Purlins

STEP lecture E3
C. Short
Consulting Engineer

Objective

To show the design and detailing of purlins in compliance with EC5.

Summary

Purlins from round pole, solid or glued laminated timber, and built up sections of I- or Box beams are discussed. The basis of design is developed, the ultimate and serviceability limit states are identified, design values of actions and resulting load cases are developed.

Methods of verification or design conditions are addressed for members with biaxial actions in relation to solid timber. Durability, structural detailing and control in relation to purlin members are discussed.

Introduction

A purlin is a horizontal member in a roof supported on the principals and supporting the common rafters.

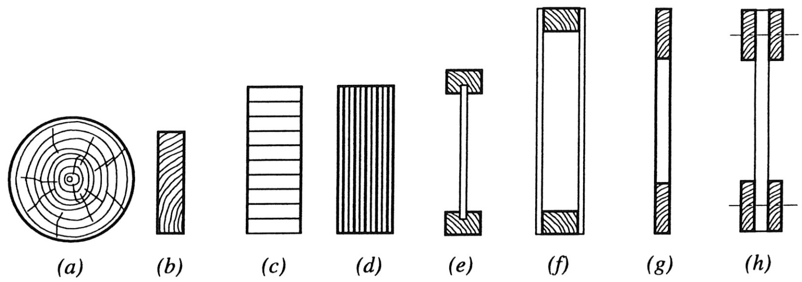


Figure 1 *Section shapes used as purlins. (a) Round solid timber or natural timber poles, (b) solid timber sections, sometimes used in groups fixed together to act as a single unit, (c) glued laminated sections, (d) Laminated Veneer Lumber, (e) I-beams, (f) box beams, (g) trussed rafters with punched metal plate connectors, sometimes fixed together and used in groups to form a multiple truss, (h) truss with glued or nailed plywood gussets, bolted, or toothed plate connections.*

Purlins can be manufactured from a variety of materials and in many configurations. The most commonly used shapes are those shown in Figure 1.

Purlins may be constructed as simply supported beams as shown in Figure 2a, or as continuous beams as shown in Figure 2b. If the timber element is continuous over three or more spans the arrangement will reduce the design moment and the effective deflection for a given section due to the continuity over the support. The limit for this structural arrangement is the maximum continuous length which the timber can be purchased or transported to the construction site.

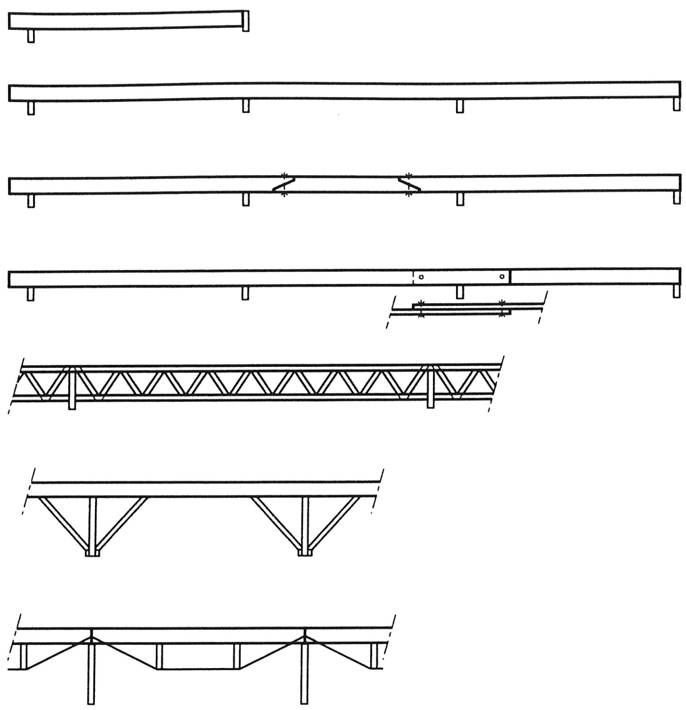


Figure 2 Typical purlin forms.

When the building length exceeds the maximum timber length one of several structural forms may be adopted to simulate the properties of the continuous beam by connecting together successive timber lengths to achieve continuity. Two methods are shown in Figure 2c and 2d. In the first case the bolted splice would occur at a point of contra-flexure and in the second case the continuity is maintained by the moment of resistance in the plated or bolted joint close to the support.

Trussed purlins, braced spans and reinforcement with tension steel are shown in Figure 2e, 2f and 2g, respectively.

Design

In all but the simply supported span shown in Figure 2a critical points for the design would contain combined stresses from bending, shear, tension and/or compression forces. Care should be taken to evaluate the effect of the geometrical orientation of the purlin and the interaction of the force vectors which can combine to generate not only combined stresses but in some cases bi-axial combined stress coexisting at the same point on the purlin.

The structural design method is similar to that for a simple or continuous beam except that the purlin is generally subject to biaxial bending and torsional effects. Refer to STEP lecture B3 for basic beam bending considerations and STEP lecture B4 for shear and torsion verification.

The normal loading arrangement on the purlin comes from the rafter and consists of the self weight of the rafter, the permanent load from the roof materials, transient load from snow or wind loads, and imposed load on the surface of the roof.

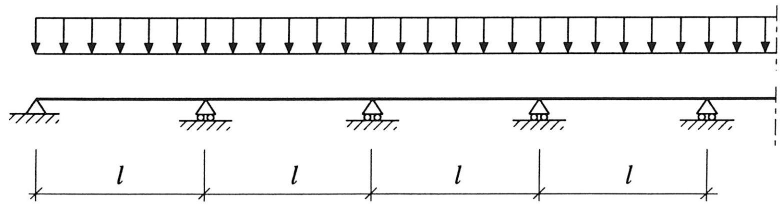
The imposed load is generally applied to the horizontal projected plan area of the roof whereas the snow, weight of the roof materials and the self weight of the rafter are applied to the true area and true length respectively. These loads have a line of action which is vertical. The wind loading is applied at right angles to the true surface area of the roof as a pressure, and can act in an upwards (suction) or downwards (positive pressure) direction depending on the prevailing wind direction and the geometric properties of the building being designed. The loads supported by the purlin should be determined from the structural action of the supported rafters.

Example

Continuous purlin over 6 spans of 6 *m* length (see Figure 2d)

roof slope $\alpha = 10°$

purlin spacing $e = 1{,}15\ m$



Timber strength class C24

Service class 1, load duration class short-term

$k_{mod} = 0{,}9$

$f_{m,d} = 16{,}6\ N/mm^2$

Characteristic values of actions

Permanent actions (self-weight)

Roofing	0,20 kN/m^2	(roof area)
Insulation	0,06 kN/m^2	(roof area)
Ceiling	0,10 kN/m^2	(roof area)
Purlins and bracing	0,10 kN/m^2	(roof area)
	0,46 kN/m^2	(roof area)
$G_k =$	0,47 kN/m^2	(horizontal area)

Variable actions (snow and wind):

$Q_k = 0{,}75\ kN/m^2$ (horizontal area)

Wind action is suction only and not governing.

Design values of actions

$$S_d = \sum \gamma_{G,j}\, G_{k,j} + \gamma_{Q,1}\, Q_{k,1} + \sum_{i>1} \gamma_{Q,i}\, \psi_{0,i}\, Q_{k,i}$$

Partial safety factors

$\gamma_G = 1{,}35$ permanent actions

$\gamma_G = 1{,}5$ variable actions

$$q_{z,d} = e \cos^2 10° \,(\gamma_G G_k + \gamma_Q Q_k)$$
$$= 1{,}15 \cos^2 10° \,(1{,}35 \cdot 0{,}47 + 1{,}5 \cdot 0{,}75) = 1{,}96\ kN/m$$

$$q_{y,d} = e \sin 10° \cos 10° (\gamma_G G_k + \gamma_Q Q_k)$$
$$= 1{,}15 \sin 10° \cos 10° \,(1{,}35 \cdot 0{,}47 + 1{,}5 \cdot 0{,}75) = 0{,}35\ kN/m$$

Bay moments:

$M_{y,d} = \eta\, q_{z,d}\, l^2$

$M_{z,d} = \eta\, q_{y,d}\, l^2$

	η	$M_{y,d}$ kNm	$M_{z,d}$ kNm
end span	0,078	5,50	0,98
second span	0,034	2,40	0,43
third span	0,043	3,03	0,54

The position of the connections between the timber members is chosen in a way that the bay moments govern the design. Although the bending moments at the supports are larger, they are not governing since two cross-sections are present at the supports. The sum of the moments in the two adjacent bays always exceeds the moment at the support for uniformly distributed loading and constant bay lengths.

From the condition that the moment at the connection position equals the value of the maximum bay moment, the following positions result (see Figure 3):
Inner bays: $l_{c1} = 0{,}10\ l$
End bay: $l_{c2} = 0{,}17\ l$

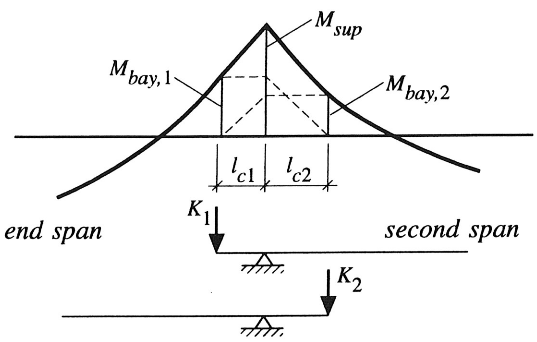


Figure 3 Simplified moment distribution and connection forces at support.

Forces in the connections (inner bays):
The forces to be transferred in the connections are determined assuming a constant moment in the single cross-section over the length l_c from the support and disregarding the influence of the uniform loading on the cantilevered part. This assumption is slightly conservative.

$$K_z = M_{z,d}/l_c = 0{,}43\ q_z\ l = 5{,}06\ kN$$
$$K_y = M_{y,d}/l_c = 0{,}43\ q_y\ l = 0{,}90\ kN$$

Ultimate limit state design

$$k_m \frac{\sigma_{m,y,d}}{f_{m,d}} + \frac{\sigma_{m,z,d}}{f_{m,d}} \leq 1$$

$$\frac{\sigma_{m,y,d}}{f_{m,d}} + k_m \frac{\sigma_{m,z,d}}{f_{m,d}} \leq 1$$

End bay: C24, b x h = 120 x 160 mm

$$\sigma_{m,y,d} = \frac{M_{y,d}}{W_y} = \frac{5{,}50 \cdot 10^6 \cdot 6}{120 \cdot 160^2} = 10{,}7\ N/mm^2$$

$$\sigma_{m,z,d} = \frac{M_{z,d}}{W_z} = \frac{0{,}98 \cdot 10^6 \cdot 6}{160 \cdot 120^2} = 2{,}55\ N/mm^2$$

$$\frac{10{,}7}{16{,}6} + 0{,}7\ \frac{2{,}55}{16{,}6} = 0{,}75 \leq 1$$

Inner bays: C24, b x h = 60 x 160 mm
The calculation corresponds to that for the end bay.

Connection in the second bay
choose 6 ringed shank nails, d = 4 mm, l = 110 mm

Lateral load-carrying capacity

β = 1,0
t_1 = 60 mm
t_2 = 50 mm
d = 4 mm
ρ_k = 350 kg/m^3
$f_{h,1,k}$ = $0{,}082\ \rho_k\ d^{-0{,}3}$ = $0{,}082 \cdot 350 \cdot 4^{-0{,}3}$ = 18,9 N/mm^2

$$f_{h,1,d} = \frac{k_{mod,1}\ f_{h,1,k}}{\gamma_M} = \frac{0{,}9 \cdot 18{,}9}{1{,}3} = 13{,}1\ N/mm^2$$

$$M_{y,k} = 180\ d^{2{,}6} = 180 \cdot 4^{2{,}6} = 6620\ Nmm$$

$$M_{y,d} = \frac{M_{y,k}}{\gamma_M} = \frac{6620}{1{,}1} = 6020\ N/mm$$

EC5: Equation 6.2.1f is governing.
$R_{la,d}$ = 873 N

Axial load-carrying capacity
Assume:
$f_{1,k}$ = $50 \cdot 10^{-6}\ \rho_k^2$ = 6,12 N/mm^2
$f_{2,k}$ = $600 \cdot 10^{-6}\ \rho_k^2$ = 73,5 N/mm^2

$$f_{1,d} = \frac{k_{mod}\ f_{1,k}}{\gamma_M} = \frac{0{,}9 \cdot 6{,}12}{1{,}3} = 4{,}24\ N/mm^2$$

$$f_{2,d} = \frac{k_{mod}\ f_{2,k}}{\gamma_M} = \frac{0{,}9 \cdot 73{,}5}{1{,}3} = 50{,}9\ N/mm^2$$

$$R_{ax,d} = \min \begin{cases} f_{1,d}\ d\ l = 4{,}24 \cdot 4 \cdot 50 = 848\ N \\ f_{2,d}\ d^2 = 50{,}9 \cdot 4^2 = 814\ N \end{cases} = 814\ N$$

Interaction

$$\left(\frac{F_{ax,d}}{R_{ax,d}}\right)^2 + \left(\frac{F_{la,d}}{R_{la,d}}\right)^2 = \left(\frac{0{,}90}{6 \cdot 0{,}814}\right)^2 + \left(\frac{5{,}06}{6 \cdot 0{,}873}\right)^2 = 0{,}97 \leq 1$$

If it is appropriate to limit deformations, the deflections may be calculated as for a continuous beam using the bending stiffness of the single cross-sections.