Robustness design of timber structures – secondary structures

Philipp Dietsch
Technische Universität München, Germany

Summary

Design rules for robustness require insensitivity to local failure and the prevention of progressive collapse. This is often verified by applying the load case “removal of a limited part of the structure”. This fact sheet will evaluate typical secondary systems for timber roof structures against these requirements, including exemplary comparative calculations for typical purlin systems. The results will be compared against typical reasons for damages and failure. Applying the finding that most failures of timber structures are not caused by random occurrences or local defects, but by global (repetitive) defects (e.g. from systematic mistakes), it is shown that the objective of load transfer - often mentioned as preferable - should be critically analysed for such structures. It is thereby demonstrated that there is no strategy which ensures robustness in all cases.

Keywords

Timber, secondary structures, determinate structures, redundant structures, local effects, global effects, human errors, robustness.

Robustness Design of Timber Structures – Secondary Structures / Purlin Systems

The requirement for a robust structure is often defined as a structure being “designed in such a way that it will not be damaged by events like fire, explosions, impact or consequences of human errors, to an extent disproportionate to the original cause” [1]. A structure shall be insensitive to local failure (disproportionate collapse), thereby including the design against progressive collapse. There are several approaches to demonstrate this, e.g. given in [1]. One of these approaches is to demonstrate that a load case “removal of a limited part of the structure” will not lead to extensive failure.
Wide-span timber structures as roof structures of arenas or halls are often composed of a primary structure, e.g. pitched cambered glulam beams, carrying a secondary structure in the form of purlins [3]. The purlins can be realized as simply supported beams (a), continuous beams (b), gerber beams (c) and lap-jointed purlins (d), see Fig. 1.

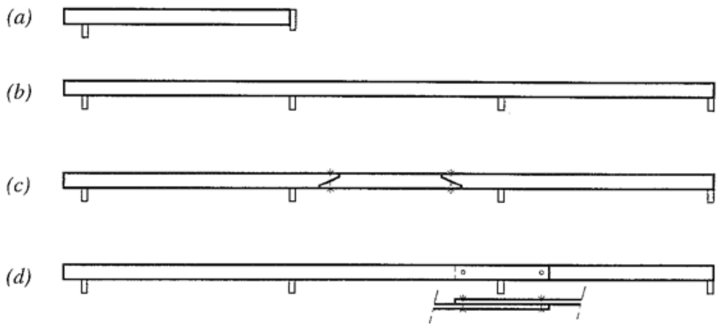


Figure 1: Typical Purlin Forms (from [3])

In this fact sheet, these systems will be evaluated against the background of above given regulations.

Exemplary comparative calculations on typical Purlin Systems in Timber

Evaluated System

To enable an evaluation of different purlin systems, it was decided to present comparative deterministic calculations based on an exemplary roof geometry as it is shown in Fig. 2.

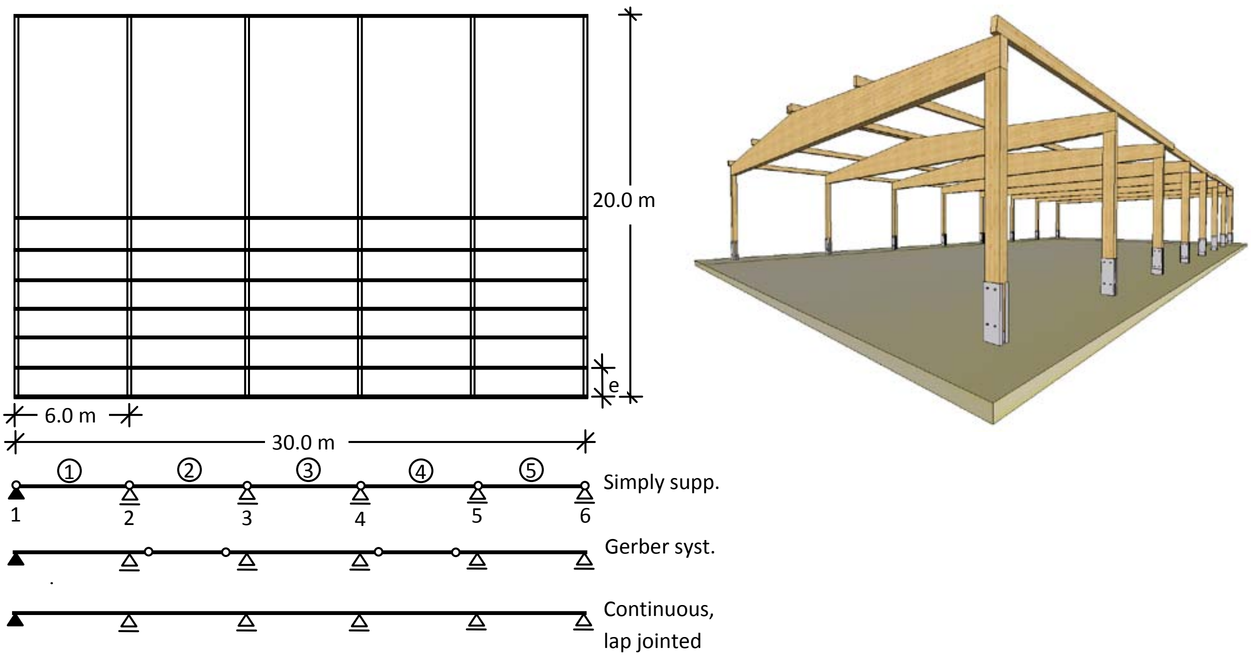


Figure 2: Schematic layout and isometric drawing of structure and possible structural systems

The chosen roof, at an angle of 10° and covering an area of ℓ/b = 30.0/20.0 m^2 is supported by 6 primary beams at a distance of e = 6.0 m. It is assumed that the beams be designed to have a utilization factor of $\eta \approx 0.95$. The dead load be g_k = 0.5 kN/m^2, the snow load be s_k = 0.8 kN/m^2, the wind load, acting as wind suction shall be neglected. The purlins, featuring a cross section of b/h = 100/200 mm^2 shall be realized with grade C24 timber. Their spacing e be chosen so that each purlin system has a utilization factor (ULS) of 0.9 <

η < 1.0. A possible change in cross section over the roof length (to adapt to the different bending moments) shall be neglected. Regarding the ULS verification for bending around both axes, this leads to the following spacings *e*:

Purlin system	Spacing *e*	Purlin System	Distance *e*
Simply supp. beam	1.0 m	Continuous beam	1.3 m
Gerber beam	1.3 m	Lap jointed purlin	1.6 m

Table 1: Realizable spacings at 0.9 < η < 1.0 for different purlin systems for given boundary conditions

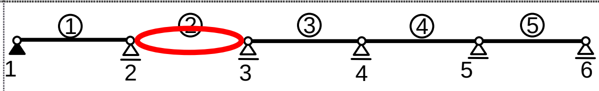
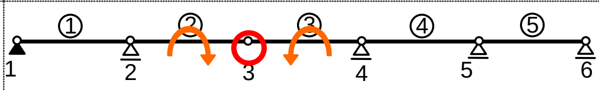
Comparative Calculations

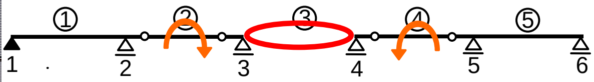
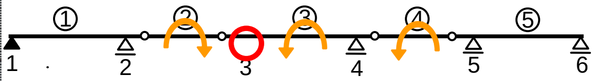
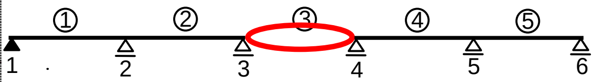
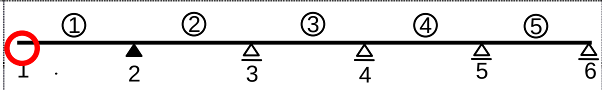
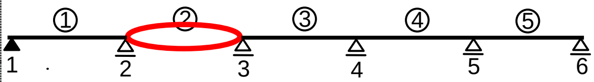
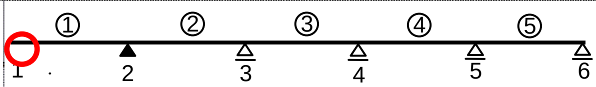
It shall now be assessed, how the removal of a limited part of the structure will affect the remaining structure. Two cases are evaluated:

a) Removal of a purlin between two supports (equivalent to the failure/rupture of one purlin)

b) Removal of one support (equivalent to the failure of one main beam).

The increase in bending stress in the remaining purlins (column 3) as well as the load increase on the main beams (column 5) are compared. Columns 4 respectively 6 list the resulting utilisation factors in the accidental load case (γ_G = γ_Q = 1.0; $\psi_{2,snow}$ = 0; $k_{mod,acc}$). Since the system is symmetrical, only elements 1 – 3 are listed.

	1	2	3	*4*	5	*6*
1	Purlin system / removed member	Removed Member Additional failing members due to system instability	Max. stress incr-ease	*Max. utili-sati-on η*	Max. stres s incr-ease	*Max. utili-sati-on η*
2			for remaining purlins		for remaining main beams (supports)	
3	Simply supp. beam					
4	a) Removal of purlin	 – no additional purlins failing due to system instability	--		--	
5	b) Removal of supp.		--		--	

6	<u>Gerber beam</u>					
7	a) Removal of purlin (worst case)		25% (field 1)	*57%*	--	
8	b) Removal of supp. (worst case)		25% (field 1)	*57%*	--	
9	<u>Continuous beam</u>					
10	a) Removal of purlin (worst case)	 - no additional purlins failing due to system instability	19% (supp. 2)	*54%*	10% (supp. 2)	*50%*
11	b) Removal of supp. (worst case)	 - no purlins failing due to system instability, - possible failure due to significant overloading of remaining purlins	475% (supp. 2)	*228%*	82% (supp. 2)	*83%*
12	<u>Lap jointed beam</u>					
13	a) Removal of purlin (worst case)		60% (field 1)	*77%*	10% (supp. 4)	*50%*
14	b) Removal of supp. (worst case)	 - no purlins failing due to system instability, - possible failure due to significant overloading of remaining purlins	520%* (field 1) * beams designed for field moment, assumed overlap of 0.10*ℓ, resp. 0.17*ℓ.	*250%**	82% (supp. 2)	*83%*

Robustness evaluation of typical Purlin Systems in Timber

Damaged Area

The comparison of damaged area(s) shows that – in the case of simply supported beams as well as continuous beams and lap jointed beams - failure of one purlin will result in local damage (no other field than the one covered by the failing member will fail due to system instability). The failure of one purlin in a gerber system will – because of system instability - in the worst case result in the additional failure of the two adjacent purlins. This extends the damaged area by 200%, compared to the area covered by the failed member.

In the case of one main member failing, simply supported beams as well as continuous beams and lap jointed beams result in the failure of the adjacent purlins (damage restricted to two fields). In the case of gerber beams, the failure of one main member will in the worst case result in the failure of 3 purlins, thereby extending the damaged area by 50%.

Load Transfer / Additional Load on remaining Members

A determinate purlin system, e.g. realized by simply supported beams has the advantage that failure of one member will not result in substantial overloading of other than the failing members. To achieve that, it is important to design the connections in such a way, that they will not transfer large additional loads in the case of failure (failing member “hinging” itself into the remaining members). Likewise, the remaining purlins in gerber systems are subjected to a comparatively small stress increase (max. 25%) after failure of a purlin or main member.

Redundant systems as continuous beams and lap jointed beams are more critical in that aspect. A failing purlin will increase the bending stress in the remaining purlin system as well as the loads on the main beams by up to 50%. A failing main beam, hinging itself into the purlin system, will theoretically increase the utilisation factor of the purlins by up to 475% resp. 520%, due to the doubled span. If the purlins shall be designed to enable load distribution, the realizable distance *e* between the purlins would decrease from 1.60 m to 0.70 m to stay below a utilization factor of $\eta \leq 1.0$ (accidental load case). This calculation includes a system factor of 1.1 permitted by EN 1995-1-1 [181], applicable for systems that enable load distribution.

A failing main member, hinging itself into a continuous secondary system, will result in an additional loading of the remaining main members of up to 82%, depending on the remaining strength and stiffness of the purlin system (achievable utilisation factor before rupture of the purlins). Applying the accidental load case, this will not result in an utilisation factor $\eta > 0.83$.

Local/Global Effects

Numerous studies of failures in timber structures e.g. [5], [6] and [7] have shown that the majority of failures were not due to statistically random occurrences, e.g. of low material weakness or local defect, e.g. local deterioration of element from local water ingress, but – in the vast majority – due to global (repetitive) defects from systematic mistakes. Structures are usually composed of repetitive elements which are connected by analogical construction principles. This systematic implies that a mistake, made during the planning or construction phase, will most likely repeat itself in all identical elements. Examples of failed structures containing systematic mistakes are e.g. given in [8] and [9]. Combining this conclusion with the concept of load distribution, it becomes evident that a structure containing systematic mistakes will not be able to withstand a large load increase due to load distribution from one failing member, meaning it is more fragile to collapse progressively (see [8]).

Conclusion

Evaluating purlin systems from a structural perspective will highlight continuous systems due to their lowered maximum bending moments, enabling the realisation of larger spacings *e* at given span and cross-section. Due to this and due to the acceleration of the construction process, the majority of purlin systems today are realized by continuous systems like lap-jointed beams.

The evaluation from a robustness perspective reveals more debatable results. Continuous systems (due to their redundancy and higher stiffness) will result in an increased load transfer in the case of failure of one structural member. Many publications on robustness mention this as preferable. Nevertheless, as recent studies have revealed, are most failures of structures not caused by local defects but by global defects from systematic mistakes. Such structures are not able to withstand a large load transfer and will therefore be more prone to progressive collapse. This idea is supported in [10], stating that the “alternate load path” approach (realized by e.g. parallel systems) may “in certain circumstances not prevent but rather promote collapse progression”. Hence, the idea of compartmentalization is introduced which is realized by a deliberate reduction of continuity at chosen compartment borders. For the systems discussed, this approach might be preferable, if the strength and/or stiffness required for the formation of an alternate load path cannot be guaranteed in case of failure of one element. Two failure examples, both featuring systematic mistakes in design and construction, emphasize this. The Siemens-Arena [9], having statically determinate secondary members, sustained a partial collapse after the failure of one main beam while the Bad Reichenhall Ice-Arena [8] suffered a progressive collapse triggered by its very stiff secondary system. These two structures and their particular failure mechanisms with respect to robustness are therefore presented in more detail in a second fact sheet “Robustness Considerations from Failures in two Large-Span Timber Roof Structures” by Munch-Andersen, J. and Dietsch, P.

In summary this means, that there is no strategy for the structural designer, which ensures robustness in all cases. When deciding on a robustness strategy one has to consider different scenarios. The major difference is weather the cause of failure is likely to be a systematic (mostly human) error or an unforeseeable (mostly local) incident. Experience tells that human errors are by far the most common cause. In order to reduce the risk of collapse and in particular progressive collapse, it is crucial to reduce the number of human errors by e.g. enhanced quality control. Only then it would be possible to choose an unambiguously beneficial robustness strategy.

Limitations and outlook to further research:

It is the belief of the author that the given statements are valid for the majority of timber structures. The numerical values given are nevertheless constricted to the example given. To put this comparison on a broader foundation, further comparative calculations on other systems should be carried out. The evaluation should also be extended to a probabilistic approach.

References

[1] EN 1990:2002-04, Eurocode: Basis of Structural Design, CEN, 2002

[2] DS-INF 146:2003, Robustness – Background and principles – Information, Danish Standards Association, 2003

[3] Short, C., Purlins, in: Timber Engineering STEP 2 - Design, details and structural systems, Centrum Hout, Netherlands, 1995

[4] EN 1995-1-1:2004-11, Eurocode 5: Design of Timber Structures – Part 1-1: General – Common rules and rules for buildings, CEN 2004

[5] Blaß, H.-J., Frese, M., "Failure Analysis on Timber Structures in Germany - A Contribution to COST Action E55", 1st Workshop, Graz University of Technology, Austria, 2007

[6] Frühwald, E., Serrano, E., Toratti, T., Emilsson, A., Thelandersson, S., Design of safe timber structures – How can we learn from structural failures in concrete, steel and timber?, Report TVBK-3053., Div. of Struct. Eng., Lund University, 2007

[7] Dietsch, P., Wolfrum, A., Winter, S.: "Evaluation of wide-span timber structures - results and recommendations, WCTE 2008 Conference Proceedings, Miyazaki, Japan, 2008

[8] Winter, S., Kreuzinger, H., 2008: "The Bad Reichenhall ice-arena collapse and the necessary consequences for wide span timber structures". Proceedings WCTE 2008 Conference 2008, Miyazaki, Japan

[9] Munch-Andersen, J.: "What is a robust construction?", 4th COST E55 Workshop, Zagreb University, Croatia, 2008

[10] Starossek, U.: "Progressive Collapse of Structures: Nomenclature and Procedures". Structural Engineering International, 2/2006, 113-117