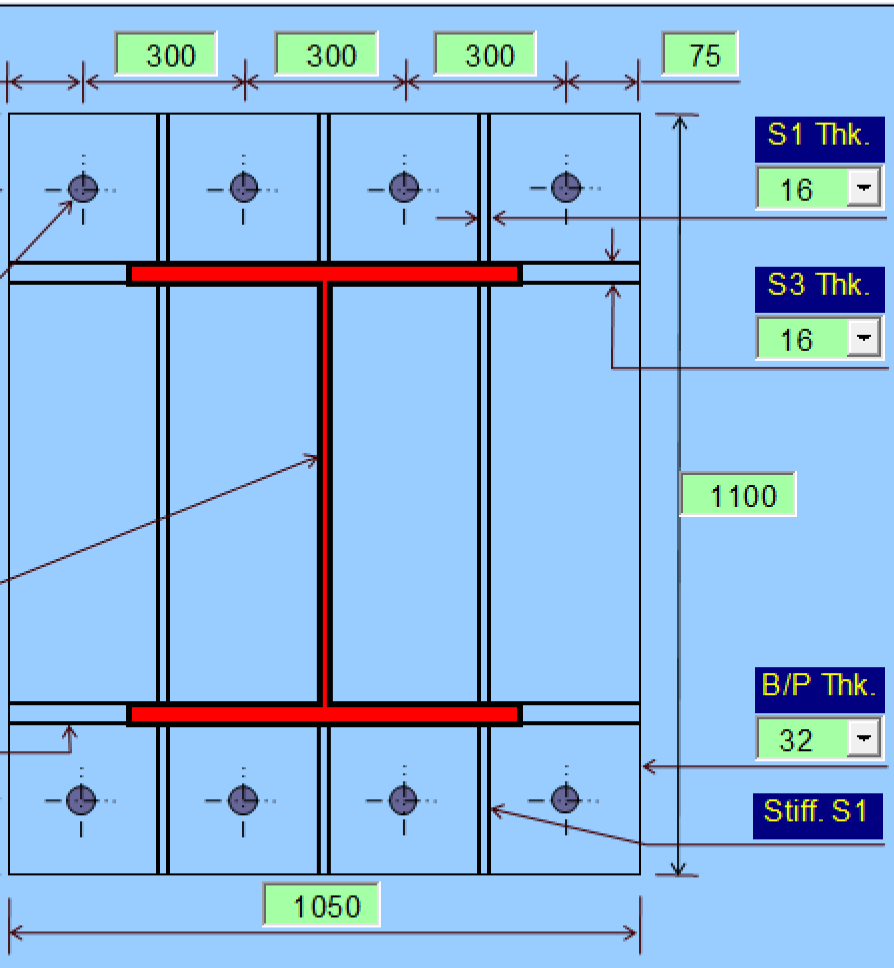
$\tau_{by} = F_y/(L_s.B)$; $\tau_{bz} = F_z/(L_s.D)$

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

$M_z = \tau_{by}.B.L_s^2/2$; $M_{dz} = 1.5\ Z_z.f_y/1.1$

$M_y = \tau_{bz}.D.L_s^2/2$; $M_{dy} = 1.5\ Z_y.f_y/1.1$

Use 150 long UC152X152X30 SK butt welded to B/P



BASEPLATE PLAN

(All dims. are in mm)