June 2014

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Calculations for
67, Brown Edge Rd
Buxton

The following details have been determined following the carrying out of the attached calculations. These calculations were carried out using information following a site visit and from plans proposed.

- **For purlin supports remove existing cavity wall and blck or prop off either a 203 x 133 (25) UB or a 152 x 152 (30) UC sat on 450 x 150 x 100 reinforced padstones.**

- **For new purlins spanning 3.75m. use a 250 x 100 C24 timber for top purlin and a 225 x 75 C24 timber for bottom purlin.**

- **If existing purlins need to be extended use a system as shown in Figure 1 at end of calculations.**

- **For main decking frame use 150 x 50 C24 joists @ 450c/s on 225 x 75 C24 main cross supports on 150 x 150 posts on 0.5m x 0.5m concrete bases as shown in figure at end of calculations.**

Any problems please ring the above office.

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Loadings kN/m²

Roof

Dead 1.1
Live 0.8 1.9

Decking floor

Live 1.5
Dead 0.5 2.0

Loadings

The loadings are as shown in the attached drawings. It is presumed that the existing gable will be taken down and the purlins either propped with minimum 150 x 100 timbers or lightweight blockwork. The middle wall in the bungalow is loadbearing and the beams are designed as single span to sit on this.

The main structural layout of the decking is also shown on the attached drawings

Steel supports

Effective span 4.1m.

This beam basically carries two purlin loads and some slight blockwork load of average 0.75m. which will provide a factor of safety.

Bottom dead purlin point load 3.5 x 1.3 x 1.1 = 5
Bottom live purlin load 3.5 x 1.3 x 0.8 = 3.65

Top dead purlin point load 3.5 x 2.1 x 1.1 = 8.1
Bottom live purlin load 3.5 x 2.1 x 0.8 = 5.9

From Pages C1 – C12 use either a 203 x 133 (25) UB or a 152 x 152 (23) UC sat on 450 x 150 x 100 reinforced padstones.

Padstone design for under beams

Maximum factored shear force = 46kN

Actual stress with padstones below $= \dfrac{4610^3}{450 \times 100}$

$= 1.02\ N/mm^2$

Allowable stress with brick $= \dfrac{1.25\ f_k}{\gamma m} = \dfrac{1.25 \times 6.4}{3.1} = 2.58\ N/mm^2$ **O.K**

Existing purlin strengthening

Existing size approx. 175 x 50

Shear at one end = 9kN

Design connection as shown in Figure 1 at end of calculations

Purlin for new roof area

Maximum span 3.75m

Dead load carried on upper purlin 2.1 x 1.1 = 2.3 (1.40 on lower)
Live load 2.1 x 0.8 = 1.7 (1.0 lower)

From Pages C13 – C18 use a 250 x 100 C24 timber sat upright for top purlin and 225 x 75 C24 timber for bottom purlin (def'n do exceed codes by 2mm but should be considered adequate).

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Decking

Worst joist position is for possible cantilever at end

B.M. = 1.5 x 1.5/2 x 0.45 x 2 = 1.01

With C24 timber

Zreqd = 1.01 10E6/7.5 = 1.35E5

With 150 x 50

Zact = 150 x 150 x 50/6 = 1.88E5 O.K.

For joists use 150 x 50 C24 joists @ 450c/s

Max. span of support beam 2.7m
Load = 4.6/2 x 2 = 4.6kN/m run (1.15 dead, 3.45 live)

From Pages C19 – C22 use 225 x 75 C24 timber for main supports

For posts maximum load = 1.25 x 12.42 = 15.5kN

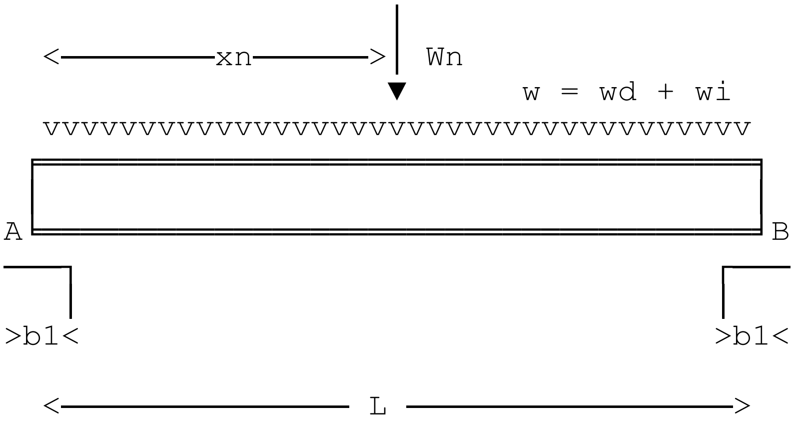
Say max. height = 1.8m

From Pages C23 – C25 use 150 x 150 C24 timber for main posts on 0.5 x 0.5 concrete bases.

Location: Brown Edge Rd

Simply supported steel beam subject to lateral torsional buckling

Calculations are in accordance with BS5950-1:2000. Simple beam with intermediate lateral restraints in accordance with Clause 4.3.5.2.



The n'th point load is shown:
Wn = Wnd + Wni
where suffices:
d denotes dead
i denotes imposed

The moment capacity of the section is determined from 4.2.5.2 or 4.2.5.3 and is based on the classification obtained from Table 11.

Beam span L=4.1 m

Section properties

203 x 133 x 25 UB.
Dimensions (mm): D=203.2 B=133.2 t=5.7 T=7.8 r=7.6
Properties (cm): Ix=2340 Iy=308 Sx=258 Sy=70.9 J=5.96
A=32 ry=3.1024 rx=8.5513

Strength of steel - Clause 3.1.1

The material thickness is 7.8 mm and the steel grade is S 275.

Design strength	py=275 N/mm^2
Young's Modulus	E=205 kN/mm^2

Loading (unfactored)

Dead UDL (including S.W)	wd=1.7 kN/m
Imposed UDL	wi=0 kN/m
Dead point load 1 (+ve down)	W1d=5 kN
Imposed point load 1	W1i=3.65 kN
Distance from L.H. supp to load 1	x1=1.3 m
Dead point load 2	W2d=8.1 kN
Imposed point load 2	W2i=5.9 kN
Distance from L.H. supp to load 2	x2=2.7 m

Factored loads

Distributed load	w'=wd*1.4+wi*1.6=1.7*1.4+0*1.6 =2.38 kN/m
Point load 1	W1'=W1d*1.4+W1i*1.6=5*1.4+3.65*1.6 =12.84 kN
Point load 2	W2'=W2d*1.4+W2i*1.6=8.1*1.4+5.9*1.6 =20.78 kN

Factored shear force

At end B	Fvb=w'*L/2+(W1'*x1+W2'*x2)/L =2.38*4.1/2+(12.84*1.3+20.78*2.7)/4.1 =22.635 kN
At end A	Fva=w'*L+W1'+W2'-Fvb =2.38*4.1+12.84+20.78-22.635 =20.743 kN
Max shear is at end B	Fve=Fvb=22.635 kN

Unfactored end shears

At end B dead	Fudb=wd*L/2+(W1d*x1+W2d*x2)/L =1.7*4.1/2+(5*1.3+8.1*2.7)/4.1 =10.405 kN
At end B imposed	Fuib=wi*L/2+(W1i*x1+W2i*x2)/L =0*4.1/2+(3.65*1.3+5.9*2.7)/4.1 =5.0427 kN
At end A dead	Fuda=wd*L+W1d+W2d-Fudb =1.7*4.1+5+8.1-10.405 =9.6655 kN
At end A imposed	Fuia=wi*L+W1i+W2i-Fuib =0*4.1+3.65+5.9-5.0427 =4.5073 kN

Factored moment

Moment at load W1 (+ve sagging)	M1=Fva*x1-w'*x1^2/2 =20.743*1.3-2.38*1.3^2/2 =24.955 kNm
Shear to left of W1	Fl=Fva-w'*x1=20.743-2.38*1.3 =17.649 kN
Shear to right of W1	Fr=Fva-w'*x1-W1' =20.743-2.38*1.3-12.84 =4.8094 kN
Moment at load W2 (+ve sagging)	M2=Fva*x2-W1'*(x2-x1)-w'*x2^2/2 =20.743*2.7-12.84*(2.7-1.3)-2.38*2.7^2/2 =29.356 kNm
Shear to left of W2	Fl=Fva-W1'-w'*x2 =20.743-12.84-2.38*2.7 =1.4774 kN
Shear to right of W2	Fr=Fva-w'*x2-W1'-W2' =20.743-2.38*2.7-12.84-20.78 =-19.303 kN

Maximum BM occurs at 2nd load M=M2=29.356 kNm
Corresponding shear force F=-Fr=19.303 kN

Classification - Clause 3.5.2

Classify outstand element of compression flange:
Parameter (Table 11 Note b) e=(275/py)^0.5=(275/275)^0.5
=1
Outstand b=B/2=133.2/2=66.6 mm
Ratio b'T=b/T=66.6/7.8=8.5385
As b/T <= 9e (9), outstand element of compression flange is classified as Class 1 plastic.
Classify web of section:
Depth between fillet radii d=D-2*(T+r)=203.2-2*(7.8+7.6)
=172.4 mm
Ratio d't=d/t=172.4/5.7=30.246
Neutral axis is assumed to be at mid-depth of section.
As d/t <= 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 Mb is first considered rather than the moment capacity Mc as a guide to selection.

Effective length - Clause 4.3.5.2

Beam is assumed to be laterally restraint at supports only.
Length of beam between restraints LT=L=4.1 m
In accordance with Clause 4.3.5.2 for beams with effective lateral restraints at intervals within their length subject to normal loading conditions
Effective length Le=LT=4.1 m

Clause 4.3.6.6 and Table 18

Equivalent uniform moment factor

The member is not loaded between restraints.
Maximum moment on segment Me=29 kNm
Far end BM betaM=24 kNm
Table 18 beta factor beta=betaM/Me=24/29=0.82759
Equivalent uniform moment factor mLT=TABLE 18 for beta=0.82759
=0.93103

Resistance to lateral-torsional buckling - Clause 4.3.6

Radius gyration about minor axis	ry=SQR(Iy/A)=SQR(308/32) =3.1024 cm
Slenderness of section	lambda=Le/ry*100=4.1/3.1024*100 =132.15
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T)/10)^2))^0.25 =(4*258^2*(1-308/2340)/(32^2*((203.2-7.8)/10)^2))^0.25 =0.87693
Torsional index	x=0.566*((D-T)/10)*(A/J)^0.5 =0.566*((203.2-7.8)/10)*(32/5.96)^0.5 =25.627
Ratio	ratio=lambda/x=132.15/25.627 =5.1569
Slenderness factor	v=1/((1+0.05*ratio^2)^0.25) =1/((1+0.05*5.1569^2)^0.25) =0.80942
Ratio βw	betaw=1.0
Equivalent slenderness	lamLT=u*v*lambda*(betaw)^0.5 =0.87693*0.80942*132.15*(1)^0.5 =93.804
Limiting slenderness	lamlo=0.4*((PI^2*E*10^3)/py)^0.5 =0.4*((3.1416^2*205*10^3)/275)^0.5 =34.31
Perry coefficient	etaLT=0.007*(lamLT-lamlo) =0.007*(93.804-34.31) =0.41646
Elastic strength	pe=PI^2*E*10^3/(lamLT^2) =3.1416^2*205*10^3/(93.804^2) =229.94 N/mm²
Factor	phiLT=(py+(etaLT+1)*pe)/2 =(275+(0.41646+1)*229.94)/2 =300.35 N/mm²
Factor	pey=pe*py=229.94*275=63232
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =(63232)/(300.35+((300.35^2-63232)^0.5)) =136.1 N/mm²
Buckling resistance moment	Mb=Sx*pb/10^3=258*136.1/10^3 =35.115 kNm
Equivalent uniform moment factor	mLT=0.93103

Since M ≤ Mb/mLT (29.356 kNm ≤ 37.716 kNm), section OK for lateral torsional buckling resistance.

Check section for combined moment and shear

Maximum moment and co-existent shear

Shear area	Av=t*D=5.7*203.2=1158.2 mm²
Shear capacity	Pv=0.6*py*Av/10^3=0.6*275*1158.2/10^3 =191.11 kN
Design shear force	Fv=F=19.303 kN

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Elastic modulus	Z=Ix/(D/20)=2340/(203.2/20)
	=230.31 cm^3

Since Fv < 0.6 Pv

Moment capacity for compact sec	Mc=py*Sx/10^3=275*258/10^3
	=70.95 kNm

Since M ≤ Mc (29.356 kNm ≤ 70.95 kNm), applied moment within moment capacity.

Maximum shear and coexistent moment

Shear capacity	Pv=191.11 kN
Design shear force	Fv=Fve=22.635 kN
Coexistent moment	M'=0 kNm

Since Fv ≤ Pv (22.635 kN ≤ 191.11 kN), shear force within shear capacity.

Check for deflection

Imposed load only considered in deflection calculation

UDL for deflection calculation	wu=wi=0 kN/m
Central UDL defln	DELw=5*wu*L^4/(384*E*Ix)*10^5
	=5*0*4.1^4/(384*205*2340)*10^5
	=0 mm
Constant for defln	const=L^3*10^5/(48*E*Ix)
	=4.1^3*10^5/(48*205*2340)
	=0.29932
Distance for defln	a=x1=1.3 m
Point load 1 for defln calc	W1=W1i=3.65 kN
Centl defln for W1	DELW1=(3*a/L-4*(a/L)^3)*W1*const
	=(3*1.3/4.1-4*(1.3/4.1)^3)*3.65 *0.29932
	=0.89993 mm
Distance for defln	a=L-x2=4.1-2.7=1.4 m
Point load 2 for defln calc	W2=W2i=5.9 kN
Centl defln for W2	DELW2=(3*a/L-4*(a/L)^3)*W2*const
	=(3*1.4/4.1-4*(1.4/4.1)^3)*5.9 *0.29932
	=1.5278 mm
Total central defln	DEL=DELw+DELW1+DELW2=0+0.89993+1.5278
	=2.4278 mm

From Table 8

Limiting deflection (brittle)	DELlim=L*1000/360=4.1*1000/360
	=11.389 mm

Since DEL ≤ DELlim (2.4278 mm ≤ 11.389 mm) OK for deflection.

SECTION	203 x 133 x 25 UKB Grade S 275	
	Shear force	22.635 kN
DESIGN	Shear capacity	191.11 kN
SUMMARY	Max. applied moment	29.356 kNm
	Moment capacity	70.95 kNm
	Buckling resistance	35.115 kNm
	Moment factor (mLT)	0.93103
	Resistance (Mb/mLT)	37.716 kNm
	Deflection	2.4278 mm
	Limiting deflection	11.389 mm
	DL shear at A	9.6655 kN
Unfactored	LL shear at A	4.5073 kN

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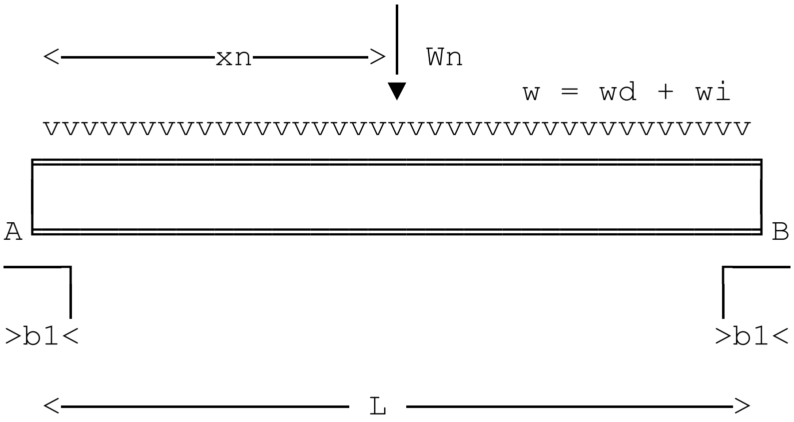
end shears	DL shear at B	10.405 kN
	LL shear at B	5.0427 kN

No436

Location: Brown Edge Rd

Simply supported steel beam subject to lateral torsional buckling

Calculations are in accordance with BS5950-1:2000. Simple beam with intermediate lateral restraints in accordance with Clause 4.3.5.2.



The n'th point load is shown:
Wn = Wnd + Wni
where suffices:
d denotes dead
i denotes imposed

The moment capacity of the section is determined from 4.2.5.2 or 4.2.5.3 and is based on the classification obtained from Table 11.

Beam span L=4.1 m

Section properties

152 x 152 x 30 UC.
Dimensions (mm): D=157.6 B=152.9 t=6.5 T=9.4 r=7.6
Properties (cm): Ix=1750 Iy=560 Sx=248 Sy=112 J=10.5
A=38.3 ry=3.8238 rx=6.7596

Strength of steel - Clause 3.1.1

The material thickness is 9.4 mm and the steel grade is S 275.
Design strength py=275 N/mm^2
Young's Modulus E=205 kN/mm^2

Loading (unfactored)

Dead UDL (including S.W)	wd=1.7 kN/m
Imposed UDL	wi=0 kN/m
Dead point load 1 (+ve down)	W1d=5 kN
Imposed point load 1	W1i=3.65 kN
Distance from L.H. supp to load 1	x1=1.3 m
Dead point load 2	W2d=8.1 kN
Imposed point load 2	W2i=5.9 kN
Distance from L.H. supp to load 2	x2=2.7 m

Factored loads

Distributed load   w'=wd*1.4+wi*1.6=1.7*1.4+0*1.6
  =2.38 kN/m
Point load 1   W1'=W1d*1.4+W1i*1.6=5*1.4+3.65*1.6
  =12.84 kN
Point load 2   W2'=W2d*1.4+W2i*1.6=8.1*1.4+5.9*1.6
  =20.78 kN

Factored shear force

At end B   Fvb=w'*L/2+(W1'*x1+W2'*x2)/L
  =2.38*4.1/2+(12.84*1.3+20.78*2.7)
  /4.1
  =22.635 kN
At end A   Fva=w'*L+W1'+W2'-Fvb
  =2.38*4.1+12.84+20.78-22.635
  =20.743 kN
Max shear is at end B   Fve=Fvb=22.635 kN

Unfactored end shears

At end B dead   Fudb=wd*L/2+(W1d*x1+W2d*x2)/L
  =1.7*4.1/2+(5*1.3+8.1*2.7)/4.1
  =10.405 kN
At end B imposed   Fuib=wi*L/2+(W1i*x1+W2i*x2)/L
  =0*4.1/2+(3.65*1.3+5.9*2.7)/4.1
  =5.0427 kN
At end A dead   Fuda=wd*L+W1d+W2d-Fudb
  =1.7*4.1+5+8.1-10.405
  =9.6655 kN
At end A imposed   Fuia=wi*L+W1i+W2i-Fuib
  =0*4.1+3.65+5.9-5.0427
  =4.5073 kN

Factored moment

Moment at load W1 (+ve sagging)   M1=Fva*x1-w'*x1^2/2
  =20.743*1.3-2.38*1.3^2/2
  =24.955 kNm
Shear to left of W1   Fl=Fva-w'*x1=20.743-2.38*1.3
  =17.649 kN
Shear to right of W1   Fr=Fva-w'*x1-W1'
  =20.743-2.38*1.3-12.84
  =4.8094 kN
Moment at load W2 (+ve sagging)   M2=Fva*x2-W1'*(x2-x1)-w'*x2^2/2
  =20.743*2.7-12.84*(2.7-1.3)-2.38
  *2.7^2/2
  =29.356 kNm
Shear to left of W2   Fl=Fva-W1'-w'*x2
  =20.743-12.84-2.38*2.7
  =1.4774 kN
Shear to right of W2   Fr=Fva-w'*x2-W1'-W2'
  =20.743-2.38*2.7-12.84-20.78
  =-19.303 kN

Maximum BM occurs at 2nd load M=M2=29.356 kNm
Corresponding shear force F=-Fr=19.303 kN

Classification - Clause 3.5.2

Classify outstand element of compression flange:
Parameter (Table 11 Note b) e=(275/py)^0.5=(275/275)^0.5
=1
Outstand b=B/2=152.9/2=76.45 mm
Ratio b'T=b/T=76.45/9.4=8.133
As b/T <= 9e (9), outstand element of compression
flange is classified as Class 1 plastic.
Classify web of section:
Depth between fillet radii d=D-2*(T+r)=157.6-2*(9.4+7.6)
=123.6 mm
Ratio d't=d/t=123.6/6.5=19.015
Neutral axis is assumed to be at mid-depth of section.
As d/t <= 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 Mb is first considered rather than the moment capacity Mc as a guide to selection.

Effective length - Clause 4.3.5.2

Beam is assumed to be laterally restraint at supports only.
Length of beam between restraints LT=L=4.1 m
In accordance with Clause 4.3.5.2 for beams with effective lateral restraints at intervals within their length subject to normal loading conditions
Effective length Le=LT=4.1 m

Clause 4.3.6.6 and Table 18

Equivalent uniform moment factor

The member is not loaded between restraints.
Maximum moment on segment Me=29 kNm
Far end BM betaM=24 kNm
Table 18 beta factor beta=betaM/Me=24/29=0.82759
Equivalent uniform moment factor mLT=TABLE 18 for beta=0.82759
=0.93103

Resistance to lateral-torsional buckling - Clause 4.3.6

Radius gyration about minor axis	ry=SQR(Iy/A)=SQR(560/38.3) =3.8238 cm
Slenderness of section	lambda=Le/ry*100=4.1/3.8238*100 =107.22
Buckling parameter	u=(4*Sx^2*(1-Iy/Ix)/(A^2*((D-T) /10)^2))^0.25 =(4*248^2*(1-560/1750)/(38.3^2 *((157.6-9.4)/10)^2))^0.25 =0.84888
Torsional index	x=0.566*((D-T)/10)*(A/J)^0.5 =0.566*((157.6-9.4)/10)*(38.3 /10.5)^0.5 =16.02
Ratio	ratio=lambda/x=107.22/16.02 =6.693
Slenderness factor	v=1/((1+0.05*ratio^2)^0.25) =1/((1+0.05*6.693^2)^0.25) =0.74537
Ratio βw	betaw=1.0
Equivalent slenderness	lamLT=u*v*lambda*(betaw)^0.5 =0.84888*0.74537*107.22*(1)^0.5 =67.843
Limiting slenderness	lamlo=0.4*((PI^2*E*10^3)/py)^0.5 =0.4*((3.1416^2*205*10^3) /275)^0.5 =34.31
Perry coefficient	etaLT=0.007*(lamLT-lamlo) =0.007*(67.843-34.31) =0.23473
Elastic strength	pe=PI^2*E*10^3/(lamLT^2) =3.1416^2*205*10^3/(67.843^2) =439.59 N/mm²
Factor	phiLT=(py+(etaLT+1)*pe)/2 =(275+(0.23473+1)*439.59)/2 =408.89 N/mm²
Factor	pey=pe*py=439.59*275=120887
Bending strength	pb=(pey)/(phiLT+((phiLT^2-pey)^0.5)) =(120887)/(408.89+((408.89^2 -120887)^0.5)) =193.71 N/mm²
Buckling resistance moment	Mb=Sx*pb/10^3=248*193.71/10^3 =48.04 kNm
Equivalent uniform moment factor	mLT=0.93103

Since M ≤ Mb/mLT (29.356 kNm ≤ 51.598 kNm), section OK for lateral torsional buckling resistance.

Check section for combined moment and shear

Maximum moment and co-existent shear

Shear area Av=t*D=6.5*157.6=1024.4 mm^2
Shear capacity Pv=0.6*py*Av/10^3=0.6*275*1024.4/10^3
=169.03 kN
Design shear force Fv=F=19.303 kN
Elastic modulus Z=Ix/(D/20)=1750/(157.6/20)
=222.08 cm^3
Since Fv < 0.6 Pv
Moment capacity for compact sec Mc=py*Sx/10^3=275*248/10^3
=68.2 kNm
Since M ≤ Mc (29.356 kNm ≤ 68.2 kNm), applied moment within moment capacity.

Maximum shear and coexistent moment

Shear capacity Pv=169.03 kN
Design shear force Fv=Fve=22.635 kN
Coexistent moment M'=0 kNm
Since Fv ≤ Pv (22.635 kN ≤ 169.03 kN), shear force within shear capacity.

Check for deflection

Imposed load only considered in deflection calculation
UDL for deflection calculation wu=wi=0 kN/m
Central UDL defln DELw=5*wu*L^4/(384*E*Ix)*10^5
=5*0*4.1^4/(384*205*1750)*10^5
=0 mm
Constant for defln const=L^3*10^5/(48*E*Ix)
=4.1^3*10^5/(48*205*1750)
=0.40024
Distance for defln a=x1=1.3 m
Point load 1 for defln calc W1=W1i=3.65 kN
Centl defln for W1 DELW1=(3*a/L-4*(a/L)^3)*W1*const
=(3*1.3/4.1-4*(1.3/4.1)^3)*3.65
*0.40024
=1.2033 mm
Distance for defln a=L-x2=4.1-2.7=1.4 m
Point load 2 for defln calc W2=W2i=5.9 kN
Centl defln for W2 DELW2=(3*a/L-4*(a/L)^3)*W2*const
=(3*1.4/4.1-4*(1.4/4.1)^3)*5.9
*0.40024
=2.0429 mm
Total central defln DEL=DELw+DELW1+DELW2=0+1.2033+2.0429
=3.2463 mm
From Table 8
Limiting deflection (brittle) DELlim=L*1000/360=4.1*1000/360
=11.389 mm
Since DEL ≤ DELlim (3.2463 mm ≤ 11.389 mm) OK for deflection.

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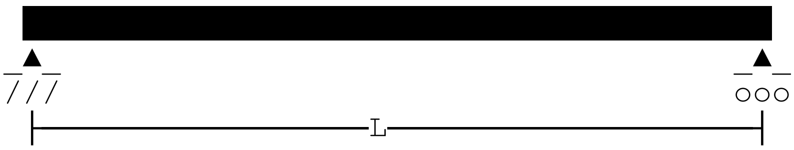
SECTION		152 x 152 x 30 UKC Grade S 275	
		Shear force	22.635 kN
DESIGN SUMMARY		Shear capacity	169.03 kN
		Max. applied moment	29.356 kNm
		Moment capacity	68.2 kNm
		Buckling resistance	48.04 kNm
		Moment factor (mLT)	0.93103
		Resistance (Mb/mLT)	51.598 kNm
		Deflection	3.2463 mm
		Limiting deflection	11.389 mm
	Unfactored end shears	DL shear at A	9.6655 kN
		LL shear at A	4.5073 kN
		DL shear at B	10.405 kN
		LL shear at B	5.0427 kN

No436

Location: Brown Edge

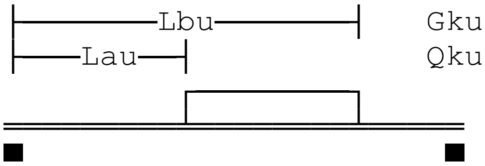
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span L=3.8 m

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



Gku Dead UDL
Qku Imposed UDL

Distances are measured from left hand support

Uniformly distributed load 1 of 1

Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=3.8 m
Dead load (unfactored)	Gku(1)=2.3 kN/m
Imposed load (unfactored)	Qku(1)=1.7 kN/m

BMs at 20th points, from left to right (sagging is positive)

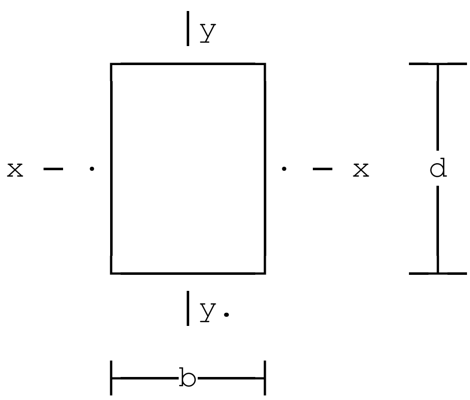
0	1.3718	2.5992	3.6822	4.6208	5.415
	6.0648	6.5702	6.9312	7.1478	7.22
	7.1478	6.9312	6.5702	6.0648	5.415
	4.6208	3.6822	2.5992	1.3718	0

Maximum span bending moment 7.22 kNm

End shears

Shear force at left hand end	7.6 kN
Shear force at right hand end	7.6 kN
Design shear force	Fve'=7.6
Unfactored dead shear at LHE	4.37 kN
Unfactored imposed shear at LHE	3.23 kN
Unfactored dead shear at RHE	4.37 kN
Unfactored imposed shear at RHE	3.23 kN

Section design parameters



Design bending moment M=7.22 kNm
Design shear force Fve=7.6 kN
Design axial load (+ve compress) Fa=0 kN
Depth of section d=250 mm
Width of section b=100 mm
Eff length for bending about xx Lex=3800 mm
Eff length for bending about yy Ley=0 mm
Length of bearing lb=75 mm
From BS5268-2 Table 18, bearing is < 75 mm from joist end.
Bearing Modification factor K4=1.0
Strength class C24 to Table 8
Timber service class adopted tmclass=1
Timber service class modification factor K2=1 as Table 16.
Modification factors:
Bending parallel to grain K2ben=1.0
Tension parallel to grain K2ten=1.0
Compression parallel to grain K2com=1.0
Compression ⊥ to grain K2per=1.0
Shear parallel to grain K2shr=1.0
Mean & min modulus of elasticity K2mod=1.0

Section properties

Inertia about xx axis Ix=b*d^3/12=100*250^3/12
=130.21E6 mm^4
Inertia about yy axis Iy=d*b^3/12=250*100^3/12
=20.833E6 mm^4
Area of cross section A=b*d=100*250=25000 mm^2
Radius of gyration about xx axis ix=SQR(Ix/A)=SQR(130.21E6/25000)
=72.169 mm
Radius of gyration about yy axis iy=SQR(Iy/A)=SQR(20.833E6/25000)
=28.868 mm

Slenderness

Slenderness ratio about xx axis lambdx=Lex/ix=3800/72.169
=52.654
Slenderness ratio about yy axis lambdy=Ley/iy=0/28.868
=0
xx axis controls slenderness at lambda=lambdx=52.654

Grade stresses

Compression parallel to grain cparg=7.9 N/mm^2
Bending parallel to grain bparg=7.5 N/mm^2
Shear parallel to grain sparg=0.71 N/mm^2
Compression perp to grain cperd=2.4 N/mm^2
Mean modulus of elasticity Emean=10800 N/mm^2
Minimum modulus of elasticity Emin=7200 N/mm^2

Modification factors

Duration of loading K3=1.25
Depth factor K7=(300/d)^0.11=(300/250)^0.11
=1.0203
Member is not in a load-sharing system as defined by Clause 2.9.
Modulus of elasticity E=Emin*K2mod=7200*1=7200 N/mm^2
Load-sharing modification factor K8=1.0
No notches exist at the support K5=1.0

Permissible stresses

Permissible bending stress sigmad=K2ben*K3*K7*K8*bparg
=1*1.25*1.0203*1*7.5
=9.5649 N/mm^2
Shear parallel to grain torad=K2shr*K3*K5*K8*sparg
=1*1.25*1*1*0.71
=0.8875 N/mm^2
Compress perp to grain (no wane) sigbad=K2per*K3*K4*K8*cperd
=1*1.25*1*1*2.4
=3 N/mm^2

Bending

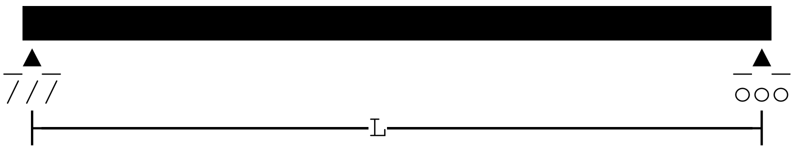
Applied bending stress sigma=M*1E6*(d/2)/Ix
=7.22*1E6*(250/2)/130.21E6
=6.9312 N/mm^2
Since sigma ≤ sigmad (6.9312 N/mm^2 ≤ 9.5649 N/mm^2) applied bending stress within permissible.

Check for deflection (including shear defln as reqd by Clause 2.10.7)

Deflection based on E=7200 N/mm^2
DL deflection without shear dld=dld=6.6609 mm
Imposed deflection without shear ild=ild=4.9232 mm
Total DL & imposed deflection 11.584 mm
Modulus of rigidity G=E/16=7200/16=450 N/mm^2
Shape factor for rect section KF=1.2
Shear area for beam Ay=d*b/KF=250*100/1.2=20833 mm^2
Total DL & imposed 15.2 kN
If total DL & imposed load applied as a UDL, additional deflection due to shear dsu=WT'*L*10^6/(8*Ay*G)
=15.2*3.8*10^6/(8*20833*450)
=0.77013 mm
Shear deflection dels=dsu*M/(WT'*L/8)
=0.77013*7.22/(15.2*3.8/8)
=0.77013 mm
Limiting deflection DELlim=0.003*L*10^3=0.003*3.8*10^3
=11.4 mm
Deflection inclusive of shear DEL=dld+ild+dels
=6.6609+4.9232+0.77013
=12.354 mm
Since DEL > DELlim (12.354 mm > 11.4 mm), deflection exceeds limiting value.

Location: Brown Edge

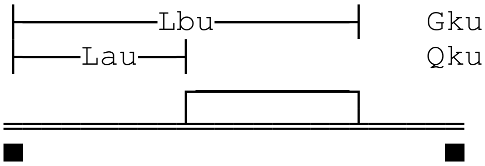
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span L=3.8 m

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



Gku Dead UDL
Qku Imposed UDL

Distances are measured from left hand support

Uniformly distributed load 1 of 1

Dist. from left support to start Lau(1)=0 m
Distance from left support to end Lbu(1)=3.8 m
Dead load (unfactored) Gku(1)=1.4 kN/m
Imposed load (unfactored) Qku(1)=1.0 kN/m

BMs at 20th points, from left to right (sagging is positive)

0	0.82308	1.5595	2.2093	2.7725	3.249
	3.6389	3.9421	4.1587	4.2887	4.332
	4.2887	4.1587	3.9421	3.6389	3.249
	2.7725	2.2093	1.5595	0.82308	0

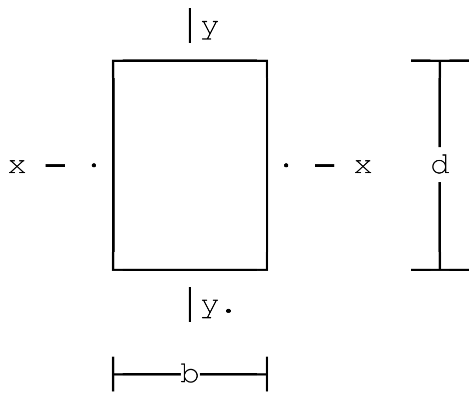
Maximum span bending moment 4.332 kNm

End shears

Shear force at left hand end 4.56 kN
Shear force at right hand end 4.56 kN
Design shear force Fve'=4.56

Unfactored dead shear at LHE 2.66 kN
Unfactored imposed shear at LHE 1.9 kN
Unfactored dead shear at RHE 2.66 kN
Unfactored imposed shear at RHE 1.9 kN

Section design parameters



Design bending moment	M=4.332 kNm
Design shear force	Fve=4.56 kN
Design axial load (+ve compress)	Fa=0 kN
Depth of section	d=225 mm
Width of section	b=75 mm
Eff length for bending about xx	Lex=3800 mm
Eff length for bending about yy	Ley=0 mm
Length of bearing	lb=75 mm

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor	K4=1.0
Strength class C24 to Table 8	
Timber service class adopted	tmclass=1

Timber service class modification factor K2=1 as Table 16.

Modification factors:

Bending parallel to grain	K2ben=1.0
Tension parallel to grain	K2ten=1.0
Compression parallel to grain	K2com=1.0
Compression ⊥ to grain	K2per=1.0
Shear parallel to grain	K2shr=1.0
Mean & min modulus of elasticity	K2mod=1.0

Section properties

Inertia about xx axis	Ix=b*d^3/12=75*225^3/12=71.191E6 mm^4
Inertia about yy axis	Iy=d*b^3/12=225*75^3/12=7.9102E6 mm^4
Area of cross section	A=b*d=75*225=16875 mm^2
Radius of gyration about xx axis	ix=SQR(Ix/A)=SQR(71.191E6/16875) =64.952 mm
Radius of gyration about yy axis	iy=SQR(Iy/A)=SQR(7.9102E6/16875) =21.651 mm

Slenderness

Slenderness ratio about xx axis	lambdx=Lex/ix=3800/64.952 =58.505
Slenderness ratio about yy axis	lambdy=Ley/iy=0/21.651 =0
xx axis controls slenderness at	lambda=lambdx=58.505

Grade stresses

Compression parallel to grain	cparg=7.9 N/mm^2
Bending parallel to grain	bparg=7.5 N/mm^2
Shear parallel to grain	sparg=0.71 N/mm^2
Compression perp to grain	cperd=2.4 N/mm^2
Mean modulus of elasticity	Emean=10800 N/mm^2
Minimum modulus of elasticity	Emin=7200 N/mm^2

Modification factors

Duration of loading K3=1.25
Depth factor K7=(300/d)^0.11=(300/225)^0.11
=1.0322
Member is not in a load-sharing system as defined by Clause 2.9.
Modulus of elasticity E=Emin*K2mod=7200*1=7200 N/mm²
Load-sharing modification factor K8=1.0
No notches exist at the support K5=1.0

Permissible stresses

Permissible bending stress sigmad=K2ben*K3*K7*K8*bparg
=1*1.25*1.0322*1*7.5
=9.6764 N/mm²
Shear parallel to grain torad=K2shr*K3*K5*K8*sparg
=1*1.25*1*1*0.71
=0.8875 N/mm²
Compress perp to grain (no wane) sigbad=K2per*K3*K4*K8*cperd
=1*1.25*1*1*2.4
=3 N/mm²

Bending

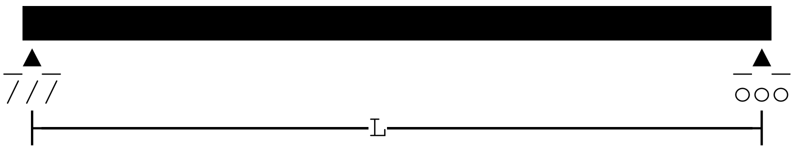
Applied bending stress sigma=M*1E6*(d/2)/Ix
=4.332*1E6*(225/2)/71.191E6
=6.8456 N/mm²
Since sigma ≤ sigmad (6.8456 N/mm² ≤ 9.6764 N/mm²) applied bending stress within permissible.

Check for deflection (including shear defln as reqd by Clause 2.10.7)

Deflection based on E=7200 N/mm²
DL deflection without shear dld=dld=7.4155 mm
Imposed deflection without shear ild=ild=5.2968 mm
Total DL & imposed deflection 12.712 mm
Modulus of rigidity G=E/16=7200/16=450 N/mm²
Shape factor for rect section KF=1.2
Shear area for beam Ay=d*b/KF=225*75/1.2=14062 mm²
Total DL & imposed 9.12 kN
If total DL & imposed load applied as a UDL, additional deflection due to shear dsu=WT'*L*10^6/(8*Ay*G)
=9.12*3.8*10^6/(8*14062*450)
=0.68456 mm
Shear deflection dels=dsu*M/(WT'*L/8)
=0.68456*4.332/(9.12*3.8/8)
=0.68456 mm
Limiting deflection DELlim=0.003*L*10^3=0.003*3.8*10^3
=11.4 mm
Deflection inclusive of shear DEL=dld+ild+dels
=7.4155+5.2968+0.68456
=13.397 mm
Since DEL > DELlim (13.397 mm > 11.4 mm), deflection exceeds limiting value.

Location: Brown Edge

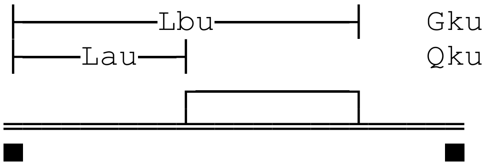
Timber beam to BS5268-2:2002



Simply supported beam subjected to vertical loads.

Beam span L=2.7 m

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



Gku Dead UDL
Qku Imposed UDL

Distances are measured from left hand support

Uniformly distributed load 1 of 1

Dist. from left support to start	Lau(1)=0 m
Distance from left support to end	Lbu(1)=2.7 m
Dead load (unfactored)	Gku(1)=1.15 kN/m
Imposed load (unfactored)	Qku(1)=3.45 kN/m

BMs at 20th points, from left to right (sagging is positive)

0	0.79643	1.509	2.1378	2.6827	3.1438
	3.5211	3.8145	4.0241	4.1498	4.1918
	4.1498	4.0241	3.8145	3.5211	3.1438
	2.6827	2.1378	1.509	0.79643	0

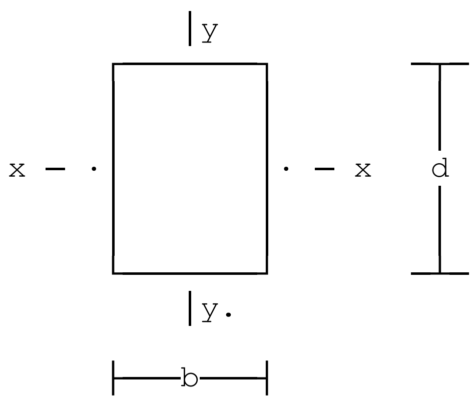
Maximum span bending moment 4.1918 kNm

End shears

Shear force at left hand end	6.21 kN
Shear force at right hand end	6.21 kN
Design shear force	Fve'=6.21

Unfactored dead shear at LHE	1.5525 kN
Unfactored imposed shear at LHE	4.6575 kN
Unfactored dead shear at RHE	1.5525 kN
Unfactored imposed shear at RHE	4.6575 kN

Section design parameters



Design bending moment	M=4.1918 kNm
Design shear force	Fve=6.21 kN
Design axial load (+ve compress)	Fa=0 kN
Depth of section	d=225 mm
Width of section	b=75 mm
Eff length for bending about xx	Lex=2700 mm
Eff length for bending about yy	Ley=0 mm
Length of bearing	lb=75 mm

From BS5268-2 Table 18, bearing is < 75 mm from joist end.

Bearing Modification factor	K4=1.0
Strength class C24 to Table 8	
Timber service class adopted	tmclass=1

Timber service class modification factor K2=1 as Table 16.

Modification factors:

Bending parallel to grain	K2ben=1.0
Tension parallel to grain	K2ten=1.0
Compression parallel to grain	K2com=1.0
Compression ⊥ to grain	K2per=1.0
Shear parallel to grain	K2shr=1.0
Mean & min modulus of elasticity	K2mod=1.0

Section properties

Inertia about xx axis	Ix=b*d^3/12=75*225^3/12=71.191E6 mm^4
Inertia about yy axis	Iy=d*b^3/12=225*75^3/12=7.9102E6 mm^4
Area of cross section	A=b*d=75*225=16875 mm^2
Radius of gyration about xx axis	ix=SQR(Ix/A)=SQR(71.191E6/16875) =64.952 mm
Radius of gyration about yy axis	iy=SQR(Iy/A)=SQR(7.9102E6/16875) =21.651 mm

Slenderness

Slenderness ratio about xx axis	lambdx=Lex/ix=2700/64.952 =41.569
Slenderness ratio about yy axis	lambdy=Ley/iy=0/21.651 =0
xx axis controls slenderness at	lambda=lambdx=41.569

Grade stresses

Compression parallel to grain	cparg=7.9 N/mm^2
Bending parallel to grain	bparg=7.5 N/mm^2
Shear parallel to grain	sparg=0.71 N/mm^2
Compression perp to grain	cperd=2.4 N/mm^2
Mean modulus of elasticity	Emean=10800 N/mm^2
Minimum modulus of elasticity	Emin=7200 N/mm^2

Modification factors

Duration of loading K3=1.00
Depth factor K7=(300/d)^0.11=(300/225)^0.11
=1.0322
Member is not in a load-sharing system as defined by Clause 2.9.
Modulus of elasticity E=Emin*K2mod=7200*1=7200 N/mm^2
Load-sharing modification factor K8=1.0
No notches exist at the support K5=1.0

Permissible stresses

Permissible bending stress sigmad=K2ben*K3*K7*K8*bparg
=1*1*1.0322*1*7.5
=7.7411 N/mm^2
Shear parallel to grain torad=K2shr*K3*K5*K8*sparg
=1*1*1*1*0.71
=0.71 N/mm^2
Compress perp to grain (no wane) sigbad=K2per*K3*K4*K8*cperd
=1*1*1*1*2.4
=2.4 N/mm^2

Bending

Applied bending stress sigma=M*1E6*(d/2)/Ix
=4.1918*1E6*(225/2)/71.191E6
=6.624 N/mm^2
Since sigma ≤ sigmad (6.624 N/mm^2 ≤ 7.7411 N/mm^2) applied bending stress within permissible.

Check for deflection (including shear defln as reqd by Clause 2.10.7)

Deflection based on E=7200 N/mm^2
DL deflection without shear dld=dld=1.5525 mm
Imposed deflection without shear ild=ild=4.6575 mm
Total DL & imposed deflection 6.21 mm
Modulus of rigidity G=E/16=7200/16=450 N/mm^2
Shape factor for rect section KF=1.2
Shear area for beam Ay=d*b/KF=225*75/1.2=14062 mm^2
Total DL & imposed 12.42 kN
If total DL & imposed load applied as a UDL, additional deflection
due to shear dsu=WT'*L*10^6/(8*Ay*G)
=12.42*2.7*10^6/(8*14062*450)
=0.6624 mm
Shear deflection dels=dsu*M/(WT'*L/8)
=0.6624*4.1918/(12.42*2.7/8)
=0.6624 mm
Limiting deflection DELlim=0.003*L*10^3=0.003*2.7*10^3
=8.1 mm
Deflection inclusive of shear DEL=dld+ild+dels=1.5525+4.6575+0.6624
=6.8724 mm
Since DEL ≤ DELlim (6.8724 mm ≤ 8.1 mm), OK for deflection.

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Shear and bearing

Shear stress (no notches) tora=3*Fve*1000/(2*b*d)
=3*6.21*1000/(2*75*225)
=0.552 N/mm²

Since tora ≤ torad (0.552 N/mm² ≤ 0.71 N/mm²) shear stress does not exceed permissible therefore OK.

Bearing stress on wall plate sigba=Fve*1000/(lb*b)
=6.21*1000/(75*75)
=1.104 N/mm²

Since sigba ≤ sigbad (1.104 N/mm² ≤ 2.4 N/mm²) bearing stress does not exceed permissible therefore OK.

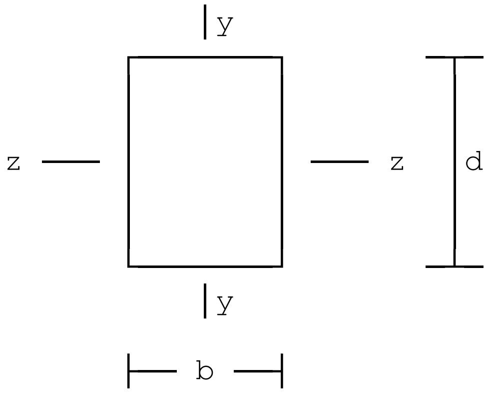
DESIGN SUMMARY

Member: 225 mm x 75 mm
Strength class C24 to Table 8
Moisture service class 1

Bending stress	6.624 N/mm²
Permissible bending	7.7411 N/mm²
Deflection	6.8724 mm
Limiting deflection	8.1 mm
Shear stress	0.552 N/mm²
Permissible shear	0.71 N/mm²
Bearing stress	1.104 N/mm²
Permissible bearing	2.4 N/mm²

No254

Location: Brown Edge



Rectangular timber member subject to axial load and bending about z-z axis

Calculations in accordance with BS5268-2:2002.

Design BM about zz (positive)	Mz=1.5 kNm
Design shear force in y direction	V=0 kN
Design axial load (+ve compress)	F=15.5 kN
Member is in compression.	
Eff length for bending about zz	Lez'=1.800 m
Depth of section	d=150 mm
Width of section	b=150 mm
	Lez=Lez'*1000=1.8*1000=1800 mm
Eff length for bending about yy	Ley'=1.8 m
	Ley=Ley'*1000=1.8*1000=1800 mm
Strength class C24 to Table 8	

Section properties

Largest section dimension	h=b=150 mm
Inertia about zz axis	Iz=b*d^3/12=150*150^3/12 =42.188E6 mm^4
Inertia about yy axis	Iy=d*b^3/12=150*150^3/12 =42.188E6 mm^4
Area of cross section	A=b*d=150*150=22500 mm^2
Radius of gyration about zz axis	iz=SQR(Iz/A)=SQR(42.188E6/22500) =43.301 mm
Radius of gyration about yy axis	iy=SQR(Iy/A)=SQR(42.188E6/22500) =43.301 mm

Slenderness

Slenderness ratio about zz axis	lambdz=Lez/iz=1800/43.301 =41.569
Slenderness ratio about yy axis	lambdy=Ley/iy=1800/43.301 =41.569
yy axis controls SR	lambda=lambdy=41.569

Grade stresses

Compression parallel to grain cparg=7.9 N/mm²
Bending parallel to grain bparg=7.5 N/mm²
Shear parallel to grain sparg=0.71 N/mm²
Mean modulus of elasticity Emean=10800 N/mm²
Minimum modulus of elasticity Emin=7200 N/mm²

Modification factors

Timber service class adopted tmclass=3
Timber stress grades and moduli adjusted as table 16.
Modification factors:
Bending parallel to grain K2ben=0.8
Tension parallel to grain K2ten=0.8
Compression parallel to grain K2com=0.6
Compression ⊥ to grain K2per=0.6
Shear parallel to grain K2shr=0.9
Mean & min modulus of elasticity K2mod=0.8
Duration of loading K3=1.00
Depth factor K7=(300/d)^0.11=(300/150)^0.11
=1.0792
Member is not in a load-sharing system as defined by Clause 2.9.
Modulus of elasticity E=Emin*K2mod=7200*0.8=5760 N/mm²
Load-sharing modification factor K8=1.0

For compression members with slenderness ratios equal to or greater than 5, the permissible stress should be calculated as the product of the grade compression parallel to the grain stress, modified as appropriate for size, moisture content, duration of load and load sharing, and the modification factor K12 given by table, or calculated as follows using the equation in annex B. The value of modulus of elasticity should be the minimum modulus of elasticity. The compression parallel to the grain stress required for entry to annex B should be the grade stress modified only for moisture content and duration of load.

Compr. parallel to grain stress sigc=K2com*K3*cparg=0.6*1*7.9
=4.74 N/mm²
Modulus of elasticity E=Emin*K2mod=7200*0.8=5760 N/mm²
Factor for annex B C=PI^2*E/(1.5*lambda^2*sigc)
=3.1416^2*5760/(1.5*41.569^2*4.74)
=4.6271
Eccentricity factor eta=0.005*lambda=0.005*41.569
=0.20785
Modification factor K12=(0.5+(1+eta)*C/2)-((0.5+(1+eta)*C/2)^2-C)^0.5
=(0.5+(1+0.20785)*4.6271/2)-((0.5+(1+0.20785)
*4.6271/2)^2-4.6271)^0.5
=0.79921

Permissible stresses

Permissible compression stress sigcad=K2com*K3*K8*K12*cparg
=0.6*1*1*0.79921*7.9
=3.7882 N/mm^2
Permissible bending stress sigmad=K2ben*K3*K7*K8*bparg
=0.8*1*1.0792*1*7.5
=6.4754 N/mm^2

Axial compression

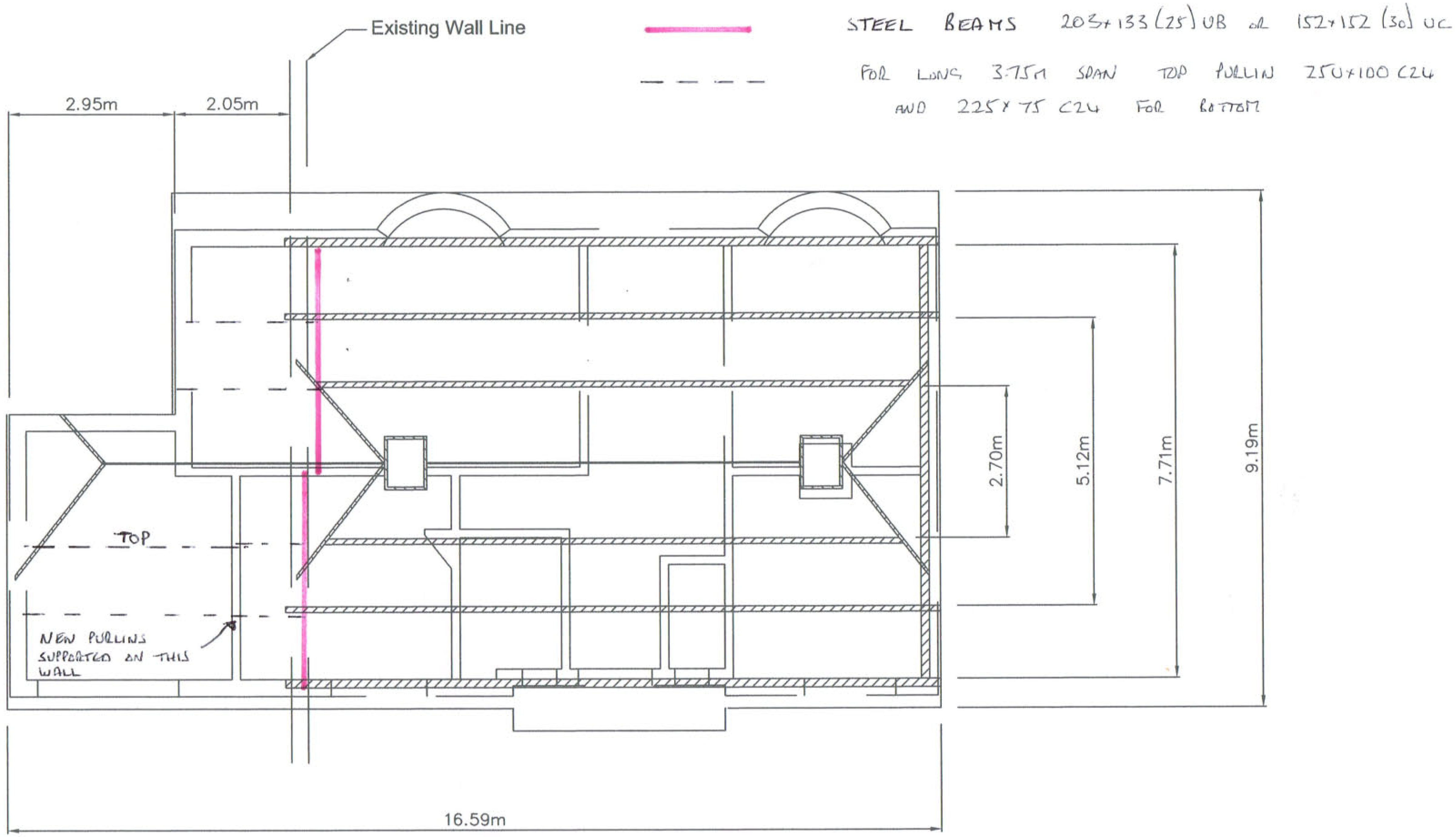
The limitation on bow in most stress grading rules are inadequate for the selection of material for columns. Particular attention should therefore be paid to the straightness of columns, e.g. by limiting bow to approximately 1/300 of the length.
Applied compression stress sigca=F*1000/A=15.5*1000/22500
=0.68889 N/mm^2

Bending

Applied bending stress sigma=Mz*1E6*(d/2)/Iz
=1.5*1E6*(150/2)/42.188E6
=2.6667 N/mm^2
Euler critical stress sige=PI^2*E/lambda^2
=3.1416^2*5760/41.569^2
=32.899 N/mm^2
Bending stress modification fact bsmf=1-1.5*sigca*K12/sige
=1-1.5*0.68889*0.79921/32.899
=0.9749
Interaction factor factor=sigma/(sigmad*bsmf)+sigca
/sigcad
=2.6667/(6.4754*0.9749)
+0.68889/3.7882
=0.60427
Interaction factor less than or equal to unity therefore OK.

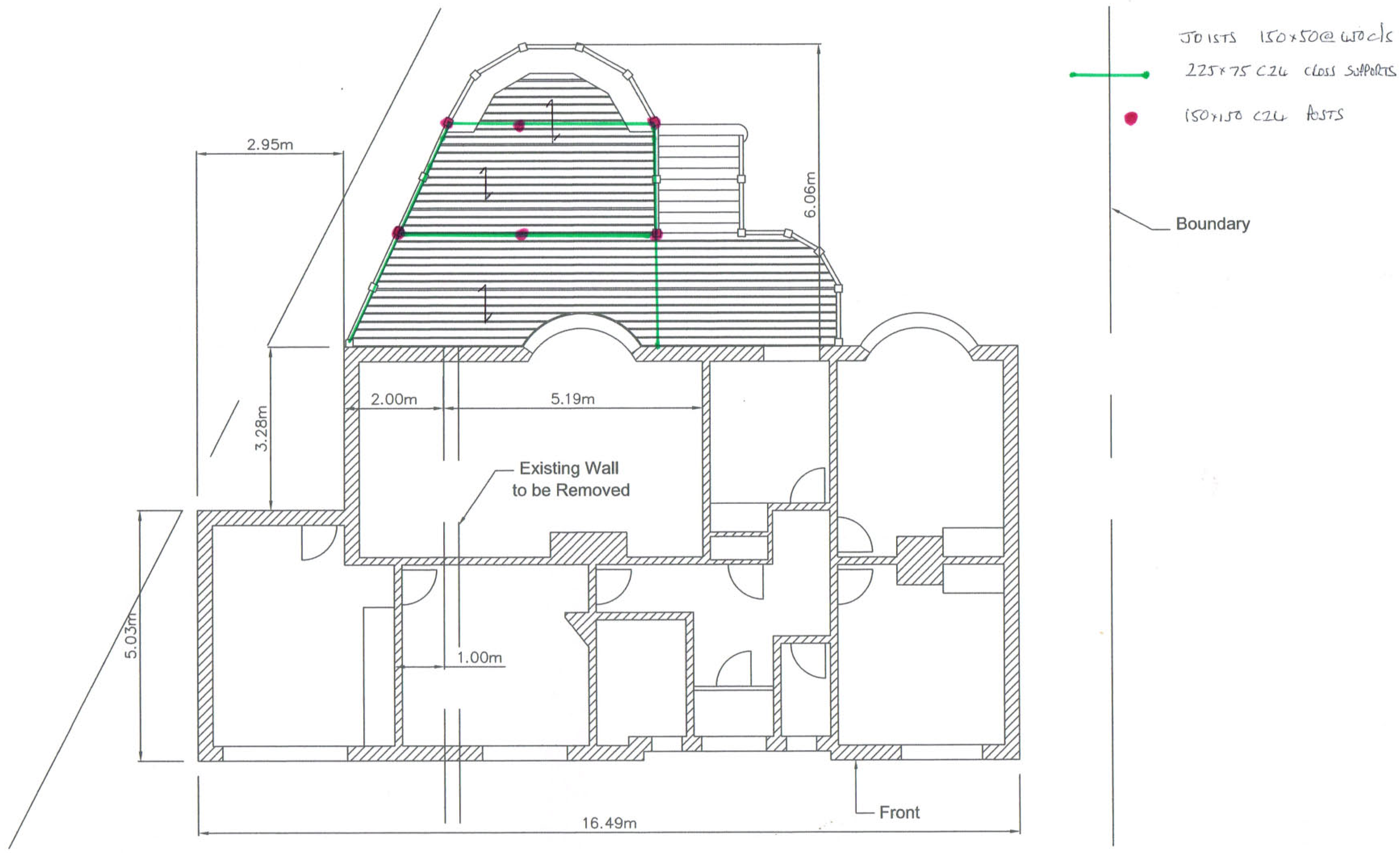
DESIGN SUMMARY	Depth of section	150 mm
	Width of section	150 mm
	Strength class C24 to Table 8	
	Timber moisture class	3
	Applied comprn stress	0.68889 N/mm^2
	Permiss comprn stress	3.7882 N/mm^2
	Applied bending stress	2.6667 N/mm^2
	Permiss bending stress	6.4754 N/mm^2
	Interaction factor	0.60427

No250



Proposed Roof Contruction

SCALE: N.T.S	MAT'L: N/A	CLIENT: 67 Brown Edge Road, Buxton, Derbyshire, SK17 7AF	
PAT. No:	CLASS:	DWG TITLE: 67 Brown Edge Road, Proposed - Roof Contruction	REVISION: 0
DRAWN: GS	DATE: 010114	SIZE: A3	DRG. No: As_Built_010114
CHECKED:	DATE:	CONTRACT No:	SHEET 6 OF 7
APPR'VD:	DATE:		



Proposed Plan View

SCALE:	N.T.S	MAT'L:	N/A
PAT. No:		CLASS:	
DRAWN:	GS	DATE:	010114
CHECKED:		DATE:	
APPR'VD:		DATE:	

CLIENT: 67 Brown Edge Road, Buxton, Derbyshire, SK17 7AF

DWG TITLE: 67 Brown Edge Road, Proposed - Plan View

REVISION: 0

SIZE: A3

DRG. No: As_Built_010114

CONTRACT No:

SHEET 5 OF 7

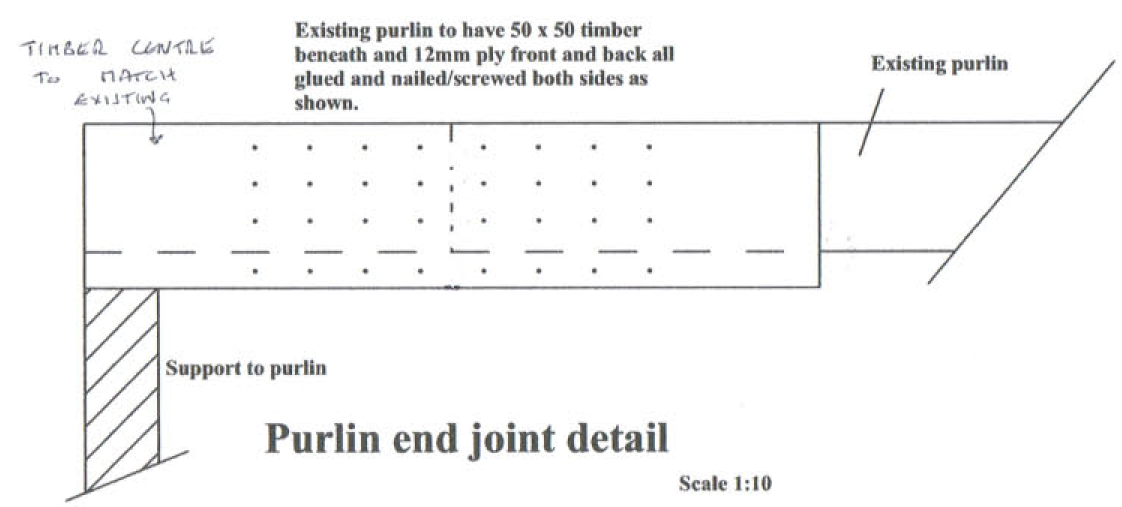

TIMBER CENTRE TO MATCH EXISTING
Existing purlin to have 50 x 50 timber beneath and 12mm ply front and back all glued and nailed/screwed both sides as shown.
Existing purlin
Support to purlin
Purlin end joint detail
Scale 1:10

FIGURE 1