ROBUSTNESS OF SECONDARY STRUCTURES IN WIDE-SPAN TIMBER STRUCTURES

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ABSTRACT: Design rules for robustness require insensitivity to local failure and the prevention of progressive collapse. This paper will evaluate typical secondary structures for timber roof structures against these requirements, comparing the results against typical reasons for damages and failure. Applying the finding that most failures of timber structures are not caused by local effects but by global (systematic) defects, it is shown that the objective of load transfer - often mentioned as preferable - should be critically analysed. Based on these findings, proposals for structural systems and details towards a robust design of wide-span timber structures are given.

KEYWORDS: timber, secondary structures, robustness, determinate structures, redundant structures, local effects, global effects, human errors

1 INTRODUCTION

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 paper 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.

2 ROBUSTNESS REQUIREMENTS FOR TIMBER STRUCTURES

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 [2]. 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.

3 SECONDARY STRUCTURES FOR WIDE-SPAN TIMBER STRUCTURES

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 Figure 1.

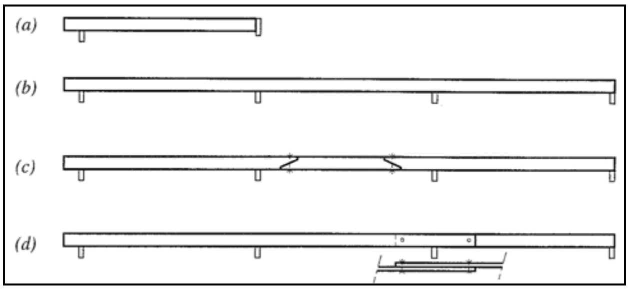

Figure 1: *Typical purlin systems (from [3])*

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

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4 EXEMPLARY COMPARATIVE CALCULATIONS ON TYPICAL PURLIN SYSTEMS IN TIMBER

4.1 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 shown in Figure 2 and Figure 3.

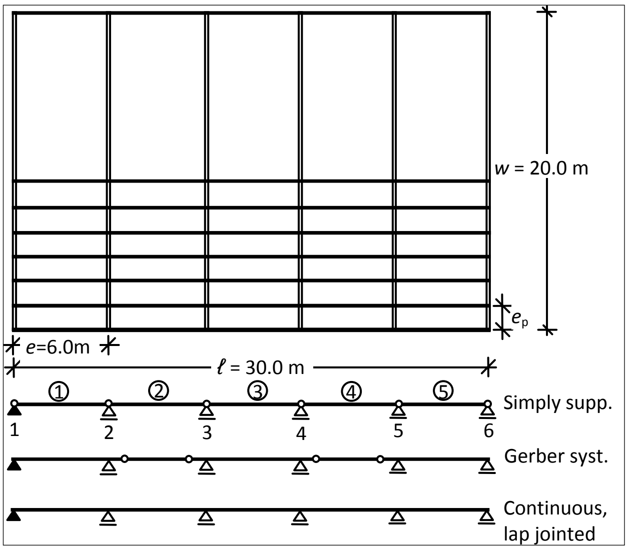


Figure 2: *Schematic layout of evaluated structure and possible purlin systems*

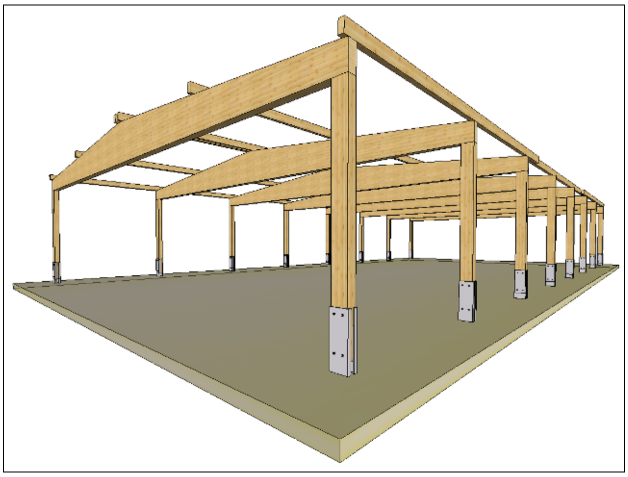


Figure 3: *Isometric sketch of structure comparable to evaluated system*

The chosen roof, featuring an angle of 10° and covering an area of ℓ/w = 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 for this evaluation. The purlins, featuring a cross section of b/h = 100/200 mm^2 shall be realized with grade C24 timber. Their spacing e_P be chosen so that each purlin system has a utilization factor (ULS) of $0.9 < \eta < 1.0$. 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 spacings e, given in Table 1.

Table 1: *Realizable spacings e_P between the purlins at $0.9 < \eta < 1.0$ for different purlin systems and given boundary conditions*

Purlin system	Spacing e_P [m]
Simply supp. beam	1.0 m
Gerber beam	1.3 m
Continuous beam	1.3 m
Lap jointed purlin	1.6 m

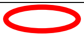
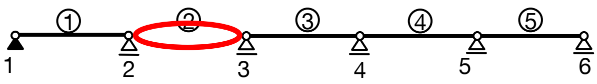
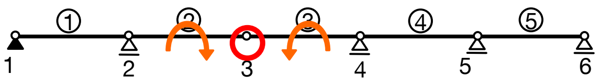
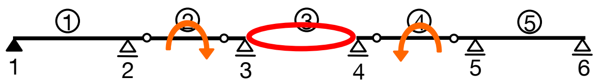
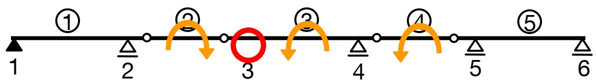
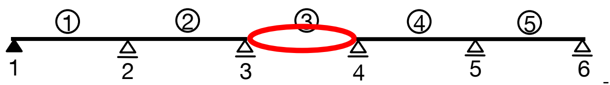
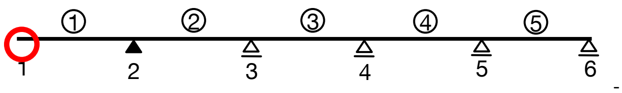
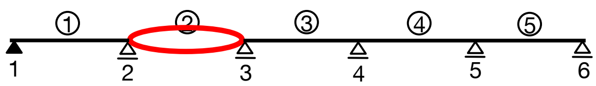
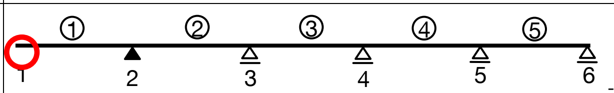
4.2 COMPARATIVE DETERMINISTIC CALCULATIONS

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).

Table 2 (given on next page) lists the purlin systems and evaluated case of failure (column 1), indicating the removed member and additional members failing due to system instability (column 2). 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 "situation after an accidental event" ($\gamma_G = \gamma_Q = \gamma_M = 1.0$; $\psi_{1,snow} = 0.5$; $k_{mod,acc}$). Since the system is symmetrical, only elements 1 – 3 are listed.

Table 2: *Evaluated purlin systems, removed members and increase in bending stresses on remaining members*

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

5 ROBUSTNESS ASSESSMENT OF TYPICAL PURLIN SYSTEMS IN TIMBER STRUCTURES

5.1 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%.

5.2 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). This subject is further evaluated in Chapter 7. 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 [4], 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 $\eta > 0.83$. In the case of failure due to local damage, this utilization factor is not critical. But if all members suffer from the same damage due to global effects, 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 [5]).

5.3 LOCAL / GLOBAL EFFECTS

Numerous studies on failures in timber structures e.g. [6], [7] and [8] have shown that the correlation of failures or developing weak spots is larger than assumed, meaning global damage from systematic (repetitive) mistakes is much more common than local damage (e.g. local deterioration of elements from local water ingress) or statistically random occurrences (e.g. low material weakness). The reason is that structures (primary and secondary 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. Figure 4 shall illustrate above given statement.

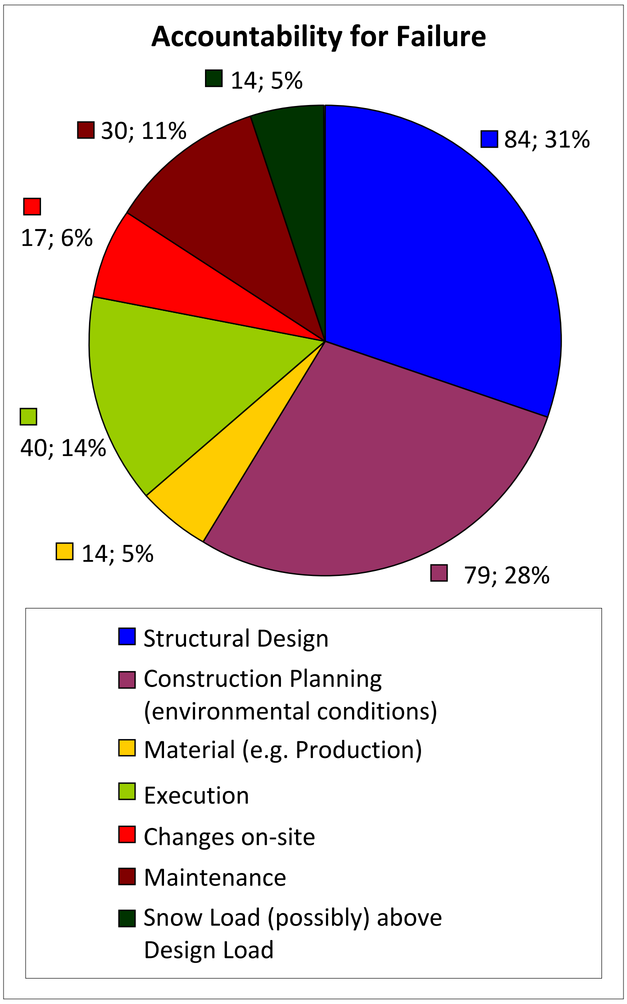


Figure 4: *Accountabilities for failures from an evaluation of 214 cases of failed timber structures [8]*

Resulting from a project carried out at the Chair for Timber Structures and Building Construction, evaluating 214 cases of failed timber structures [8], Figure 4 pictures the accountabilities for evaluated failures. It can be concluded that 70% of the errors (Design, Planning and Maintenance) will - with an utmost probability - have a global effect while the remaining 30% of failures can either result in global or local damages. These numbers are comparable with the data given in [9], comparing multiple studies on failures in structures of all building materials. That study by Ellingwood reveals an average of 45% failures due to errors in design and planning, 38% due to construction errors and 17% due to errors in the utilization phase (maintenance).

6 DISCUSSION

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 the redistribution of loads as preferable, which is true in the case of local effects, e.g.

- local deterioration of element from e.g. local water ingress
- local weakening of element from e.g. holes
- local overloading from e.g. local snow accumulation.

Nevertheless, as recent studies have revealed, are most failures of structures not caused by local defects but by global defects from systematic mistakes (see Chapter 5.3). Such effects can include:

- global weakening of structural elements due to systematic (repetitive) mistakes
- global deterioration of elements from e.g. wrong assumption of ambient climate
- global overloading from e.g. addition of green roof without structural verification

Structures suffering from global damaging effects 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 errors in design and construction, emphasize this.

The Siemens-Arena (described in detail in [5] and [11]) suffered from gross errors in the structural design, reducing the load-carrying capacity of the heel joint of the fish-shaped truss to 25% - 30% of its required strength. Due to this, two of the 72 m long trusses collapsed without warning and under very low variable loads, shortly after the opening of the arena (see Figure 5). During design it was decided "that the 12 m long purlins between the trusses should only be moderately fastened to the trusses, such that a failure of one truss should not initiate progressive collapse. This strategy proved to work fairly well as "only" two of the 12 trusses collapsed. Considering that all trusses had a much lower strength than required it might be fair to conclude that the extent of the collapse was *not* disproportionate to the cause." (from [5])

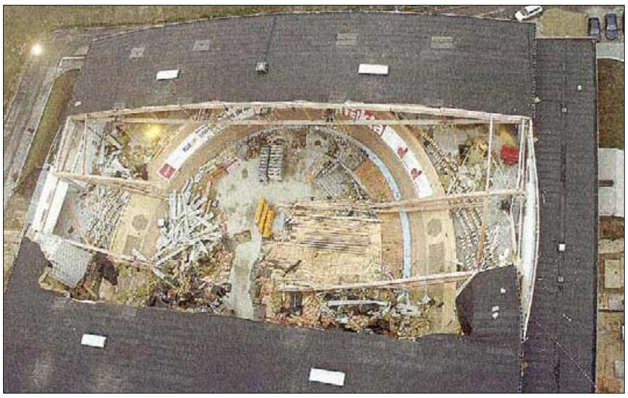


Figure 5: *Top view on Siemens-Arena Ballerup, Denmark, collapse of 2 out of 12 main trusses [11]*

The Bad Reichenhall Ice-Arena (further described in [12]), featuring timber box-girders with lateral web boards made from so-called "Kämpf web boards", suffered from multiple errors and defects, including cumulative degradation processes in the glue-lines and finger joints due to the humidity exposure over the years.

Figure 6: Top view on Bad Reichenhall Ice-Arena after progressive collapse of all main girders [12]

This eventually triggered a progressive collapse of the whole roof structure after approximately 34 years of use (see Figure 6) under a large but not exceptional snow load. The investigation concluded that the failure most probably initiated in one of the three main girders on the east side. Due to fact that the secondary system, which was realized as a K-bracing to also function against lateral-torsional buckling, was not only strong but also very stiff, the loads were shifted from the girder that failed first to the neighbouring girders. Since these girders suffered from the same errors and degradation processes as the girder failing first, they could not sustain the additional load. Consequently, this developed into a progressive collapse which realized within seconds. The very stiff secondary system also resulted in the fact that a weak girder (containing e.g. general finger joints having lost their adhesion) could transfer its loads to the adjacent girders without large deformations, meaning that this form of advance warning of failure was impeded.

In summary it can be stated 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.

7 PROPOSALS TOWARDS DESIGNING FOR ROBUSTNESS IN TIMBER STRUCTURES

Robustness strategies can be ambiguous, as outlined in the previous Chapter, since they depend on the failure scenario. In this Chapter, some ideas are outlined with the aim to reduce failures of primary structural elements while decreasing the possibility of a progressive collapse in the case of one failing element. For this reflection, seismic situations have not been considered since they oftentimes require a different treatment. The ideas presented are solely based on structural considerations, not on the objective of efficiency and cost-effectiveness.

Structural systems for wide-span timber structures are often composed of a statically determinate primary structure (e.g. single-span beams), carrying a statically indeterminate secondary structure. Against the background of above given statements, this scenario of redundancy should be reversed, meaning that primary elements should become more redundant while secondary structures should be designed as determinate systems to achieve the objective of compartmentalization.

7.1 Primary Structures

Amongst the most typical statically indeterminate structural systems for timber halls are frame systems, oftentimes realized with V-shaped columns (see Figure 7). Another possibility to increase redundancy of primary structural elements is to introduce internal indeterminacy as in beams which are trussed with sag rods (see Figure 8).

For such systems it seems feasible to consider the failure of one structural element (e.g. the steel rods), designing the remaining element to withstand the stress resultants in the changed structural system, applying the accidental load case. However, this could imply that the elements would be over-designed in the ULS.

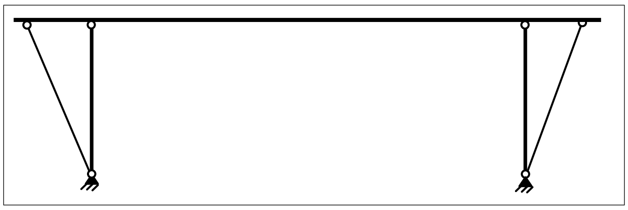


Figure 7: *Indeterminate frame system with V-shaped frame corner*

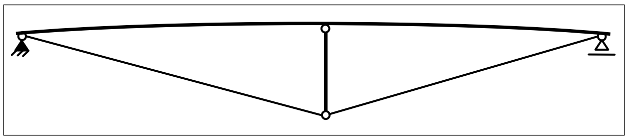


Figure 8: *Example of system with internal indeterminacy, beam trussed with sag rod*

7.2 Secondary Structures

Typical purlin systems for timber structures have to fulfil two requirements:

a) to carry the vertical loads from the roof structure (e.g. self-weight and snow) and to transfer them to the primary structural elements

b) to perform as part of the bracing system, transferring the horizontal loads (stability and wind loads) to the vertical bracing system (e.g. the exterior walls).

This dual function causes the main difficulties when considering robustness and the objective of realizing compartmentalization. To obtain functionality as bracing against wind loads and lateral torsional buckling of the primary members, the purlin systems are realized to transfer horizontal (axial) loads in tension and compression. This implies that, in the case of one main member failing, the purlin systems will develop into a tie member, thereby transferring the vertical loads from the failing member to the adjacent members (see Figure 9).

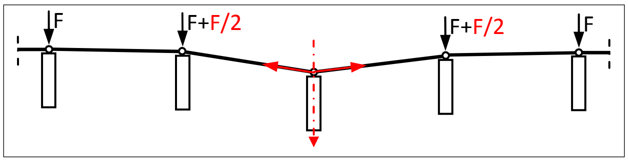


Figure 9: *Example of purlin system acting as tie member in the case of failing main member (to be avoided)*

A first design approach would be to differentiate between the purlins according to their functions since not every purlin has to act as part of the bracing system. This means that purlins which are not part of the bracing system should be designed as to not transfer axial (horizontal loads) while purlins that act supplementary as part of the bracing system should not “hinge” themselves into the remaining main members in case of failure. The latter could be achieved by not “over-designing” the connections between the determinate purlins for axial forces, meaning that the purlins would only develop a tie member until the system reaches the axial design forces from horizontal loads. This could be achieved by e.g. matching the amount of nails used in hangers to these forces. Another connection type which would enable this and potentially lead to a detachment of the purlin system and the main member in case of failure is sketched in Figure 10. Many producers of connectors offer systems which feature illustrated mechanisms since such connector types are also known to decrease assembly time.

For such cases in which vertical loads should basically only be transferable in compression it is important to check if lifting forces like wind-suction are compensated by the self-weight of the roof. If this is not the case (e.g. in edge regions), these lifting forces have to be locally anchored whereby the anchoring device should adhere to the mentioned requirements (easy detachment in case of failure).

It is self-evident that the roof cladding be constructed so as to not support the formation of a tie member in the case of failure. While it might be worth considering a roof cladding which can carry the loads of a failing purlin, it should definitely not support load transfer in the case of a failing main member.

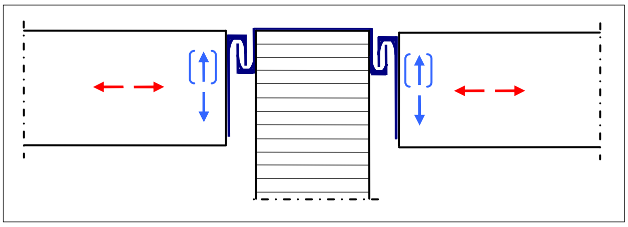


Figure 10: *Connection to transfer horizontal and vertical loads, potentially enabling detachment in case of failure*

Pushing these ideas further it seems plausible to consider a clear separation of both functions stated above. This would result in a purlin system, designed to only transfer vertical loads and one separate bracing system to carry all horizontal loads. A possible layout of this detail is sketched in Figure 11.

The supports of the purlins on the main members could then be designed to only carry vertical loads, while simply a slight horizontal fastening would be needed to secure their position. The bracing system would still be designed to transfer horizontal loads in tension and compression but - due to the separation of both systems - would not transfer any vertical loads to the neighbouring beams in the case of one main member failing. Nevertheless can such a design only be fully beneficial if easy detachment of main member and bracing system is enabled (as indicated by the channel section, only horizontally stabilizing the main beam) and the purlins and bracing elements are not placed within the same plane. Although only bracing elements perpendicular to the main member are sketched in Figure 11, it is self-explanatory that cross bracing is needed in at least one, generally two fields to transfer the horizontal loads to the vertical bracing elements. Load transfer in case of failure will be more pronounced between two primary members adjacent to a horizontal cross bracing, making these members key elements which should be given special consideration during design.

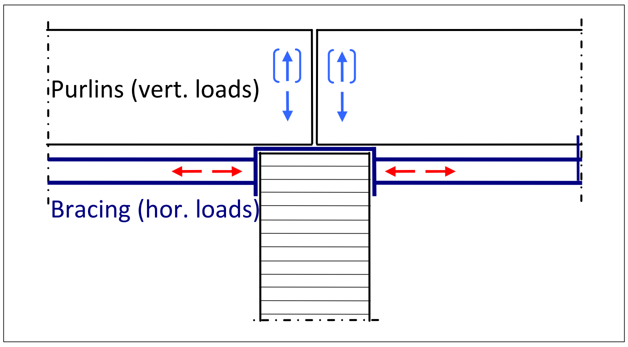


Figure 11: *Separation of load bearing structure for horizontal and vertical loads, enabling detachment in case of Failure*

Modifying the above given possibilities, it seems feasible to consider a bracing system which is designed to only carry axial (horizontal) loads in compression as given in Figure 12. This would involve the construction of at least two cross-bracings since horizontal loads could only be transferred unidirectional (see Figure 13).

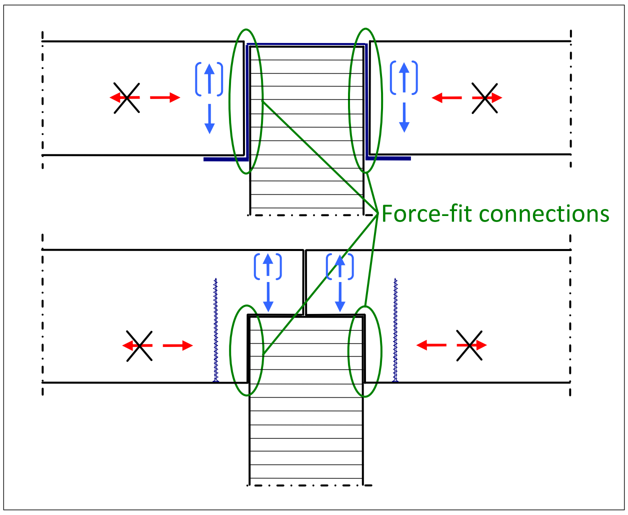


Figure 12: *Connections to transfer axial compression forces and vertical loads, enabling detachment in case of failure*

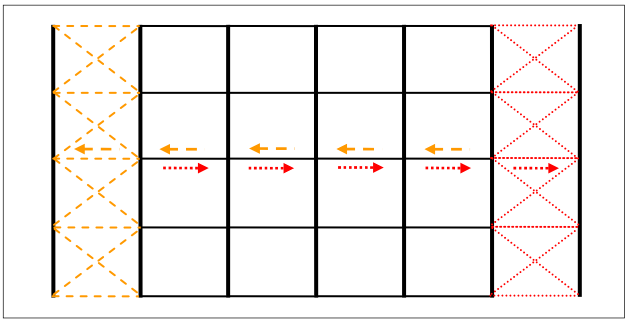


Figure 13: *Plan view of necessary cross bracings in the case of unidirectional transfer of horizontal loads due to connection type sketched in Figure 12*

Such a system needs exact execution to obtain force-fit connections. Nevertheless it would eliminate the necessity to design connections for axial tension forces, thereby avoiding the development of tie members in the case of failure. This alternative can be realized with two separate systems or one purlin system fulfilling both functions (transfer of vertical and horizontal loads).

A comparable system, enabling easy detachment between secondary members, hangers and main members was applied in the roof structure of an exhibition centre. When one main beam failed due to the corrosion failure of its appendant steel suspension cable, did the purlins not develop into tie members but developed hinges at the supports, limiting the failure to one field (see Figure 14).

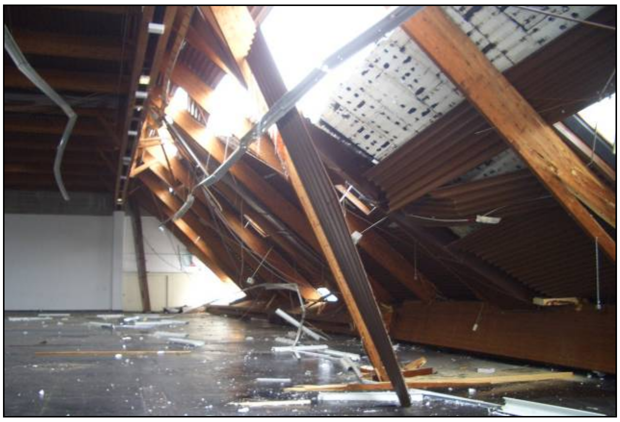


Figure 14: *Exhibition centre, failure of one main beam, development of hinges at supports of purlins [MPA BAU, TUM]*

The final alternative to realize connections between the primary beams and the purlins which only transfer vertical forces, thereby enabling easy detachment in the case of failure of one element is to design the primary beams as internally stable against lateral torsional buckling, also being capable to transfer external horizontal loads (e.g. wind loads). This is only achievable if the primary beams be designed less slender or with a T-section as sketched in Figure 15.

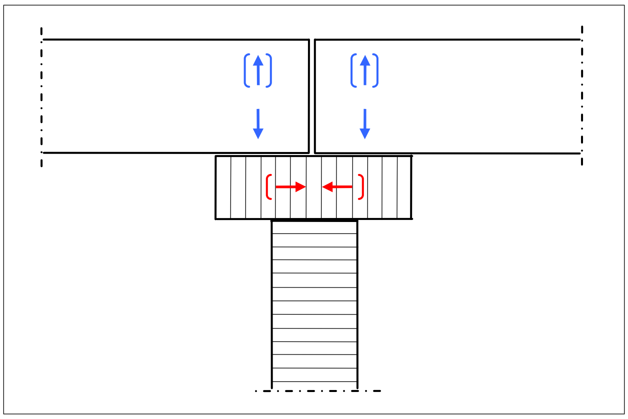


Figure 15: *Primary beam with cross-section to enable internal stability against lateral torsional buckling, also capable to transfer external horizontal loads (e.g. wind loads)*

8 OUTLOOK AND FURTHER RESEARCH

It is the belief of the authors that given statements are valid for the majority of wide-span timber structures. The numerical values given are nevertheless constricted to the example given in Chapter 4. To put this comparison on a broader foundation, also considering a probabilistic approach, a research project “risk-based assessment of robustness and collapse behavior of secondary Structures in wide-span timber structures” has been started by the authors in collaboration with Prof. Dr.-Ing. Daniel Straub, Engineering Risk Analysis Group at TUM.

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