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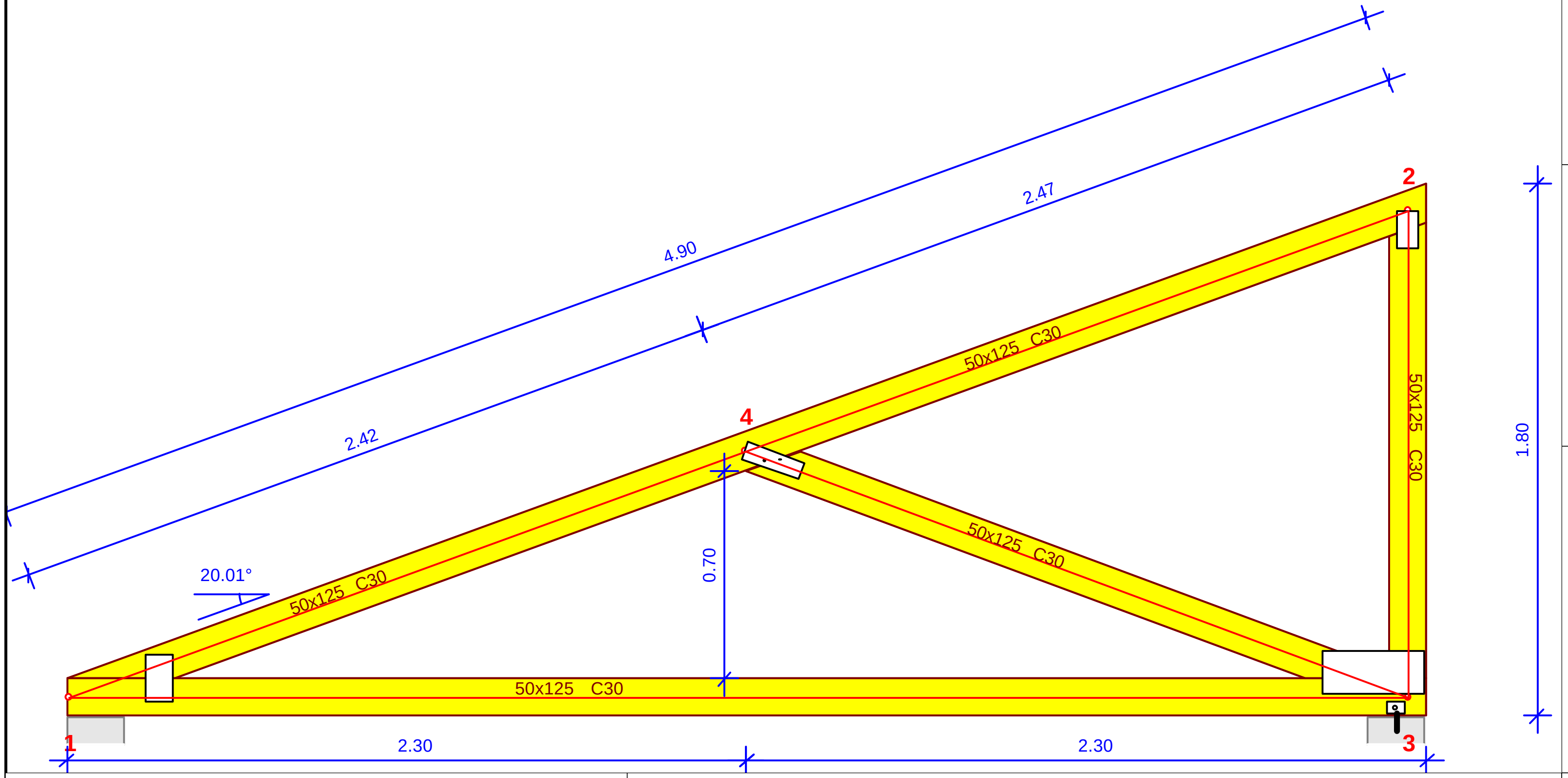

General information

Timber class for trusses C30

Truss spacing C/C 0.60 m

Purlins C30, 50x50 mm, at C/C 0.30 m

Service classes (EN1995-1-1, §2.3.1.3): Class 2

Material factor: 1.30 (EC5 EN1995-1-1:2009, Table 2.3)

Truss volume =0.084 m³

Design codes

EN1990-1-1:2002 Basis of structural design

EN1991-1-1:2003 Actions on structures

EN1991-1-3:2003 Snow loads

EN1991-1-4:2005 Wind actions

EN1995-1-1:2009 Design of timber structures

Distributed roof loads

Permanent load of roof covering	0.100 kN/m²
Purlins, finishing, insulation	0.100 kN/m²
Load of ceiling under the roof	0.300 kN/m²
Snow load on the ground	2.000 kN/m²
Wind pressure on vertical surface	1.000 kN/m²

Truss elements

elem	size	class	length(L)	(Lmax)
El 1-2 :	50x125	C30	L1-2 =4.82 m	Lmax =4.90 m
El 1-3 :	50x125	C30	L1-3 =4.53 m	Lmax =4.60 m
El 2-3 :	50x125	C30	L2-3 =1.65 m	Lmax =1.80 m
El 3-4 :	50x125	C30	L3-4 =2.39 m	Lmax =2.21 m

Connection plates

node	type	size (BxL)mm	nails
Nd 2 :	Steel plate 2.0mm	2x80x135mm	Nails 4.0/35 :8 [4+4]
Nd 1 :	Steel plate 2.0mm	2x95x165mm	Nails 4.0/35 :10 [5+5]
Nd 4 :	Steel plate 2.0mm	2x200x60mm	Nails 4.0/35 :8 [4+4]
Nd 3 :	Steel plate 2.0mm	2x350x155mm	Nails 4.0/35 :14 [4+6+4]

Project: Example of Monopitch roof

ROOF -002

Scale : **1:25** **Date:** 09/09/2011

Designer: **Draw.No.:**

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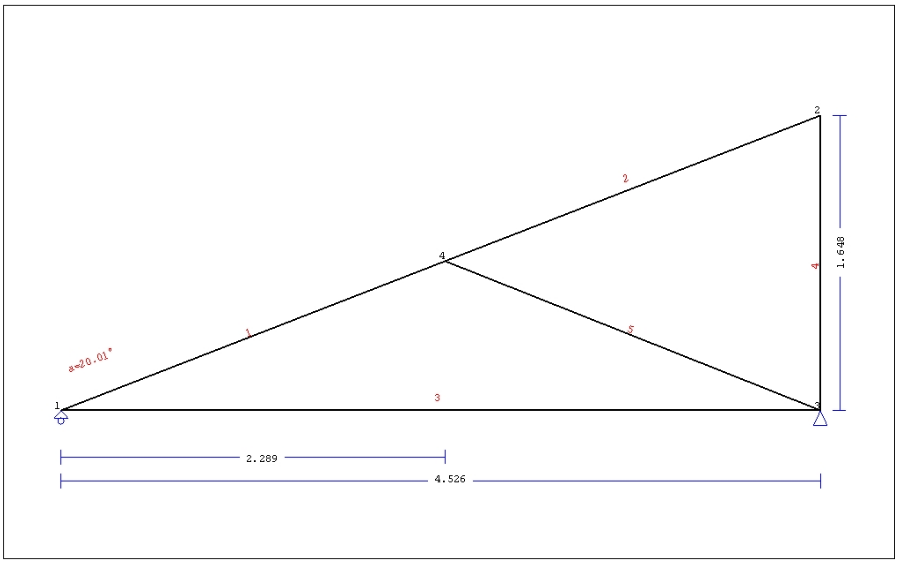
Example of Monopitch roof

1. ROOF -002

Trussed rafter (monopitch) roof type N



1.1. General description, assumptions, materials, loads

1.1.1. Construction type

Timber roof, from trusses with timber C30. The truss type as sketch above.
Truss span 4.526 m, height 1.648 m, roof pitch 20.01°, truss spacing 0.600m
Purlins from timber C30, with dimensions 50x50 mm, in spacing 0.300 m
Truss element cross sections BxH [mm]
Elements 1, 2, cross section 50x125 [mm]
Elements 3, cross section 50x125 [mm]
Elements 4, cross section 50x125 [mm]
Elements 5, cross section 50x125 [mm]
Truss volume =0.084 m^3, truss weight =0.312 kN

1.1.2. Design codes

EN1990-1-1:2002, Eurocode 0 Part 1-1, Basis of structural design
EN1991-1-1:2003, Eurocode 1 Part 1-1, Actions on structures
EN1991-1-3:2003, Eurocode 1 Part 1-3, Snow loads
EN1991-1-4:2005, Eurocode 1 Part 1-4, Wind actions
EN1995-1-1:2009, Eurocode 5 Part 1-1, Design of timber structures

1.1.3. Design methodology

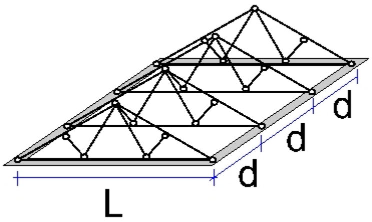
The internal forces of the roof trusses are computed with finite element analysis. The truss is considered as a two dimensional frame. The stiffness of the connections is adjusted according to the selected degree of stiffness. In order to compute the design values for internal forces in various loading conditions, the internal forces are first computed in unit loading, and then from their combination the internal forces in various loading conditions are obtained.
All the load combinations according to Eurocode 1 and Eurocode 5 are taken into account, and the checks are performed in the most unfavourable loading conditions, for combined action, in ultimate limit state, according to EC5 EN1995-1-1:2009, §6. The connections are designed as nailed connections with metal plates according to EC5 EN1995-1-1:2009, §8.
The deflections are checked in serviceability limit condition, according to EC5 EN1995-1-1:2009, §7.

1.1.4. Material properties (truss, purlins) (EC5 EN1995-1-1:2009, §3)

Timber class : C30
Service classes : Class 1, moisture content<=12% (EC5 §2.3.1.3)
Material factor γM=1.30 (EC5 Table 2.3)
Characteristic material properties for timber
fmk = 30.0 MPa, ft0k = 18.0 MPa, ft90k= 0.4 MPa
fc0k= 23.0 MPa, fc90k= 2.7 MPa, fvk = 4.0 MPa
E0m =12000 MPa, E005 = 8000 MPa, E90m = 400 MPa
Gm = 750 MPa, ρk = 380 Kg/m^3

1.1.5. Distributed roof loads

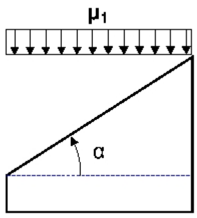
Permanent load of roof covering	Ge= 0.100 kN/m^2	(Thatch c
Purlins, finishing, insulation	Gt= 0.100 kN/m^2	Ge+Gt= 0.:
Load of ceiling under the roof	Gc= 0.300 kN/m^2	
Snow load on the ground	Sk= 2.000 kN/m^2	
Wind pressure on vertical surface	Qw= 1.000 kN/m^2	
Imposed load (category H)	Qi= 0.400 kN/m^2	



1.2. Snow load (EC1 EN1991-1-3:2003, §5)

Characteristic value of snow load on the ground: sk=2.000 kN/m^2

Snow load on the roof (EC1 EN1991-1-3:2003, §5)
Angle of pitch of roof : α=20.008°
Exposure coefficient : Ce=1.000
Thermal coefficient : Ct=1.000
Shape factors, α=20.01°, μ1=0.800 (Table 5.2)
Snow is prevented from sliding off the roof , μ1(α1)=0.800

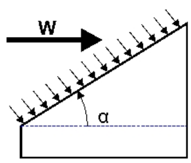


Snow load (EC1 EN1991-1-3:2003, §5.3.2)
S1=μ1.Ce·Ct·Sk=0.800x1.000x1.000x2.000=1.600 kN/m^2

1.3. Wind loading (EC1 EN1991-1-4:2005 §5)

Pick velocity pressure Q(z)=Qref·Ce(z), Qref=Vref^2/1.6 (EC1 EN1991-1-4:2005 §4.5)
Wind pressure on vertical surface Qref.Ce(z)= 1.000 kN/m^2

Wind pressure on roof we=Qref·Ce(z).Cpe (EC1 EN1991-1-4:2005, §5.2)
External pressure coefficients (EC1 EN1991-1-4:2005 Table 7.4)
For pitch angle α=20.01°, Cpe=0.27
Wind pressure we=0.267 kN/m^2



1.4. Design of purlins

Structural system for purlins

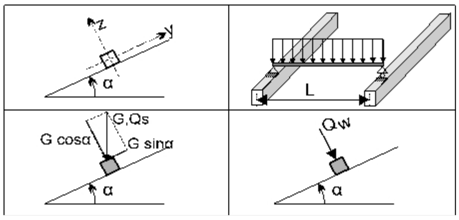
The purlins are designed as simply supported beams with span length L=0.600m the distance between the trusses. They are loaded with a surface load of width L1=0.300m (purlin spacing). The purlin axis has inclination a=20.01° with the vertical. The vertical loads (self weight, snow, concentrated load) are decomposed in two components in the directions z-z P.cosa, and y-y P.sina, the wind load acts in the z-z direction.

Dimensions of purlins

Timber of purlins: C30, Class 1, moisture content<=12%, cross section of purlins BxH:50x50mm
Spacing of purlins L1=0.300m, roof pitch α=20.01°, spacing of trusses L=0.600m.

Uniform loading of purlins kN/m²

Roof covering	Ge=	0.100 kN/m²
Finishing+self weight	G1=	0.100 kN/m²
Snow load	Qs=	1.600 kN/m²
Wind load	Qw=	0.267 kN/m²
Concentrated load	Qp=	1.000 kN



Line loading of purlins (kN/m) in z-z and y-y

Roof covering+self weight	Gk = 0.060 kN/m,	Gkz = 0.056 kN/m,	Gkez= 0.021 kN/m
Snow load	Qks= 0.480 kN/m,	Qksz= 0.451 kN/m,	Qksz= 0.164 kN/m
Wind load	Qkw= 0.080 kN/m,	Qkwz= 0.080 kN/m,	Qkwy= 0.000 kN/m
Concentrated load	Qkp= 1.000 kN,	Qkpz= 0.940 kN,	Qkpz= 0.342 kN

Internal forces of purlins (span L=0.600 m, BxH: 50x50 mm)

Loading			action	γg	γq	ψo	Qz[kN]	Qy[kN]	My[kNm]	Mz[kNm]
(Gk)	Permanent	Gk =0.060[kN/m]	Permanent	1.35	0.00	1.00	0.017	0.006	0.003	0.001
(Qk1)	Snow	Qks=0.480[kN/m]	Short-term	0.00	1.50	0.60	0.135	0.049	0.020	0.007
(Qk2)	Wind	Qkw=0.080[kN/m]	Short-term	0.00	1.50	0.50	0.024	0.000	0.004	0.000
(Qk3)	Concentr.	Qkp=1.000[kN]	Instantaneous	0.00	1.00	0.00	0.470	0.171	0.141	0.051

1.4.1. Serviceability limit state (EC5 EN1995-1-1:2009, §2.2.3, §7)

Control of deflection (EC5 §7.2)

Loading	[kN/m]		u[mm]	action	ψ0	ψ1	ψ2	Kdef
(Gk)	Permanent	Gk =0.056[kN/m]	0.006	Permanent	1.00	1.00	1.00	0.60
(Qk1)	Snow	Qks=0.451[kN/m]	0.051	Short-term	0.60	0.20	0.00	0.60
(Qk2)	Wind	Qkw=0.080[kN/m]	0.009	Short-term	0.50	0.20	0.00	0.60

Load combination	w.inst	w.fin [mm]
1 Gk	0.006	0.010
2 Gk + Qk1	0.057	0.061
3 Gk + Qk2	0.015	0.019
4 Gk + Qk1 + ψo.Qk2	0.061	0.065
5 Gk + Qk2 + ψo.Qk1	0.046	0.049

w.fin,g=w.inst,g(1+kdef), w.fin,q=w.inst,q(1+ψ2·kdef)(EC5 §2.2.3, Eq.2.3, Eq.2.4)

Maximum deflection values

w.inst = 0.061 mm, w.fin = 0.065 mm

Check according to EC5 EN1995-1-1:2009 §7.2, Tab.7.2

Final deflections

w.inst = 0.061 mm < L/300=600/300= 2.000 mm
w.net,fin = 0.065 mm < L/250=600/250= 2.400 mm
w.fin = 0.065 mm < L/150=600/150= 4.000 mm
The check is satisfied

1.4.2. Check of purlins, Ultimate limit state of design (EC5 EN1995-1-1:2009, §6)

L.C.	Load combination	duration class	kmod	Qz/Kmod	Qy/Kmod	My/Kmod	Mz/Kmod
1	γg.Gk	Permanent	0.60	0.038	0.014	0.006	0.002
2	γg.Gk + γq.Qk1	Short-term	0.90	0.251	0.091	0.038	0.014
3	γg.Gk + γq.Qk2	Short-term	0.90	0.065	0.009	0.010	0.001
4	γg.Gk + γq.Qk3	Instantaneous	1.10	0.448	0.163	0.131	0.048
5	γg.Gk + γq.Qk1 + γq.ψo.Qk2	Short-term	0.90	0.271	0.091	0.041	0.014
6	γg.Gk + γq.Qk2 + γq.ψo.Qk1	Short-term	0.90	0.201	0.059	0.030	0.009
	Maximum values			0.448	0.163	0.131	0.048

Purlin, load combination No 4

Shear, Fv=0.493 kN (EC5 §6.1.7)

Rectangular cross section, bef=0.67x50=34 mm, h=50 mm, A= 1 700 mm²
Modification factor Kmod=1.10 (Table 3.1), material factor γM=1.30 (Table 2.3)
fvk=4.00 N/mm², fvd=Kmod·fvk/γM=1.10x4.00/1.30=3.38N/mm² (EC5 Eq.2.14)
Fv=0.493 kN, τv0d=1.50Fv0d/Anetto=1000x1.50x0.493/1700=0.43N/mm² < 3.38N/mm²=fv0d (Eq.6.13)
The check is satisfied

Purlin, load combination No 4

Shear, Fv=0.179 kN (EC5 §6.1.7)

Rectangular cross section, bef=0.67x50=34 mm, h=50 mm, A= 1 700 mm²
Modification factor Kmod=1.10 (Table 3.1), material factor γM=1.30 (Table 2.3)
fvk=4.00 N/mm², fvd=Kmod·fvk/γM=1.10x4.00/1.30=3.38N/mm² (EC5 Eq.2.14)
Fv=0.179 kN, τv0d=1.50Fv0d/Anetto=1000x1.50x0.179/1700=0.16N/mm² < 3.38N/mm²=fv0d (Eq.6.13)
The check is satisfied

Purlin, load combination No 4

Bending, Myd=0.144 kNm, Mzd=0.053 kNm (EC5 §6.1.6)

Rectangular cross section, b=50mm, h=50mm, A=2.500E+003mm², Wy=2.083E+004mm³, Wz=2.083E+004mm³
Modification factor Kmod=1.10 (Table 3.1), material factor γM=1.30 (Table 2.3)
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=1.10x30.00/1.30=25.38N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=1.10x30.00/1.30=25.38N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σmyd=Myd/Wmy,netto=1E+06x0.144/2.083E+004= 6.93 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.053/2.083E+004= 2.52 N/mm²

σmyd/fmyd+Km.σmzd/fmzd=0.273+0.070= 0.34 < 1 (EC5 Eq.6.11)
Km.σmyd/fmyd+σmzd/fmzd=0.191+0.099= 0.29 < 1 (EC5 Eq.6.12)
The check is satisfied

Purlin, load combination No 4

Lateral torsional stability of beams, Myd=0.144 kNm, Mzd=0.053 kNm (EC5 §6.3.3)

Rectangular cross section, b=50mm, h=50mm, A=2.500E+003mm², Wy=2.083E+004mm³, Wz=2.083E+004mm³
Modification factor Kmod=1.10 (Table 3.1), material factor γM=1.30 (Table 2.3)
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=1.10x23.00/1.30=19.46N/mm²
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=1.10x30.00/1.30=25.38N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=1.10x30.00/1.30=25.38N/mm²

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Rectangular cross section Km=0.70 (EC5  §6.1.6.(2))
σmyd=Myd/Wmy,netto=1E+06x0.144/2.083E+004= 6.93 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.053/2.083E+004= 2.52 N/mm²


Buckling length Sk
Sky= 1.00x0.600=0.600 m=  600 mm
Skz= 1.00x0.600=0.600 m=  600 mm


Slenderness
iy=√(Iy/A)=0.289x  50= 14 mm, λy=  600/ 14= 42.86
iz=√(Iz/A)=0.289x  50= 14 mm, λz=  600/ 14= 42.86


σm,crit=0.78.b²·E005/(h·Lef)=0.78x50²x 8000/(50x600)= 520.00N/mm² (EC5 Eq.6.32)
σm,crit=0.78.b²·E005/(h·Lef)=0.78x50²x 8000/(50x600)= 520.00N/mm² (EC5 Eq.6.32)
Critical stresses
σm,crity=  520.00 N/mm², λrel,my=     √(fmyk/σm,crity)=  0.24 (EC5 Eq.6.30)
σm,critz=  520.00 N/mm², λrel,mz=     √(fmzk/σm,critz)=  0.24 (EC5 Eq.6.30)


λrel,my=0.24, (λrel<=0.75), Kcrity=1.00 (EC5 Eq.6.34)
λrel,mz=0.24, (λrel<=0.75), Kcritz=1.00 (EC5 Eq.6.34)


σmyd/(Kcrity·fmyd)+Km.σmzd/(Kcritz·fmzd)=0.273+0.070= 0.34 < 1 (EC5 Eq.6.33)
Km.σmyd/(Kcrity·fmyd)+σmzd/(Kcritz·fmzd)=0.191+0.099= 0.29 < 1 (EC5 Eq.6.33)
The check is satisfied
```

1.5. Truss design

Truss geometric characteristics

Length L=4.526 m, height H=1.648 m, truss spacing d=0.600 m
Pitch =36.41%, angle α=20.01 °, tanα=0.364, sinα=0.342, cosα=0.940
Number of nodes = 4, number of elements =5, supports 2

Nodal coordinates **Truss element properties**

Node	x[m]	y[m]	Sup.	Element	K1	K2	bxh[mm]	L[m]	A[mm²]	Iy[mm4]	Wy[mm³]
1	0.000	0.000	01	1	1	4	50x125	2.436	6.250E+003	8.138E+006	1.302E+005
2	4.526	1.648		2	4	2	50x125	2.381	6.250E+003	8.138E+006	1.302E+005
3	4.526	0.000	11	3	1	3	50x125	4.526	6.250E+003	8.138E+006	1.302E+005
4	2.289	0.833		4	3	2	50x125	1.648	6.250E+003	8.138E+006	1.302E+005
				5	4	3	50x125	2.388	6.250E+003	8.138E+006	1.302E+005

Line loads per truss

Timber density =380.00 kg/m³, truss self weight =0.312 kN
Truss spacing d=0.60 m, weight of truss connections =0.031 kN

Permanent line loads (kN/m) on truss

Roof covering+self weight Gk1= 0.196 kN/m
Ceiling under roof Gk2= 0.180 kN/m

Variable line loads of short term action (kN/m) on truss

Imposed Qki= 0.40x0.600= 0.240 kN/m
Snow load Qk1= 0.960 kN/m
Wind load Qk2= 0.160 kN/m

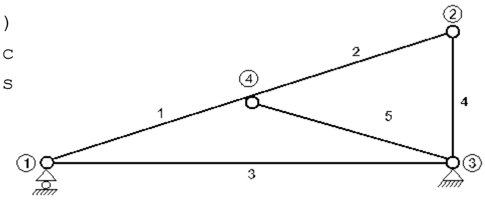
Design load combinations

(γg=1.35, γq=1.50, ψo(live Qf)=0.70, ψo(snow Q1)=0.60, ψo(wind Q2)=0.50)

L.C.	Actions Permanent-Variable	Duration classes
1	γg.Gk	Permanent
2	γg.Gk+γq.Qk1	Short-term
3	γg.Gk+γq.Qk2	Short-term
4	γg.Gk+γq.Qki	Short-term
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term

1.6. Truss static analysis

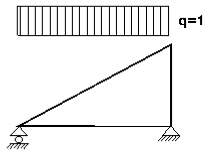
Design for connections with reduced stiffness (factor 0.20)
The truss is designed as frame structure (EN1995-1-1 §5.4.1)
with reduced connection stiffness according to the above fac
The rafter and the tie are considered as continuous elements
The truss is first solved for various unit load conditions,
and from them are computed the internal forces
for the various loading conditions and load combinations.
Number of nodes = 4, number of elements =5, supports 2



1.6.1. Static solutions for unit loads

Internal forces for unit loading (1 kN/m left rafter downwards)

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]
1	1	4	-4.42	0.80	0.02	-3.63	-1.35	-0.65
2	4	2	-0.49	1.33	-0.65	0.28	-0.77	0.01
3	1	3	3.88	0.00	0.00	3.88	0.00	0.00
4	3	2	-0.82	0.00	0.00	-0.82	0.00	0.01
5	4	3	-4.13	0.00	0.00	-4.13	0.00	0.00



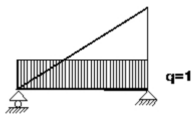
Element end forces for unit loading (1 kN/m left rafter downwards)

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	3.88	2.26	0.02	-3.88	0.03	0.65
2	4	2	0.00	1.42	-0.65	0.00	0.82	-0.01
3	1	3	-3.88	0.00	0.00	3.88	0.00	0.00
4	3	2	0.00	0.82	0.00	0.00	-0.82	-0.01
5	4	3	3.87	-1.44	0.00	-3.87	1.44	0.00

(element end forces in global coordinate system x-y)

Internal forces for unit loading (1 kN/m tie downwards)

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]
1	1	4	-0.14	-0.05	0.03	-0.14	-0.05	-0.09
2	4	2	-0.03	0.04	-0.09	-0.03	0.04	0.00
3	1	3	0.15	2.26	0.07	0.15	-2.27	0.06
4	3	2	0.04	0.02	-0.03	0.04	0.02	0.00
5	4	3	-0.14	0.01	0.00	-0.14	0.01	0.02



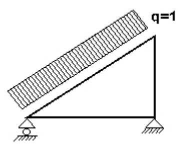
Element end forces for unit loading (1 kN/m tie downwards)

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	0.15	0.00	0.03	-0.15	0.00	0.09
2	4	2	0.02	0.04	-0.09	-0.02	-0.04	0.00
3	1	3	-0.15	2.26	0.07	0.15	2.27	-0.06
4	3	2	-0.02	-0.04	-0.03	0.02	0.04	0.00
5	4	3	0.14	-0.04	0.00	-0.14	0.04	-0.02

(element end forces in global coordinate system x-y)

Internal forces for unit loading (1 kN/m left rafter pressure)

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]
1	1	4	-3.26	0.90	0.02	-3.26	-1.53	-0.75
2	4	2	0.31	1.51	-0.75	0.31	-0.87	0.01
3	1	3	2.75	0.00	0.00	2.75	0.00	0.00
4	3	2	-0.92	0.00	0.00	-0.92	0.00	0.01
5	4	3	-4.69	0.00	0.00	-4.69	0.00	0.00



Element end forces for unit loading (1 kN/m left rafter pressure)

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	2.75	1.96	0.02	-3.59	0.33	0.75
2	4	2	-0.81	1.31	-0.75	0.00	0.92	-0.01
3	1	3	-2.75	0.00	0.00	2.75	0.00	0.00
4	3	2	0.00	0.92	0.00	0.00	-0.92	-0.01
5	4	3	4.40	-1.64	0.00	-4.40	1.64	0.00

(element end forces in global coordinate system x-y)

1.6.2. Internal forces for applied loads

Internal forces, Loading: (Gk) Dead Gk1 = 0.196, Gk2 = 0.180 [kN/m]

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]	Nm[kN]	Vm[kN]	Mm[kNm]
1	1	4	-0.95	0.16	0.01	-0.78	-0.29	-0.15	-0.89	0.00	0.08
2	4	2	-0.11	0.28	-0.15	0.05	-0.15	0.00	0.00	0.00	0.07
3	1	3	0.83	0.41	0.01	0.83	-0.41	0.01	0.83	0.00	0.47
4	3	2	-0.16	0.00	0.00	-0.16	0.00	0.00	-0.16	0.00	0.00
5	4	3	-0.89	0.00	0.00	-0.89	0.00	0.00	-0.89	0.00	0.00

(m point of maximum span moment for permanent load, or element middle point)

Internal forces, Loading: (Qk1) Snow Qks = 0.960 [kN/m]

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]	Nm[kN]	Vm[kN]	Mm[kNm]
1	1	4	-4.24	0.77	0.02	-3.49	-1.30	-0.63	-3.98	0.04	0.36
2	4	2	-0.47	1.28	-0.63	0.27	-0.74	0.01	0.01	-0.03	0.34
3	1	3	3.72	0.00	0.00	3.72	0.00	0.00	3.72	0.00	0.00
4	3	2	-0.79	0.00	0.00	-0.79	0.00	0.01	-0.79	0.00	0.00
5	4	3	-3.97	0.00	0.00	-3.97	0.00	0.00	-3.97	0.00	0.00

(m point of maximum span moment for permanent load, or element middle point)

Internal forces, Loading: (Qk2) Wind Qkw = 0.160 [kN/m]

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]	Nm[kN]	Vm[kN]	Mm[kNm]
1	1	4	-0.52	0.14	0.00	-0.52	-0.25	-0.12	-0.52	0.01	0.07
2	4	2	0.05	0.24	-0.12	0.05	-0.14	0.00	0.05	-0.01	0.06
3	1	3	0.44	0.00	0.00	0.44	0.00	0.00	0.44	0.00	0.00
4	3	2	-0.15	0.00	0.00	-0.15	0.00	0.00	-0.15	0.00	0.00
5	4	3	-0.75	0.00	0.00	-0.75	0.00	0.00	-0.75	0.00	0.00

(m point of maximum span moment for permanent load, or element middle point)

Internal forces, Loading: (Qki) Imposed (H) Qi = 0.240 [kN/m]

elem.	node-1	node-2	N1[kN]	V1[kN]	M1[kNm]	N2[kN]	V2[kN]	M2[kNm]	Nm[kN]	Vm[kN]	Mm[kNm]
1	1	4	-1.06	0.19	0.00	-0.87	-0.32	-0.16	-0.99	0.01	0.09
2	4	2	-0.12	0.32	-0.16	0.07	-0.19	0.00	0.00	-0.01	0.08
3	1	3	0.93	0.00	0.00	0.93	0.00	0.00	0.93	0.00	0.00
4	3	2	-0.20	0.00	0.00	-0.20	0.00	0.00	-0.20	0.00	0.00
5	4	3	-0.99	0.00	0.00	-0.99	0.00	0.00	-0.99	0.00	0.00

(m point of maximum span moment for permanent load, or element middle point)

1.6.3. Element end forces for applied loads

Element end forces, Loading: (Gk) Dead Gk1 = 0.196, Gk2 = 0.180 [kN/m]

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	0.83	0.47	0.01	-0.83	0.00	0.15
2	4	2	0.00	0.30	-0.15	0.00	0.16	0.00
3	1	3	-0.83	0.41	0.01	0.83	0.41	-0.01
4	3	2	0.00	0.16	0.00	0.00	-0.16	0.00
5	4	3	0.83	-0.31	0.00	-0.83	0.31	0.00

(element end forces in global coordinate system x-y)

Element end forces, Loading: (Qk1) Snow Qks = 0.960 [kN/m]

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	3.72	2.17	0.02	-3.72	0.02	0.63
2	4	2	0.00	1.36	-0.63	0.00	0.79	-0.01
3	1	3	-3.72	0.00	0.00	3.72	0.00	0.00
4	3	2	0.00	0.79	0.00	0.00	-0.79	-0.01
5	4	3	3.72	-1.38	0.00	-3.72	1.38	0.00

(element end forces in global coordinate system x-y)

Element end forces, Loading: (Qk2) Wind Qkw = 0.160 [kN/m]

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	0.44	0.31	0.00	-0.57	0.05	0.12
2	4	2	-0.13	0.21	-0.12	0.00	0.15	0.00
3	1	3	-0.44	0.00	0.00	0.44	0.00	0.00
4	3	2	0.00	0.15	0.00	0.00	-0.15	0.00
5	4	3	0.70	-0.26	0.00	-0.70	0.26	0.00

(element end forces in global coordinate system x-y)

Element end forces, Loading: (Qki) Imposed (H) Qi = 0.240 [kN/m]

elem.	node-1	node-2	F1x[kN]	F1y[kN]	M1[kNm]	F2x[kN]	F2y[kN]	M2[kNm]
1	1	4	0.93	0.54	0.00	-0.93	0.01	0.16
2	4	2	0.00	0.34	-0.16	0.00	0.20	0.00
3	1	3	-0.93	0.00	0.00	0.93	0.00	0.00
4	3	2	0.00	0.20	0.00	0.00	-0.20	0.00
5	4	3	0.93	-0.35	0.00	-0.93	0.35	0.00

(element end forces in global coordinate system x-y)

1.6.4. Vertical nodal displacements (in mm)

node	Gk	Qk1	Qk2	Qki
1	0.00	0.00	0.00	0.00
2	0.00	-0.02	0.00	0.00
3	0.00	0.00	0.00	0.00
4	-0.15	-0.68	-0.10	-0.17

1.6.5. Support reactions (kN)

node	react.	Gk	Qk1	Qk2	Qki
1	Fx	0.00	0.00	0.00	0.00
1	Fy	0.88	2.17	0.31	0.54
3	Fx	0.00	0.00	-0.26	0.00
3	Fy	0.88	2.17	0.41	0.54

1.7. Support reactions for load combinations (kN)

Loading [kN/m]	action	γg	γq	ψo
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	Permanent	1.35	0.00	1.00
(Qk1) Snow Qks = 0.960	Short-term	0.00	1.50	0.60
(Qk2) Wind Qkw = 0.160	Short-term	0.00	1.50	0.50
(Qki) Imposed (H) Qi = 0.240	Short-term	0.00	1.50	0.00

1.7.1. Reactions at node : 3 (kN)

L.C.	Load combination	duration class	kmod	Fx	Fy	Fx/Kmod	Fy/Kmod
1	γg.Gk	Permanent	0.60	0.000	1.187	0.000	1.978
2	γg.Gk+γq.Qk1	Short-term	0.90	0.000	4.446	0.000	4.940
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.396	1.802	-0.440	2.002
4	γg.Gk+γq.Qki	Short-term	0.90	0.000	2.001	0.000	2.224
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.198	4.754	-0.220	5.282
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-0.396	3.758	-0.440	4.175
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	-0.198	4.265	-0.220	4.739
	Maximum values			0.396	4.754	0.440	5.282
8	γg.Gk+γq.Qk2=0.9Gk+1.5Qk2, (EQU)	Short-term	0.90	-0.396	1.407	-0.440	1.563

1.7.2. Reactions at node : 1 (kN)

L.C.	Load combination	duration class	kmod	Fx	Fy	Fx/Kmod	Fy/Kmod
1	γg.Gk	Permanent	0.60	0.000	1.186	0.000	1.977
2	γg.Gk+γq.Qk1	Short-term	0.90	0.000	4.445	0.000	4.939
3	γg.Gk+γq.Qk2	Short-term	0.90	0.000	1.658	0.000	1.842
4	γg.Gk+γq.Qki	Short-term	0.90	0.000	2.001	0.000	2.223
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	0.000	4.681	0.000	5.201
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	0.000	3.613	0.000	4.014
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	0.000	4.192	0.000	4.658
	Maximum values			0.000	4.681	0.000	5.201
8	γg.Gk+γq.Qk2=0.9Gk+1.5Qk2, (EQU)	Short-term	0.90	0.000	1.262	0.000	1.402

1.8. Serviceability limit state

1.8.1. Serviceability limit state (EC5 EN1995-1-1:2009, §2.2.3, §7)

Control of deflection at node 4 (EC5 §7.2)

Loading [kN/m]	u[mm]	action	ψ0	ψ1	ψ2	Kdef
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	-0.152	Permanent	1.00	1.00	1.00	0.60
(Qk1) Snow Qks = 0.960	-0.681	Short-term	0.60	0.20	0.00	0.00
(Qk2) Wind Qkw = 0.160	-0.097	Short-term	0.50	0.20	0.00	0.00

Load combination	w.inst	w.fin [mm]
1 Gk	0.152	0.244
2 Gk + Qk1	0.833	0.925
3 Gk + Qk2	0.249	0.341
4 Gk + Qk1 + ψo.Qk2	0.882	0.973
5 Gk + Qk2 + ψo.Qk1	0.658	0.749

w.fin,g=w.inst,g(1+kdef), w.fin,q=w.inst,q(1+ψ2·kdef)(EC5 §2.2.3, Eq.2.3, Eq.2.4)

Maximum deflection values at node 4

w.inst = 0.882 mm, w.fin = 0.973 mm

Check according to EC5 EN1995-1-1:2009 §7.2, Tab.7.2

Final deflections at node 4

w.inst = 0.882 mm < L/300=4526/300= 15.088 mm
w.net,fin = 0.973 mm < L/250=4526/250= 18.106 mm
w.fin = 0.973 mm < L/150=4526/150= 30.177 mm
The check is satisfied

1.8.2. Serviceability limit state (EC5 EN1995-1-1:2009, §2.2.3, §7)

Control of deflection in middle of element 1 (EC5 §7.2)

Loading [kN/m]	u[mm]	action	ψ0	ψ1	ψ2	Kdef
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	0.359	Permanent	1.00	1.00	1.00	0.60
(Qk1) Snow Qks = 0.960	1.762	Short-term	0.60	0.20	0.00	0.00
(Qk2) Wind Qkw = 0.160	0.000	Short-term	0.50	0.20	0.00	0.00

Load combination	w.inst	w.fin [mm]
1 Gk	0.359	0.575
2 Gk + Qk1	2.121	2.337
3 Gk + Qk2	0.359	0.575
4 Gk + Qk1 + ψo.Qk2	2.121	2.337
5 Gk + Qk2 + ψo.Qk1	1.417	1.632

w.fin,g=w.inst,g(1+kdef), w.fin,q=w.inst,q(1+ψ2·kdef)(EC5 §2.2.3, Eq.2.3, Eq.2.4)

Maximum deflection values in middle of element 1

w.inst = 2.121 mm, w.fin = 2.337 mm

Check according to EC5 EN1995-1-1:2009 §7.2, Tab.7.2

Final deflections in middle of element 1

w.inst = 2.121 mm < L/300=2436/300= 8.120 mm
w.net,fin = 2.337 mm < L/250=2436/250= 9.744 mm
w.fin = 2.337 mm < L/150=2436/150= 16.240 mm
The check is satisfied

1.8.3. Serviceability limit state (EC5 EN1995-1-1:2009, §2.2.3, §7)

Control of deflection in middle of element 3 (EC5 §7.2)

Loading [kN/m]	u[mm]	action	ψ0	ψ1	ψ2	Kdef
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	10.075	Permanent	1.00	1.00	1.00	0.60
(Qk1) Snow Qks = 0.960	0.000	Short-term	0.60	0.20	0.00	0.00
(Qk2) Wind Qkw = 0.160	0.000	Short-term	0.50	0.20	0.00	0.00

Load combination	w.inst	w.fin [mm]
1 Gk	10.075	16.120
2 Gk + Qk1	10.075	16.120
3 Gk + Qk2	10.075	16.120
4 Gk + Qk1 + ψo.Qk2	10.075	16.120
5 Gk + Qk2 + ψo.Qk1	10.075	16.120

w.fin,g=w.inst,g(1+kdef), w.fin,q=w.inst,q(1+ψ2·kdef)(EC5 §2.2.3, Eq.2.3, Eq.2.4)

Maximum deflection values in middle of element 3

w.inst = 10.075 mm,   w.fin = 16.120 mm

Check according to EC5 EN1995-1-1:2009 §7.2, Tab.7.2

Final deflections in middle of element 3

w.inst = 10.075 mm < L/300=4526/300= 15.088 mm
w.net,fin = 16.120 mm < L/250=4526/250= 18.106 mm
w.fin = 16.120 mm < L/150=4526/150= 30.177 mm
The check is satisfied

1.9. Characteristic structural natural frequencies (self weight + permanent loads)

After a dynamic analysis the basic natural frequencies of the structure are computed. For the computation of natural frequencies, we consider mass corresponding to the self weight and the permanent loads.

No.	Frequency[Hz]	Period[sec]
1	13.72817	0.07284
2	32.87006	0.03042
3	40.79107	0.02452
4	43.16529	0.02317

1.9.1. Ultimate limit state (EC5 EN1995-1-1:2009, §6)

Rafter, elements: 1, 2

Loading [kN/m]	action	γg	γq	ψo
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	Permanent	1.35	0.00	1.00
(Qk1) Snow Qks = 0.960	Short-term	0.00	1.50	0.60
(Qk2) Wind Qkw = 0.160	Short-term	0.00	1.50	0.50
(Qki) Imposed (H) Qi = 0.240	Short-term	0.00	1.50	0.00

L.C.	Load combination	duration class	kmod	-N/Kmod	+N/Kmod	V/Kmod	M/Kmod
1	γg.Gk	Permanent	0.60	-2.128	0.118	0.653	0.342
2	γg.Gk+γq.Qk1	Short-term	0.90	-8.485	0.523	2.595	1.272
3	γg.Gk+γq.Qk2	Short-term	0.90	-2.288	0.162	0.844	0.428
4	γg.Gk+γq.Qki	Short-term	0.90	-3.186	0.190	0.975	0.489
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-8.920	0.564	2.800	1.372
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-6.528	0.428	2.140	1.054
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	-7.860	0.498	2.476	1.215
	Maximum values			-8.920	0.564	2.800	1.372

1.9.2. Check of cross section Rafter, elements: 1, 2

Rafter, elements: 1, 2 , load combination No 5
Tension parallel to the grain, Ft0d=0.508 kN (EC5 §6.1.2)
Rectangular cross section, b=50 mm, h=125 mm, A= 6 250 mm²
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
ft0k=18.00 N/mm², ft0d=Kmod·ft0k/γM=0.90x18.00/1.30=12.46N/mm² (EC5 Eq.2.14)
Ft0d=0.508 kN, σt0d=Ft0d/Anetto=1000x0.508/6250=0.08N/mm² < 12.46N/mm²=ft0d (Eq.6.1)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5
Compression parallel to the grain, Fc0d=-8.028 kN (EC5 §6.1.4)
Rectangular cross section, b=50 mm, h=125 mm, A= 6 250 mm²
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm² (EC5 Eq.2.14)
Fc0d=-8.028 kN, σc0d=Fc0d/Anetto=1000x8.028/6250=1.28N/mm² < 15.92N/mm²=fc0d (Eq.6.2)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5
Shear, Fv=2.520 kN (EC5 §6.1.7)
Rectangular cross section, bef=0.67x50=34 mm, h=125 mm, A= 4 250 mm²
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fvk=4.00 N/mm², fvd=Kmod·fvk/γM=0.90x4.00/1.30=2.77N/mm² (EC5 Eq.2.14)
Fv=2.520 kN, τv0d=1.50Fv0d/Anetto=1000x1.50x2.520/4250=0.89N/mm² < 2.77N/mm²=fv0d (Eq.6.13)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5
Bending, Myd=1.235 kNm, Mzd=0.000 kNm (EC5 §6.1.6)
Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σmyd=Myd/Wmy,netto=1E+06x1.235/1.302E+005= 9.48 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

σmyd/fmyd+Km.σmzd/fmzd=0.457+0.000= 0.46 < 1 (EC5 Eq.6.11)
Km.σmyd/fmyd+σmzd/fmzd=0.320+0.000= 0.32 < 1 (EC5 Eq.6.12)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5
Combined bending and axial compression, Fc0d=-8.028kN, Myd=1.235kNm, Mzd=0.000kNm (EC5 §6.2.4)
Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm²
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σc0d=Fc0d/Anetto=1000x8.028/6250= 1.28 N/mm²
σmyd=Myd/Wmy,netto=1E+06x1.235/1.302E+005= 9.48 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

(σc0d/fc0d)²+σmyd/fmyd+Km.σmzd/fmzd=0.007+0.457+0.000= 0.46 < 1 (EC5 Eq.6.19)
(σc0d/fc0d)²+Km.σmyd/fmyd+σmzd/fmzd=0.007+0.320+0.000= 0.33 < 1 (EC5 Eq.6.20)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5
Column stability with bending, Fc0d=-8.028kN, Myd=1.235kNm, Mzd=0.000kNm (EC5 §6.3.2)
Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3, E005=8000N/mm²
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm²
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σc0d=Fc0d/Anetto=1000x8.028/6250= 1.28 N/mm²
σmyd=Myd/Wmy,netto=1E+06x1.235/1.302E+005= 9.48 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

Buckling length Sk
Sky= 1.00x2.436=2.436 m= 2436 mm (most unfavourable)
Skz= 0.12x2.436=0.300 m= 300 mm (effective length/total length=0.30/2.44=0.12)

Slenderness
iy=√(Iy/A)=0.289x 125= 36 mm, λy= 2436/ 36= 67.67
iz=√(Iz/A)=0.289x 50= 14 mm, λz= 300/ 14= 21.43

Critical stresses
σc,crity=π²E005/λy²= 17.24 N/mm², λrel,y= √(fc0k/σc,crity)= 1.15 (EC5 Eq.6.21)
σc,critz=π²E005/λz²= 171.93 N/mm², λrel,z= √(fc0k/σc,critz)= 0.37 (EC5 Eq.6.22)

βc=0.20 (solid timber)
ky=0.5[1+βc(λrely-0.3)+λrely²]= 1.25, Kcy=1/(ky+ √(ky²-λrely²))=0.576 (Eq.6.27 6.25)
kz=0.5[1+βc(λrelz-0.3)+λrelz²]= 0.57, Kcz=1/(kz+ √(kz²-λrelz²))=0.985 (Eq.6.28 6.26)

σc0d/(Kcy·fc0d)+σmyd/fmyd+Km.σmzd/fmzd=0.140+0.457+0.000= 0.60 < 1 (EC5 Eq.6.23)
σc0d/(Kcz·fc0d)+Km.σmyd/fmyd+σmzd/fmzd=0.082+0.320+0.000= 0.40 < 1 (EC5 Eq.6.24)
The check is satisfied

Rafter, elements: 1, 2 , load combination No 5

Lateral torsional stability of beams, Myd=1.235 kNm, Mzd=0.000 kNm (EC5 §6.3.3)

Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm²
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σmyd=Myd/Wmy,netto=1E+06x1.235/1.302E+005= 9.48 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

Buckling length Sk
Sky= 1.00x2.436=2.436 m= 2436 mm (most unfavourable)
Skz= 0.12x2.436=0.300 m= 300 mm (effective length/total length=0.30/2.44=0.12)

Slenderness
iy=√(Iy/A)=0.289x 125= 36 mm, λy= 2436/ 36= 67.67
iz=√(Iz/A)=0.289x 50= 14 mm, λz= 300/ 14= 21.43

σm,crit=0.78.b²·E005/(h·Lef)=0.78x50²x 8000/(125x2192)= 56.92N/mm² (EC5 Eq.6.32)
σm,crit=0.78.b²·E005/(h·Lef)=0.78x125²x 8000/(50x300)=6500.00N/mm² (EC5 Eq.6.32)
Critical stresses
σm,crity= 56.92 N/mm², λrel,my= √(fmyk/σm,crity)= 0.73 (EC5 Eq.6.30)
σm,critz= 6500.00 N/mm², λrel,mz= √(fmzk/σm,critz)= 0.07 (EC5 Eq.6.30)

λrel,my=0.73, (λrel<=0.75), Kcrity=1.00 (EC5 Eq.6.34)
λrel,mz=0.07, (λrel<=0.75), Kcritz=1.00 (EC5 Eq.6.34)

σmyd/(Kcrity·fmyd)+Km.σmzd/(Kcritz·fmzd)=0.457+0.000= 0.46 < 1 (EC5 Eq.6.33)
Km.σmyd/(Kcrity·fmyd)+σmzd/(Kcritz·fmzd)=0.320+0.000= 0.32 < 1 (EC5 Eq.6.33)
The check is satisfied

Negligible tensile stress, combined bending-tension check is omitted (EC5 §6.2.3)

1.9.3. Ultimate limit state (EC5 EN1995-1-1:2009, §6)

Tie, elements: 3

Loading [kN/m]	action	γg	γq	ψo
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	Permanent	1.35	0.00	1.00
(Qk1) Snow Qks = 0.960	Short-term	0.00	1.50	0.60
(Qk2) Wind Qkw = 0.160	Short-term	0.00	1.50	0.50
(Qki) Imposed (H) Qi = 0.240	Short-term	0.00	1.50	0.00

L.C.	Load combination	duration class	kmod	-N/Kmod	+N/Kmod	V/Kmod	M/Kmod
1	γg.Gk	Permanent	0.60	0.000	1.878	0.918	1.063
2	γg.Gk+γq.Qk1	Short-term	0.90	0.000	7.454	0.613	0.712
3	γg.Gk+γq.Qk2	Short-term	0.90	0.000	1.987	0.612	0.709
4	γg.Gk+γq.Qki	Short-term	0.90	0.000	2.803	0.612	0.710
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	0.000	7.821	0.613	0.712
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	0.000	5.708	0.613	0.711
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	0.000	6.891	0.613	0.712
	Maximum values			0.000	7.821	0.918	1.063

1.9.4. Check of cross section Tie, elements: 3

Tie, elements: 3 , load combination No 5

Tension parallel to the grain, Ft0d=7.039 kN (EC5 §6.1.2)

Rectangular cross section, b=50 mm, h=125 mm, A= 6 250 mm²
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
ft0k=18.00 N/mm², ft0d=Kmod·ft0k/γM=0.90x18.00/1.30=12.46N/mm² (EC5 Eq.2.14)
Ft0d=7.039 kN, σt0d=Ft0d/Anetto=1000x7.039/6250=1.13N/mm² < 12.46N/mm²=ft0d (Eq.6.1)
The check is satisfied

Tie, elements: 3 , load combination No 1

Shear, Fv=0.551 kN (EC5 §6.1.7)

Rectangular cross section, bef=0.67x50=34 mm, h=125 mm, A= 4 250 mm²
Modification factor Kmod=0.60 (Table 3.1), material factor γM=1.30 (Table 2.3)
fvk=4.00 N/mm², fvd=Kmod·fvk/γM=0.60x4.00/1.30=1.85N/mm² (EC5 Eq.2.14)
Fv=0.551 kN, τv0d=1.50Fv0d/Anetto=1000x1.50x0.551/4250=0.19N/mm² < 1.85N/mm²=fv0d (Eq.6.13)
The check is satisfied

Tie, elements: 3 , load combination No 1

Bending, Myd=0.638 kNm, Mzd=0.000 kNm (EC5 §6.1.6)

Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.60 (Table 3.1), material factor γM=1.30 (Table 2.3)
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.60x30.00/1.30=13.85N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.60x30.00/1.30=13.85N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σmyd=Myd/Wmy,netto=1E+06x0.638/1.302E+005= 4.90 N/mm²
σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

σmyd/fmyd+Km.σmzd/fmzd=0.354+0.000= 0.35 < 1 (EC5 Eq.6.11)
Km.σmyd/fmyd+σmzd/fmzd=0.248+0.000= 0.25 < 1 (EC5 Eq.6.12)
The check is satisfied

Tie, elements: 3 , load combination No 5

Combined bending and axial tension, Ft0d=7.039kN, Myd=0.641kNm, Mzd=0.000kNm (EC5 §6.2.3)

Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³

Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)

ft0k=18.00 N/mm², ft0d=Kmod·ft0k/γM=0.90x18.00/1.30=12.46N/mm²

fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²

fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))

σt0d=Ft0d/Anetto=1000x7.039/6250= 1.13 N/mm²

σmyd=Myd/Wmy,netto=1E+06x0.641/1.302E+005= 4.92 N/mm²

σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

σt0d/ft0d+σmyd/fmyd+Km.σmzd/fmzd=0.090+0.237+0.000= 0.33 < 1 (EC5 Eq.6.17)

σt0d/ft0d+Km.σmyd/fmyd+σmzd/fmzd=0.090+0.166+0.000= 0.26 < 1 (EC5 Eq.6.18)

The check is satisfied

Tie, elements: 3 , load combination No 1

Combined bending and axial tension, Ft0d=1.127kN, Myd=0.638kNm, Mzd=0.000kNm (EC5 §6.2.3)

Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³

Modification factor Kmod=0.60 (Table 3.1), material factor γM=1.30 (Table 2.3)

ft0k=18.00 N/mm², ft0d=Kmod·ft0k/γM=0.60x18.00/1.30=8.31N/mm²

fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.60x30.00/1.30=13.85N/mm²

fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.60x30.00/1.30=13.85N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))

σt0d=Ft0d/Anetto=1000x1.127/6250= 0.18 N/mm²

σmyd=Myd/Wmy,netto=1E+06x0.638/1.302E+005= 4.90 N/mm²

σmzd=Mzd/Wmz,netto=1E+06x0.000/5.208E+004= 0.00 N/mm²

σt0d/ft0d+σmyd/fmyd+Km.σmzd/fmzd=0.022+0.354+0.000= 0.38 < 1 (EC5 Eq.6.17)

σt0d/ft0d+Km.σmyd/fmyd+σmzd/fmzd=0.022+0.248+0.000= 0.27 < 1 (EC5 Eq.6.18)

The check is satisfied

1.9.5. Ultimate limit state (EC5 EN1995-1-1:2009, §6)

Elements: 4

Loading [kN/m]	action	γg	γq	ψo
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	Permanent	1.35	0.00	1.00
(Qk1) Snow Qks = 0.960	Short-term	0.00	1.50	0.60
(Qk2) Wind Qkw = 0.160	Short-term	0.00	1.50	0.50
(Qki) Imposed (H) Qi = 0.240	Short-term	0.00	1.50	0.00

L.C.	Load combination	duration class	kmod	-N/Kmod	+N/Kmod	V/Kmod	M/Kmod
1	γg.Gk	Permanent	0.60	-0.366	0.000	0.008	0.011
2	γg.Gk+γq.Qk1	Short-term	0.90	-1.556	0.000	0.011	0.011
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.244	0.000	0.005	0.007
4	γg.Gk+γq.Qki	Short-term	0.90	-0.572	0.000	0.007	0.007
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-1.680	0.000	0.011	0.011
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-1.032	0.000	0.009	0.007
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	-1.483	0.000	0.010	0.010
	Maximum values			-1.680	0.000	0.011	0.011

1.9.6. Check of cross section Elements: 4

Elements: 4 , load combination No 5

Compression parallel to the grain, Fc0d=-1.512 kN (EC5 §6.1.4)

Rectangular cross section, b=50 mm, h=125 mm, A= 6 250 mm²

Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)

fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm² (EC5 Eq.2.14)

Fc0d=-1.512 kN, σc0d=Fc0d/Anetto=1000x1.512/6250=0.24N/mm² < 15.92N/mm²=fc0d (Eq.6.2)

The check is satisfied

Negligible shear stress, shear check is omitted (EC5 §6.1.7)

Negligible bending moment, bending check is omitted (EC5 §6.1.6)

Negligible bending moment, combined bending-compression check is omitted (EC5 §6.2.4)

Elements: 4 , load combination No 5

Column stability, Fc0d=-1.512 kN (EC5 §6.3.2)

Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³

Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3, E005=8000N/mm²

fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm²

fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²

fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))

σc0d=Fc0d/Anetto=1000x1.512/6250= 0.24 N/mm²

Buckling length Sk

Sky= 1.00x1.648=1.648 m= 1648 mm (most unfavourable)

Skz= 1.00x1.648=1.648 m= 1648 mm (most unfavourable)

Slenderness

iy=√(Iy/A)=0.289x 125= 36 mm, λy= 1648/ 36= 45.78

iz=√(Iz/A)=0.289x 50= 14 mm, λz= 1648/ 14=117.73

Critical stresses

$\sigma c,crity=\pi^2 E005/\lambda y^2=$ 37.67 N/mm^2, $\lambda rel,y=\sqrt{(fc0k/\sigma c,crity)}=$ 0.78 (EC5 Eq.6.21)

$\sigma c,critz=\pi^2 E005/\lambda z^2=$ 5.70 N/mm^2, $\lambda rel,z=\sqrt{(fc0k/\sigma c,critz)}=$ 2.01 (EC5 Eq.6.22)

$\beta c=0.20$ (solid timber)

$ky=0.5[1+\beta c(\lambda rely-0.3)+\lambda rely^2]=$ 0.85, $Kcy=1/(ky+\sqrt{(ky^2-\lambda rely^2)})=0.836$ (Eq.6.27 6.25)

$kz=0.5[1+\beta c(\lambda relz-0.3)+\lambda relz^2]=$ 2.69, $Kcz=1/(kz+\sqrt{(kz^2-\lambda relz^2)})=0.223$ (Eq.6.28 6.26)

$\sigma c0d/(Kcy\cdot fc0d)=$ 0.02 < 1 (EC5 Eq.6.23)

$\sigma c0d/(Kcz\cdot fc0d)=$ 0.07 < 1 (EC5 Eq.6.24)

The check is satisfied

Negligible bending moment, lateral stability check is omitted (EC5 §6.3.3)

1.9.7. Ultimate limit state (EC5 EN1995-1-1:2009, §6)
Elements: 5

Loading [kN/m]	action	γg	γq	ψo
(Gk) Dead Gk1 = 0.196, Gk2 = 0.180	Permanent	1.35	0.00	1.00
(Qk1) Snow Qks = 0.960	Short-term	0.00	1.50	0.60
(Qk2) Wind Qkw = 0.160	Short-term	0.00	1.50	0.50
(Qki) Imposed (H) Qi = 0.240	Short-term	0.00	1.50	0.00

L.C.	Load combination	duration class	kmod	-N/Kmod	+N/Kmod	V/Kmod	M/Kmod
1	γg.Gk	Permanent	0.60	-1.995	0.000	0.003	0.007
2	γg.Gk+γq.Qk1	Short-term	0.90	-7.942	0.000	0.002	0.005
3	γg.Gk+γq.Qk2	Short-term	0.90	-1.330	0.000	0.002	0.005
4	γg.Gk+γq.Qki	Short-term	0.90	-2.983	0.000	0.002	0.005
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-7.942	0.000	0.002	0.005
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-5.297	0.000	0.002	0.005
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk4	Short-term	0.90	-6.950	0.000	0.002	0.005
	Maximum values			-7.942	0.000	0.003	0.007

1.9.8. Check of cross section Elements: 5

Elements: 5 , load combination No 5
Compression parallel to the grain, Fc0d=-7.148 kN (EC5 §6.1.4)
Rectangular cross section, b=50 mm, h=125 mm, A= 6 250 mm²
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3)
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm² (EC5 Eq.2.14)
Fc0d=-7.148 kN, σc0d=Fc0d/Anetto=1000x7.148/6250=1.14N/mm² < 15.92N/mm²=fc0d (Eq.6.2)
The check is satisfied

Elements: 5 , load combination No 5
Column stability, Fc0d=-7.148 kN (EC5 §6.3.2)
Rectangular cross section, b=50mm, h=125mm, A=6.250E+003mm², Wy=1.302E+005mm³, Wz=5.208E+004mm³
Modification factor Kmod=0.90 (Table 3.1), material factor γM=1.30 (Table 2.3, E005=8000N/mm²
fc0k=23.00 N/mm², fc0d=Kmod·fc0k/γM=0.90x23.00/1.30=15.92N/mm²
fmyk=30.00 N/mm², fmyd=Kmod·fmyk/γM=0.90x30.00/1.30=20.77N/mm²
fmzk=30.00 N/mm², fmzd=Kmod·fmzk/γM=0.90x30.00/1.30=20.77N/mm²

Rectangular cross section Km=0.70 (EC5 §6.1.6.(2))
σc0d=Fc0d/Anetto=1000x7.148/6250= 1.14 N/mm²

Buckling length Sk
Sky= 1.00x2.388=2.388 m= 2388 mm (most unfavourable)
Skz= 1.00x2.388=2.388 m= 2388 mm (most unfavourable)

Slenderness
iy=√(Iy/A)=0.289x 125= 36 mm, λy= 2388/ 36= 66.32
iz=√(Iz/A)=0.289x 50= 14 mm, λz= 2388/ 14=170.55

Critical stresses
σc,crity=π²E005/λy²= 17.95 N/mm², λrel,y= √(fc0k/σc,crity)= 1.13 (EC5 Eq.6.21)
σc,critz=π²E005/λz²= 2.71 N/mm², λrel,z= √(fc0k/σc,critz)= 2.91 (EC5 Eq.6.22)

βc=0.20 (solid timber)
ky=0.5[1+βc(λrely-0.3)+λrely²]= 1.22, Kcy=1/(ky+ √(ky²-λrely²))=0.592 (Eq.6.27 6.25)
kz=0.5[1+βc(λrelz-0.3)+λrelz²]= 5.00, Kcz=1/(kz+ √(kz²-λrelz²))=0.110 (Eq.6.28 6.26)

$\sigma c0d/(Kcy \cdot fc0d) = 0.12 < 1$ (EC5 Eq.6.23)
$\sigma c0d/(Kcz \cdot fc0d) = 0.65 < 1$ (EC5 Eq.6.24)
The check is satisfied

1.10. Truss connections

1.10.1. Lateral Load-carrying capacity of connections (EC5 EN1995-1-1:2009, §8)

Connection nails and connection plates

Selected nails 4.0/35 mm (d=4.0mm, L=35mm). Metal plates , t=2.0 mm.
Yield strength for plate steel fy=240 N/mm². Net plate area (minus holes) Anet=(0.75)·b·t

Cross section properties

Thickness of timber d=50.0 mm, thickness of steel plate t=2.0 mm
Nail properties (EC5 §8.3.1)
Smooth nails, round cross section, no pre-drilling
Nail diameter d=4.0 mm, nail length l=35 mm.
Distance between nails (EC5 Table 8.2)
as most unfavourable is chosen a1=14d=14x4.0=56 mm, a2=5d=20 mm

Characteristic value for yield moment (EC5 §8.3.1.1)
Myrk=0.30fud^2.6=0.30x600x4.0^2.6=6617 Nmm (fu=600N/mm²) (EN1995-1-1 Eq.8.14)

Characteristic value of embedment strength (EC5 §8.3.1.1)
fhk=0.082ρk/d^0.3=20.56N/mm², (ρk=380kg/m³, d=4.0mm) (EN1995-1-1 Eq.8.15)

Permanent action
Capacity of laterally loaded nails -Single shear connection (EC5 §8.2.3)
t2=33.0 mm (nail depth), thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.9(a), 8.9(b))
0.40fhk·t2·d=1.086 kN
1.15√[2Myrk·fhk·d]=1.200 kN
Lateral load-carrying capacity of nail Rd=Kmod·Fvrk/γM=0.60x1.086/1.30=0.501 kN

Medium-term action
Capacity of laterally loaded nails -Single shear connection (EC5 §8.2.3)
t2=33.0 mm (nail depth), thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.9(a), 8.9(b))
0.40fhk·t2·d=1.086 kN
1.15√[2Myrk·fhk·d]=1.200 kN
Lateral load-carrying capacity of nail Rd=Kmod·Fvrk/γM=0.80x1.086/1.30=0.668 kN

Short-term action
Capacity of laterally loaded nails -Single shear connection (EC5 §8.2.3)
t2=33.0 mm (nail depth), thickness of steel plate t=2.0<=0.5d=0.5x4.0=2.0 mm
Fvrk=the minimum of the values (EC5 EN1995-1-1:2009 Eq.8.9(a), 8.9(b))
0.40fhk·t2·d=1.086 kN
1.15√[2Myrk·fhk·d]=1.200 kN
Lateral load-carrying capacity of nail Rd=Kmod·Fvrk/γM=0.90x1.086/1.30=0.752 kN

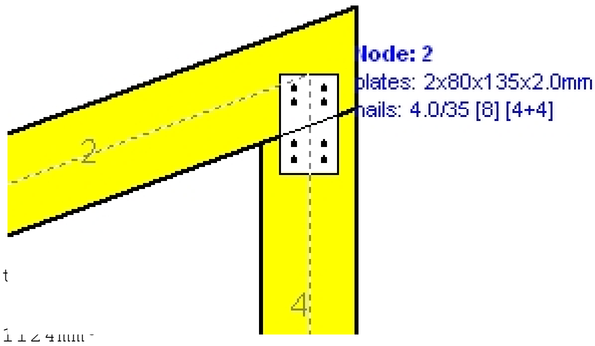
Assumptions for the design of nailed connections
The design of connections is based on plastic analysis. The forces at the nails are all reaching the same limit value. The metal plate capacity is based on plastic section modulus. The compressive design force is reduced to 0.50xFd

1.10.2. Ultimate limit state

Design of nailed connection at node : 2 (EC5 EN1995-1-1:2009, §8.3)
Connection with double (2) metal plates on the two faces of the truss.

Connection check between elements 2 and 4, at node 2

Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=135mmx80mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
4 nails on each of the connected elements
Distance between nails a1=56 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm²
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 4, (kef=1.0, neff=n), A=50mm², r=16mm, Wp =1124mm³
σ and σd plate normal and bearing stress N/mm²



Forces at node 2 ,from element 2, at the center of the joint F(force) M(moment)

Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.110	-0.003	0.027	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-0.700	-0.018	0.174	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.221	-0.006	0.055	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-0.258	-0.007	0.064	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.756	-0.019	0.188	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-0.575	-0.015	0.143	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.667	-0.017	0.166	<	0.752

Check capacity of connection plate

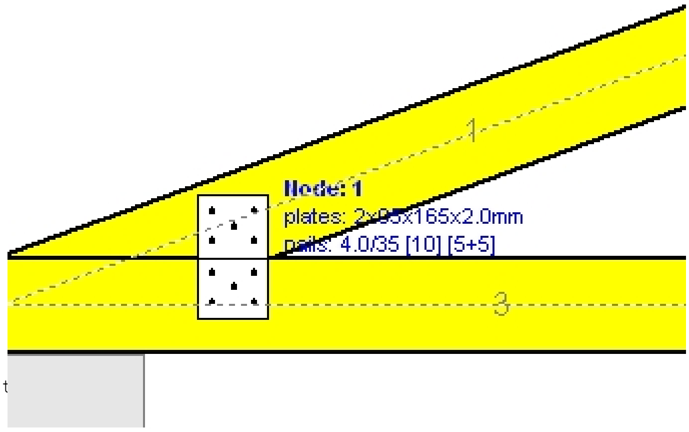
L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm²)
1	γg.Gk	Permanent	0.60	-0.110	-0.003	1	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-0.700	-0.018	7	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.221	-0.006	2	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-0.258	-0.007	2	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.756	-0.019	7	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-0.575	-0.015	5	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.667	-0.017	6	<	196

1.10.3. Ultimate limit state

Design of nailed connection at node : 1 (EC5 EN1995-1-1:2009, §8.3)
Connection with double (2) metal plates on the two faces of the truss.

Connection check between elements 1 and 3, at node 1

Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=95mmx165mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
5 nails on each of the connected elements
Distance between nails a1=56 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm²
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 5, (kef=1.0, neff=n), A=63mm², r=29mm, Wp =2146mm³
σ and σd plate normal and bearing stress N/mm²



Forces at node 1 ,from element 1, at the center of the joint F(force) M(moment)

Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.647	-0.011	0.089	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.879	-0.083	0.583	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-1.052	-0.018	0.148	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-1.455	-0.029	0.212	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-4.081	-0.087	0.613	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.991	-0.062	0.445	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.596	-0.076	0.539	<	0.752

Check capacity of connection plate

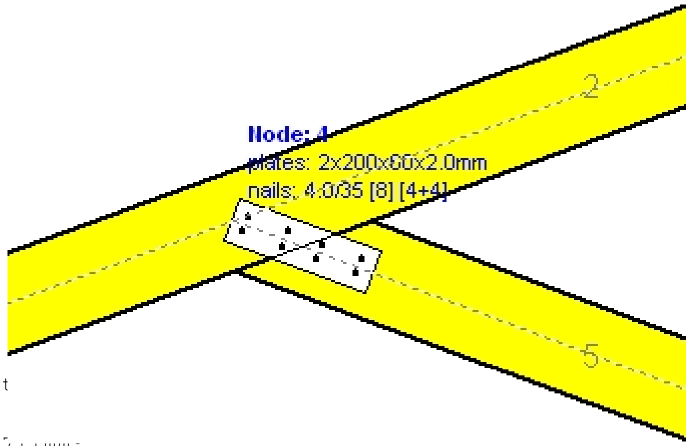
L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm²)
1	γg.Gk	Permanent	0.60	-0.647	-0.011	4	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.879	-0.083	26	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-1.052	-0.018	6	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-1.455	-0.029	9	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-4.081	-0.087	27	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.991	-0.062	20	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.596	-0.076	24	<	196

1.10.4. Ultimate limit state

Design of nailed connection at node : 4 (EC5 EN1995-1-1:2009, §8.3)
Connection with double (2) metal plates on the two faces of the truss.

Connection check of element 5, with elements 1 and 2, at node 4

Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=200mmx60mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
4 nails on each of the connected elements
Distance between nails a1=20 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm²
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 4, (kef=1.0, neff=n), A=50mm², r=10mm, Wp =7
σ and σd plate normal and bearing stress N/mm²



Forces at node 4 ,from element 5, at the center of the joint F(force) M(moment)

Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.598	0.000	0.075	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.574	0.000	0.447	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.598	0.000	0.075	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-1.342	0.000	0.168	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.574	0.000	0.447	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.384	0.000	0.298	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.128	0.000	0.391	<	0.752

Check capacity of connection plate

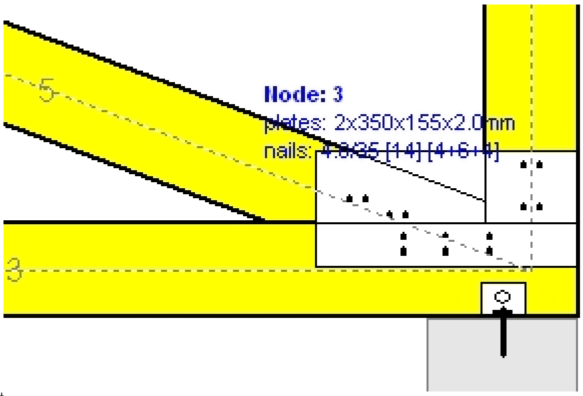
L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm²)
1	γg.Gk	Permanent	0.60	-0.598	0.000	3	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.574	0.000	20	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.598	0.000	3	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-1.342	0.000	7	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.574	0.000	20	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.384	0.000	13	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.128	0.000	17	<	196

1.10.5. Ultimate limit state

Design of nailed connection at node : 3 (EC5 EN1995-1-1:2009, §8.3)

Connection with double (2) metal plates on the two faces of the truss.

Connection check of element 4 and 5, with element 3, at node 3



Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=350mmx155mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
6 nails on each of the connected elements
Distance between nails a1=56 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm^2
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 6, (kef=1.0, neff=n), $A=75mm^2$, r=41mm, Wp =5562mm^3
σ and σd plate normal and bearing stress N/mm^2

Forces at node 3 ,from elements 4, 5, at the center of the joint F(force) M(moment)

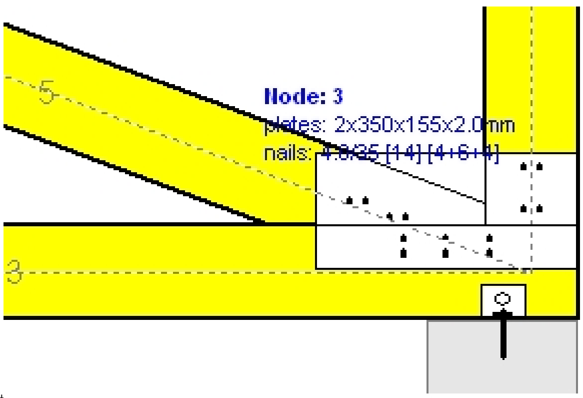
Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.647	0.012	0.072	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.878	0.095	0.473	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.647	0.012	0.072	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-1.455	0.032	0.172	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.907	0.095	0.475	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.586	0.062	0.313	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.422	0.083	0.414	<	0.752

Check capacity of connection plate

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm^2)
1	γg.Gk	Permanent	0.60	-0.647	0.012	1	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.878	0.095	5	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.647	0.012	1	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-1.455	0.032	2	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.907	0.095	5	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.586	0.062	3	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.422	0.083	4	<	196

Connection check of element 5, with elements 3 and 4, at node 3



Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=350mmx155mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
4 nails on each of the connected elements
Distance between nails a1=56 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm^2
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 4, (kef=1.0, neff=n), $A=50mm^2$, r=10mm, Wp =711mm^3
σ and σd plate normal and bearing stress N/mm^2

Forces at node 3 ,from element 5, at the center of the joint F(force) M(moment)

Check capacity of connection

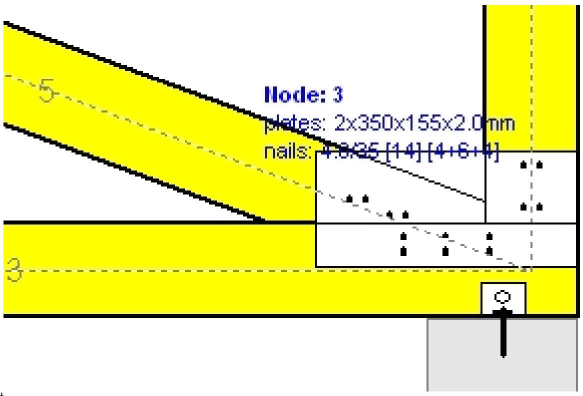
L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.598	-0.002	0.089	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.574	-0.002	0.460	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.598	-0.002	0.089	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-1.342	-0.002	0.182	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.574	-0.002	0.460	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.384	-0.002	0.312	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.128	-0.002	0.405	<	0.752

Check capacity of connection plate

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm²)
1	γg.Gk	Permanent	0.60	-0.598	-0.002	1	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-3.574	-0.002	8	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.598	-0.002	1	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-1.342	-0.002	3	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.574	-0.002	8	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-2.384	-0.002	5	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-3.128	-0.002	7	<	196

Connection check of element 4, with elements 3 and 5, at node 3

Fastener characteristics:
Two(2) metal 2.0mm plates with dimensions
BxH=350mmx155mm, and thickness 2.0mm
Nails 4.0/35 mm (d=4.0mm, L=35mm),
4 nails on each of the connected elements
Distance between nails a1=56 mm, a2=20 mm
Yield strength for plate steel fy=240 N/mm²
Net plate area (minus holes) Anet=(0.75)·b·t
Fa= force at the center of the connection
Ma= moment at the center of the connection
Maximum force at corner nail Fn=Fa/n+Ma/Wp
n: number of nails, a: nail section area
A=nxa: total area of nails
r: distance of corner nail from connection cent
Wp: section modulus of connection
n= 4, (kef=1.0, neff=n), A=50mm², r=21mm, Wp =149[illegible]
σ and σd plate normal and bearing stress N/mm²



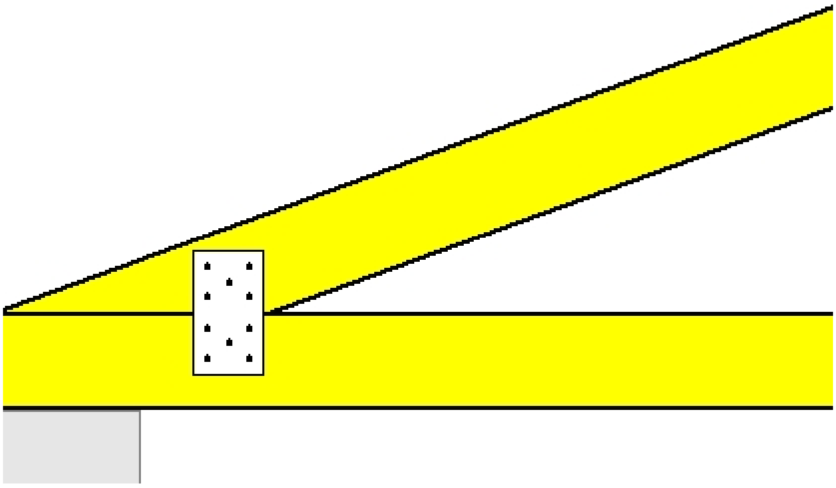
Forces at node 3 ,from element 4, at the center of the joint F(force) M(moment)

Check capacity of connection

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	Fn(kN)		Rd(kN)
1	γg.Gk	Permanent	0.60	-0.110	-0.003	0.026	<	0.501
2	γg.Gk+γq.Qk1	Short-term	0.90	-0.700	-0.003	0.100	<	0.752
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.110	-0.003	0.026	<	0.752
4	γg.Gk+γq.Qki	Short-term	0.90	-0.258	-0.003	0.045	<	0.752
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.756	-0.003	0.106	<	0.752
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-0.464	-0.003	0.070	<	0.752
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.667	-0.003	0.095	<	0.752

Check capacity of connection plate

L.C.	Load combination	duration class	kmod	Fa(kN)	Ma(kNm)	σ		σd(N/mm²)
1	γg.Gk	Permanent	0.60	-0.110	-0.003	0	<	131
2	γg.Gk+γq.Qk1	Short-term	0.90	-0.700	-0.003	1	<	196
3	γg.Gk+γq.Qk2	Short-term	0.90	-0.110	-0.003	0	<	196
4	γg.Gk+γq.Qki	Short-term	0.90	-0.258	-0.003	0	<	196
5	γg.Gk+γq.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.756	-0.003	1	<	196
6	γg.Gk+γq.Qk2+γq.ψo.Qk1	Short-term	0.90	-0.464	-0.003	0	<	196
7	γg.Gk+γq.Qki+γq.ψo.Qk1+γq.ψo.Qk2	Short-term	0.90	-0.667	-0.003	1	<	196



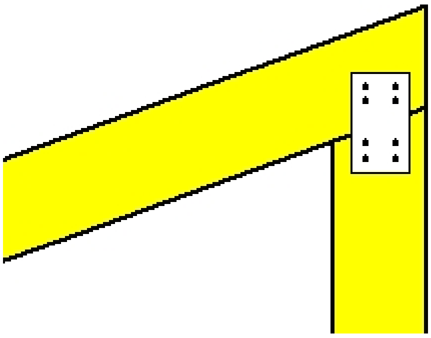
Connection at node 1

(node at x=0.307 m, y=0.063 m)

plates: 2x95x165x2.0mm

Nails

nails: 4.0/35 [10] [5+5]



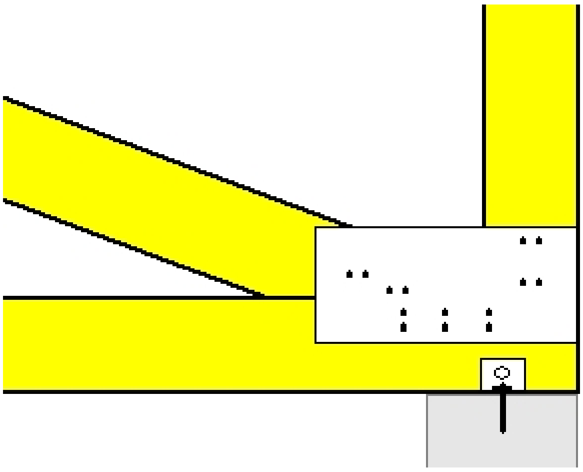
Connection at node 2

(node at x=4.526 m, y=1.648 m)

plates: 2x80x135x2.0mm

Nails

nails: 4.0/35 [8] [4+4]



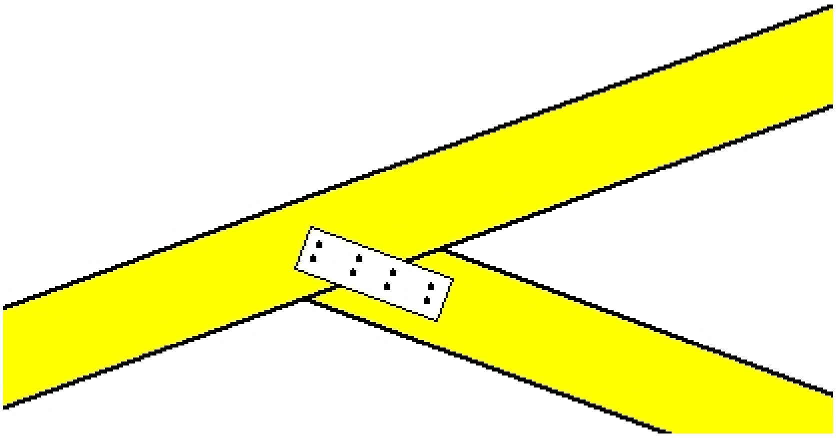
Connection at node 3

(node at x=4.526 m, y=0.000 m)

plates: 2x350x155x2.0mm

Nails

nails: 4.0/35 [14] [4+6+4]



Connection at node 4

(node at x=2.289 m, y=0.833 m)

plates: 2x200x60x2.0mm

Nails

nails: 4.0/35 [8] [4+4]