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A holistic framework for designing for structural robustness in tall timber buildings

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ABSTRACT

With the ever-increasing popularity of engineered wood products, larger and more complex structures made of timber have been built, such as new tall timber buildings of unprecedented height. Designing for structural robustness in tall timber buildings is still not well understood due the complex properties of timber and the difficulty in testing large assemblies, making the prediction of tall timber building behaviour under damage very difficult. This paper discusses briefly the existing state-of-the-art and suggests the next step in considering robustness holistically. Qualitatively, this is done by introducing the concept of scale, that is to consider robustness at multiple levels within a structure: in the whole structure, compartments, components, connections, connectors, and material. Additionally, considering both local and global exposures is key in coming up with a sound conceptual design. Quantitatively, the method to calculate the robustness index in a building is presented. A novel framework to quantify robustness and find the optimal structural solution is presented, based on the calculation of the scenario probability-weighted average robustness indices of various design options of a building. A case study example is also presented in the end.

1. Introduction

1.1. Structural Robustness

If damage happens to a structure (for example column failure), which progressively triggers further consequences disproportionate to the extent of this initial damage (for example partial or total collapse), this structure is said to lack structural robustness. As the potential consequences of the damages become larger, so does the importance of structural robustness. Structures whose purpose is to protect human lives must incorporate robustness design [1].

There are some famous structural collapses where a more robust design could have avoided severe consequences, for example the Ronan Point partial collapse in London in 1968. A small gas explosion on the 18th floor (of 22) triggered the partial collapse of an entire corner of the structure, causing the death of 4 people [2].

Other such famous collapses are the partial collapse of the Alfred Murrah Building in Oklahoma in 1995 following a terrorist bombing [3], and the collapse of the roof of the Bad Reichenhall Ice Arena in Germany in 2006 [4]. Further examples are presented in Agarwal et al. [5].

1.2. Tall Timber Buildings

In the late 19th and early 20th century, the old growth forests of the Pacific North West of the USA and Canada had large enough trees to produce solid timber building components of sizes that are unheard of today. This allowed impressively large buildings to be constructed in North America more than one hundred years ago. This typology, called “heavy timber”, has left examples that are still in use today, such as the Leckie Building in Vancouver [6].

Due to height limitations imposed by building codes in North America from the 1940s, combined with the abundance of cheaper material alternatives like concrete, the traditional heavy timber buildings fell out of favour in the second half of the 20th century [7]. However, environmental concerns, coupled with modern manufacturing technologies and prefabrication in timber that are able to produce

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building components that far exceed their old growth ancestors, have enabled engineers to design and build larger and taller timber buildings than ever before. With the title for the world's tallest timber building having broken 4 times in the years 2013 to 2019, the timber industry is seeing a trend of unprecedented pace. Modern tall timber buildings can be constructed sustainably, fast, and at high quality [7]. At 85.4 meters, Mjøstårnet (the Tower of Lake Mjøsa) in Norway opened its doors to tenants in March 2019, boasting the title of the tallest timber building in the world (Figure 1) [8,9]. Timber buildings of much larger size have already been proposed and preliminarily studied, for example the Oakwood tower in London, UK [10], SOM's 40 storey timber tower project [11], and River Beech Tower in Chicago, USA [12].

Wood is a complex material [13,14]. Among its numerous beneficial properties and benefits of timber buildings, there are some important challenges that must be highlighted, particularly when such timber buildings are becoming taller. These are:

- Sensitivity to moisture: timber is a natural grown, hygroscopic material which degrades significantly when it remains wet for a long time. From light swelling to complete loss of structural strength due to fungi attack, it is very important to design timber to remain protected from high moisture in its structural lifetime, particularly for highly loaded components.
- Light weight: timber used in construction is approximately 5 times less dense than reinforced concrete and 15 times less dense than structural steel. The direct advantage of a lighter building with smaller foundations has the pitfall of being much more sensitive to critical lateral loads as the height of the building increases.
- Orthotropic: As a natural grown material, the properties of timber are not the same in every direction. Timber is strong along the fibres, but very weak across them. Failing to address this can have catastrophic consequences.
- Low stiffness: timber used in construction is approximately 3 times less stiff than reinforced concrete and 20 times less stiff than structural steel. At increasing heights this can have a severe impact on deflections, accelerations, and occupant comfort. Furthermore, and combined with the orthotropic behaviour, it becomes challenging to construct stiff, moment resisting connections.
- Brittleness: timber under tension, bending and shear is brittle in failure, although with careful design ductility can be achieved, in particular in connections with steel fasteners. Brittle behaviour is particularly unwelcome when a structure is called to redistribute loads, for example in the case of an accidental event requiring robustness.
- System effects: although this applies to all materials, the way loads are distributed in a system is less clear, particularly for heterogeneous timber components. In the case of large timber structures subject to abnormal loads and potential damage, better knowledge of system behaviour is important.
- Size effects: A significant overall strength reduction is possible in the case of a large structural element. This applies to all materials, however the size effects in very large timber elements are still rather unknown, with preliminary indications that they can be significant [15].
- Time effects: timber creeps with time, which can be critical in heavily loaded structures like tall timber buildings. Differential settlement in hybrid buildings including loadbearing timber elements is even more challenging [16].

Figure 1. Mjøstårnet, the world's tallest timber building to date (Photo credits ©Moelven)

The above properties make the design of a timber building everything but straightforward, particularly at larger scales. There is still little experience with tall timber buildings, and the difficulty to test large assemblies makes their understanding even more difficult. The behaviour of tall timber buildings when damaged is a largely unknown area. Therefore, designing them for robustness is a crucial research topic that requires more attention in order to avoid potential catastrophic failures happening due to poor understanding of systems when scaled up, as for example with the Ronan Point Tower [2].

In this paper, the state-of-the-art, state-of-research, and the existing practice regarding structural robustness are presented. The main focus of the paper is the discussion of an improved framework for considering robustness in a structure, which stems out of the consideration of scale within a structure, and organising the design approaches based on the scale and the type of exposure in question (see Figs. 2 and 3).

2. Robustness state-of-the-art and existing practice

2.1. Definitions of Robustness

There are currently numerous definitions of structural robustness in the literature, which imply the same things but have slight differences, often causing confusion. The differences are largely in the context. For example, Ellingwood [17] provides the following definition:

"A progressive collapse of a building is initiated by an event that causes local damage that the structural system cannot absorb or contain, and that subsequently propagates throughout the structural system, or a major portion of it, leading to a final damage state that is disproportionate to the local damage that initiated it".

The Eurocode definition is rather event specific [18]:

"Robustness: the ability of a structure to withstand events like fire, explosions, impact or the consequences of human error, without being damaged to an extent disproportionate to the original cause".

Confusion mainly arises from the lack of precise definition of terms such "damage" and "disproportionate". Regardless, the widely accepted notion of robustness is that of the relation between direct ("initial") and indirect ("subsequent") consequences from any event. Huber, et al. [19]

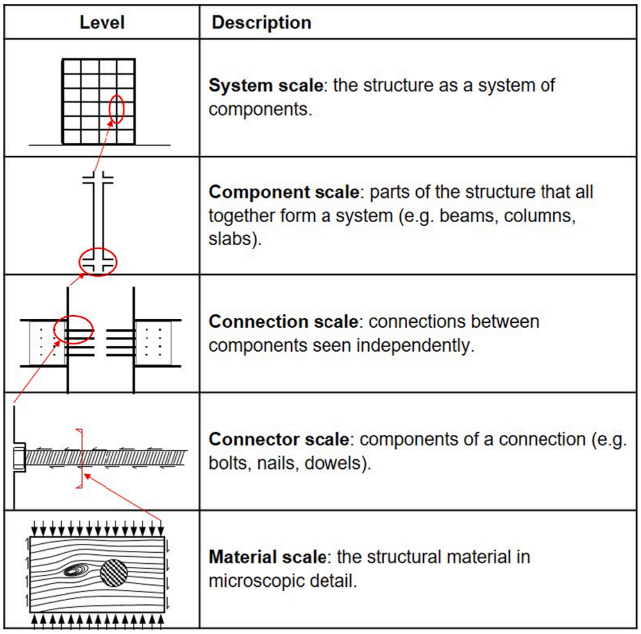


Figure 2. Levels of a building

and Adam, et al. [20] discuss the various definitions and make useful clarifications in their respective review papers on structural robustness.

2.2. Formal Definitions

Further clarifications of the robustness-related terms are presented in [17,21], and [22]. It is generally accepted that robustness is only one of three parts of the overall disproportionate collapse resistance, as indicated in Equation (1) from [21]:

$$P(C) = P(E) \times P(D|E) \times P(C|D) \tag{1}$$

The equation reads: "The probability of collapse equals the probability of an exposure to the structure (P(E), or "Exposure"), times the probability of initial damage occurring in the structure given the exposure has occurred (P(D|E), or "Vulnerability"), times the probability of subsequent disproportional collapse, given the initial damage has occurred (P(C|D), or "Robustness").

It is therefore important to realise that a structure can be designed to resist disproportionate collapse by decreasing its exposure and/or vulnerability, leaving robustness unaffected. As a consequence of that, a structure can resist collapse without being robust. Given, however, that this last term of the equation is what the designer can best control, increasing robustness is currently the preferred method to minimise the probability of disproportionate collapse [23].

The equation has been defined in the structural engineering context, but it can be adapted to any context where P(C) is the probability of consequences (rather than collapse) related to an exposure. For example, the consequences in question can be the rotting of a building in terms of moisture, with the exposure representing the water ingress, the vulnerability representing the moisture damage in a component of the building, and robustness representing the spread of the moisture damage in the rest of the building.

An extended version of this equation will be used in this paper that considers consequences together with probabilities, in line with [24] and [25]. This is the only way to consider the (dis)proportionality of consequences, which is at the heart of the robustness definitions. Thus, the value of interest is the expectation of consequences, given by the sum of expected direct consequences (C_{Dir}) and expected indirect consequences (C_{Ind}):

$$\mathbb{E}[C] = \mathbb{E}[C_{Dir}] + \mathbb{E}[C_{Ind}] \tag{2}$$

The expectation of direct consequences is related by definition to the exposure and vulnerability terms of Equation (1), whereas the expectation of indirect consequences is related to all three terms of the original equation. Therefore Equation (2) becomes:

$$\mathbb{E}[C] = (P(E) \times P(D|E)) \times C_{Dir} + (P(E) \times P(D|E) \times P(C|D)) \times C_{Ind} \tag{3}$$

A clarification in the definition of the equation is desirable, to maintain consistency of the terms: P(C|D) is by definition the complement of robustness, since higher robustness means lower probability of indirect consequences given a direct effect (that is, P(C|D) reduces).

To actually quantify robustness, several risk-based measures exist, described in detail by Adam, et al. [20]. In this paper, we will use the robustness index defined by Baker, et al. [24], which is the fraction of the total risk (due to direct plus indirect consequences) caused by the risk by the direct consequences only:

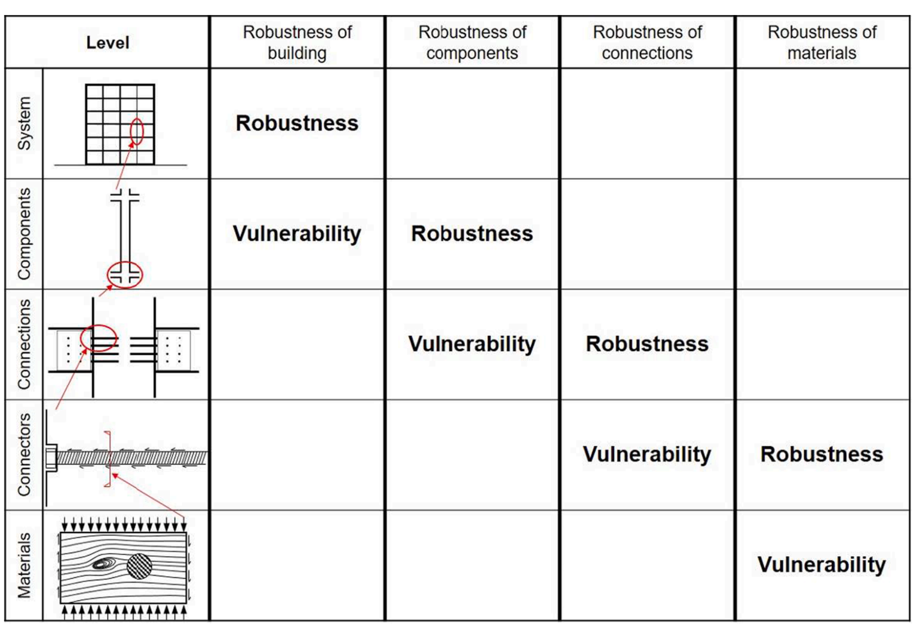


Figure 3. Multi-level robustness

$$I_{Rob} \stackrel{def}{=} \frac{\mathbb{E}[C_{Dir}]}{\mathbb{E}[C_{Dir}] + \mathbb{E}[C_{Ind}]} = \frac{(P(E) \times P(D|E)) \times C_{Dir}}{(P(E) \times P(D|E)) \times C_{Dir} + (P(E) \times P(D|E) \times P(C|D)) \times C_{Ind}} \tag{4}$$

Equation (5) gives the simplified expression, where the probabilities of the exposure and the initial damage cancel out:

$$I_{Rob} = \frac{C_{Dir}}{C_{Dir} + P(C|D) \times C_{Ind}} \tag{5}$$

An index of zero means that all the risk is due to the indirect consequences (no robustness). An index of one means that all the risk is due to the direct consequences only (excellent robustness). The cancellation, however, of $(P(E) \times P(D|E))$ means that this robustness index is a relative measure of robustness for a specific damage scenario. If, for example, a given structure has a robustness index close to one (very high robustness) for an extremely improbable event, this does *not* mean that the structure is robust for all events, as there might exist a much more likely damaged state with a smaller robustness index. The cancelled out probabilities of exposure and initial damage must be considered in the overall design of the building, as it will be explained in section 4 of this paper.

2.3. Existing Practice

There are three broad categories of design approaches to structural robustness, in order of decreasing complexity: a risk approach, where the risks of direct versus indirect consequences of an action are compared [24,25], a reliability approach, where the probabilities of failure of the structure at the undamaged and damaged stages are compared [26], and a deterministic approach, which evaluates the structural reaction of a building for assumed damage scenarios [19]. The risk approach is the only one that attempts to quantify the relation between cause and effect and is therefore considered the most complete and comprehensive approach for the analysis of a structure in an accidental design situation. In practice, however, deterministic analyses are the most common, with most attention focused on the robustness component of the collapse resistance equation and by employing primarily structural design methods, despite there being numerous non-structural design methods, for which more details are provided in [21].

These deterministic structural design methods are either direct or indirect. Indirect methods aim to enhance a building's robustness without considering specific components or scenarios. Examples are tying some or all structural components together to enhance system performance, and providing redundancy at certain locations, or globally. Direct methods are applied for specific damage scenarios, for example a failed column due an explosion. Examples are the Alternate Load Path Analysis (ALPA), designing key elements, and splitting the structure into different compartments to limit the spread of structural damage. A detailed review of the deterministic design methods is included in [27] and [19].

The Finite Element Method (FEM) is most popularly used to model and design with the above direct methods. The complexity of modelling can vary from simple, conservative linear static analyses, to complex, more accurate non-linear dynamic analyses. In addition, there are other methods available such as the Discrete Element Method (DEM), and the Applied Element Method (AEM). Huber, et al. [19] and Adam, et al. [20] provide further details on the history and application of the various analysis methods.

The topic of robustness is addressed in most major design building codes around the world, but instructions are typically generic and without much guidance to the engineer. In the Eurocodes robustness is only addressed in the accidental design part of Eurocode 1 [28], where the designer chooses from the suggested methods depending on the consequence class (1, 2A, 2B, 3) of the building. No material-specific provisions exist, however there are plans to include timber-specific provisions in the new Eurocode 5 [29]. Design methods such as alternate load path analysis (ALPA), minimum tie forces, and key element design are recommended, but no guidance or explicit design criteria are given. A systematic risk assessment is recommended for consequence class 3 buildings [18]. In Switzerland, robustness is a basic requirement from the conceptual stage of a project [28], with further guidance given specific to each material in the other parts of the SIA suite (for example ductility addressed in SIA 265 for timber). In the USA, the Department of Defense (DoD) [30] splits the buildings into four risk categories, similar to the Eurocode's consequence classes. The same methods (ALPA, tying, key elements) are again available in the engineer's palette, however it includes much more detailed guidance on how to perform such designs. More details on the code provisions regarding robustness can be found in [27] and [31]. The latter carried out a global survey on the existing structural engineering practice with regard to robustness design. In the survey, the authors found that not all structural engineers consider robustness in their designs, and many are not even familiar with the topic. There exists a general dissatisfaction with the provisions of the building codes, amplified in countries where the code does not include clauses on robustness. In general, there is a preference towards more performance-based code provisions rather than prescriptive rules, with more guidance on robustness and specific examples on case studies.

2.4. Robustness Research in Tall Timber Buildings

For many decades there has been very little research on the robustness of multi-storey timber buildings with the main research project being the Timber Frame (TF2000) project in the UK [32,33]. The project involved the testing of a six-storey light-frame timber building by removing different wall panels. The results were used to give connection detailing recommendations and calculate tie force provisions. However, these conclusions do not necessarily apply to modern tall timber buildings made with mass timber elements such as glued laminated beams and columns and cross-laminated timber (CLT) slabs.

Mpidi Bita [34] in Vancouver, Canada, has been leading robustness related research on tall timber buildings. He has performed disproportionate collapse analyses on both a twelve-storey CLT building with platform construction, and a nine-storey flat-plate CLT building. In the 12 storey CLT platform construction study, non-linear dynamic ALPA using ground floor wall removals were used to model the reaction of the building to initial damage and the strength and stiffness demands of the components to resist disproportionate collapse. A reliability analysis was carried out at the components, concluding that the structure has a probability of failure of up to 32% if designed without considering structural robustness. Connection detailing has been identified as the most critical robustness aspect, as it is largely responsible for developing catenary action in the floor system to resist collapse. In the 9 storey flat plate CLT study, non-linear dynamic ALPA using 11 different ground floor column removals was used to model the reaction of the building to initial damage and the strength and stiffness demands of the components to resist disproportionate collapse. In a similarly high probability of failure, the critical areas of improvement identified were in the axial tension capacity of the column-to-column connections, the rotational capacity of the floor-to-column connections, and the floor-to-floor axial and shear resistances.

Mpidi Bita has also carried out experiments on mass-timber floor systems in an attempt to develop the necessary catenary action required for a collapse resistance mechanism [35]. By comparing assemblies using conventional (self-tapping screws and angle brackets) and novel (steel tubes and rods) connections, he found that using the novel connection design, the proposed floor systems can achieve enough strength, stiffness, and ductility to activate catenary action and resist disproportionate collapse.

Mpidi Bita [36] is also researching on the application of the alternative load path analysis method in a six storey timber office structure, in order to give guidance to industry practitioners with simplified analysis tools.

Huber et al. [37] at Luleå University, Sweden, have looked into the detailed modelling of typical connections in platform type CLT buildings. Specifically, finite element analyses were run for self-tapping screws (STS) and nail-fastened angle brackets. The goal was the understanding of the connection mechanics between timber elements and individual connectors in order to accurately model the resistance mechanisms to progressive collapse in a CLT building. Initial models showed that progressive collapse can be induced by a zipper-like progressive failure of the floor-to-floor connectors.

Lyu et al. [38] at Griffith University, Australia, have carried out large deformation experiments on ¼ scale, 2-bay 2D timber frame substructures in the middle column loss scenario to investigate the possibility of the timber beams developing catenary action. During a quasi-static internal column removal of the two span frame, the midspan deflection and end rotations of the beam were measured and plotted in graphs, comparing three connections with existing commercially available beam-to-column connectors and one proposed novel connection. The connections are evaluated with respect to the amount of catenary action they are able to activate. It was found that the commercially available connectors may be able to develop some catenary action, but the novel connection had a substantially increased ductility. None of the connectors, however, allowed the frame to survive a column loss scenario as required in the DoD guideline [30]. In contrast, the novel connection allowed the frame to survive a column loss scenario as required in the IStructE guidelines and the Australia/New Zealand standards, which do not consider a dynamic amplification factor.

3. Qualitative Framework

3.1. Structures on Different Levels

Designing structural components can be a simple cross section check for a small member like a timber joist, or a very lengthy, elaborate process for very large structures. For example, the design of skyscraper mega-columns are entire projects themselves. This paper does not discuss structures of extreme size like skyscrapers, however the concept of the different scales, or levels of the structure, is used to consider robustness more holistically.

Consider a building structure in the following levels, in top-down order from global to local:

3.2. Robustness on Different Levels

In literature and practice, the interpretation of Equation (1) (Collapse resistance = Exposure x Vulnerability x Complement of Robustness) is at the scale of the entire building: vulnerability corresponds to the initial damage as a failure at the component level, and robustness corresponds to the disproportionality of consequences in the whole building level. The most commonly discussed example is the column loss scenario: vulnerability corresponds to the column failure, and robustness corresponds to the whole building as a system arresting disproportionate collapse due to the initial column failure. This is one valid application of the equation, *but not the only one.*

Consider the equation for a single component only: vulnerability would correspond to the initial damage at a certain location within the component (local failure), and robustness would correspond to the disproportionality of consequences in the entire component. An example is buckling in a steel column: vulnerability corresponds to the initial local buckling at, say, a part of the flange, and robustness corresponds to the ability of the entire column as a system to prevent disproportionate failure, or collapse, by preventing the local buckling in question to spread into global buckling failure (for example with web stiffeners).

By combining the two examples above it can be seen that the robustness failure of the steel column (component level) and the vulnerability failure of the column (whole building level) are the same event seen from different perspectives. It is therefore possible to express Equation (1) in all levels of the structure by expanding it into the following matrix, and considering robustness in different scales:

Further examples of robustness failures on various levels are:

<u>Whole building level</u>: column loss scenario, or slab collapse, leading to building collapse.

<u>Component level</u>: local buckling leading to global buckling; propagation of initial crack at beam soffit to the collapse of the beam.

<u>Connection level</u>: shearing of a bolt, or pullout of a glued-in rod, leading to failure of the entire connection.

<u>Connector level</u>: crack in a bolt leading to snapping of the bolt; crack on a glue line leading to a zipper-effect failure of a glued connection.

<u>Material level</u>: timber knot substantially reducing the strength of timber; martensite in steel leading to local brittle cracks.

Based on the above, one can talk about robust buildings, robust components, robust connections, et cetera. For each level of the scale, different methods are applied to ensure robustness. The existing design methods for robustness (primarily the Alternative Load Path Analysis) focus on the whole building system robustness, while there are more design methods for more local considerations. An example is the key element design, which is generally unfavourable in practice ("method of last resort", [19]), because it does not address system robustness. It can, however, address component robustness and should be further developed to design critical elements in a building. Simple overstrength, the current provision of Eurocode 1 [18] for key elements does not however suffice to design a robust element, very much in the same way that any kind of overstrength does not necessarily make a system robust. Alternative load paths, monitoring, and easy reparation are additional qualitative principles that must be employed to successfully design a robust component or building. A lot can be learnt from very large scale components, for example the design of mega-columns in skyscrapers [39].

Following the above, the robustness definition can be expanded to consider its scale dependency:

> "Robustness is a property of a structure or structural component that prevents consequences spreading to a higher level in the structural scale"

3.3. Limitations of the Scale Approach

Dividing a structure into scales is a simple approach of a complex reality and thus requires care when used. For example, different exposures affect different levels of the structure, and certain exposures can affect multiple levels. An exceptional earthquake is a systematic exposure to the entire structure, all of its components and connections. Manufacturing errors can affect entire components, or just a few connectors. On the other hand, a malicious action, like a terrorist explosion on the ground floor, is an exposure at the component level only. As such, designing for robustness at a certain level in the scale does not necessarily eliminate the risk of different exposures affecting the structure on the scale above.

Another important aspect of the scale approach is that levels can be correlated. For example, corrosion failure of a steel strand within a cable may well mean that the adjacent strands may be corroded, leading to failure of the entire cable. Similarly, failure of a screw due to material defects may mean that screws of the same batch may also fail. If all screws in a connection are made from the same batch, the connection may fail as a consequence of an exposure to a lower level in the scale.

Considering the different types of exposures is necessary in order to design using the multi-level scale approach

3.4. Localised vs Systematic Exposures

Since different levels of a structure are affected by different scenarios, it is important to understand the different types of abnormal exposures before considering robustness. Abnormal exposures can be loads (e.g. explosion) or effects (e.g. moisture damage); localised (one/few components), or systematic (many/all components); static (e.g. foundation settlement) or dynamic (e.g. earthquake). Among all types of abnormal exposures, a clear distinction can be made between localised and systematic.

Localised exposures are defined as those that affect a limited part of the structure, such as one or a few components. Examples of such exposures are explosions, confined fire, and impact. Systematic exposures are defined as those that affect a large group of elements in a structure, or the entire structure. The inclusion of consequences in the consideration of collapse resistance becomes particularly applicable here. Examples of abnormal systematic exposures are earthquakes, defective materials, and human design error. The latter has been found to be the main cause of failure in large-span timber structures [40].

Methods to address localised exposures, such as alternative load paths and structural tying, can have a devastating effect in the face of systematic exposures. The collapse of the Bad Reichenhall ice arena is a case study where the presence of alternative load paths in an already globally weakened structure lead to disproportionate collapse [4]. Its long span timber roof collapsed without warning, killing 15 people and injuring another 34 (Figure 4). The entire roof used urea-formaldehyde glue for the production of the 48m long main timber box girders. The continuous exposure to moisture during the lifetime of the structure (34 years until the collapse) severely weakened the girders, something which was not anticipated at the time of construction in 1971. On the 2nd January 2006, a failure started at one of the main girders, whose collapse pulled the rest of the roof down with it, via the strong secondary purlin connections between the main girders. There were more errors that contributed to this catastrophic collapse, which are discussed in detail in [4] and [41].

On the other hand, the partial collapse of the Siemens Arena in Ballerup, Denmark, is an example of a long span timber roof structure whose primary members are only simply connected with each other and no transfer of load is possible between them [26]. The roof comprised 12 glulam trusses with a clear span of 73m. Two trusses collapsed without warning in January 2003, just months after the arena's inauguration, fortunately when the building was empty (Figure 5). Several design errors were found to have caused the collapse, a systematic exposure. The collapse remained partial, since the purlins were only simply supported between the trusses. Had the secondary connection system been strong and stiff like in the Bad Reichenhall arena, the entire roof would have collapsed.

Systematic exposures require the structure to be segmented into parts, such that collapse cannot progress beyond predefined boundaries. The comparison of these two examples [42] highlights how different methods to design for structural robustness are required for localised

Figure 4. The Bad Reichenhall Ice Arena collapse (Photo credits ©LKA Bayern)

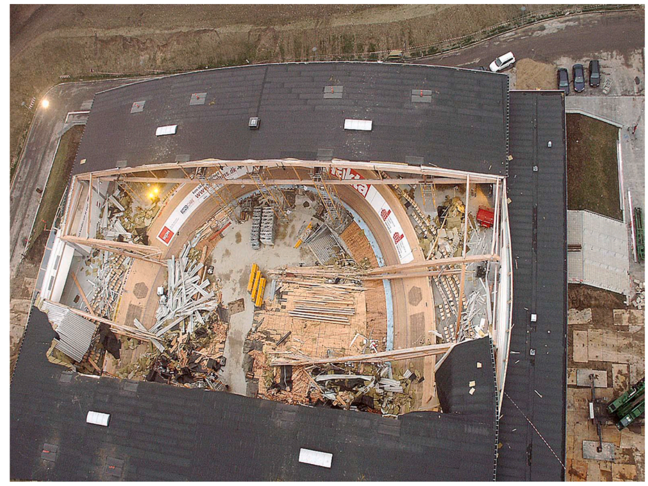


Figure 5. Partial collapse of the Siemens Arena in Ballerup, Denmark (Photo credits ©Peter M. Thorup)

and systematic exposures. The well-known ALPA method alone is not sufficient to address systematic exposures. This distinction is very important to make and not immediately obvious.

3.5. Proposed Qualitative Framework

It is suggested to combine structural methods for robustness design in different levels of the scale in order to address localised and systematic exposures simultaneously. For example, both the ALPA and designing for key elements are good at addressing localised exposures when each is seen in isolation. Consider however their combination to create compartments: each compartment is able to arrest damage internally by activating alternative load paths, while key elements forming the strong borders between compartments prevent any failure from propagating outside of the compartment. The whole building system can therefore be seen as a system of robust compartments. How this is addressing systematic exposures is best demonstrated in the following example of a tall building.

A hypothetical 25 storey building is split into five compartments of five storeys each. Each compartment comprises a structural frame that employs the ALPA method to arrest collapse from localised damage. The components within the compartments are designed at the Ultimate Limit State as per normal practice. The compartment boundaries are formed from key elements that provide both vertical and horizontal support ("mega columns" and "mega braces"). Each key element is designed to be robust in itself, which requires more than just overstrength: availability of alternative load paths within the key elements in case of damage to sub-elements (such as confinement or stiffener plates), continuous inspection and monitoring for damage, and accessibility for maintenance and repair. As illustrated in Figure 6 below, the structure addresses systematic exposures by suffering damage within a compartment, where a collapse can be contained.

This is how combining robustness measures at the component (key elements) and system (ALPA) levels can address both localised and systematic exposures. This systematic approach shall apply to other levels of the scale, for example at the connection level: certain connectors can, or must be designed as "key connectors", and the entire connection shall be able to survive initial damage (i.e. be robust) by redistributing loads in other connectors. For very large connections, compartmentalisation is possible, for example in skyscraper core wall-to-beam connections which comprise several parts separated by large stiffener and confinement plates. Using "key connectors" only is also a possible method, which is frequently used to realise architectural ideas. When "key connectors" are correctly designed, the saying "one bolt is no

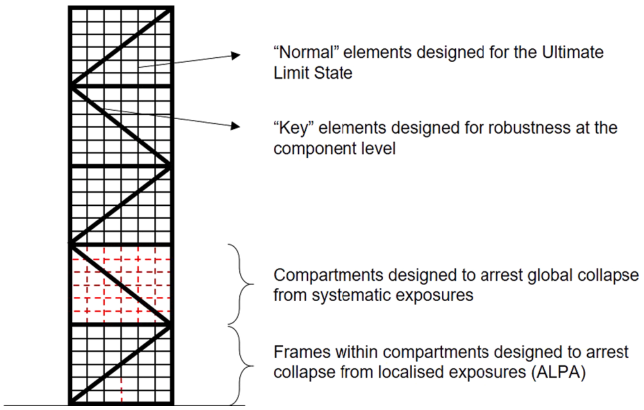


Figure 6. Conceptual example of a robust tall building. Damaged members in dashed lines

bolt" does not hold true anymore, like in the façade of Heathrow Airport's Terminal 5 (Figure 7).

The table of the different levels of a structure can now be revised to include compartments, which are nothing more but the next step in the system scale, when large structures are considered (Figure 8). The existing robustness design methods are noted in the appropriate level of the scale. It is also important to recognise that different parties are ultimately responsible for designing for robustness at different levels of the scale. Material suppliers, for example, shall be responsible to provide products to specification and agreed safety standards. The responsibility list shown is not exhaustive, but indicates that the structural designer need not control everything. Conceptual design is ultimately a collaboration of engineers, architects, contractors, and many others.

This example illustrates the principles of this qualitative approach, which applies to all construction materials. The actual design decisions of a tall timber building would be influenced by the complex timber properties discussed in the introduction.

3.6. *Existing Structures*

The above concept has been deduced from the structural robustness methodology discussed in the paper. Nevertheless, just by simple observation one can realise that there exist tall buildings which already apply this compartmentalisation concept in various forms. Below are three examples, one of which is a tall timber building (Fig. 9). In the next section we would like to provide a quantitative framework in order to be able to increase structural robustness by calculation, via appropriate and economical design decisions.

4. Quantitative Framework

While the above theoretical framework can be very useful to come up with a sound conceptual design for a robust building, it does not allow the designer to quantify the said building's robustness. To achieve this quantification, we have to go back to the robustness index in Equation (5).

4.1. *Calculation of the Robustness Index*

The three technical steps to calculate this index are outlined below. The fine details and application of each step are beyond the scope of this paper, however the appropriate references are provided for the interested reader.

- A sufficiently realistic structural analysis method, to investigate the response of the building to a given initial damage. This could be, but is not limited to, a Finite Element model. Since the behaviour of interest involves propagation of damage and potential collapse, the model should ideally be dynamic, or at least quasi-static or static with the appropriate dynamic amplification factors. Nonlinearities in the materials (e.g. yielding of steel, crushing of wood), as well as geometrical nonlinearities (e.g. to capture catenary action) are essential. Full details on the Finite Element Method are provided in Bathe [43].
- An uncertainty propagation method, to calculate the reliability of the above model, that is the probability that a given damage causes further indirect consequences, $P(C|D)$, given appropriate probabilistic model inputs. This may be achieved with a traditional Monte Carlo Simulation, where the structural model is run multiple times, each one with a different realisation of the probabilistic input

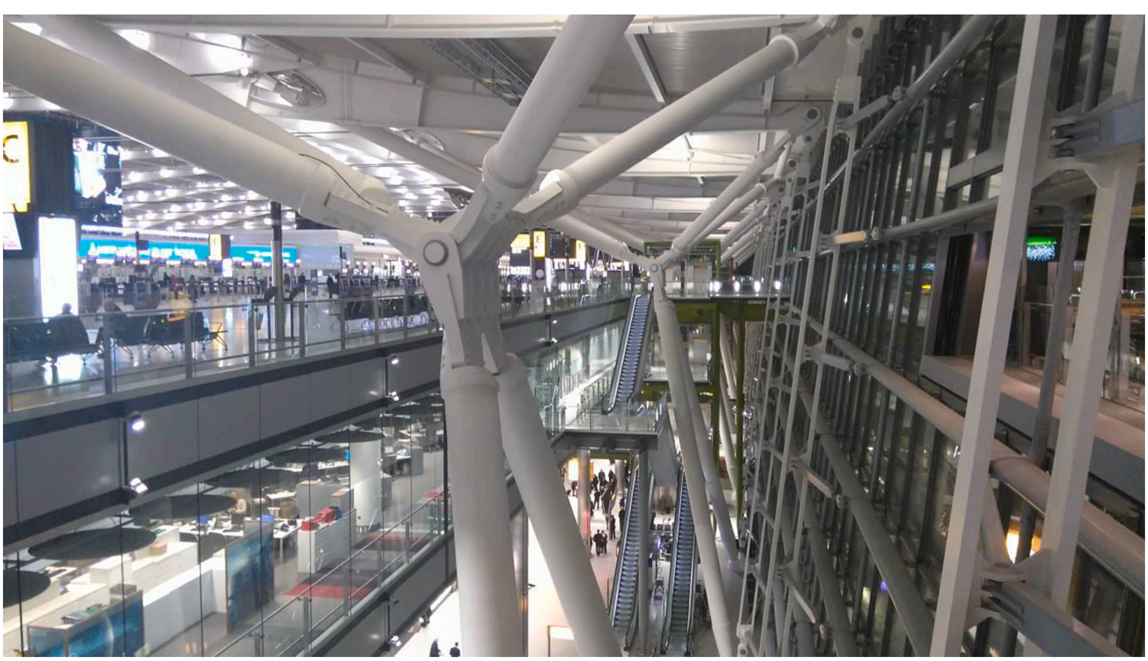


Figure 7. Heathrow Airport Terminal 5 steel node connection (author's photo)

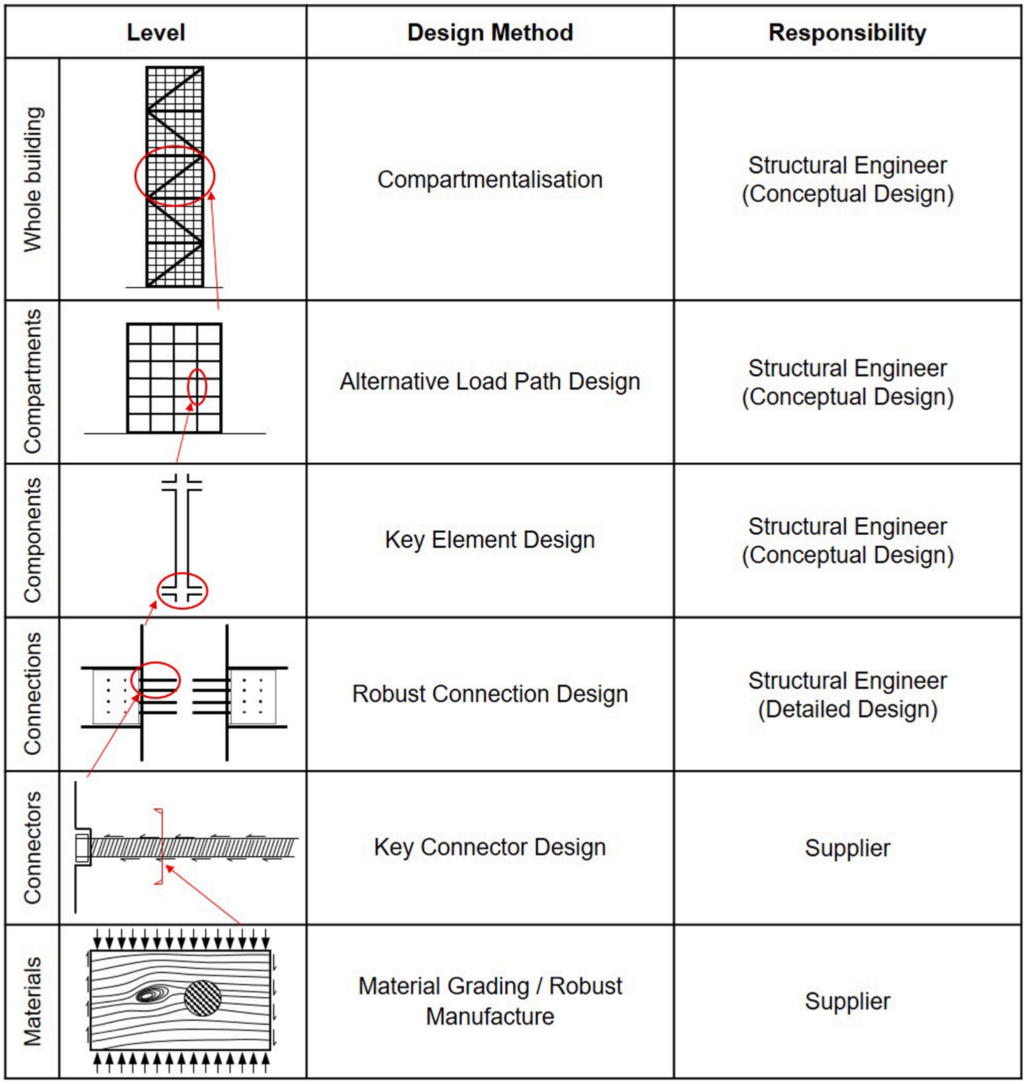


Figure 8. Multi-scale robustness approach with design methods and responsibilities

variables, and where the probability of failure can be simply calculated by the ratio of models that failed to all models. More details on Monte Carlo Simulations is provided in Ditlevsen & Madsen [44]. However, given that finite element models are computationally intensive processes, this method is likely too slow in this context. It is rather recommended to surrogate the structural model, that is to construct a copy of it (a surrogate, or "metamodel") that runs much faster while achieving sufficient accuracy. This may be done mathematically by constructing an appropriate Polynomial Chaos Expansion. This method is outlined in Blatman & Sudret [45]. In addition, the Polynomial Chaos Expansion method can yield analytically the Sobol' indices for a global sensitivity analysis of the structure in question. This is described in Sudret, at al. [46]. The sensitivity analysis is not required for the calculation of the robustness index, it is however a very useful way to identify the model inputs which contribute the most (or the least) to the model output uncertainty.

- A consequence estimation method, to quantify the direct and indirect consequences of the damage scenarios analysed (C_{Dir}, C_{Ind}). Calculating these explicitly is a very difficult task that involves putting a price on human lives, in the case where collapse leads to fatalities. However, only the ratio of direct to indirect consequences plays a role in the calculation of the robustness index, therefore we recommend to express failure consequences as the product of the failed building areas A_{Fail} and their base cost of reconstruction c (cost per unit area). This reconstruction cost may in turn be scaled according to the magnitude of the failure area (scaling factor α_A) in order to take into account the nonlinear increase in consequences as the failed building area increases (larger failures are more likely to involve many fatalities, while smaller failures may involve no fatalities).

$$C_{Dir/Ind} = A_{Fail,Dir/Ind} \times \alpha_A \times c \tag{6}$$

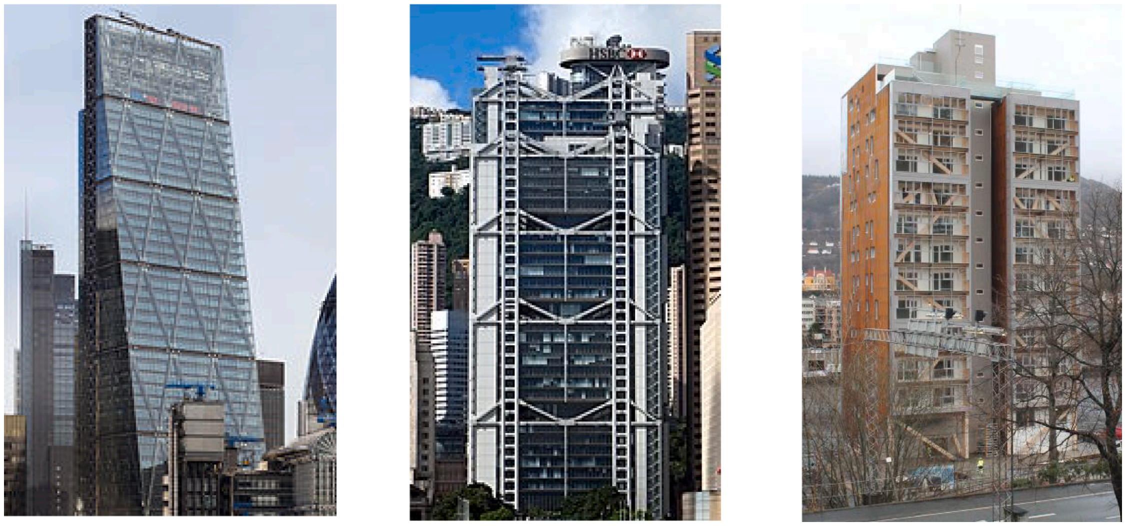
The robustness index therefore becomes

$$I_{Rob} = \frac{(A_{Fail,Dir} \times \alpha_{A_{Dir}} \times c)}{(A_{Fail,Dir} \times \alpha_{A_{Dir}} \times c) + (P(C|D) \times A_{Fail,Ind} \times \alpha_{A_{Ind}} \times c)} \tag{7}$$

and the base cost of reconstruction, c, cancels out. Therefore, to calculate the robustness index in this simplified expression, the structural model shall only be used to calculate the extent of the damage and the probability that this occurs. An example is illustrated in Figure 10: two almost identical frames are subject to the same column removal scenario, and the areas of direct and indirect failure are noted. The structure on the right, which is assumed to be able to redistribute loads via alternative load paths, has an indirect failure area of almost zero.

4.2. Complete Framework Proposal

These three technical steps lead to the calculation of the robustness



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Treet, Bergen, Norway
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Figure 9. Existing tall buildings following a multi-scale approach

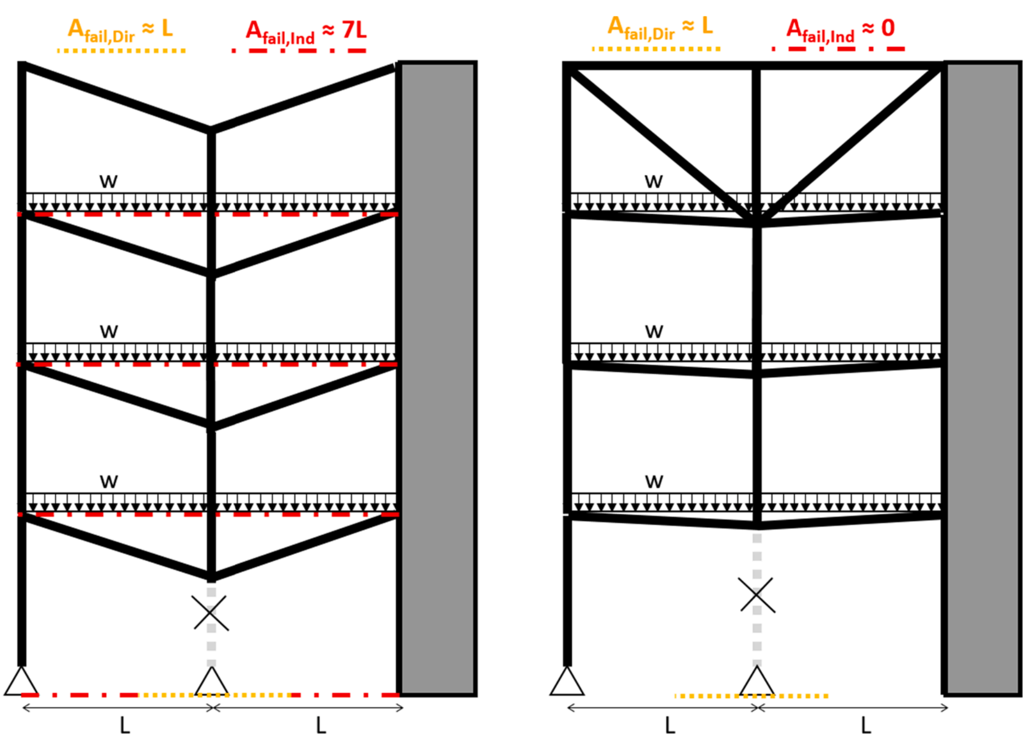


Figure 10. Comparison of two similar structures, assuming the right one is able to redistribute loads due to component failure, and their respective areas of failure (direct and indirect)

index, but only for a specific damage scenario. For this reason, one run of this sequence of steps is not sufficient to quantify the robustness of the building: rather, this sequence must be run multiple times for different damage scenarios. This is illustrated in Figure 11 below.

Therefore, multiple robustness indices are calculated for a given structure. It becomes obvious that the analysis model must be fast enough for the sequence to be possible with a reasonable computational budget. To quantify the actual robustness of the whole building, all the indices from all the scenarios shall be weighted with the probability of that damage scenario occurring, $P_S = \mathrm{P(E)} \times \mathrm{P(D|E)}$. Now consider the building in question being the base geometry, and with our goal to explore design options which improve the building's robustness. Let us recommend k different options that slightly depart from the base geometry, for example the addition of frame diagonals, or the moment stiffening of the connections, et cetera. We end up with k different concepts, each needing an analysis sequence as per Figure 11. The

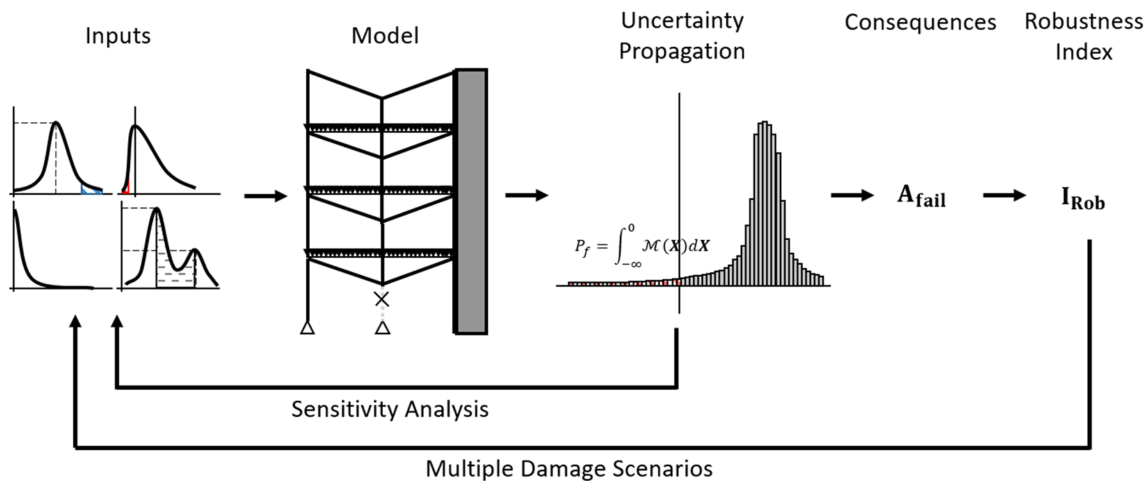


Figure 11. Robustness index calculation workflow for every damage scenario

average robustness index of a given geometry k for n damage scenarios is given by Equation (8):

$$I_{Rob(k,av)} = \sum\nolimits_{i=1}^{n} (W_i \times I_{Rob(k,i)}) \tag{8}$$

Where W_i is the weighting factor for the i^{th} scenario, given in Equation (9):

$$W_i = \frac{P_{Si}}{\sum_{i=1}^{n} P_{Si}} = \frac{(P(E) \times P(D|E))_i}{\sum_{i=1}^{n} (P(E) \times P(D|E))_i} \tag{9}$$

By comparing the average robustness indices of the different options we can draw conclusions about the increase (or decrease) in robustness of a building in different design decisions. *This is exactly the basis of the proposed framework: the goal is not to analyse a large number of completely different structures which fit the remit of a building design and compare their robustness performance, but rather explore the effectiveness of design decisions for improving the robustness of an initial conceptual design, whose base geometry has been selected according to numerous other factors such as location, project size, architecture, local materials and expertise, et cetera.*

In order to provide appropriate guidelines for the design of buildings for robustness, a minimum average robustness index must be required, considering the exposures (and hence damage scenarios) that apply to that particular building. The value of the minimum average robustness ("how robust is robust enough"), as well as the choice of damage scenarios is a complex topic related to the building location and importance, and needs a comprehensive risk analysis beyond the scope of this paper. For now we will focus on engineering decisions to improve the robustness, rather than trying to recommend a minimum value for robustness.

Let us define the Absolute Geometry Rating of a given concept k, AGR_k, which is simply the ratio of the average robustness index of that concept to the average robustness index of the base geometry:

$$AGR_k = \frac{I_{Rob(k,av)}}{I_{Rob(1,av)}} \tag{10}$$

By definition, $AGR_1 = 1$. If $AGR_k > 1$, it means that concept design k is more robust than the base geometry and therefore a potentially good option. If $AGR_k < 1$, it means that concept design k is less robust than the base geometry and therefore a potentially undesirable option. Before, however, rejecting the less robust options, their cost effectiveness should be calculated: let us use a measure of the base cost of each concept design. In order to avoid complex and unreliable cost calculations, it is recommended to use simple indications of cost, such as the total volume of materials, V_k, required in each concept design k, normalised by the total volume of materials of the base geometry, V_1. By taking into account the Absolute Geometry Rating as well as the total volume of materials required for each concept design k, and comparing them with those of the base geometry, we define the Normalised Geometry Rating (NGR_k) as per Equation (11):

$$NGR_k = AGR_k \times \frac{V_1}{V_k} = \frac{I_{Rob(k,av)}}{I_{Rob(1,av)}} \times \frac{V_1}{V_k} \tag{11}$$

By definition, $NGR_1 = 1$. If $NGR_k < 1$, it means that concept design k is a worse option overall. This may not necessarily mean that this option has poorer robustness: the case may be that this option has slightly higher robustness than the base geometry, however at a huge material volume expense. To the contrary, if $NGR_k > 1$, it means that concept design k is a better option overall. That may equivalently mean that this option either has better robustness at a reasonable cost increase, or may have slightly worse robustness, but leading to high material (equiv. cost) savings.

The above procedure is conveniently summarised in Figure 12 below. Given the importance of the distinction between localised and systematic exposures, $NGR_{k,loc}$ and $NGR_{k,sys}$ may both be calculated separately. It is obvious that both a lower limit on the average robustness index, as well as the normalised geometry ratings are required in order to make the optimum decision.

4.3. Connection to the Scale Approach

The last step is to connect the complete framework with the qualitative scale approach presented in chapter 3. Let us categorise each of the k concepts in their respective building scale shown in Figure 8. For example, if the robustness improvement of a building in question is the addition of diagonals in certain parts of the building, then this concept belongs in the compartment scale; if the improvement is the design of certain connections for ductility, this concept belongs in the connection scale; et cetera. Provided enough concepts are studied at all the scales of the building, the maximum NGR in each scale can be determined, separately for the localised and systematic exposures. For the given base geometry, we can now identify which scale of the building contributes the most in the cost effective (or simply effective, in the case of AGR) robustness improvement. One may find, for example, that the biggest positive effect on robustness against systematic exposures is mainly caused by changes in the whole building scale (via compartmentalisation), while the biggest positive effect on robustness against localised exposures is mainly caused by changes in the compartment scale (via the alternative load path analysis method). Solutions on lower levels of the scale should not be disregarded, since they may be improving the robustness of the building slightly, but at a very low cost compared to whole building solutions (for example, increasing the timber grade). The calculated NGR can capture this.

Robustness Optimisation	Scenario 1	Scenario 2	...	Scenario n	**Average I_{Rob}** (Separate for localised and systematic exposures)	Measure of base cost, e.g. material volume	**Normalised Geometry Rating (NGR)**
Base Concept	$I_{Rob(1,1)}$	$I_{Rob(1,2)}$	...	$I_{Rob(1,n)}$	$\mathbf{I_{Rob(1,av)}}$	V_1	$\frac{\mathbf{I_{Rob(1,av)}}}{\mathbf{I_{Rob(1,av)}}} \times \frac{\mathbf{V_1}}{\mathbf{V_1}} = \mathbf{1}$
Concept 2	$I_{Rob(2,1)}$	$I_{Rob(2,2)}$	...	$I_{Rob(2,n)}$	$\mathbf{I_{Rob(2,av)}}$	V_2	$\frac{\mathbf{I_{Rob(2,av)}}}{\mathbf{I_{Rob(1,av)}}} \times \frac{\mathbf{V_1}}{\mathbf{V_2}} = ?$
...	...	...	...	...	...	...	...
Concept k	$I_{Rob(k,1)}$	$I_{Rob(k,2)}$	...	$I_{Rob(k,n)}$	$\mathbf{I_{Rob(k,av)}}$	V_k	$\frac{\mathbf{I_{Rob(k,av)}}}{\mathbf{I_{Rob(1,av)}}} \times \frac{\mathbf{V_1}}{\mathbf{V_k}} = ?$
	$\mathbf{I_{Rob,av(1)}}$	$\mathbf{I_{Rob,av(2)}}$	...	$\mathbf{I_{Rob,av(n)}}$			

Figure 12. Robustness optimisation sequence as described in section 4.2

In addition, sensitivity factors may be calculated for each scale, in order to quantify the contribution of each scale in the robustness improvement, or the contribution of multiple scales simultaneously. This is shown in Figure 13 below.

4.4. Case Study Example

A simplified tall timber building case study has been analysed based on the proposed framework, in order to demonstrate how the procedure works.

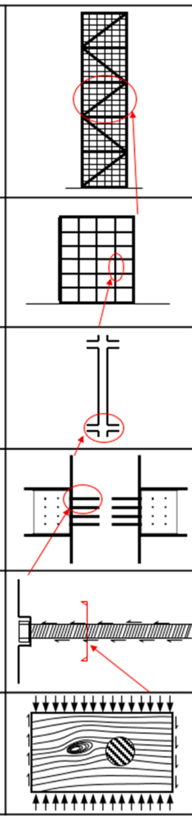
A simple 3x3 bay, 10-storey timber building was modelled in Dassault Systèmes' Finite Element software Abaqus®. The bay size is 5m and the storey height is 3m. The structure was generated using a Python® script for speed and parametrisation, as well as to be able to interface with Matlab® for the uncertainty propagation. The building comprises 100x320mm beams and 200x200mm columns, as well as 200mm thick, 2-way spanning 5-ply Cross-Laminated Timber (CLT) panels for the core walls and 120mm thick, 2-way spanning 3-ply CLT panels for the floors. The timber material is Spruce (Picea abies) with stiffness properties taken from [47]. A linear elastic behaviour was assumed for the material; plasticity and failure were concentrated in the connections by defining connector behaviours in Abaqus®. The ground level walls are rigidly fixed to the ground, and the ground level columns are pinned to the ground (translational fixity only). In addition to the members' selfweight, the floors are loaded with an additional 1kN/m^2 dead load to take into account floor finishes, and a 0.5kN/m^2 imposed load (from w = 2.4kN/m^2 and $\psi_2 = 0.2$, simulating the acting live load for a residential building in an accidental scenario). The members have been approximately sized to not reach failure stress (assuming C24 softwood and partial safety factors) under full loading conditions, without considering wind and snow loads: this calculation is not shown and only serves to have an approximately realistic member size for a building of this scale.

Three versions of the building were created: concept 1 (the base structure) has pinned beam-to-column and beam-to-wall connections. The ability to develop alternative load paths has been introduced in two further concepts of the same building: In concept 2, moment resistance was introduced in the beam-to-column and beam-to-wall connections. In concept 3, the connections remained all pinned and diagonal members were introduced in the corner bays of the top floor. Both improved concepts have substantially strengthened column-to-column vertical connections in order to be able to carry loads upwards in tension. Necessarily, the column dimensions were also increased by 50%. The structural differences are outlined in Figure 14. All concepts have pinned floor-to-beam and floor-to-wall connections, and rigid wall-to-wall connections.

All but the wall-to-wall connections were modelled as linear elastic with failures at the loads corresponding to the section strength of the respective degree of freedom (uncoupled). This simplification does not affect the robustness framework conclusions, which depend on the relative, rather than absolute, analysis results.

The solver employed for the analysis is the Abaqus/Standard implicit dynamic solver with a quasi-static application, which is very powerful in following the (potential) collapse of the structure incrementally. The mass matrix of the structure is assembled using the density of the materials. The model is analysed in a time length of three seconds, with load ramping, in small, automatic time increments to follow the acceleration of parts upon connection failure.

Two damaged versions of the building are analysed: a ground floor corner and edge column removal respectively. The output variables of each analysis which were saved for post-processing were the vertical deflections at the centre nodes of each floor panel. A post-processor was written in Matlab® in order to determine whether collapse has occurred, and to distinguish between different collapse types.



Level	Design Method	Responsibility	Max(NGR)	Level Sensitivity
Whole building	Compartmentalisation	Structural Engineer (Conceptual Design)	NGR_{system} NGR_{local}	$S_{building}$
Compartments	Alternative Load Path Design	Structural Engineer (Conceptual Design)	NGR_{system} NGR_{local}	$S_{compartment}$
Components	Key Element Design	Structural Engineer (Conceptual Design)	NGR_{system} NGR_{local}	$S_{component}$
Connections	Robust Connection Design	Structural Engineer (Detailed Design)	NGR_{system} NGR_{local}	$S_{connection}$
Connectors	Key Connector Design	Supplier	NGR_{system} NGR_{local}	$S_{connector}$
Materials	Material Grading / Robust Manufacture	Supplier	NGR_{system} NGR_{local}	$S_{material}$

Figure 13. Scale-dependent robustness optimisation framework

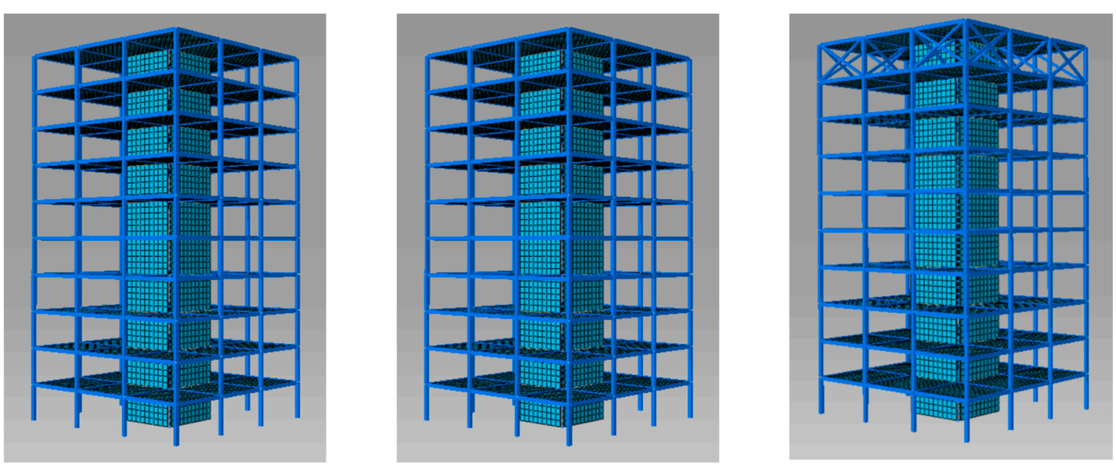


Figure 14. Comparison of the three building versions

Indirect failure has been simplified to a binary status of each building bay: by measuring the resulting deflection at the centre of each floor slab, a flag of zero (no collapse) or one (collapse) is given. We believe the coarseness of this approach is realistic for massive elements such as glued laminated beams and columns and CLT floors, where collapses of fractions of a bay are unlikely. The deflection limit for failure used here is 1m. This was chosen since all normal deflections of a healthy structure are well below this value, while deflections of a collapsed floor are well above it.

The absolute deflections and failure flags are printed in a three-dimensional matrix corresponding to the bays of the building, in terms of their x-y-z position. Each unique 3D failure matrix of zeros and ones corresponds to a unique collapse scenario, which can be named or numbered accordingly and used as a unique collapse class identifier. In this case study, only one collapse class per damage scenario occurred, therefore a final flag of one or zero was used to identify a general case of collapse or no collapse respectively. This result is observed when the strength or stiffness properties vary identically for all members. For models of higher uncertainty dimensionality, multiple types of collapse may occur for the same initial damage scenario.

Calculating the indirect failure area, $A_{Fail,Ind}$, is a straightforward process of multiplying the failure matrix of zeros and ones with the bay area (a matrix with bay areas may be used for non-symmetric buildings) and summing all the terms. The process is shown in Figure 15 (for a simple 2D case).

In the case of a column removal scenario, the direct failure area, $A_{Fail,Dir}$, is assumed to be the area at a radius of half a bay size from the failed column. For a corner column this corresponds to a quarter bay, while for an edge column this corresponds to a half bay. This assumption is independent of the structure type around the failed column (e.g. the floor type).

While systematic exposures are of great interest for holistic robustness, they have not yet been investigated in this case study.

All geometry variables are treated as constants (deterministic). For simplicity, the only inputs that we vary probabilistically are the timber stiffness and imposed live load, as shown below with their probabilistic properties. These properties apply to all members throughout, we are therefore dealing with a mathematically two-dimensional probabilistic model.

We did not vary the actual connection strength explicitly to keep the model uncertainty dimensionality to a minimum. Connection strength is however already affected probabilistically from the material stiffness: the failure initiation is force-based and a structure that deflects more will, with the geometrical nonlinearities, put more load on the connections (see Table 1).

The Abaqus® Python script is run externally via Matlab® while editing the input file on every run to carry out the simulations for different realisations of the input variables. The uncertainty quantification software developed at ETH Zürich, UQLab®, is used to propagate the uncertainties [48]. While the software is very powerful in using regression and classification techniques to propagate uncertainties in highly dimensional and nonlinear models with non-smooth outputs, we restricted our study to Monte Carlo Simulations for the sake of simplicity and to demonstrate the working of the framework without unnecessarily adding confusion with advanced surrogate models. The interested reader may refer to [49].

The six models (three concepts in two damage scenarios each) were run 30 times each, for the same random input vector. Due to the small

Table 1
Model probabilistic variables

Variable	Distribution	Mean	CoV
Timber Stiffness (E)	Lognormal	11000 MPa	10%
Imposed UD Load (W)	Gumbel	0.5 kN/m2	22%

sample set, each concept/damage pair either certainly collapsed (concept 1 – Figure 14), or certainly did not collapse (concepts 2 & 3). This meant using P(C|D) equal to one and zero respectively. The collapse of concept 1 in damage scenario 2 is shown in Figure 16. While in a surrogate-based, large sample analysis the probability of collapse values may be more accurate (close to but not exactly zero and close to but not exactly one), the difference this makes to the results is minimal and the application of the framework is demonstrated clearly. The post-processing of the data according to Figure 15 and assuming equiprobable damage scenarios yielded the results summarised in Table 2 below:

Clearly, both concepts 2 and 3 offer a substantial robustness improvement for the base geometry, with little added timber volume, V_t. It is not surprising that very little added material cost is involved in building truss diagonals in order to avoid the potentially problematic moment resisting connections in timber. However, a comparison beyond the material volume is certainly required for a fair choice. Construction costs, architectural considerations, local expertise, and maintenance issues must all be involved in a more holistic “cost” parameter to be used in Equation (11).

4.5. Case Study Remarks

The two improved concepts studied (connection stiffening and truss floor addition) are typical robustness conceptual measures that create alternative load paths and the increase in robustness is quantified based on the proposed framework. For a more complete study of robustness according to the proposed framework, further damage scenarios and conceptual designs should be evaluated. The scripting of the models allows for this by simply editing variables in the code: since the procedure is object-oriented, the model inputs, structure, uncertainty propagation, and post-processing can each be adapted and improved independently. This offers the opportunity to study a large range of building configurations and to incorporate external models, such as those of CLT floors, advanced material models for wood, or entire tall timber building assemblies.

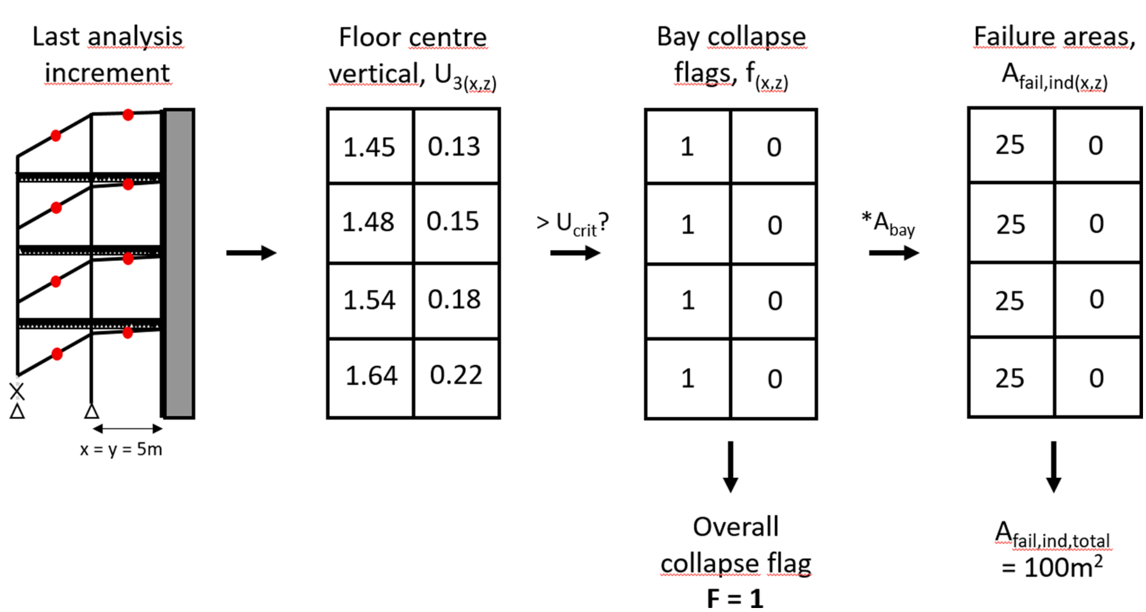


Figure 15. Failure area calculation procedure (2D example)

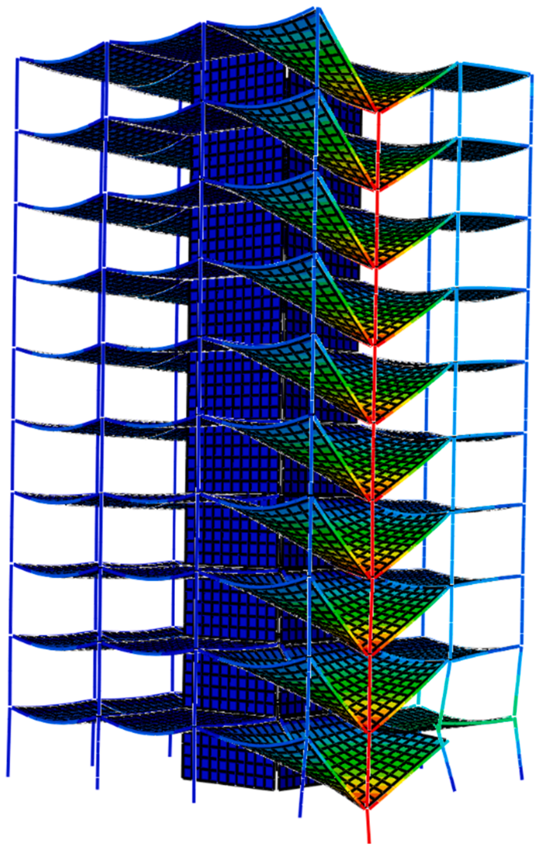


Figure 16. Collapse of the base geometry at damage scenario 2

Table 2
Robustness results for the case study

	Scenario 1	Scenario 2	$I_{Rob(av)}$	V_t	*NGR*
Concept 1	I_{Rob}= 0.0168	I_{Rob}= 0.0138	0.0153	406.4	1
Concept 2	I_{Rob}= 1	I_{Rob}= 1	1	424.4	62.6
Concept 3	I_{Rob}= 1	I_{Rob}= 1	1	431.9	61.6
W_S	0.5	0.5			
$A_{fail,Dir}$	6.25	12.5			

5. Conclusion

Robustness, or in a structural context the ability to prevent disproportionate collapse, is an important property of any structure. It becomes crucial when large or unusual structures are considered, whose behaviour is not: fully understood and where the consequences of a disproportionate collapse are high. Tall timber buildings, seeing a renaissance with the technological improvements of engineered wood products, are one such type of large and unusual structure.

To date, research and understanding of structural robustness is mainly restricted to concrete and steel structures. This paper has summarised the current state-of-the-art and state-of-research for robustness design of tall timber buildings. With the majority of the building codes not explicitly providing guidance on designing for robustness, particularly for timber, the aim of this paper has been to introduce an improved way of thinking about robustness that clarifies many of the difficulties in tackling the topic. The six main conclusions, which apply to all types of structures and materials, are:

1) Increasing robustness is only one of three ways to prevent disproportionate collapse of a structure, the other two being reducing the exposure and vulnerability, per Equation (1). The effect of each method must be quantified by considering both probabilities of occurrence and subsequent consequences.
2) The concept of scale is very important: any structure can be separated into various distinct levels of the scale: the whole structure, compartments making up the structure, components making up the compartments, sub-components and connections linking the components, connectors making up the connections, materials making up the connectors and components.
3) To design for "total" robustness, one has to consider robustness at multiple levels of the scale, as opposed to the current view that robustness is a property of the whole building only functioning as a one system of components. Different methods can be used to ensure robustness at different levels: for example, alternative load paths can ensure a robust structural compartment, whereas correct key element design can ensure a robust component, like a column.
4) By considering the two types of exposures (localised and systematic) and recognising that each requires a different design strategy, it is very important to realise that the alternative load path method can have a detrimental effect in robustness against systematic exposures. A combination of robustness solutions at different scales (for example key elements and alternative load paths together) must be used to design a structure that is robust against any exposure. The importance of a sound conceptual design is paramount.
5) To quantify the robustness of a building, a sufficiently detailed model must be analysed, and the probability of failure, together with the consequences of that failure, shall be calculated. This yields the robustness index of that building specific to the damage scenario analysed. An average robustness index shall be calculated by repeating the process for many damage scenarios, and averaging the respective robustness indices weighted with the probability of their equivalent damage scenario occurring. A lower limit on the average robustness index could be imposed by building codes given the building's consequence class, to check that the building is robust enough.
6) To take the optimal design decisions for robustness, the average robustness indices and base costs of a building and several other improved options of the same building may be used to calculate the Normalised Geometry Ratings (NGR). This is an effective way to identify which design solutions increase robustness most economically. In addition, if the improved options are categorised according to their building scale (e.g. compartment, connection, et cetera), then the maximum NGR per scale gives an indication on where the design should look first to increase robustness most economically.

This "holistic" approach to robustness can be applied to any structure. Its usefulness is maximised in large and unusual structures, such as tall timber buildings.

CRediT authorship contribution statement

Konstantinos Voulpiotis: Conceptualization, Formal analysis, Methodology, Visualization, Writing - original draft, Writing - review & editing. **Jochen Köhler:** Methodology, Supervision, Writing - review & editing. **Robert Jockwer:** Funding acquisition, Supervision, Writing - review & editing. **Andrea Frangi:** Project administration, Funding acquisition, Writing - review & editing.

Declaration of Competing Interest

The authors declare that they have no known competing financial interests or personal relationships that could have appeared to influence the work reported in this paper.

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