

20 SHIRE COURT,
QUAKERS YARD,
TREHARRIS.

STRUCTURAL CALCULATIONS.

PROPOSED EXTENSION AND ALTERATIONS :

**STEEL BEAMS AND BEAM WITH WELDED
PLATE DESIGN.**

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A PORCH EXTENSION AND INTERNAL AND EXTERNAL ALTERATIONS ARE PROPOSED AT THE ABOVE PROPERTY.

A DESIGN IS REQUIRED FOR A NUMBER OF STEEL BEAMS OVER THE LOUNGE / KITCHEN / DINING TO PROVIDE AN OPEN PLAN LAYOUT ON THE GROUND FLOOR AND TO SUPPORT THE FLOOR CONSTRUCTION WITH PARTITIONS ABOVE.

A DESIGN IS ALSO PROVIDED FOR A STEEL BEAM WITH WELDED PLATE OVER AN APPROX. 4.3m WIDE EXTERNAL OPENING IN THE ASSUMED 300mm WIDE CAVITY WALL FOR BIFOLD DOORS.

200mm MINIMUM BEAM / PLATE END BEARING ONTO THE SUPPORTING WALLS IS REQUIRED WITH 400mm LONG x 100mm WIDE x 150mm DEEP PADSTONES (CONCRETE LINTELS) ON THE ASSUMED DENSE BLOCKWORK INNER LEAF AND THE BEAM / PLATE SPAN LENGTHS SHOULD BE CHECKED BEFORE ORDERING.

THE BEAM / PLATE IS TO BE GALVANISED OR RECEIVE SUITABLE EXTERNAL PROTECTION.

THE BEAM / PLATE MUST BE POSITIONED WITH A 15mm CLEARANCE ABOVE THE PROPOSED BIFOLDS TO ACCOMMODATE DEFLECTION.

1. LOADING DATA:

PITCHED ROOF (TILES):

DEAD LOAD:	TILES	0.48 kn/m ²
	BATTENS & FELT	0.05 kn/m ²
	CEILING	0.25 kn/m ²
	INSULATION	0.05 kn/m ²
	PLASTERBOARD & SKIM	0.15 kn/m ²
	RAFTERS & JOISTS	<u>0.12 kn/m²</u>
		1.10 kn/m ²

LIVE LOAD: 0.75kn/m²

FLOOR:

DEAD LOAD :	BOARDS	0.12 kn/m ²
	JOISTS	0.08 kn/m ²
	PARTITIONS	0.50 kn/m ²
	PLASTERBOARD & SKIM	<u>0.15 kn/m²</u>
		0.85 kn/m ²

LIVE LOAD : 1.5 kn/m²

WALL:

DEAD LOAD: (BRICK / BLOCK) 2.1 kn/m²

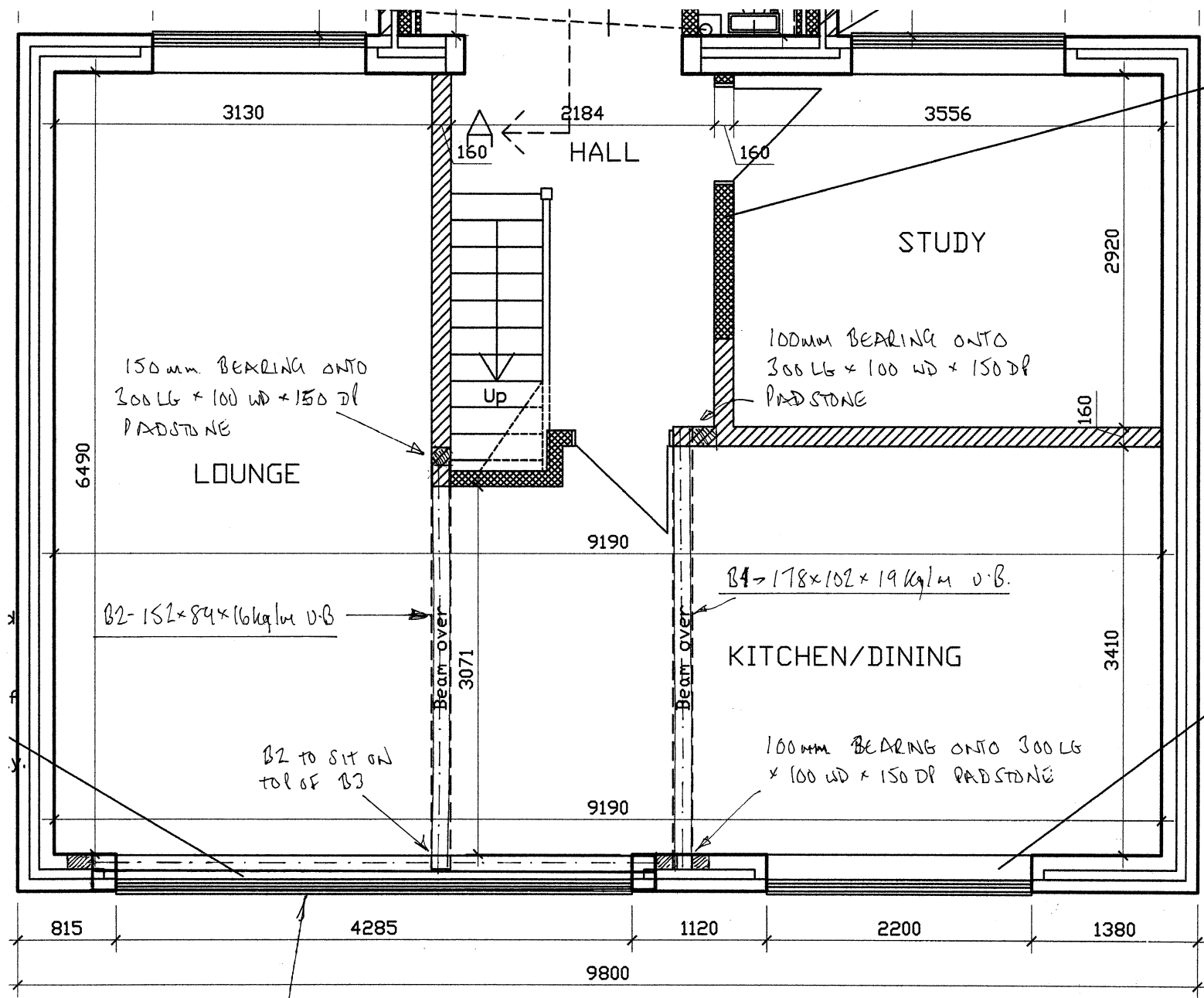
2. BEAM OVER KITCHEN / DINING – B1 :

DESIGN SPAN = 3.4m

FLOOR LOAD: LOADED DISTANCE = 5.8m / 2 = 2.9m

D.L. 0.85 kn/m² x 2.9m = 2.465 kn/m L.L. 1.5 kn/m² x 2.9m = 4.35 kn/m

FROM THE COMPUTER PRINTOUT: A 178 x 102 x 19 kg/m U.B. IS ADEQUATE



BEAM WITH PLATE OVER BIFOLDS - B3

254 x 146 x 37 kg/m U.B. WITH 280mm WIDE x 8mm THICK PLATE WELDED TO BOTTOM FLANGE WITH 6mm FILLET WELD BOTH SIDES (200mm WELD, 200mm GAP) SEE PAGE 16
PROVIDE 200mm BEARING ONTO 400 LG x 100 WD x 150 DP PADSTONES ON INNER LEAF.

3. BEAM OVER LOUNGE – B2 :

DESIGN SPAN = 3.1m

FLOOR LOAD:

LOADED DISTANCE = 5.0m / 2 = 2.5m

D.L. $0.85 \text{ kn/m}^2 \times 2.5\text{m} = 2.125 \text{ kn/m}$

L.L. $1.5 \text{ kn/m}^2 \times 2.5\text{m} = 3.75 \text{ kn/m}$

FROM THE COMPUTER PRINTOUT: **A 152 x 89 x 16 kg/m U.B. IS ADEQUATE**

4. BEAM WITH PLATE OVER BIFOLDS – B3:

DESIGN SPAN = 4.3m

DEAD POINT LOAD FROM BEAM B2 AT 2.7m = 3.2939 kn

LIVE POINT LOAD FROM BEAM B2 AT 2.7m = 5.8125 kn

ROOF LOAD:

LOADED DISTANCE = 4.0m / 2 = 2.0m

D.L. $1.10 \text{ kn/m}^2 \times 2.0\text{m} = 2.2 \text{ kn/m}$

L.L. $0.75 \text{ kn/m}^2 \times 2.0\text{m} = 1.5 \text{ kn/m}$

WALL LOAD:

LOADED HEIGHT = 2.3m

D.L. $(2.1 + 2.1 \text{ kn/m}^2) \times 2.3\text{m} = 9.66 \text{ kn/m}$

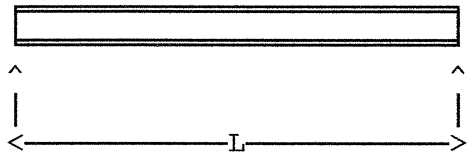
FROM THE PRINTOUT: **A 254 x 146 x 37 kg/m U.B. WITH 8mm THICK x 280mm WIDE PLATE
WELDED TO THE BOTTOM FLANGE WITH A 6mm FILLET WELD IS
ADEQUATE. (WELD APPLIED FOR 200mm, THEN A 200mm GAP.)**

THE BEAM / PLATE IS TO BE GALVANISED OR RECEIVE SUITABLE EXTERNAL PROTECTION.

Office:

Location: BEAM OVER KITCHEN / DINING - B1

Simply supported steel beam



Calculations in accordance
 with BS5950-1: 2000

Beam span $L=3.4$ m

Section properties

178 x 102 x 19 UB.
 Dimensions (mm): $D=177.8$ $B=101.2$ $t=4.8$ $T=7.9$ $r=7.6$
 Properties (cm): $I_x=1360$ $I_y=137$ $S_x=171$ $S_y=41.6$ $J=4.41$
 $A=24.3$ $r_y=2.3744$ $r_x=7.4811$

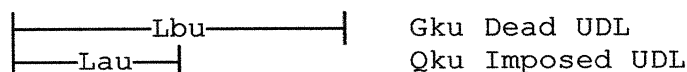
Strength of steel Clause 3.1.1

For material thickness 7.9 mm (Grade S 275)
 Design strength $p_y=275$ N/mm²
 Young's Modulus $E=205$ kN/mm²

Loading

Dead load factor $\gamma_{md}=1.4$
 Imposed load factor $\gamma_{mi}=1.6$

All loads are positive downwards, reactions are positive upwards,
 sagging moments are positive.



Distances are measured
 from left hand support

Uniformly distributed load 1 of 1
 Dist. from left support to start $L_{au}(1)=0$ m
 Distance from left support to end $L_{bu}(1)=3.4$ m
 Dead load (unfactored) $G_{ku}(1)=2.465$ kN/m
 Imposed load (unfactored) $Q_{ku}(1)=4.35$ kN/m

BMs at 40th points, from left to right (sagging is positive)

0	1.4668	2.8583	4.1747	5.4158	6.5817
	7.6724	8.6878	9.6281	10.493	11.283
	11.998	12.637	13.201	13.69	14.104
	14.442	14.705	14.893	15.006	15.044
	15.006	14.893	14.705	14.442	14.104
	13.69	13.201	12.637	11.998	11.283
	10.493	9.6281	8.6878	7.6724	6.5817
	5.4158	4.1747	2.8583	1.4668	0

Maximum span bending moment 15.044 kNm

Office:

End shears

Shear force at left hand end 17.699 kN
Shear force at right hand end 17.699 kN
Design shear force $F_v=17.699$ kN

Unfactored dead shear at LHE 4.1905 kN
Unfactored imposed shear at LHE 7.395 kN
Unfactored dead shear at RHE 4.1905 kN
Unfactored imposed shear at RHE 7.395 kN

Unfactored dead load deflection 1.5384 mm
Unfactored imposed load deflectn 2.7149 mm
Total DL & imposed deflection 4.2533 mm
Span:defln ratio for dead load 2210
Span:defln ratio for imposed load 1252.4
Span:defln ratio for total load 799.38

>From Table 8

Limiting deflection (brittle) $DEL_{lim}=L*1000/360=3.4*1000/360$
 $=9.4444$ mm

Since imposed load deflection $\leq DEL_{lim}$ (2.7149 mm $\leq$ 9.4444 mm)
deflection within limiting value.

Classification Clause 3.5.2

Classify outstand element of compression flange:

Parameter (Table 11 Note b) $e=(275/p_y)^{0.5}=(275/275)^{0.5}$
 $=1$

Outstand $b=B/2=101.2/2=50.6$ mm

Ratio $b'/T=b/T=50.6/7.9=6.4051$

As $b/T \leq 9e$ (9), outstand element of compression
flange is classified as Class 1 plastic

Classify web of section:

Depth between fillets $d=D-2*(T+r)=177.8-2*(7.9+7.6)$
 $=146.8$ mm

Ratio $d'/t=d/t=146.8/4.8=30.583$

For sections with neutral axis at mid-depth

As $d/t \leq 80e$ (80), web is classified as Class 1 plastic

Shear buckling

Since $d/t < 70e$ no check for shear buckling is required.

Buckling resistance

Since the beam is subject to possible lateral torsional buckling, the buckling resistance moment M_b is first considered rather than the moment capacity M_c as a guide to selection. A conservative analysis is adopted, taking the worst moment with the greatest effective length.

Office:

Effective length - Clause 4.3.5.2

As no point loads are applied to the beam the effective length, L_e , is found by considering the restraint conditions at the supports from Table 13 based on the length between the restraints equal to the beam span $L_T = L = 3.4 \text{ m}$

Compression flange laterally restrained.

Nominal torsional restraint about longitudinal axis.

Both flanges free to rotate on plan.

Normal loading conditions therefore

Effective length (Table 13) $L_e = 1.0 * L_T = 1.0 * 3.4 = 3.4 \text{ m}$

Clause 4.3.6.6 and Table 18

Equivalent uniform moment factor

The member is loaded between restraints.

For a beam carrying a distributed load over its entire span.

Equivalent uniform moment factor $m_{LT} = 0.925$

Resistance to lateral-torsional buckling Cl.4.3.6

Radius gyration about minor axis $r_y = \sqrt{I_y/A} = \sqrt{137/24.3}$
 $= 2.3744 \text{ cm}$

Slenderness of section $\lambda = L_e/r_y * 100 = 3.4/2.3744 * 100$
 $= 143.19$

Buckling parameter $u = (4 * S_x^2 * (1 - I_y/I_x) / (A^2 * ((D-T)/10)^2))^0.25$
 $= (4 * 171^2 * (1 - 137/1360) / (24.3^2 * ((177.8 - 7.9)/10)^2))^0.25$
 $= 0.88631$

Torsional index $x = 0.566 * ((D-T)/10) * (A/J)^0.5$
 $= 0.566 * ((177.8 - 7.9)/10) * (24.3/4.41)^0.5$
 $= 22.573$

Ratio $\text{ratio} = \lambda/x = 143.19/22.573$
 $= 6.3435$

Slenderness factor $v = 1 / ((1 + 0.05 * \text{ratio}^2)^0.25)$
 $= 1 / ((1 + 0.05 * 6.3435^2)^0.25)$
 $= 0.75908$

Ratio β_w $\beta_{tw} = 1.0$

Equivalent slenderness $\lambda_{mLT} = u * v * \lambda * (\beta_{tw})^0.5$
 $= 0.88631 * 0.75908 * 143.19 * (1)^0.5$
 $= 96.337$

Limiting slenderness $\lambda_{mlo} = 0.4 * ((\pi^2 * E * 10^3) / p_y)^0.5$
 $= 0.4 * ((3.1416^2 * 205 * 10^3) / 275)^0.5$
 $= 34.31$

Perry coefficient $\eta_{LT} = 0.007 * (\lambda_{mLT} - \lambda_{mlo})$
 $= 0.007 * (96.337 - 34.31)$
 $= 0.43419$

Elastic strength $p_e = \pi^2 * E * 10^3 / (\lambda_{mLT}^2)$
 $= 3.1416^2 * 205 * 10^3 / (96.337^2)$
 $= 218.01 \text{ N/mm}^2$

Factor $\phi_{LT} = (p_y + (\eta_{LT} + 1) \cdot p_e) / 2$
 $= (275 + (0.43419 + 1) \cdot 218.01) / 2$
 $= 293.83 \text{ N/mm}^2$

Factor $p_{ey} = p_e \cdot p_y = 218.01 \cdot 275 = 59951$

Bending strength $p_b = (p_{ey}) / (\phi_{LT} + ((\phi_{LT}^2 - p_{ey})^{0.5}))$
 $= (59951) / (293.83 + ((293.83^2 - 59951)^{0.5}))$
 $= 131.4 \text{ N/mm}^2$

Buckling resistance moment $M_b = S_x \cdot p_b / 10^3 = 171 \cdot 131.4 / 10^3$
 $= 22.469 \text{ kNm}$

Equivalent uniform moment factor $m_{LT} = 0.925$

Since $M \leq M_b / m_{LT}$ ($15.044 \text{ kNm} \leq 24.291 \text{ kNm}$) section O.K. for lateral torsional buckling resistance.

Check section for combined moment and shear

Conservatively the maximum shear is checked with the maximum moment

Shear area $A_v = t \cdot D = 4.8 \cdot 177.8 = 853.44 \text{ mm}^2$

Shear capacity $P_v = 0.6 \cdot p_y \cdot A_v / 10^3 = 0.6 \cdot 275 \cdot 853.44 / 10^3$
 $= 140.82 \text{ kN}$

Design shear force $F_v = 17.699 \text{ kN}$

Elastic modulus $Z_x = I_x / (D/20) = 1360 / (177.8/20)$
 $= 152.98 \text{ cm}^3$

Since $F_v < 0.6 P_v$

Moment capacity for compact sec $M_c = p_y \cdot S_x / 10^3 = 275 \cdot 171 / 10^3$
 $= 47.025 \text{ kNm}$

Limiting moment capacity $M_{clim} = 1.2 \cdot p_y \cdot Z_x / 10^3$
 $= 1.2 \cdot 275 \cdot 152.98 / 10^3$
 $= 50.484 \text{ kNm}$

Since $M \leq M_c$ ($15.044 \text{ kNm} \leq 47.025 \text{ kNm}$) applied moment within moment capacity.

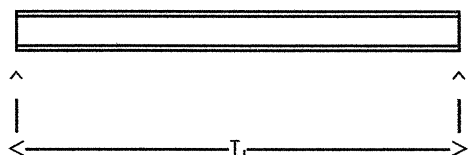
N.B. Buckling resistance is less than moment capacity and therefore buckling resistance controls the design.

UNIVERSAL BEAM	178 x 102 x 19 UB Grade S 275
DESIGN	Max. shear force 17.699 kN
SUMMARY	Shear capacity 140.82 kN
	Max. applied moment 15.044 kNm
	Moment capacity 47.025 kNm
	Buckling resistance 22.469 kNm
	Moment factor (m_{LT}) 0.925
	Resistance (M_b / m_{LT}) 24.291 kNm
	Unfactored DL defln 1.5384 mm
	Unfactored LL defln 2.7149 mm
	Limiting deflection 9.4444 mm
Unfactored end shears	DL shear at LHE 4.1905 kN
	LL shear at LHE 7.395 kN
	DL shear at RHE 4.1905 kN
	LL shear at RHE 7.395 kN

Office:

Location: BEAM OVER LOUNGE - B2

Simply supported steel beam



Calculations in accordance
with BS5950-1: 2000

Beam span $L=3.1$ m

Section properties

152 x 89 x 16 UB.

Dimensions (mm): $D=152.4$ $B=88.7$ $t=4.5$ $T=7.7$ $r=7.6$

Properties (cm): $I_x=834$ $I_y=89.8$ $S_x=123$ $S_y=31.2$ $J=3.56$
 $A=20.3$ $r_y=2.1032$ $r_x=6.4097$

Strength of steel Clause 3.1.1

For material thickness 7.7 mm (Grade S 275)

Design strength $p_y=275$ N/mm²

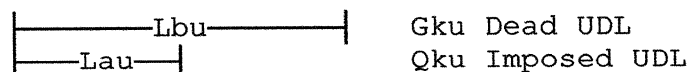
Young's Modulus $E=205$ kN/mm²

Loading

Dead load factor $\gamma_{md}=1.4$

Imposed load factor $\gamma_{mi}=1.6$

All loads are positive downwards, reactions are positive upwards,
sagging moments are positive.



Distances are measured
from left hand support

Uniformly distributed load 1 of 1

Dist. from left support to start $L_{au}(1)=0$ m

Distance from left support to end $L_{bu}(1)=3.1$ m

Dead load (unfactored) $G_{ku}(1)=2.125$ kN/m

Imposed load (unfactored) $Q_{ku}(1)=3.75$ kN/m

BMs at 40th points, from left to right (sagging is positive)

0	1.0512	2.0484	2.9918	3.8812	4.7168
	5.4984	6.2262	6.9	7.5199	8.0859
	8.598	9.0562	9.4605	9.8109	10.107
	10.35	10.539	10.673	10.754	10.781
	10.754	10.673	10.539	10.35	10.107
	9.8109	9.4605	9.0562	8.598	8.0859
	7.5199	6.9	6.2262	5.4984	4.7168
	3.8812	2.9918	2.0484	1.0512	0

Maximum span bending moment 10.781 kNm

Office:

End shears

Shear force at left hand end 13.911 kN
Shear force at right hand end 13.911 kN
Design shear force $F_v=13.911$ kN

Unfactored dead shear at LHE 3.2938 kN
Unfactored imposed shear at LHE 5.8125 kN
Unfactored dead shear at RHE 3.2938 kN
Unfactored imposed shear at RHE 5.8125 kN

Unfactored dead load deflection 1.4946 mm
Unfactored imposed load deflectn 2.6375 mm
Total DL & imposed deflection 4.1321 mm
Span:defln ratio for dead load 2074.1
Span:defln ratio for imposed load 1175.3
Span:defln ratio for total load 750.22

>From Table 8

Limiting deflection (brittle) $DEL_{lim}=L*1000/360=3.1*1000/360$
 $=8.6111$ mm

Since imposed load deflection $\leq DEL_{lim}$ (2.6375 mm $\leq$ 8.6111 mm)
deflection within limiting value.

Classification Clause 3.5.2

Classify outstand element of compression flange:

Parameter (Table 11 Note b) $e=(275/p_y)^{0.5}=(275/275)^{0.5}$
 $=1$

Outstand $b=B/2=88.7/2=44.35$ mm

Ratio $b'T=b/T=44.35/7.7=5.7597$

As $b/T \leq 9e$ (9), outstand element of compression
flange is classified as Class 1 plastic

Classify web of section:

Depth between fillets $d=D-2*(T+r)=152.4-2*(7.7+7.6)$
 $=121.8$ mm

Ratio $d't=d/t=121.8/4.5=27.067$

For sections with neutral axis at mid-depth

As $d/t \leq 80e$ (80), web is classified as Class 1 plastic

Shear buckling

Since $d/t < 70e$ no check for shear buckling is required.

Buckling resistance

Since the beam is subject to possible lateral torsional buckling, the buckling resistance moment M_b is first considered rather than the moment capacity M_c as a guide to selection. A conservative analysis is adopted, taking the worst moment with the greatest effective length.

Office:

Effective length - Clause 4.3.5.2

As no point loads are applied to the beam the effective length, L_e , is found by considering the restraint conditions at the supports from Table 13 based on the length between the restraints equal to the beam span $L_T = L = 3.1 \text{ m}$

Compression flange laterally restrained.

Nominal torsional restraint about longitudinal axis.

Both flanges free to rotate on plan.

Normal loading conditions therefore

Effective length (Table 13) $L_e = 1.0 * L_T = 1.0 * 3.1 = 3.1 \text{ m}$

Clause 4.3.6.6 and Table 18

Equivalent uniform moment factor

The member is loaded between restraints.

For a beam carrying a distributed load over its entire span.

Equivalent uniform moment factor $m_{LT} = 0.925$

Resistance to lateral-torsional buckling Cl.4.3.6

Radius gyration about minor axis $r_y = \sqrt{I_y/A} = \sqrt{89.8/20.3}$
 $= 2.1032 \text{ cm}$

Slenderness of section $\lambda = L_e/r_y * 100 = 3.1/2.1032 * 100$
 $= 147.39$

Buckling parameter $u = (4 * S_x^2 * (1 - I_y/I_x) / (A^2 * ((D-T)/10)^2))^0.25$
 $= (4 * 123^2 * (1 - 89.8/834) / (20.3^2 * ((152.4 - 7.7)/10)^2))^0.25$
 $= 0.88944$

Torsional index $x = 0.566 * ((D-T)/10) * (A/J)^0.5$
 $= 0.566 * ((152.4 - 7.7)/10) * (20.3/3.56)^0.5$
 $= 19.557$

Ratio $\text{ratio} = \lambda/x = 147.39/19.557$
 $= 7.5364$

Slenderness factor $v = 1 / ((1 + 0.05 * \text{ratio}^2)^0.25)$
 $= 1 / ((1 + 0.05 * 7.5364^2)^0.25)$
 $= 0.71437$

Ratio β_w $\beta_{tw} = 1.0$

Equivalent slenderness $\lambda_{mLT} = u * v * \lambda * (\beta_{tw})^0.5$
 $= 0.88944 * 0.71437 * 147.39 * (1)^0.5$
 $= 93.65$

Limiting slenderness $\lambda_{mlo} = 0.4 * ((\pi^2 * E * 10^3) / p_y)^0.5$
 $= 0.4 * ((3.1416^2 * 205 * 10^3) / 275)^0.5$
 $= 34.31$

Perry coefficient $\eta_{LT} = 0.007 * (\lambda_{mLT} - \lambda_{mlo})$
 $= 0.007 * (93.65 - 34.31)$
 $= 0.41538$

Elastic strength $p_e = \pi^2 * E * 10^3 / (\lambda_{mLT}^2)$
 $= 3.1416^2 * 205 * 10^3 / (93.65^2)$
 $= 230.69 \text{ N/mm}^2$

Office:
 Factor $\phi_{LT} = (p_y + (\eta_{LT} + 1) \cdot p_e) / 2$
 $= (275 + (0.41538 + 1) \cdot 230.69) / 2$
 $= 300.76 \text{ N/mm}^2$
 Factor $p_{ey} = p_e \cdot p_y = 230.69 \cdot 275 = 63441$
 Bending strength $p_b = (p_{ey}) / (\phi_{LT} + ((\phi_{LT}^2 - p_{ey})^{0.5}))$
 $= (63441) / (300.76 + ((300.76^2 - 63441)^{0.5}))$
 $= 136.4 \text{ N/mm}^2$
 Buckling resistance moment $M_b = S_x \cdot p_b / 10^3 = 123 \cdot 136.4 / 10^3$
 $= 16.777 \text{ kNm}$
 Equivalent uniform moment factor $m_{LT} = 0.925$
 Since $M \leq M_b / m_{LT}$ ($10.781 \text{ kNm} \leq 18.137 \text{ kNm}$) section O.K. for lateral torsional buckling resistance.

Check section for combined moment and shear

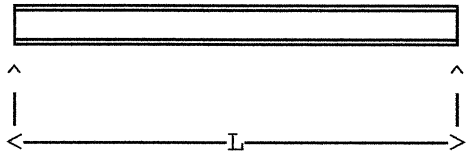
Conservatively the maximum shear is checked with the maximum moment
 Shear area $A_v = t \cdot D = 4.5 \cdot 152.4 = 685.8 \text{ mm}^2$
 Shear capacity $P_v = 0.6 \cdot p_y \cdot A_v / 10^3 = 0.6 \cdot 275 \cdot 685.8 / 10^3$
 $= 113.16 \text{ kN}$
 Design shear force $F_v = 13.911 \text{ kN}$
 Elastic modulus $Z_x = I_x / (D/20) = 834 / (152.4/20)$
 $= 109.45 \text{ cm}^3$
 Since $F_v < 0.6 P_v$
 Moment capacity for compact sec $M_c = p_y \cdot S_x / 10^3 = 275 \cdot 123 / 10^3$
 $= 33.825 \text{ kNm}$
 Limiting moment capacity $M_{clim} = 1.2 \cdot p_y \cdot Z_x / 10^3$
 $= 1.2 \cdot 275 \cdot 109.45 / 10^3$
 $= 36.118 \text{ kNm}$
 Since $M \leq M_c$ ($10.781 \text{ kNm} \leq 33.825 \text{ kNm}$) applied moment within moment capacity.
 N.B. Buckling resistance is less than moment capacity and therefore buckling resistance controls the design.

UNIVERSAL BEAM	152 x 89 x 16 UB Grade S 275
	Max. shear force 13.911 kN
DESIGN	Shear capacity 113.16 kN
SUMMARY	Max. applied moment 10.781 kNm
	Moment capacity 33.825 kNm
	Buckling resistance 16.777 kNm
	Moment factor (m_{LT}) 0.925
	Resistance (M_b / m_{LT}) 18.137 kNm
	Unfactored DL defln 1.4946 mm
	Unfactored LL defln 2.6375 mm
	Limiting deflection 8.6111 mm
	DL shear at LHE 3.2938 kN
Unfactored	LL shear at LHE 5.8125 kN
end shears	DL shear at RHE 3.2938 kN
	LL shear at RHE 5.8125 kN

Office:

Location: BEAM WITH PLATE OVER BIFOLDS - B3

Simply supported steel beam



Calculations in accordance
with BS5950-1: 2000

Beam span

$L=4.3$ m

Section properties

254 x 146 x 37 UB.

Dimensions (mm): $D=256$ $B=146.4$ $t=6.3$ $T=10.9$ $r=7.6$

Properties (cm): $I_x=5540$ $I_y=571$ $S_x=483$ $S_y=119$ $J=15.3$

$A=47.2$ $r_y=3.4781$ $r_x=10.834$

Strength of steel Clause 3.1.1

For material thickness 10.9 mm (Grade S 275)

Design strength $p_y=275$ N/mm²

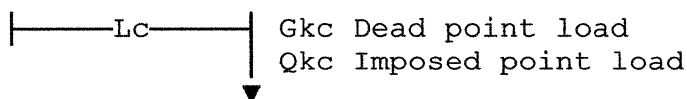
Young's Modulus $E=205$ kN/mm²

Loading

Dead load factor $\gamma_{md}=1.4$

Imposed load factor $\gamma_{mi}=1.6$

All loads are positive downwards, reactions are positive upwards,
sagging moments are positive.



Distances are measured
from left hand support

Concentrated load 1 of 1

Distance from left support

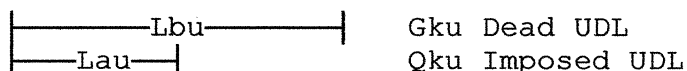
$L_c(1)=2.7$ m

Dead load (unfactored)

$G_{kc}(1)=3.2939$ kN

Imposed load (unfactored)

$Q_{kc}(1)=5.8125$ kN



Distances are measured
from left hand support

Uniformly distributed load 1 of 2

Dist. from left support to start $L_{au}(1)=0$ m

Distance from left support to end $L_{bu}(1)=4.3$ m

Dead load (unfactored)

$G_{ku}(1)=2.2$ kN/m

Imposed load (unfactored)

$Q_{ku}(1)=1.5$ kN/m

Uniformly distributed load 2 of 2

Office:

Dist. from left support to start $L_{au}(2)=0$ m
Distance from left support to end $L_{bu}(2)=4.3$ m
Dead load (unfactored) $G_{ku}(2)=9.66$ kN/m
Imposed load (unfactored) $Q_{ku}(2)=0$ kN/m

BMs at 40th points, from left to right (sagging is positive)

0	4.839	9.4583	13.858	18.038	21.999
	25.739	29.261	32.562	35.644	38.507
	41.15	43.573	45.776	47.76	49.525
	51.069	52.395	53.5	54.386	55.052
	55.499	55.726	55.733	55.521	55.089
	53.116	50.75	48.164	45.358	42.332
	39.088	35.623	31.939	28.035	23.911
	19.568	15.006	10.223	5.2215	0

Maximum span bending moment 55.733 kNm

End shears

Shear force at left hand end 46.035 kN
Shear force at right hand end 49.594 kN
Design shear force $F_v=49.594$ kN

Unfactored dead shear at LHE 26.725 kN
Unfactored imposed shear at LHE 5.3878 kN
Unfactored dead shear at RHE 27.567 kN
Unfactored imposed shear at RHE 6.8747 kN

Unfactored dead load deflection 5.086 mm
Unfactored imposed load deflectn 1.3596 mm
Total DL & imposed deflection 6.4456 mm
Span:defln ratio for dead load 845.46
Span:defln ratio for imposed load 3162.8
Span:defln ratio for total load 667.13

>From Table 8

Limiting deflection (brittle) $DEL_{lim}=L*1000/360=4.3*1000/360$
 $=11.944$ mm

Since imposed load deflection $\leq DEL_{lim}$ (1.3596 mm $\leq$ 11.944 mm)
deflection within limiting value.

Classification Clause 3.5.2

Classify outstand element of compression flange:

Parameter (Table 11 Note b) $e=(275/p_y)^{0.5}=(275/275)^{0.5}$
 $=1$

Outstand $b=B/2=146.4/2=73.2$ mm

Ratio $b'T=b/T=73.2/10.9=6.7156$

As $b/T \leq 9e$ (9), outstand element of compression
flange is classified as Class 1 plastic

Classify web of section:

Depth between fillets $d=D-2*(T+r)=256-2*(10.9+7.6)$
 $=219$ mm

Ratio $d't=d/t=219/6.3=34.762$

For sections with neutral axis at mid-depth

As $d/t \leq 80e$ (80), web is classified as Class 1 plastic

Office:

Shear buckling

Since $d/t < 70e$ no check for shear buckling is required.

Buckling resistance

Since the beam is subject to possible lateral torsional buckling, the buckling resistance moment M_b is first considered rather than the moment capacity M_c as a guide to selection. A conservative analysis is adopted, taking the worst moment with the greatest effective length.

Effective length - Clause 4.3.5.2

As the point load does not provide restraint to the compression flange, the effective length, L_e , is found by considering the restraint conditions at the supports from Table 13 based on the length between the restraints equal to the beam span $L_T=L=4.3$ m

Compression flange laterally restrained.

Nominal torsional restraint about longitudinal axis.

Compression flange partially restrained against rotation on plan.

Normal loading conditions therefore

Effective length (Table 13) $L_e=0.85*L_T=0.85*4.3=3.655$ m

Clause 4.3.6.6 and Table 18

Equivalent uniform moment factor

The member is loaded between restraints.

Moments on beam, all moments are taken as positive

Start $m_1=0$ kNm

Quarter point $m_2=39$ kNm

Mid-span $m_3=55$ kNm

Three quarter point $m_4=43$ kNm

End $m_5=0$ kNm

Equivalent uniform moment factor $m_{LT}=0.2+(0.15*m_2+0.5*m_3+0.15*m_4)/M$
 $=0.2+(0.15*39+0.5*55+0.15*43)$
 $/55.733$
 $=0.91412$

Resistance to lateral-torsional buckling Cl.4.3.6

Radius gyration about minor axis $r_y=\text{SQR}(I_y/A)=\text{SQR}(571/47.2)$
 $=3.4781$ cm

Slenderness of section $\lambda=L_e/r_y*100=3.655/3.4781*100$
 $=105.08$

Buckling parameter $u=(4*S_x^2*(1-I_y/I_x)/(A^2*((D-T)/10)^2))^0.25$
 $=(4*483^2*(1-571/5540)/(47.2^2$
 $*((256-10.9)/10)^2))^0.25$
 $=0.88927$

Torsional index $x=0.566*((D-T)/10)*(A/J)^0.5$
 $=0.566*((256-10.9)/10)*(47.2$
 $/15.3)^0.5$
 $=24.366$

Office:

Ratio $\text{ratio} = \lambda / x = 105.08 / 24.366$
 $= 4.3128$

Slenderness factor $v = 1 / ((1 + 0.05 * \text{ratio}^2)^{0.25})$
 $= 1 / ((1 + 0.05 * 4.3128^2)^{0.25})$
 $= 0.84842$

Ratio β_w $\text{betaw} = 1.0$

Equivalent slenderness $\lambda_{mLT} = u * v * \lambda * (\text{betaw})^{0.5}$
 $= 0.88927 * 0.84842 * 105.08 * (1)^{0.5}$
 $= 79.284$

Limiting slenderness $\lambda_{mlo} = 0.4 * ((\pi^2 * E * 10^3) / p_y)^{0.5}$
 $= 0.4 * ((3.1416^2 * 205 * 10^3) / 275)^{0.5}$
 $= 34.31$

Perry coefficient $\eta_{LT} = 0.007 * (\lambda_{mLT} - \lambda_{mlo})$
 $= 0.007 * (79.284 - 34.31)$
 $= 0.31482$

Elastic strength $p_e = \pi^2 * E * 10^3 / (\lambda_{mLT}^2)$
 $= 3.1416^2 * 205 * 10^3 / (79.284^2)$
 $= 321.87 \text{ N/mm}^2$

Factor $\phi_{LT} = (p_y + (\eta_{LT} + 1) * p_e) / 2$
 $= (275 + (0.31482 + 1) * 321.87) / 2$
 $= 349.1 \text{ N/mm}^2$

Factor $p_{ey} = p_e * p_y = 321.87 * 275 = 88514$

Bending strength $p_b = (p_{ey}) / (\phi_{LT} + ((\phi_{LT}^2 - p_{ey})^{0.5}))$
 $= (88514) / (349.1 + ((349.1^2 - 88514)^{0.5}))$
 $= 166.46 \text{ N/mm}^2$

Buckling resistance moment $M_b = S_x * p_b / 10^3 = 483 * 166.46 / 10^3$
 $= 80.401 \text{ kNm}$

Equivalent uniform moment factor $m_{LT} = 0.91412$

Since $M \leq M_b / m_{LT}$ ($55.733 \text{ kNm} \leq 87.955 \text{ kNm}$) section O.K. for lateral torsional buckling resistance.

Check section for combined moment and shear

Conservatively the maximum shear is checked with the maximum moment

Shear area $A_v = t * D = 6.3 * 256 = 1612.8 \text{ mm}^2$

Shear capacity $P_v = 0.6 * p_y * A_v / 10^3 = 0.6 * 275 * 1612.8 / 10^3$
 $= 266.11 \text{ kN}$

Design shear force $F_v = 49.594 \text{ kN}$

Elastic modulus $Z_x = I_x / (D / 20) = 5540 / (256 / 20)$
 $= 432.81 \text{ cm}^3$

Since $F_v < 0.6 P_v$

Moment capacity for compact sec $M_c = p_y * S_x / 10^3 = 275 * 483 / 10^3$
 $= 132.82 \text{ kNm}$

Limiting moment capacity $M_{clim} = 1.2 * p_y * Z_x / 10^3$
 $= 1.2 * 275 * 432.81 / 10^3$
 $= 142.83 \text{ kNm}$

Since $M \leq M_c$ ($55.733 \text{ kNm} \leq 132.82 \text{ kNm}$) applied moment within moment capacity.

N.B. Buckling resistance is less than moment capacity and therefore buckling resistance controls the design.

UNIVERSAL BEAM

DESIGN
 SUMMARY

Office:	254 x 146 x 37 UB Grade S 275
Max. shear force	49.594 kN
Shear capacity	266.11 kN
Max. applied moment	55.733 kNm
Moment capacity	132.82 kNm
Buckling resistance	80.401 kNm
Moment factor (mLT)	0.91412
Resistance (Mb/mLT)	87.955 kNm
Unfactored DL defln	5.086 mm
Unfactored LL defln	1.3596 mm
Limiting deflection	11.944 mm
DL shear at LHE	26.725 kN
LL shear at LHE	5.3878 kN
DL shear at RHE	27.567 kN
LL shear at RHE	6.8747 kN

Unfactored
end shears

No408

