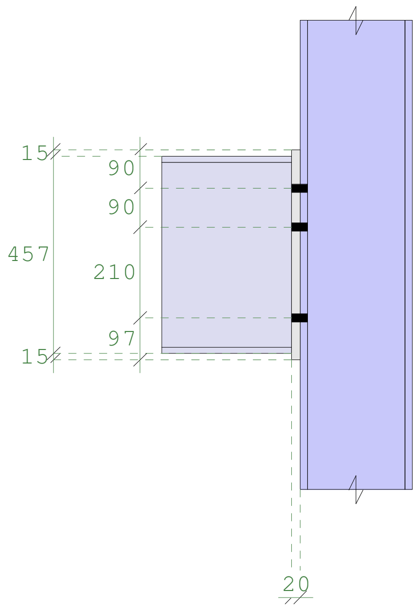
3 D Steel Ltd
Draughting & Design Service
Windsor
Ovens
Co Cork
Ireland
Tel: 021 4826464

Job No:	Sheet No:	Rev:
08265	1	
Beam to Column Moment Connection		
Made By:	Date:	Chd:
G. O'M.	23/May/08	

Job Title:

Conn Design



Beam: 457X191X74UB S275JR
End plate: S275JR t=20 B=200
Plate weld to: beam top flg: FW. leg=6
beam web : FW. leg=6 beam bot flg : FW. leg=6
Bolts: M20 8.8 Rows = 3 c/crs = 100.0

Column: 254X254X89UC S275JR

DESIGN TO BS5950-1:2000 and SCI's GREEN BOOK
File: scon2.dat 23/May/08 11:18:00

Loadcase 1: loadcase1

Items	Unity Ratios Left Side	Unity Ratios Right Side
Moment Capacity	0.65 Pass	
Bolts in Shear	0.91 Pass	
Weld: Beam Top Flange	0.44 Pass	
Weld: Beam Web	0.36 Pass	
Weld: Beam Bottom Flange	Direct Bearing.	

DATA:

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Column      : 254X254X89UC  S275JR  Top = 2500.0 mm Base = -2500.0 mm
Left beam   : 457X191X74UB  S275JR  TOS = 0.0 mm  Angle = 0.0 degrees
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Left plate : S275JR B = 200.0 mm t = 20.0 mm
Height above = 15 mm Height below = 15 mm

Left bolt rows:
Grade = 8.8 Size = M20 End distance: Top = 90 mm Bottom = 97 mm g = 100 mm
Number = 3 Spacings: 90 mm 210 mm
For shear only: 1 row(s) under +ve moment and 1 row(s) under -ve moment

Left plate welds to: Beam top flange...... Fillet. leg = 6 mm
Beam web............. Fillet. leg = 6 mm
Beam bottom flange... Fillet. leg = 6 mm

Applied loads for all load cases:

		Left beam	
case	Axial(kN)	Shear(kN)	Moment(kNm)
1	150.0	300.0	100.0

CONNECTION GEOMETRY: Left Beam

Column side: (g = 100 mm t_c = 10.3 mm r_c = 12.7 mm B_c = 256.3 mm)
m = 34.7 mm e = 78.2 mm n = 43.4 mm

Beam side: (t_b = 9.0 mm b_p = 200.0 mm S_{ww} = 6 mm)
m = 40.7 mm e = 50.0 mm n = 50.0 mm

End plate:
e = 50.0 mm

Top end plate:
Bolt end distance = 90 mm Plate above beam = 15 mm Beam flange = 14.5 mm

Bottom end plate:
Bolt end distance = 97 mm Plate below beam = 15 mm Beam flange = 14.5 mm

Plastic distribution limit:
Bolt: Grade = 8.8 Dia = 20 mm U_f = 785.0 N/mm^2
Design strengths: P_{yp} (end plate) = 0.0 N/mm^2 P_{yc} (column) = 265.0 N/mm^2

Eq 2.5: t_p = 20.0 mm Limit = 18.1 mm (limit exceeded)
Eq 2.6: T_c = 17.3 mm Limit = 18.1 mm

Triangular distribution limit line need not be imposed

STEP 1. RESISTANCE OF BOLTS IN TENSION ZONE [Left]

Bolt Row 1 (row 1 alone) [Left]

Column flange bending:
(Bolt row 1 next to free end - 6.)
(2575 mm below top of column & 90 mm above a bolt)

Table 2.5: Min{v, ii, i}
(e = 78.2 mm e_x = 2575.0 mm m = 34.7 mm)

pattern(i) = 218.0 mm
pattern(ii) = 236.4 mm
pattern(v) = 2693.2 mm
L_{eff} = 218.0 mm (T_c = 17.3 mm P_{yc} (column) = 265.0 N/mm^2)

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$M_p$ for column flange = 4322 kNmm
(n = 43.4 mm  capacity of one bolt ($p_t$') = 137.2 kN)

Critical failure modes:
  Mode 1: 498.3 kN  Mode 2: 263.2 kN  Mode 3: 274.4 kN
  Potential resistance ($c_{fb}$ alone).................. $P_r$ = 263 kN
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Column web tension:
  (L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
  L_t - above bolt. = 86.6 mm
  L_t - below bolt. = 86.6 mm
  L_t (total) = 173.2 mm
  Potential resistance (c_{wt} alone).................... P_t = 473 kN

End plate bending:
  (Bolt row 1 below beam flange of flush end plate. -3)
  (68 mm below beam top flange & 90 mm above bolt)

  Table 2.5: Min{ Max{(ii+iii)/2}, ii}, i}
  (e = 50.0 mm m & m_1 = 40.7 mm m_2 = 55.7 mm λ = 0.45 λ_2 = 0.61 α = 5.88)
  (g = 100.0 mm B_b = 190.4 mm T_b = 14.5 mm t_p = 20.0 mm)

  pattern(i) = 255.7 mm
  pattern(ii) = 225.3 mm
  pattern(iii) = 239.5 mm
  L_{eff} = 232.4 mm (t_p = 20.0 mm P_{yp} = 265.0 N/mm^2)

  M_p for end plate = 6158 kNmm
  (n = 50.0 mm capacity of one bolt (p_t') = 137.2 kN)

  Critical failure modes:
    Mode 1: 605.3 kN Mode 2: 287.1 kN Mode 3: 274.4 kN
    Potential resistance (e_{pb} alone).................. P_r = 274 kN

Beam web tension:
  (L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
  L_t above bolt row = 86.6 mm -- beam top flange within L_t (at 67.8 mm from bolt)
  N/A

Potential resistance of bolt row 1 .... = 263.2 kN

Bolt Row 2 (row 2 alone) [Left]

Column flange bending:
  (Bolt row 2 not influenced by a stiffener or a free end - 1.)
  (90 mm below a bolt & 210 mm above a bolt)

  Table 2.5: Min{ii, i}
  (e = 78.2 mm m = 34.7 mm)

  pattern(i) = 218.0 mm
  pattern(ii) = 236.4 mm
  L_{eff} = 218.0 mm (T_c = 17.3 mm P_{yc} (column) = 265.0 N/mm^2)

  M_p for column flange = 4322 kNmm
  (n = 43.4 mm capacity of one bolt (p_t') = 137.2 kN)

  Critical failure modes:
    Mode 1: 498.3 kN Mode 2: 263.2 kN Mode 3: 274.4 kN
    Potential resistance (c_{fb} alone).................. P_r = 263 kN

Column web tension:
  (L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
  L_t - above bolt. = 86.6 mm

L_t - below bolt. = 86.6 mm
L_t (total) = 173.2 mm
Potential resistance (c_{wt} alone).................... P_t = 473 kN

End plate bending:
(Bolt row 2 not influenced by a stiffener or a free end. - 1)
(90 mm below bolt & 210 mm above bolt)

Table 2.5: Min{ii, i}
(e = 50.0 mm m = 40.7 mm)

pattern(i) = 255.7 mm
pattern(ii) = 225.3 mm
L_{eff} = 225.3 mm (t_p = 20.0 mm P_{yp} = 265.0 N/mm^2)

M_p for end plate = 5970 kNmm
(n = 50.0 mm capacity of one bolt (p_t') = 137.2 kN)

Critical failure modes:
Mode 1: 586.8 kN Mode 2: 282.9 kN Mode 3: 274.4 kN
Potential resistance (e_{pb} alone).................. P_r = 274 kN

Beam web tension:
(L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
L_t - above bolt. = 86.6 mm
L_t - below bolt. = 86.6 mm
L_t (total) = 173.2 mm
Potential resistance (b_{wt} alone).................... P_t = 429 kN

Bolt Rows 2 + 1 (rows combined) [Left]

Column flange bending:

(Bolt row 2 bottom of a group along a clear length - 1)
(90 mm below a bolt & 210 mm above a bolt)
Table 2.6: {ii/2 + p/2}
(e = 78.2 mm m = 34.7 mm)

pattern(ii) = 236.4 mm
p = 90.0 mm
L_{eff} of row 2 = 163.2 mm

(Bolt row 1 top of a group next to a free edge - 5)
(2575 mm below top of column & 90 mm above a bolt)
Table 2.6: Min{ex, ii/2} + p/2
(e = 78.2 mm m = 34.7 mm e_x = 2575.0 mm)

pattern(ii) = 236.4 mm
p = 90.0 mm
L_{eff} of row 1 = 163.2 mm
L_{eff}(total) = 326.4 mm (T_c = 17.3 mm P_{yc} (column) = 265.0 N/mm^2)

M_p for column flange = 6473 kNmm
(n = 43.4 mm capacity of one bolt (p_t') = 137.2 kN)

Critical failure modes:
Mode 1: 746.4 kN Mode 2: 470.7 kN Mode 3: 548.8 kN
Potential resistance (c_{fb} combined)............... P_r = 471 kN
Potential resistance (c_{fb} alone).................. P_r = 208 kN

Column web tension:
(L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
L_t - above top bolt. = 86.6 mm
L_t - within bolts. = 90.0 mm
L_t - below bottom bolt. = 86.6 mm

L_t (total) = 263.2 mm
Potential resistance (b_{wt} combined)................. P_t = 718 kN
Potential resistance (c_{wt} alone).................... P_t = 455 kN

End plate bending:

(Bolt row 2 bottom of a group along a clear length. - 1)
(90 mm below bolt & 210 mm above bolt)
Table 2.6: {ii/2 + p/2}
(e = 50.0 mm m = 40.7 mm)

pattern(ii) = 225.3 mm
p = 90.0 mm
L_{eff} of row 2 = 157.7 mm

(Bolt row 1 top below the beam flange of flush end plate.- 3)
(68 mm below beam top flange & 90 mm above bolt)
(g = 100.0 mm B_b = 190.4 mm T_b = 14.5 mm t_p = 20.0 mm)
Table 2.6: Max{ii/2, iii/2) + p/2
(e = 50.0 mm m = 40.7 mm)
(e = 50.0 mm m = 40.7 mm m_2 = 55.7 mm λ_1 = 0.45 λ_2 = 0.61 α = 5.88)

pattern(ii) = 225.3 mm
pattern(iii) = 239.5 mm
p = 90.0 mm
L_{eff} of row 1 = 164.7 mm
L_{eff}(total) = 322.4 mm (t_p = 20.0 mm P_{yp} = 265.0 N/mm^2)

M_p for end plate = 8543 kNmm
(n = 50.0 mm capacity of one bolt (p_t') = 137.2 kN)

Critical failure modes:
Mode 1: 839.7 kN Mode 2: 490.9 kN Mode 3: 548.8 kN
Potential resistance (e_{pb} combined).............. P_r = 491 kN
Potential resistance (e_{pb} alone).................. P_r = 228 kN

Beam web tension:
(L_t = Tensile length of web assuming 1:1.732 load spread from the bolt)
L_t above bolt row = 86.6 mm -- beam top flange within L_t (at 67.8 mm from bolt)
N/A

Potential resistance of bolt row 2 .... = 207.6 kN

Bolt Row Capacity Summary: [Left]

	c_{fb} (kN)	c_{wt} (kN)	e_{pb} (kN)	b_{wt} (kN)	bolt-rows		P_r (kN)
1	263.2	472.7	274.4	0.0	1		
						row 1	263.2
2	263.2	472.7	274.4	428.7	2		
3	207.6	455.2	227.7	0.0	2+1		
						row 2	207.6

STEP 2. RESISTANCE OF COLUMN WEB IN COMPRESSION ZONE (to BS5950)[Left]

Full depth compression stiffener:

No active compression stiffener to column web

Bearing of unstiffened web(cl 4.5.2.1)
t(beam flange) = 14.5 mm s(beam flange weld) = 6 mm
t_c(column web) = 10.3 mm T(plate) = 20 mm
b_1 = 54.3 mm b_e = 2028.0 mm n = 5.0

Rolled section: T = 17.3 mm root r. = 12.7 mm k = 30.0 mm
P_{yc} (column) = 265.0 N/mm^2 p_y (stiffener) = 0.0 N/mm^2 p_y = 265.0 N/mm^2
P_{bw} (bearing capacity) .... = 557.6 kN

Buckling of unstiffened column web(cl 4.5.3.1)
d (effective depth of web) = 200.3 mm t (web) = 10.3 mm
P_{yc} (column) = 265 N/mm^2 eps. $\{(275/p_y)^{1/2}\}$ = 1.02
a_e (reaction pt.to member end) = 2050.3 mm .7d = 140.2 mm
P_x (buckling capacity) .... = 723.1 kN

Beam flange crushing (bearing)
T_b = 14.5 mm B_b = 190.4 mm p_y = 275.0 N/mm^2
P_c (beam flange crushing) .... = 1062.9 kN

Resistance in the compression zone .... = 557.6 kN

STEP 3. PANEL SHEAR [Left]

Column Web
t_c = 10.3 mm D_c = 260.3 mm P_{yc} (column) = 265.0 N/mm^2
Shear resistance of column web: .... = 426.3 kN

Total shear resistance (including stiffeners)............ = 426.3 kN

STEP 4. MOMENT CAPACITY [Left]
(Compression in beam flange only)

Total bolt tension force .....(P_{ri}) = 470.7 kN
Applied axial force ....(N) = 150.0 kN (compression)
Bolt force + axial ....(P_{ri} + N) = 620.7 kN

Total compression zone force (P_c) = 557.6 kN
web panel shear capacity ......(P_v) = 426.3 kN

F_c = Min (620.7 kN, 426.3 kN) = 426.3 kN
****Shear critical for column****

Bolt force to be reduced by (620.7 kN - 426.3 kN) = 194.5 kN

Bolt row	Original P_r(kN)	Modified P_r(kN)	Distance(mm)
1	263.2	263.2	374.8
2	207.6	13.1	284.8

Moment capacity M_c .... = 102.4 kNm

Applied moment .... (M) = 100.0 kNm
Applied axial force .... (N) = 150.0 kN
hN .... = 221.3 mm
Modified moment M_m .... = 66.8 kNm
(Unity factor = 0.653)

STEP 5. VERTICAL SHEAR FORCE [Left]

Strength N/mm^2:
Bolt shear: p_s = 375.0 [BS_Tab30 grd = 8.8]
Bolt bearing: p_{bb} = 1000.0 [BS_Tab31 grd = 8.8]
Column bearing: p_{bc} = 460.0 [BS_Tab32 grd = S275]
Plate bearing: p_{bp} = 460.0 [BS_Tab32 grd = S275]

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    Bolt diameter = M20  Area = 245  mm²
    Column flange = 17.3 mm
    Plate thickness = 20.0 mm

  Capacity:
    Bolt shear (ps*As)                          =  91.9 kN
    Bolt bearing on end plate (d*tp*Pbp)        = 184.0 kN
    Bolt bearing on column flange (d*Tf*Pbc)    = 159.2 kN
    Pss (Shear capacity of a single bolt in shear zone) ... =  91.9 kN
    Pts (Shear capacity of a single bolt in tension zone) . =  36.8 kN

  Capacity of Top Bolt:
    Plate top end distance                     =  90.0 mm
    Column top end distance                    = 2575.0 mm
    Bolt shear (ps*As)                         =  91.9 kN
    Bolt bearing on end plate                  =   0.0 kN
    Bolt bearing on column flange              =   0.0 kN
    Pss (Shear capacity of top bolt if in shear zone) ..... =  91.9 kN
    Pts (Shear capacity of top bolt if in tension zone) ... =  36.8 kN

  Capacity of bottom bolt:
    Plate bottom end distance                  =  97.0 mm
    Column bottom end distance                 = 2125.0 mm
    Bolt shear (ps*As)                         =  91.9 kN
    Bolt bearing on end plate                  =   0.0 kN
    Bolt bearing on column flange              =   0.0 kN
    Pss (Shear capacity of top bolt if in shear zone) ..... =  91.9 kN
    Pts (Shear capacity of top bolt if in tension zone) ... =  36.8 kN

  Total number of bolts: in shear zone = 2 (at the bottom+)
                         in tension zone = 4

  Shear capacity Pv ...................................... = 330.8 kN
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STEP 7. DESIGN OF WELDS: [Left]

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  Design strength of welds:
    Grades: Beam/Haunch = S275  Plate = S275
    Electrode = E42
    pw - fillet weld (Table 37)........................... = 220 N/mm²

  Capacity of joint:
    Moment capacity Mc = 102.4 kNm
    Applied moment  M  = 100.0 kNm
    Modified moment Mm =  66.8 kNm
    Unity factor (moment) ................................. = 0.653

  Beam top flange to plate:
    Beam: B = 190.4 mm  T = 14.5 mm  Grade = S275  py = 0.0 kN
    End plate: Grade = S275
    Tension capacity of beam flange = 759.2 kN
    Tension capacity of top 2 bolt row(s) = 276.3 kN
    Applied tension load in top 2 bolt row(s) = 180.3 kN
    Design force to tension flange welds ................. = 180.3  kN

    B(beam) = 190.4 mm  B(plate) = 200.0 mm  B = 190.4 mm
    Max weld length on each side = 178.4 mm
    s = 6 mm
    Angle between plate and beam flange = 90.0 degrees
    Above flg: angle between Ft and throat = 45.0 degrees  K = 1.25  a = 4.2 mm
    Below flg: angle between Ft and throat = 45.0 degrees  K = 1.25  a = 4.2 mm
    Max weld capacity = 412.1 kN  (required = 180.3 kN)
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Unity factor... = 0.44

Beam web to plate:
m_1 = 40.7 mm g = 100.0 mm 1.732g/2 = 86.6 mm
No of tension bolts = 2 No of shear bolts = 1 +ve moment
Beam: T_{Flg} = 14.5 mm R_{Rad} = 10.2 mm
Beam welds: Top flange: Fillet Leg = 6 mm Bottom flange: Fillet Leg = 6 mm

Row	Above the row	m_{2A}(mm)	m_{3A}(mm)	topL(mm)	Below the row	m_{2B}(mm)	m_{3B}(mm)	botL(mm)
1	flg. at 60.5 mm	55.7	50.3	50.3	bolt at 90.0 mm	45.0	.....	45.0
2	bolt at 90.0 mm	45.0	.....	45.0	bolt at 210.0 mm	86.6	.....	86.6
3	bolt in shear-							

Row	Capacity	Load	Ld ratio	Ld length	Transverse load on web	
1	263.2 kN	171.8 kN	0.578	95.3 mm	99.3 kN	0.52 kN/mm
2	13.1 kN	8.6 kN	1.000	131.6 mm	8.6 kN	0.03 kN/mm

F_t - Transverse stress = 0.52 kN/mm

Shear Load = 300.0 kN
Overall depth = 457.0 mm
Beam flange = 14.5 mm
Beam root radius = 10.2 mm
L_{ws} - Shear zone length = 407.6 mm
F_l - Longitudinal stress = 0.37 kN/mm

a (web) = 4.2 mm
coefficient K = 1.25

P_l - Longitudinal capacity = 0.92 kN/mm
P_t - Transverse capacity = 1.15 kN/mm

$[(F_l/P_l)^2 + (F_t/P_t)^2]$ .... = 0.36