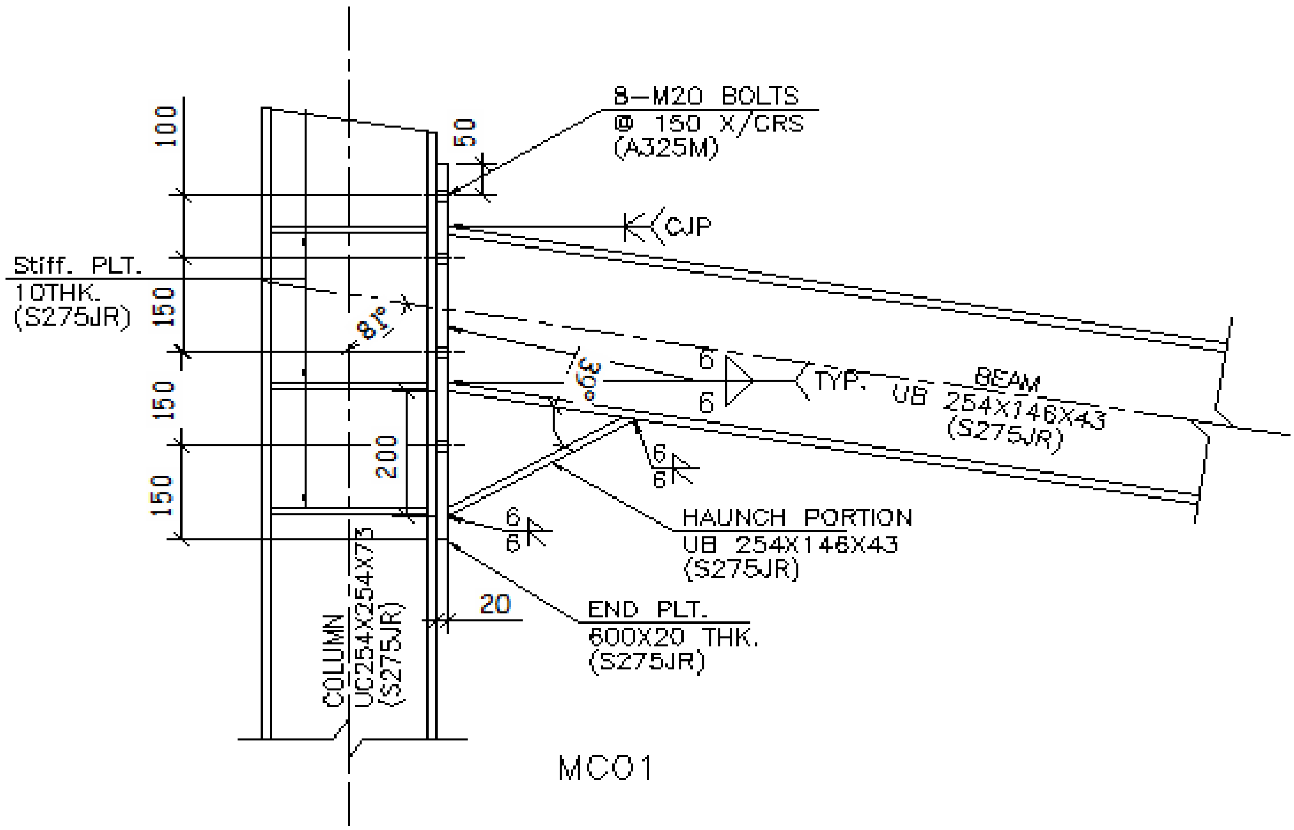
PROFILE SIZES:

BEAM :			**UB254X146X43**											
D_b =	259.60	mm	W_b =	147.30	mm	T_b =	12.70	mm	t_{wb} =	7.30	mm	r_b =	7.60	mm
COLUMN :			**UC254X254X73**											
D_C =	254.00	mm	W_c =	254.00	mm	T_C =	14.20	mm	t_{wC} =	8.60	mm	r_C =	12.70	mm

CONNECTION FORCES	**BEAM :**	**UB254X146X43**		**0.4M+0.9V**	**0.85M+0.55V**	**Worst Case**	
Axial Ten./Comp. Force,		F_x	=	**40**	**0**	**40**	kN
Major Axis Shear - Vertical,		F_y	=	**187.6129**	**177.19**	**187.6129**	kN
Minor Axis Shear - Horizontal,		F_z	=	**1**	**0**	**1**	kN
Minor Axis Moment,		M_y	=	**1**	**0**	**1**	kN-m
Major Axis Moment,		M_z	=	**41.0916**	**56.50095**	**41.0916**	kN-m

Full Tension Capacity = 0.6 X Fyp X As/1000	=	903.71	kN
Full Shear Capacity = 0.4 x Db x twb x Fyp / 1000	=	208.46	kN
Full Moment Capacity = 0.66 x Fyp x Zx / 10^6	=	102.73	kN-m

CONNECTION DETAIL:



MATERIAL GRADE:

STRUCTURAL STEEL 'W', 'WT'- SHAPES & BASE PLT. : **S275JR**

STRUCTURAL STEEL 'C', 'L' - SHAPES & CONNECTION PLT. : **S275JR**

STRUCTURAL BOLTS : **ASTM A325M**
(BEARING TYPE CONNECTIONS, BOLT THREADS INCLUDED IN SHEAR PLANE)

DESIGN PROCEDURE : **AISC -360-5 (ASD)**

						REMARKS
MATERIAL PROPERTIES:						
Design strength of grade S275JR material,	F_y	=	275	N/mm^2		
Minimum tensile strength of S275JR material,	F_{up}	=	410	N/mm^2		
Nominal shear stress of grade A325M Bolt,	F_{nv}	=	330	N/mm^2	Bearing type conn.,	Table (J3.2), pp 16.1-104
Nominal tensile stress of grade A325M Bolt,	F_{nt}	=	620	N/mm^2	Bolt threads included in shear plane.	
Allowable shear strength of weld,	F_w	=	144	N/mm^2	(Where, $Fw = 0.6 \times F_{EXX} / \Omega$)	Table (J2.5), pp 16.1-99
(For electrode grade E70xx)					(Ω =2.00 for ASD)	

BOLT PROPERTIES			Flange			
Grade of Bolt,	G_y	=	A325M			
Diameter of Bolt,	d	=	20.00	mm		
Diameter of Bolt Hole,	d_h	=	22.00	mm		
Number of Bolt Rows,	n_r	=	4			
Number of Bolt Columns,	n_c	=	2			
Total Number of Bolts,	n	=	8			
Shear Stress Area of Bolt,	A_b	=	314.2	mm^2	(Where, $A_b = \pi \times d^2 / 4$)	Eqn. (J3-1), pp 16.1-108
Allowable Shear Capacity of M20 Bolt,	R_{nv}	=	51.836	kN	(Where, $R_{nv} = F_{nv} \times A_b/\Omega$)	
Allowable Tension Capacity of M20 Bolt,	R_{nt}	=	97.389	kN	(Where, $R_{nt} = F_{nt} \times A_b/\Omega$)	(Ω =2.00 for ASD)

PLATE PROPERTIES								
Pitch Distance,	p	=	100.0	mm	150.0	mm	150.0	mm
Gauge Distance,	g	=	150.0	mm				
Plate End Distance,	e_v	=	50.0	mm				
Plate Edge Distance,	e_d	=	50.0	mm				
Plate Edge Distance (Bottom),	e_{db}	=	150.0	mm				
Haunch depth of beam,	D_{ha}	=	200.0	mm				
Width of end Plate,	W_p	=	200.0	mm				
Thickness of end Plate,	t_p	=	20.0	mm				
Thickness of stiffener plate,	t_s	=	10.0	mm				
Dist. from beam top flange to 1st bolt row below beam top flange,			P_{1b}	=	50.0	mm		
Dist. from haunch flange to bolt row just above haunch flange,			P_{2b}	=	113.1	mm		

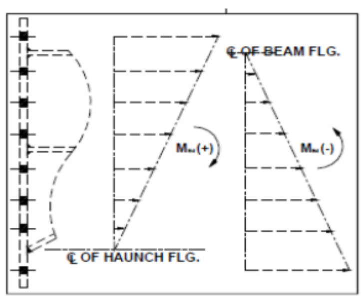
1. CHECK FOR BOLTS

Check bolt Tension using triangular force distribution

Taking moment about centre of haunch flange - clockwise moment & ,

Taking moment about centre of beam top flange - Anti-clockwise moment .

Bolt rows	Lever Arm	Clockwise		Anti-Clockwise	
1^{st}	L_{a1}	505.77	mm	-56.44	mm
2^{nd}	L_{a2}	405.77	mm	43.56	mm
3^{rd}	L_{a3}	255.77	mm	193.56	mm
4^{th}	L_{a4}	105.77	mm	343.56	mm
5^{th}	L_{a5}	0.00	mm	0.00	mm
6^{th}	L_{a6}	0.00	mm	0.00	mm
7^{th}	L_{a7}	0.00	mm	0.00	mm
8^{th}	L_{a8}	0.00	mm	0.00	mm
9^{th}	L_{a9}	0.00	mm	0.00	mm
10^{th}	L_{a10}	0.00	mm	0.00	mm



$L_{a.n}$ 343.6 mm

Section modulus of bolt group, (MZ - Major Axis Moment)

Clockwise moment,

Z_{bg} = n_c x ($L_{a1}^2 + L_{a2}^2 + L_{a3}^2 + L_{a4}^2 + L_{a5}^2 + L_{a6}^2 + L_{a7}^2 + L_{a8}^2 + L_{a9}^2 + L_{a10}^2$) / L_{a1}

= 2 x (505.77^2 + 405.77^2 + 255.77^2 +105.77^2 + 0^2 + 0^2 + 0^2 + 0^2 + 0^2 + 0^2) / 505.77

= 1965.55 mm x Unit Bolt Area

Anti-Clockwise moment,

Z_{bga} = n_c x ($L_{a1}^2 + L_{a2}^2 + L_{a3}^2 + L_{a4}^2 + L_{a5}^2 + L_{a6}^2 + L_{a7}^2 + L_{a8}^2 + L_{a9}^2 + L_{a10}^2$) / $L_{a.n.}$

= 2 x (-56.44^2 + 43.56^2 + 193.56^2 +343.56^2 + 0^2 + 0^2 + 0^2 + 0^2 + 0^2 + 0^2) / 343.56

= 934.81 mm x Unit Bolt Area

Tensile force on extreme bolt due to Mz

Clockwise Moment			Anti-Clockwise Moment		
F_{tm}	=	M_Z / Z_{bg}	F_{tma}	=	M_Z / Z_{bga}
	=	41.0916 x 1000 / 1965.55		=	41.0916 x 1000 / 934.81
	=	**20.9059** kN		=	**44.0** kN

Tensile force per bolt due to Beam Axial Tension Force (Fx),

F_{ta} = F_x / n

= 40 / 8

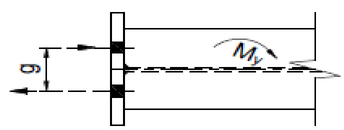
= 5 kN

Section modulus of bolt group, (My - Minor Axis Moment)

Z_{bgy} = n_r x g

= 4 x 150

= 600 mm



Tensile force per bolt due to My,

F_{tmy} = M_y / Z_{bgy}

= 1 / 600

= 0.0 kN

Maximum tensile force on bolts due to M_Z, M_y & F_x,

Bolt rows		**Clockwise Moment**	**Anti-Clockwise Moment**	**Maximum bolt tension**
1st	F_{t1}	= F_{tm} x (L_{a1} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a1} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(505.77 / 505.77) +0 + 5	= 43.96(-56.44 / 343.56) +0 + 5	
		= 25.9076 kN	= -2.219602	= 25.908 kN
2nd	F_{t2}	= F_{tm} x (L_{a2} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a2} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(405.77 / 505.77) +0 + 5	= 43.96(43.56 / 343.56) +0 + 5	
		= 21.7741 kN	= 10.57499	= 21.774 kN
3rd	F_{t3}	= F_{tm} x (L_{a3} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a3} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(255.77 / 505.77) +0 + 5	= 43.96(193.56 / 343.56) +0 + 5	
		= 15.5739 kN	= 29.76688	= 29.767 kN
4th	F_{t4}	= F_{tm} x (L_{a4} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a4} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(105.77 / 505.77) +0 + 5	= 43.96(343.56 / 343.56) +0 + 5	
		= 9.37365 kN	= 48.95877	= 48.959 kN
5th	F_{t5}	= F_{tm} x (L_{a5} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a5} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(0 / 505.77) +0 + 5	= 43.96(0 / 343.56) +0 + 5	
		= 5 kN	= 5	= 5 kN
6th	F_{t6}	= F_{tm} x (L_{a6} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a6} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(0 / 505.77) +0 + 5	= 43.96(0 / 343.56) +0 + 5	
		= 5 kN	= 5	= 5 kN
7th	F_{t7}	= F_{tm} x (L_{a7} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a7} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(0 / 505.77) +0 + 5	= 43.96(0 / 343.56) +0 + 5	
		= 5 kN	= 5	= 5 kN
8th	F_{t8}	= F_{tm} x (L_{a8} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a8} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(0 / 505.77) +0 + 5	= 43.96(0 / 343.56) +0 + 5	
		= 5 kN	= 5	= 5 kN
9th	F_{t9}	= F_{tm} x (L_{a9} / L_{a1}) + F_{tmy} + F_{ta}	= F_{tma} x ($L_{a9} / L_{a.n}$) + F_{tmy} + F_{ta}	
		= 20.91(0 / 505.77) +0 + 5	= 43.96(0 / 343.56) +0 + 5	
		= 5 kN	= 5	= 5 kN

10th	F_{t10}	=	F_{tm} x (L_{a10} / L_{a1}) + F_{tmy} + F_{ta}	=	F_{tma} x (L_{a10} / $L_{a.n}$) + F_{tmy} + F_{ta}		
		=	20.91(0 / 505.77) +0 + 5	=	43.96(0 / 343.56) +0 + 5		
		=	5 kN	=	5	=	5 kN

$F_{t.Max}$ = 48.96 kN

Allowable tension capacity per bolt, Rnt (97.39kN) > Ft.Max (48.96kN) : Therefore O.K.

Check bolt Shear

Vertical Shear per bolt, f_{vb} = F_y / n
= 187.61292 / 8
= 23.5 kN

Horizontal Shear per bolt, f_{vb} = F_z / n
= 1 / 8
= 0.1 kN

Resultant Shear per bolt, f_{rb} = Sqrt [$f_{vb}^2 + f_{hb}^2$]
= Sqrt [23.45^ 2 + 0.125^ 2]
= 23.5 kN

Allowable shear capacity per bolt, Rnv (51.84kN) > Frb (23.45kN) : Therefore O.K.

Check combined Tension & Shear

Actual Shear stress per bolt, f_v = f_{rb} / A_b
= 23.45 / 314.16
= 74.65 N/mm^2

Nominal tensile stress modified to include the effects of shearing stress,

F'_{nt} = Min[(1.3 x F_{nt}) - (Ω x F_{nt} x f_v / F_{nv}) , F_{nt}] — Eqn. (J3-3a), pp 16.1-109
= Min[(1.3 x 620) - (2 x 620 x 74.65 / 330),620]
= 525.49746 N/mm^2

Available tensile strength of bolt subjected to combined tension and shear,

R_n = F'_{nt} x A_b /Ω — Eqn. (J3-2), pp 16.1-109
= 525.5 x 314.16/2
= 82.544948

Available tensile strength per bolt, Rn (82.54kN) > Ft.Max (48.96kN) : Therefore O.K.

2. CHECK FOR END PLATE

Check Bearing capacity of end plate

Clear distance in the direction of forces,

L_c = Min[(p - d_h) , (e_v - d_h/2) , (e_d - d_h/2)]
= Min[(100 - 22) , (50 - 22/ 2), (50 - 22/ 2)
= Min[78,39,39]
= 39.00 mm

Available Bearing strength at bolt hole,

R_{nb} = Min[1.2 x L_c x t_p x F_{u36} , 2.4 x d x t_p x F_{u36}]/Ω — Eqn. (J3-6a), pp 16.1-111
= Min[1.2 x 39 x 20 x 410 , 2.4 x 20 x 20 x 410] / 2
= Min[383760 , 393600] / 2
= 191.88 kN

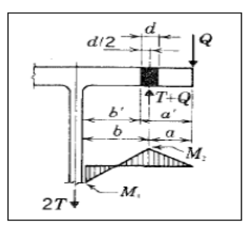
Available bearing strength at bolt hole, Rnb (191.88kN) > Frb (23.45kN) : Therefore O.K.

Check Bending capacity of end plate - to eliminate prying action

Refer AISC ASD Manual
Hanger Type Connection
Part 4, pp 4-89
Prying Action

Distance from bolt centre to the face of the T-stem,

b = 71.4 mm
a = 50.00 mm
b' = b - d / 2
= 71.4 - 20/ 2
= 61.4 mm

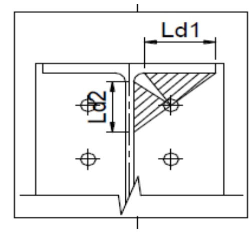


a' = Min[(a + (d / 2)) , ((1.25 x b) + (d / 2))]
= Min[(50 + (20 / 2)) , ((1.25 x 71.4) + (20/ 2))]
= Min[60 , 99.25]
= 60.00 mm

Tributary length of bolts (dispersion length of bolt tension force),

L_{d1} = Min{ [(W_b - t_{wb})/2 - r_b] , [2 x (p_{1b} - T_b) x tan60˚] ,
[(g - twb)/2 - rb + (p1b - Tb) x tan60˚] }
= Min{ [(147.3 - 7.3)/2 - 7.6] , [2 x (50 - 12.7) x tan60˚]
[(150 - 7.3)/2 - 7.6 + (50 - 12.7) x tan60˚] }
= Min{ 62.4 , 129.21 , 128.36 }
= 62.4 mm

L_{d2} = Min{ [(p /2) + (p_{1b} - T_b - r_b)] , [2 x (g - t_{wb})/2 x tan60˚] ,
[(g - t_{wb})/2 x tan60˚ + (p_{1b} - T_b - r_b)] }
= Min{ [(100 /2) + (50 - 12.7 - 7.6)] , [2 x (150 - 7.3)/2 x tan60˚] ,
[(150 - 7.3)/2 x tan60˚ + (50 - 12.7 - 7.6)] }
= Min{ 79.7 , 247.16 , 153.28 }
= 79.70 mm



L_d = L_{d1} + L_{d2}
= 62.4 + 79.7
= 142.10 mm

Required / Applied tensile strength per bolt,
T = $F_{t.max}$
= 48.96 kN (Max. tension force on top most bolts)

Available tension per bolt, B = Rn
= 82.545 KN

δ = 1 - d_h / L_d (Where, Tributary length of bolts, p = L_d)
= 1 - 22/ 142.1
= 0.85

ρ = b' / a'
= 61.4/ 60
= 1.023

β = (1 / ρ) x [(B / T) - 1]
= (1 /1.02) x [(82.54 /48.96) - 1]
= 0.6703675

α' = Min[1 , ((1 / δ) x (β / (1 - β)))]
= Min[1 ,((1 /0.85) x (0.67 / (1 - 0.67)))]
= Min[1 ,2.41]
= 1 not less than "0"
= 0

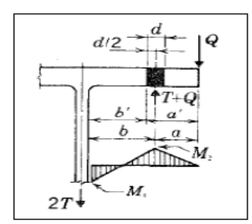
The endplate thickness required to eliminate prying action,
t_{min} = Sqrt[(4 x Ω x T x b') / (Ld x Fu36 x (1+ (δ x α')))]
= Sqrt[(4 x 1.67 x 48.96 x 61.4) / (142.1 x 410 x (1+ (0.85 x 0)))]
= 18.565 mm

Provided endplate thickness, tp (20mm) > tmin (18.57mm) : Therefore O.K.

Check Bending capacity of end plate - Prying Action

Distance from bolt centre to the face of the T-stem,
b = 71.4 mm
a = 50.00 mm
b' = b - d / 2
= 71.4 - 20/ 2
= 61.4 mm

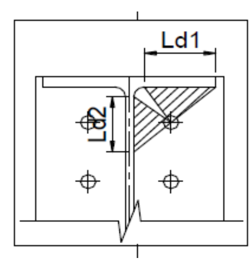
a' = Min[(a + (d / 2)) , ((1.25 x b) + (d / 2))]
= Min[(50 + (20 / 2)) , ((1.25 x 71.4) + (20/ 2))]
= Min[60 , 99.25]
= 60.00 mm



Tributary length of bolts (dispersion length of bolt tension force),
L_{d1} = 62.4 mm L_{d2} = 79.70 mm L_d = 142.10 mm

δ = 1 - d_h / L_d (Where, Tributary length of bolts, p = L_d)
= 1 - 20/ 142.1
= 0.86

ρ = b' / a'
= 61.4/ 60
= 1.023



Available tension per bolt, B = Rn
= 82.545 KN

Required / Applied tensile strength per bolt,
T = $F_{t.Max}$ (Max. tension force on bottom most bolts)
= 48.96 kN

β = $(1 / \rho) \times [(B / T) - 1]$
= $(1 /1.02) \times [(82.54 /48.96) - 1]$
= 0.6703675

α' = $Min[1 , ((1 / \delta) \times (\beta / (1 - \beta)))]$
= $Min[1 ,((1 /0.86) \times (0.67 / (1 - 0.67)))]$
= $Min[1 ,2.37]$
= 1 not less than "0"
= 0

The thickness required to ensure an acceptable combination of fitting strength, stiffeness and bolt strength,

t_{min} = $Sqrt[(4 \times 1.67 \times T \times b') / (L_d \times F_{u36} \times (1+ (\delta \times \alpha')))]$
= $Sqrt[(4 \times 1.67 \times 48.96 \times 61.4) / (142.1 \times 410 \times (1+ (0.86 \times 0)))]$
= 18.57 mm

<u>Provided endplate thickness, tp (20mm) > tmin (18.57mm) : Therefore O.K.</u>

The thickness required to develop the available strength of the bolt, B, with no prying action,

t_{cal} = $Sqrt[(4 \times 1.67 \times B \times b') / (L_d \times F_{u36} \times (1+ (\delta \times \alpha')))]$
= $Sqrt[(4 \times 1.67 \times 82.54 \times 61.4) / (142.1 \times 410 \times (1+ (0.86 \times 0)))]$
= 24.11 mm

α = $Max[(1 / \delta) \times [(T / B) \times (t_{cal} /t_p)^2 - 1] , 0]$
= Max[(1 / 0.86) x [(48.96 / 82.54) x (18.57 / 20)^2 - 1] , 0]
= Max[-0.57, 0]
= 0

q = $B \times [\delta \times \alpha \times \rho \times (t_p / t_{cal})^2]$ (When, q = 0 ; no prying)
= 82.54 x [0.86 x 0 x 1.02 x (20 / 24.11) ^2]
= 0 kN

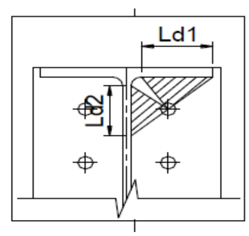
Total force per bolt including the effects of prying action,

T_{tot} = $T + q$
= 48.96 + 0 kN
= 48.96 kN

<u>Available tensile strength per bolt, Rn (82.54kN) > Ttot (48.96kN) : Therefore O.K.</u>

<u>3. CHECK FOR BEAM</u>

<u>Check for beam web tension</u>



L_{d1} = 62.4 mm L_{d2} = 79.70 mm L_d = 142.10 mm

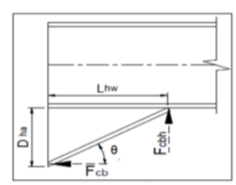
Force on beam web due to bolt tension,

F_{tw} = $2 \times T_{tot} \times (L_{d2} / L_d)$
= $2 \times 48.96 \times (79.7 / 142.1)$
= 54.92 kN

Beam web tension capacity, R_{nbw} = $L_{d2} \times t_{wb} \times F_{y50} /\Omega$ Eqn. (J4-1), pp 16.1-112
= $79.7 \times 7.3 \times 275/ 1.67$
= 95.81 kN

<u>Rnbw (95.81kN) > Ftw (54.92kN) : Therefore O.K.</u>

<u>Check beam/haunch flange in Compression</u>



Force acting on Beam flange due to Compression,
considering force due to Major axis moment(Mz) & Axial Compression (Fx)

F_{cb} = $[M_z / (D_b + D_{ha} -T_b))] + (F_x / 3)$
= $[41.0916 / (259.6 + 200 - 12.7)]+ (40 / 3)$
= 105.28 kN

Beam Flange Capacity in compression,
(when, $(k \times L / r) \le 25$) R_{nbf} = $T_b \times W_b \times F_{y50} / \Omega$
= $12.7 \times 147.3 \times 275 / 1.67$
= 308.05 kN

<u>Rnbf (308.05kN) > Fcb (105.28kN) : Therefore O.K.</u>

Check Beam web Panel shear capacity

Angle of Haunch flange to the end plate,

θ = 39 °

L_{hw} = D_{ha} / TAN(θ)

= 200 / TAN (39)

= 246.98 mm

Haunch compression force at beam bottom flange,

F_{cbh} = F_{cb} x sin(θ)

= 105.28 x Sin(39)

= 66.256 kN

Beam Web panel -Shear capacity , R_{nbwp} = 0.60 x F_{y50} x t_{wb} x D_b /Ω — Eqn. (J10-9), pp 16.1-119

= 0.6 x 275 x 7.3 x 259.6 / 1.67

= 187.24 kN

Fcbh (66.26kN) < Rnbwp (187.24kN) : Therefore O.K.

Check Beam web local yielding

Distance from outer face of the flange to the web toe of the fillet,

k = T_b + r_b

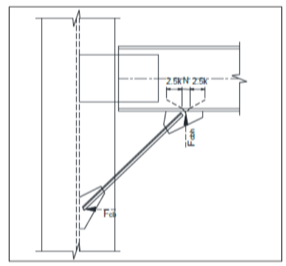
= 12.7 + 7.6

= 20.30 mm

N = (2 x Tb)

= (2 x 12.7)

= 25.40 mm



Beam web local yielding capacity , R_{nbwy} = (5 x k + N) x F_{y50} x t_{wb} / Ω — Eqn. (J10-2), pp 16.1-11

= (5 x 20.3 + 25.4) x 275 x 7.3 / 1.50

= 169.83 kN

Fcbh (66.26kN) < Rnbwy (1

Modulus of elasticity of steel,

E = 200000 N/mm^2 — Refer, pp 16.1-23

Check Beam web crippling

Beam web crippling capacity , R_{nbwc} = 0.8 x t_{wb}^2 x [1 + 3 x (N / D_b) x (t_{wb} / T_b)$^{1.5}$] x SQRT[E x F_{y50} x T_b / t_{wb}] / Ω — Eqn. (J10-4), pp 16.1-11

= (0.8 x 7.3^ 2 x [1 + 3 x (25.4 / 259.6) x (7.3 / 12.7)^1.5] x SQRT[200000 x 275 x 12.7 / 7.3]/ 2

= 235.77 kN

Fcbh (66.26kN) < Rnbwc (235.77kN) : Therefore O.K.

Check Beam web compression buckling

Clear distance between flanges less the fillet or root radius,

h = D_b - 2 x (T_b + r_b)

= 259.6- 2 x (12.7 + 7.6)

= 219 mm

Beam web compression buckling capacity ,

R_{nbwb} = 24 x t_{wbc}^3x SQRT[E x F_{y50}] / (h x Ω) — Eqn. (J10-8), pp 16.1-119

= 24 x 7.3^3 x SQRT [200000 x 275] / (219 x 1.67)

= 189.32 kN

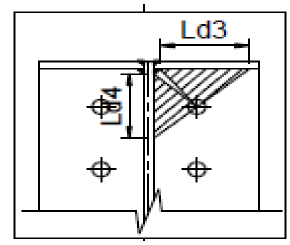
Rcbh (66.26kN) < Rnbwb(189.32kN) : Therefore O.K.

4. CHECK FOR COLUMN

Check for column web tension

Tributary length of bolts (dispersion length of bolt tension force),

L_{d3} = Min{ [(W_c - t_{wc})/2 - r_c] , [2 x (p_{1b} - t_s) x tan60˚] ,
[(g - t_{wc})/2 - r_c + (p_{1b} - t_s) x tan60˚] }
= Min{ [(254 - 8.6)/2 - 12.7] , [2 x (50 - 10) x tan60˚],
[(150 - 8.6)/2 - 12.7 + (50 - 10) x tan60˚] }
= Min{ 110 , 138.6 , 127.28 }
= 110.00 mm



L_{d4} = Min{ [(p /2) + (p_{1b} - t_s - r_c)] , [2 x (g - t_{wc})/2 x tan60˚] ,
[(g - t_{wc})/2 x tan60˚ + (p_{1b} - t_s - r_c)] }
= Min{ [(100 /2) + (50 - 10 - 12.7)] , [2 x (150 - 8.6)/2 x tan60˚] ,
[(150 - 8.6)/2 x tan60˚ + (50 - 10 - 12.7)] }
= Min{ 77.3,244.91,149.76]
= 77.30 mm

L_{d5} = L_{d3} + L_{d4}
= 110 + 77.3
= 187.30 mm

Force on col. web due to bolt tension,

F_{tcw} = 2 x T_{tot} x (L_{d4} / L_{d5}) Eqn. (J4-1), pp 16.1-112
= 2 x 48.96 x (77.3/187.3)
= 40.41 kN

Column web tension capacity, R_{ncw} = L_{d4} x t_{wc} x F_{y50} /Ω
= 77.3 x 8.6 x 275 / 1.67
= 109.46976 kN

Rncw (109.47kN) > Ftcw(40.41kN) : Therefore O.K.

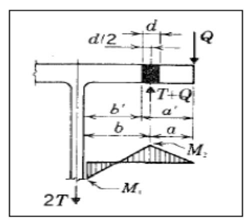
Check Bending capacity of column flange - Prying Action

Distance from bolt centre to the face of the T-stem

Refer AISC ASD Manual
Hanger Type Connection
Part 4, pp 4-89
Prying Action

b = (g - t_{wc}) / 2
= (150 - 8.6)/2
= 70.7 mm

a = (W_c - g) / 2
= (254 - 150)/2
= 52 mm



b' = b - d / 2
= 70.7 - 20/ 2
= 60.7 mm

a' = Min[(a + (d / 2)) , ((1.25 x b) + (d / 2))]
= Min[(52 + (20 / 2)) , ((1.25 x 70.7) + (20/ 2
= Min[62 , 98.375]
= 62.00 mm

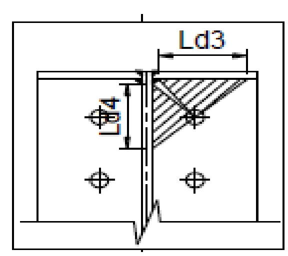
δ = 1 - d_h / L_{d5} (Where, Tributary length of bolts, p = L_{d5})
= 1 - 22/ 187.3
= 0.88

ρ = b' / a'
= 60.7/ 62
= 0.979

Available tension per bolt, B = Rn
= 82.545 kN

Required / Applied tensile strength per bolt,

T = $F_{t.Max}$ (Max. tension force on bottom most bolts)
= 48.96 kN



β = (1 / ρ) x [(B / T) - 1]
= (1 /0.98) x [(82.54 /48.96) - 1]
= 0.7007015

α' = Min[1 , ((1 / δ) x (β / (1 - β)))]
= Min[1 ,((1 /0.88) x (0.7 / (1 - 0.7)))]
= Min[1 ,2.65]
= 1 not less than "0"
= 0

Tributary length of bolts (dispersion length of bolt tension force),

L_{d3} = 110.00 mm L_{d4} = 77.30 mm L_{d5} = 187.30 mm

The thickness required to ensure an acceptable combination of fitting strength, stiffeness and bolt strength,

t_{min} = Sqrt[(4 x 1.67 x T x b') / (L_{d5} x F_{u36} x (1+ (δ x α')))]

= Sqrt[(4 x 1.67 x 48.96 x 60.7) / (187.3 x 410 x (1+ (0.88 x 0)))]

= 0.51 mm

Available thickness of column flange, Tc (14.2mm) > tmin (0.51mm) : Therefore O.K.

The thickness required to develop the available strength of the bolt, B, with no prying action,

t_{cal} = Sqrt[(4 x 1.67 x B x b') / (L_{d5} x F_{u36} x (1+ (δ x α')))]

= Sqrt[(4 x 1.67 x 82.54 x 60.7) / (187.3 x 410 x (1+ (0.88 x 0)))]

= 20.88 mm

α = Max[(1 / δ) x [(T / B) x (t_{cal} /t_p)^2 - 1] , 0]

= Max[(1 / 0.88) x [(48.96 / 82.54) x (20.88 / 20)^2 - 1] , 0]

= Max[-0.4, 0]

= 0

q = B x [δ x α x ρ x (t_p / t_{cal})^2] (When, q = 0 ; no prying)

= 82.54 x [0.88 x 0 x 0.98 x (20 / 20.88) ^2]

= 0 kN

Total force per bolt including the effects of prying action,

T_{tot} = T + q

= 48.96 + 0 kN

= 48.96 kN

Available tensile strength per bolt, Rn (82.54kN) > Ttot (48.96kN) : Therefore O.K.

Check Column flange local bending

Column flange local bending capacity ,

R_{ncf} = 6.25 x T_c^2 x F_{y50} /Ω

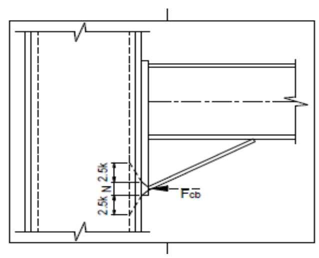
= 6.25 x 14.2^ 2 x275 /1.67

= 207.5 kN

Fcb (105.28kN) < Rncf (207.53kN) : Therefore O.K.

Check Column web local yielding

Distance from outer face of the flange to the web toe of the fillet,



k = T_c + r_c

= 14.2 + 12.7

= 26.90 mm

N = 2 x (t_p + T_c)

= 2 x (20 + 14.2)

= 68.4 mm

Beam web local yielding capacity , R_{ncwy} = (5 x k + N) x F_{y50} x t_{wc} / Ω Eqn. (J10-2), pp 16.1-116

= (5 x 26.9 + 68.4) x 275 x 8.6 / 1.50

= 319.9 kN

Fcb (105.28kN) < Rncwy (319.91kN) : Therefore O.K.

Check Column web crippling

Column web crippling capacity , R_{ncwc} = 0.8 x twc2 x [1 + 3 x (N / Dc) x (twc / Tc)1.5] x SQRT[E x Fy50 x Tc / twc] / Ω Eqn. (J10-4), pp 16.1-117

= (0.8 x 8.6^2 x [1 + 3 x (68.4 / 254) x (8.6 / 14.2)^1.5] x SQRT[200000 x 275 x 14.2 / 8.6] /2

= 389.27225 kN

Fcb (105.28kN) < Rncwc (389.27kN) : Therefore O.K.

Check Column web compression buckling

Clear distance between flanges less the fillet or root radius,

h = D_b - 2 x (T_b + r_b)

= 254 - 2 x (14.2 + 12.7)

= 200.2 mm

Col. web comp. buckling capacity ,

R_{ncwb} = 24 x t_{wc}^3x SQRT[E x F_{y50}] / (h x Ω) Eqn. (J10-8), pp 16.1-119

= 24 x 8.6^3 x SQRT [200000 x 275] / (200.2 x 1.67)

= 338.62 kN

Fcb (105.28kN) < Rncwb(338.62kN) : Therefore O.K.

Check Column web Panel shear capacity

Column Web panel -Shear capacity ,

R_{ncwp} = 0.6 x F_{y50} x t_{wc} x D_c /Ω Eqn. (J10-9), pp 16.1-119

= 0.6 x 275 x 8.6 x 254 / 1.67

= 215.8 kN

Fcb (105.28kN) < Rncwp(215.82kN) : Therefore O.K.

5. CHECK FOR WELD

Check for weld - (At beam top flange to end plate)

Compression/Tension per mm on weld due to axial,

F_{ct} = $[F_x / 3] / [(2 \times W_b - (t_{wb} + 2 \times r_b))]$

= [40/ 3] / [(2 x 147.3-(7.3 + 2 x 7.6))]

= 49.00 N/mm

(Axial comp./tension force, F_x distributed to each flange)

Tension per mm on weld due to M_z,

F_{mz} = $[M_z / (D_b + D_{ha} - T_b)] / [(2 \times W_b - (t_{wb} + 2 \times r_b))]$

= [41.0916 / (259.6 + 200 - 12.7)]/[(2 x 147.3 - (7.3 + 2 x 7.6))]

= 337.92 N/mm

Calculate Section modulus of weld,

w_e = $(W_b - t_{wb}) / 2$

= (147.3-7.3) / 2

= 70.0 mm

Section Modulus per mm on weld,

Z_w = $4 \times \{ [W_e^3 / 12] + W_e \times [(W_e / 2) + (t_{wb} / 2)]^2 \} / (W_b / 2)$

= 4 x {[70^3 / 12] +70 x [(70/2) + (7.3/2)]^2}/(147.3/ 2)

= 7231.5497 mm^2

Comp./Ten. per mm on weld due to My,

(Minor axis moment, M_y distributed to each flange)

F_{my} = $[M_y / 3] / Z_w$

= [1 / 3] / 7231.55

= 5E-05 N/mm

(Horizontal shear force, F_z distributed to each flange)

Horizontal shear per mm on weld,

F_{hw} = $(F_z / 3) / [(2 \times W_b - (t_{wb} + 2 \times r_b))]$

= (1 / 3) / [(2 x147.3- (7.3 + 2 x7.6))]

= 1.23 N/mm

Resultant force per mm on weld,

F_{wR} = $Sqrt[(F_{my} + F_{hw})^2 + (F_{ct} + F_{mz})^2]$

= Sqrt[(0 + 1.23)^2 + (49 + 337.92)^2]

= 386.92 kN/mm

Size of weld required, $\mathbf{S_{reqd}}$ = $F_{wR} / (0.707 \times F_w)$

= 386.92 / (0.707 x 144)

= 3.80 mm

Therefore, Provide full strength butt weld (CJP).

Check for weld - (At beam web to end plate)

Effective dispersion length of bolt,

L_{d1} = 62.4 mm L_{d2} = 79.70 mm L_d = 142.10 mm

Vertical shear per mm on weld, F_{vw} = $F_y / [2 \times (D_b + D_{ha} - 3 \times (T_b + r_b))]$

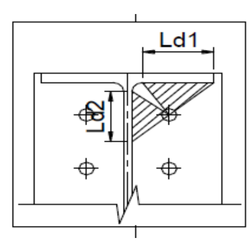
= 187.61292 / [2 x (259.6 + 200 - 3 x (12.7 + 7.6))]

= 235.281 N/mm

Tension per mm on weld, F_{vt} = $F_{t.Max} / Ld$

= 48.96 / 142.1

= 344.53744 N/mm



Resultant force per mm on weld,

F_{wR1} = $Sqrt[F_{vw}^2 + F_{wt}^2]$

= Sqrt[235.28^2 + 344.54^2]

= 417.21 N/mm

Size of weld required, S_{reqd} = $F_{wR1} / (0.707 \times F_w)$

= 417.21 / (0.707 x 144)

= 4.10 mm

Therefore, Provide 6mm CFW.

Check for weld - (At beam bottom flange to end plate)

Resultant force per mm on weld due to My, Fx & Fz,

F_{wR2} = Sqrt[$F_{hw}{}^2 + (F_{ct} + F_{my})^2$]
= Sqrt[1.23^2 + (49 + 0)^2]
= 49.02 N/mm

Size of weld required, S_{reqd} = F_{wR1} / (0.707 x F_w)
= 49.02 / (0.707 x 144)
= 0.48 mm

Therefore, Provide 6mm CFW.

Check for weld - (At beam haunch flange to end plate)

Compression per mm on weld due to M_z & F_x,

F_{MzFx} = F_{cb} / W_b
= 105.28 / 147.3
= 714.74 N/mm

Section Modulus per mm on weld,

Z_{w1} = $W_b{}^2$ / 6
= 147.3^2 / 6
= 3616.215 mm^2 per mm

Compression/Tension per mm on weld due to M_y,

F_{my1} = [M_y / 3] / Z_{w1} (Minor axis moment, M_y distributed to each flange)
= [1 / 3] / 3616.215
= 92.17741 N/mm

Horizontal Shear per mm on weld due to M_y & F_z,

F_{hw1} = (F_z / 3) / W_b (Horizontal shear force, F_z distributed to each flange)
= (1/ 3)/147.3
= 2.2629554 N/mm

Resultant force per mm on weld,

F_{wR3} = Sqrt[$(F_{MzFx})^2 + (F_{my1} + F_{hw1})^2$]
= Sqrt[(714.74^ 2 + (92.18 + 2.26)^2]
= 720.95 N/mm

Size of weld required, S_{reqd} = 2 x F_{wR3} / (F_{yb})
= 2 x 720.95 / 275
= 5.24 mm

Therefore, Provide full strength butt weld (CJP).

6. CHECK FOR STIFFENERS (Applicable only when column flange & web fails)

Check for Tension Stiffeners - (At beam top flange)

Effective dispersion length of bolt,

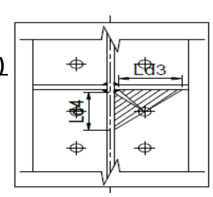
L_{d3} = 110.00 mm L_{d4} = 77.30 mm L_{d5} = 187.30 mm

Force on column web stiffeners due to bolt tension,

F_{tcws} = 2 x T_{totc} x (L_{d3} / L_{d5})
= 2 x 48.96 x (110 / 187.3)
= 57.51 kN

Stiffener tension capacity, R_{nst} = L_{d3} x t_s x F_{y36} /Ω Eqn. (J4-1), pp 16.1-112
= 110 x 10 x 275 /1.67
= 181.1 kN

Rnst (181.14kN) > Ftcws (57.51kN)



Check for Tension Stiffeners weld - (At beam top flange)

Tension per mm on weld, F_{wt1} = T_{totc} / L_{d5}
= 48.96 / 187.3
= 261.39226 N/mm

Size of weld required, S_{reqd} = F_{wt1} / (0.707 x F_w)
= 261.39 / (0.707 x 144)
= 2.57 mm

Therefore, Provide 6mm CFW.

Check for Compression Stiffeners - (In column at haunch flange)

Buckling length of the stiffener plate,

L = 225.6 mm where, $L = D_c - 2 \times Tc$

Governing radius of Gyration,

r_{min} = Sqrt [t_s^2 / 12] r_{min} = Sqrt [I_{min} / A]

= Sqrt [10^2/ 12] = Sqrt [($W_s \times t_s^3$ / 12) / ($W_s \times t_s$)]

= 2.89 mm = Sqrt [t_s^2 / 12]

Effective length factor, K = 1.00 Table (C-C2.2), pp 16.1-240

Modulus of elasticity of steel, Refer, pp 16.1-23

E = 200000 N/mm^2 Clause (E3-4), pp 16.1-33

Elastic critical buckling stress,

F_e = (π^2 x E) / (K x L / r_{min})2

= (3.142^2 x 200000) / (1 x 225.6 / 2.89)^2

= 323.2 N/mm^2

When, Fe ≥ 0.44 x Fy

323.2 ≥ 0.44 x 275

323.2 ≥ 121

Flexural buckling stress, F_{cr} = 0.658^(F_y / F_e)] x F_y Clause (E3-2), pp 16.1-33

= (0.658^(275 / 323.2)] x 275

= 192.60469

Width of one stiffener plate, W_s = ($W_c - t_{wc}$) / 2 - r_c

= (254 - 8.6) / 2 - 12.7

= 110 mm

Gross area of C/S, A_s = ($W_s \times t_s$)

= (110 x 10)

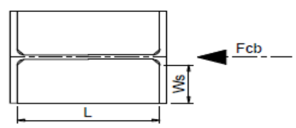
= 1100 mm^2

Allowable compressive strength of stiffeners (considering both sides),

P_n = 2 x F_{cr} x A_s / Ω Clause (E3-1), pp 16.1-33

= 2 x 192.6 x 1100 / 1.67

= 240.75586 kN



Max. Force on haunch flange comp., : Column flange local yieding

F_{cb} = 105.28 kN . = Column web local yieding

Min. capacity of column at haunch, R_{ncwc} = Column web crippling

R_{nmin} = Min[R_{ncf} , R_{ncwy} , R_{ncwc} , R_{ncwb} , R_{ncwp}] R_{ncwb} = Column web buckling

= Min[207.53 , 319.91 , 389.27 , 338.62 , 215.82] R_{ncwp} = Column web panel shear

= 207.5 kN

Force taken by compression stiffeners,

F_{cs} = $F_{cb} - R_{nmin}$

= 105.28 - 207.53

= -102.24 kN

Pn (240.76kN) > Fcs (-102.24kN) : Therefore O.K.

Check for compression Stiffeners weld - (in Column at haunch flange)

Compression per mm on weld, F_{wc} = F_{cs} / (4 x W_s)

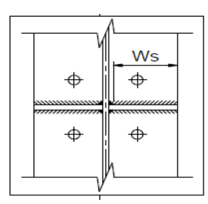
= -102.24 / (4 x 110)

= -0.232 kN/mm

Size of weld required, S_{reqd} = F_{wc} / (0.707 x F_w)

= -0.23 / (0.707 x 144)

= 0.00 mm



Therefore, Provide 6mm CFW.