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QUANTIFYING ROBUSTNESS IN TALL (TIMBER) BUILDINGS: A CASE STUDY

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ABSTRACT: Robustness research has become popular, however very little is known on its explicit quantification. This paper summarises a quantification method previously published by the main author and proceeds in demonstrating its step-by-step application with a case study tall timber building. A hypothetical 15-storey timber building is designed for normal loads and four improved options are designed to account for abnormal loads in order to increase the building's robustness. A detailed, nonlinear dynamic Finite Element model is set up in Abaqus® to model three ground floor column removal scenarios, and a Random Forest classifier is set up to propagate uncertainties and efficiently calculate the probability of certain collapse classes occurring and the importance of each input parameter. The results show how design improvements at the whole building scale (e.g., strong floors) have a higher impact on robustness performance than just improving the strength and ductility of some selected connections, although these results are exclusive to the building studied. The whole procedure is put in context of the practicing engineer, with a suggestion for a calculation-free, purely qualitative robustness framework. The case study reinforces the importance of a sound conceptual design for achieving robustness in tall timber buildings.

KEYWORDS: Disproportionate collapse, Finite Element Analysis, Random Forest classification, machine learning

1 INTRODUCTION

Our profession is seeing a paradigm shift towards buildings of lower carbon footprint: tall timber buildings are a fine example of this shift. Any departure from the "familiar waters" of the common structural typologies runs a higher risk of not having identified or anticipated certain structural behaviours: similar to the scaling issues that led to the partial collapse of the Ronan Point in London in 1968 [1], we must understand how timber buildings scale to the new heights constructed in the last 10 years, particularly regarding their disproportionate collapse behaviour.

Structural robustness, or disproportionate collapse resistance, is the ability of a structure to withstand damage without disproportionate further consequences, and it is an important and yet not so widely understood quality of our building stock. While a lot of work has been put into understanding structural robustness at a qualitative level and regarding concrete and steel buildings, little is known on the robustness of timber buildings and even less on how to specifically quantify how robust is a building, and whether this is enough or not.

In this paper, we build on the framework presented by Voulpiotis et al. [2] on robustness quantification followed by a case study tall timber building. How to connect this complex work to design in practice is presented in the end. Only essentials are discussed here – for more details and the literature review on the topic please refer to the doctoral thesis of Konstantinos Voulpiotis [3].

2 QUANTIFICATION FRAMEWORK

Since it belongs to the category of extreme (rare) events, disproportionate collapse is best studied in a probabilistic manner. Of the frameworks that have been proposed to quantify robustness in the last 30 years [4–7], the robustness index is a common measurement of the disproportionality of risk: the importance of secondary, or indirect consequences (C_{Ind}) compared to the direct ones (C_{Dir}) given the probability of them occurring following a damage scenario ($P(C|D)$):

$$I_{Rob} = \frac{C_{Dir}}{C_{Dir} + P(C|D) \times C_{Ind}} \quad (1)$$

According to Voulpiotis et al. (2021) [2,8], the consequence can be measured in terms of extent of collapse area ($C_{Dir/Ind} \rightarrow A_{Fail,Dir/Ind}$). Since a damaged building can fail in different ways (let us assume n different "collapse classes"), we calculate the robustness index for a given damage scenario by summing the indirect risk occurring from each collapse class i:

$$I_{Rob} = \frac{A_{Fail,Dir}}{A_{Fail,Dir} + \sum_{i=0}^{n} (P(C_i|D) \times A_{Fail,Ind,i})} \quad (2)$$

Given several damage scenarios, we can obtain a weighted robustness index which is unique to that building design. Assuming several building design

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improvements, we can use the increase (or decrease) of the average robustness index to decide on the best design solution to increase robustness. This is demonstrated in the case study below.

3 CASE STUDY ANALYSIS

3.1 BUILDING DESIGN

A hypothetical 15-storey timber building skeleton structure for offices in Zürich, Switzerland, is designed to the Swiss standards to quantify its robustness using the method described in the previous section. The design is kept as simple as possible to focus on the collapse performance (Figures 2 & 3): post-and-beam construction with glulam timber beams and columns, and timber-concrete composite floors, all designed for category B loads (office) in service class 1 and also checking for fire, buckling, deflections, and creep. The loads are assumed to go back to a CLT core which functions are a vertical cantilever (not explicitly designed). The materials used are GL32h glulam timber (SIA 265:2012) [9], C25/30 concrete with B500B rebar (SIA 262:2013) [10], and 5.6 connector steel for screws and dowels (SIA 263:2013) [11]. The substructure has not been designed.

The main design focus has been put on the connections, as there is evidence that they play a significant role in increasing structural robustness [12]. We used dowelled connections with slotted-in steel plates for the beam-column connections, glued-in rods for the column-column connections and screwed steel angle brackets for the floor-floor connections. They are all assumed to be pinned for the design of the building, however, their actual stiffness and resistance on all degrees of freedom have been estimated using first principles. An idealized elastic-plastic curve has been assumed for each degree of freedom, defined by only four values: elastic stiffness (K_e), yield load (F_y), plastic deformation (δ_p), and ultimate load (F_u). Full details are in Voulpiotis [3].

Four improved versions of the building have also been designed for comparison: options 2 & 3 employ a diagonally-braced "strong floor" at the 15th or both 7th and 15th floor respectively with the columns designed to work in tension should a support be compromised. Options 4 & 5 employ an increase in the ductility of the beam-column and column-column connections respectively. This increased the size of the connections, which in turn also affected the size of the columns. The buildings have been parametrically defined in Abaqus® and studied in ground floor column removal scenarios using both implicit and explicit solver to consider dynamic and nonlinear effects.

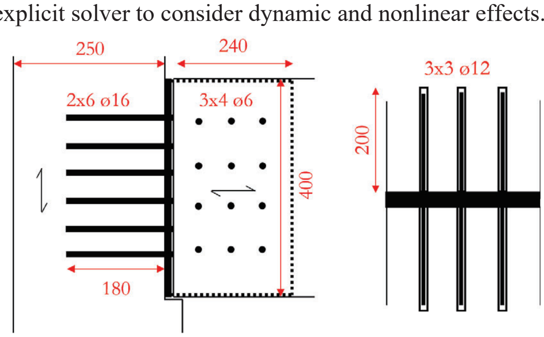

Figure 1: *Beam-column (l) and column-column (r) connections*

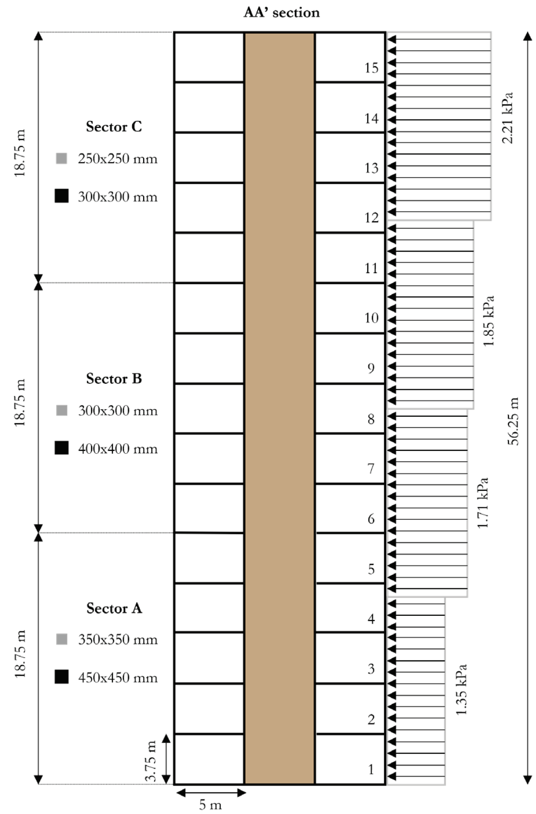

Figure 2: *Case study building section*

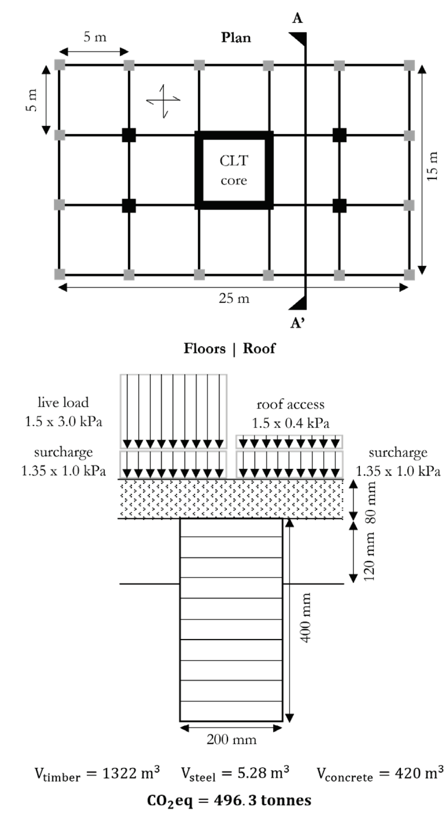

Figure 3: *Case study building plan and detail*

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3.2 SURROGATE MODELLING

A surrogate model is a simplified, computationally cheap-to-evaluate mathematical representation of the complex finite element model, necessary to calculate the probabilities of each collapse class occurring in a reasonable amount of time.

Three independent damage scenarios were studied: the removal at the corner, edge, and internal ground floor columns with a removal time of 2 ms:

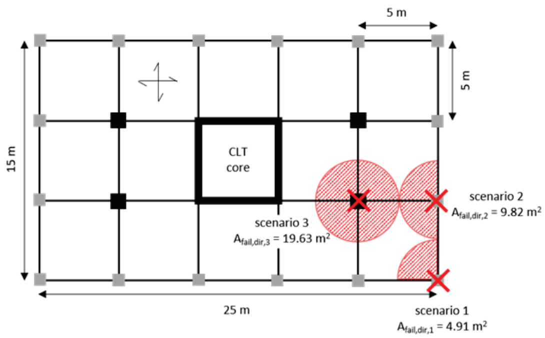


Figure 4: *The three damage scenarios studied*

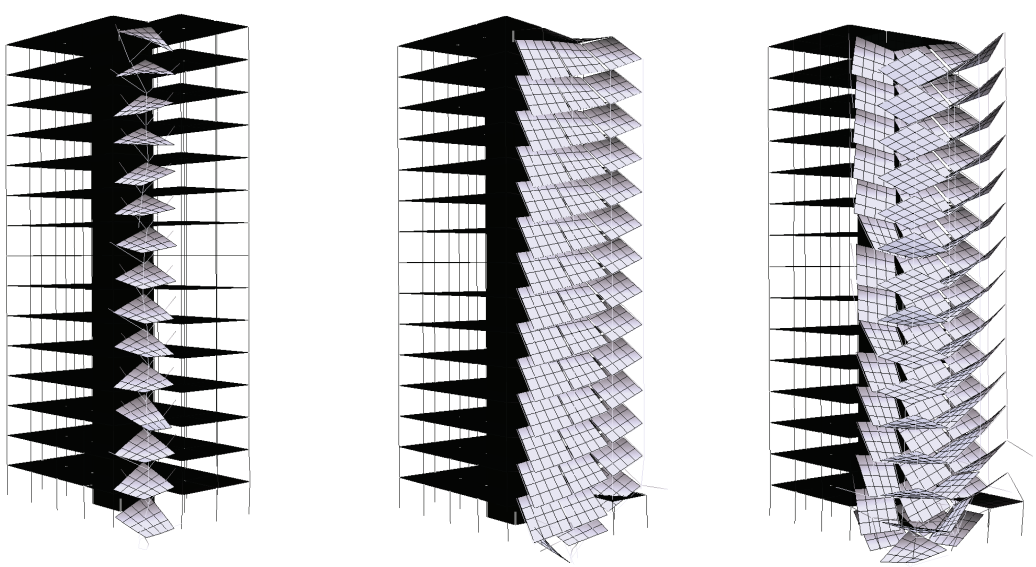
A probabilistic input vector was set up for each building design and used to train a Random Forest classifier [13], later enriched in the domains of the minority classes using synthetic sampling (SMOTE) [14]. This way the probability of each collapse class occurring (for a given damage scenario) could be calculated with only around 1,500 model runs for each building design. Performance scores (accuracy and macro F1 score) of the classifiers were obtained using k-fold cross-validation [15] and sensitivity analyses were carried out using the Impurity-Based Importance (IBI) [16]. Everything was set up in Python®, using the Scikit-learn [17] and Imbalanced-learn [18] modules. Full details of the algorithm setup and hyperparameter optimisation are provided in Voulpiotis [2]. This learning algorithm can efficiently find patterns in computationally expensive models with chaotic tendencies and is opening up new horizons in the numerical analysis of large, complex structures, such as highly nonlinear analyses of tall timber buildings.

3.3 RESULTS

All analyses were run on the "EULER" High Performance Computer Cluster of ETH Zürich [19]. Using master scripts in Python and a batching system, Abaqus input files and job submissions were performed in parallel by both Intel® and AMD® compute nodes. The model deformations were extracted at the 1.3 second timestamp before the results showed chaotic tendencies (a small change in the inputs led to large, inconsistent changes in the outputs). It is assumed that capturing the initial collapse stages using the method from section 2 suffices to explore cost-effective robustness solutions. Further collapse does occur beyond 1.3 seconds, however, modelling this is both unreliable and also unnecessary.

3.3.1 Design option 1

The code-compliant design option 1 partially collapses in all of the three damage scenarios shown in Figure 4. Additionally, a dominant collapse class is always present (375 m² for scenario 1; 1,100 m² for scenario 2; and 2,200 m² for scenario 3, see Figure 5). The extent of the collapse is increasingly worse by scenario: a corner column removal causes the entire corner of the building to collapse, an edge column removal causes the entire edge of the building to collapse, and an internal column removal causes half the building to collapse (until the assumed rigid core).



Scenario 1, severity = 375 m²   Scenario 2, severity = 1,100 m²   Scenario 3, severity = 2,200 m²

Figure 5: *Abaqus® images and severity* ($\boldsymbol{A_{Fail,Ind,i}}$) *values for the dominant collapse classes for design option 1*

The mechanics of the collapses in design option 1 are simple to explain by scenario:

<u>Scenario 1</u>: The load of the 14 unsupported floors is initially transferred towards the neighbouring columns via the beams in cantilever action, and via the floors in in-

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plane shear action. The beam-column and floor-beam connections break very quickly, and the entire corner of the building, with the floor slabs detached, is accelerating towards the ground.

Scenario 2: The mechanism is similar to scenario 1, however this time the unsupported area is double in size and the forces are carried by three beams and two slabs per storey. The horizontal forces developing in the edge beams pull on the adjacent corner column, causing it to buckle and collapse too.

Scenario 3: With an even larger initial failure (four times that of scenario 1), a larger portion of the internal frame spreads loads to its surrounding beams and slabs in the same manner as scenarios 1 & 2. However, horizontal resistance is only provided from one side, the core, causing everything on the outer side of the building to buckle and collapse as well.

3.3.2 Design option 2

No collapse was observed for damage scenario 1 in design option 2, where the top floor is a trussed "strong floor". Although to a significantly lesser extent than in design option 1, the failure of an edge or an internal column again caused a progressive collapse. The truss structure is unable to carry the weight of the unsupported building when its area exceeds half a bay. It is therefore not surprising that scenario 3 leads to a collapse, albeit of lower initial extent than in the original design option. Upon removal of the column, the column-column connection breaks axially, and the membrane action that develops in the slabs is pulling the surrounding structure inward (Figure 6). Since stiffness is asymmetric (the core side is much stiffer), the edge of the building buckles and a substantial collapse initiates.

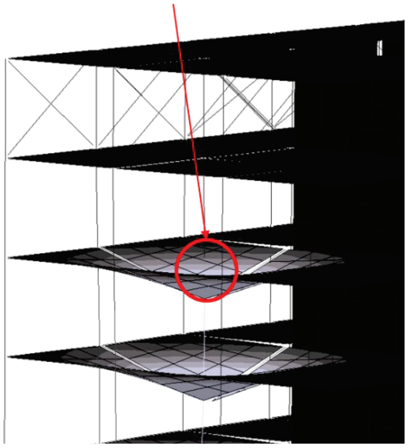

Figure 6: *Scenario 3 of design option 2 @0.5 s, column connection axial failure*

Scenario 2 is more marginal in that it could have supported the weight of the collapsing building had dynamic factors been included in the sizing of the stronger connections. However, the design of the strong floor was static, and the fast column removal speed is causing larger, dynamic force reactions. The failure that caused the collapse was the column-column connection in shear at the floor below the truss (Figure 7). This indicates that the strong floor is not stiff enough to prevent large deformations that will induce very large forces in the surrounding connections. Also, even the much stronger column-column connections are not particularly strong in shear: an alternative for this degree of freedom is an option worth exploring.

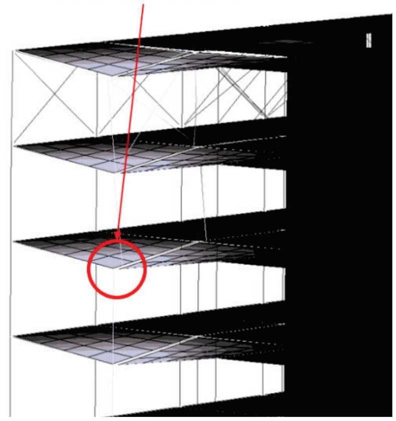

Figure 7: *Scenario 2 of design option 2 @0.4 s, column connection shear failure*

3.3.3 Design option 3

Design option 3 was similar to design option 2 in that scenario 1 was fully arrested. Scenario 2, however, showed a wider and more severe response spectrum (severities up to 1,300 m^2 compared to 950 m^2 for design option 2). The mechanics of the collapse in scenario 2 are similar to design option 2, with the difference that shear failure occurred both above and below the truss strong floor in the middle of the building and destabilised the edge of the building. Collapse very quickly spreads to the lower half. Scenario 3 showed mixed results: in most cases the building survived. No axial failure at the column connections was observed. There were, however, cases where again the shear failure of the column-column connection under the middle strong floor caused the initially contained collapse to spread downward. Unlike design option 1, the horizontal connectivity with the truss reduced the initial spread of the damage to the adjacent corner column in both design options 2 & 3.

3.3.4 Design option 4

No resistance improvement compared to design option 1 was observed for design option 4: although scenario 1 sometimes survived the damage, the spread of collapse classes was much wider with the majority collapse class still being the corner bay as with design option 1, with similar mechanics described earlier and shown in Figure 5. Damage scenarios 2 & 3 showed very consistent behaviour despite the variability of the probabilistic inputs. A possible explanation is that although alternative load paths changed with the improved connections, they could not find their way back to the core. A closer look at the simultaneous improvement of the floor slab design and connectivity, together with the beam and column connectivity, is a worthwhile investment.

3.3.5 Design option 5

Finally, in design option 5, the small change of the column-column connection to increase its ductility did not improve the robustness performance at all. Collapse mechanics were also similar to option 1.

3.3.6 Robustness indices

The results show that the performance of the original design is disappointing ($I_{Rob,av}$ close to zero) and so is the performance of options 4 & 5 with an improved connection ductility. On the other hand, options 2 & 3 perform much better, although at a higher cost in terms of embodied carbon due to the substantially larger columns and connections to accommodate the strong floor forces. Option 4 performs equally well as option 1, but it also costs more, therefore it is not a cost-effective solution.

The summary of the results is clear in the following table which compares the robustness indices for each damage scenario in each design option, as well as their cost in terms of CO_2-equivalent based on the volume of the structural materials. Surrogate modelling performance scores are also provided below.

Table 1: *Robustness comparison between all design options, including accuracy and marco F1 scores*

	$I_{Rob,sc1}$	$I_{Rob,sc2}$	$I_{Rob,sc3}$	CO_2eq (t)
Concept 1 scores	0.0129 (0.94 / 0.75)	0.0088 (0.73 / 0.40)	0.0088 (0.90 / 0.63)	**496.3**
Concept 2 scores	1.00 (0.83 / 0.36)	0.0182 (0.58 / 0.46)	0.0144 (0.76 / 0.71)	**519.6 (+4.7%)**
Concept 3 scores	1.00 (1.00 / 1.00)	0.0133 (0.56 / 0.43)	0.0274 (0.70 / 0.49)	**520.8 (+5.0%)**
Concept 4 scores	0.0119 (0.68 / 0.49)	0.0091 (1.00 / 1.00)	0.0090 (1.00 / 1.00)	**505.4 (+2.4%)**
Concept 5 scores	0.0095 (0.69 / 0.63)	0.0059 (0.33 / 0.33)	0.0088 (1.00 / 1.00)	**496.27 (-0.002%)**
Average	0.4069	0.0110	0.0137	
$A_{Fail,Dir}$	4.91	9.82	19.63	

With a 34-fold increase in the average robustness index, design options 2 & 3 with structural improvements in the whole building scale ("strong floors") are much better solutions for this particular building. They owe this improvement due to the full arrest of collapse in scenario 1 ($I_{Rob,sc1} = 1.0$) and the marginal, but insufficient improvement of collapse in scenarios 2 and 3 ($I_{Rob,sc2/3} < 0.03$, which is practically not different from zero).

This result is in line with the observations of Mpidi Bita et al. (2019) [20], who also proved the benefit of designing a strong floor from which columns can hang the floors below in case of damage. The benefit of the conceptual design is evident despite their study being on a different structural typology (flat-plate CLT building).

Design option 4, equally robust as option 1, requires more steel in the connections and thus becomes uneconomic in the given assumptions. This is not to say that an improvement in the connections cannot increase robustness; rather, the specific solution implemented does not provide sufficient alternative load paths. Design option 5 is marginally worse than the starting option in terms of performance. These results are in line with the alternative load paths and collapse mechanisms discussed in the previous paragraphs. They highlight once again the significance of understanding how alternative load paths are formed, and making sure they lead to the ground. Increasing the ductility of only two connections (e.g. beam-column and column-column in option 4) allows loads to better redistribute in these parts of the structure, however, collapse resistance is also dependent on the floor-beam and floor-floor connections, global stiffness symmetry, and the buckling of columns. One should have a clear understanding of the flow of loads in a structure at the global scale (conceptual design) in order to make the right decisions regarding connection and component detailing.

3.3.7 Sensitivity studies

The properties of the columns and the column-column connections dominate the importance for external damage (scenarios 1 & 2), while the properties of the beams and floors dominate the importance for internal damage (scenario 3). However, the spread of importance values throughout the Random Forest is high, and the importance values themselves are neither high, nor very different from each other. This indicates an absence of an overall dominant feature, which reflects the observations in the collapse mechanisms, and explains why design options 4 & 5 do not perform better in terms of robustness. Collapse is arrested by the structure functioning as a large, complex system and an improvement on many variables, rather than just a few, is necessary to achieve an overall better robustness performance. Full details of the sensitivity study with figures is provided in Voulpiotis [3].

4 FROM RESEARCH TO PRACTICE

4.1 QUALITATIVE DESIGN (SCALE APPROACH)

The procedure outlined so far and the example case study are fairly complex, time-consuming activities. We do not suggest that practicing engineers must go through all this to ensure robustness; rather, simplified, qualitative techniques shall be used for simple or well-established structural typologies, and the full quantification procedure shall only be used for new, unknown typologies (of any material) whose collapse would have very serious consequences to the society.

A reinforced concrete, cast in-situ midrise building is a simple, known structure where robustness can be implicitly assumed. A method to assess its robustness more rigorously without performing any calculations is detailed in Voulpiotis et al. (2021) [2] and is based on considering structural robustness on different levels of the scale (Figure 8): whole building, compartments, components, connections, connectors, and material microstructure. What is important to understand is that current robustness design methods are largely based on preventing collapse to propagate from the component to the whole building level. If we are able to prevent collapse propagation also from connections to components, and from components to building compartments, the disproportionate collapse probability dramatically decreases. This is demonstrated by an example "stacked compartment" structure in Figure 9.

The qualitative approach may of course be also used for complex, unknown structures such as midrise timber buildings like the one studied in this paper. In

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combination with advanced quantitative techniques, collaborative work of researchers and practitioners can

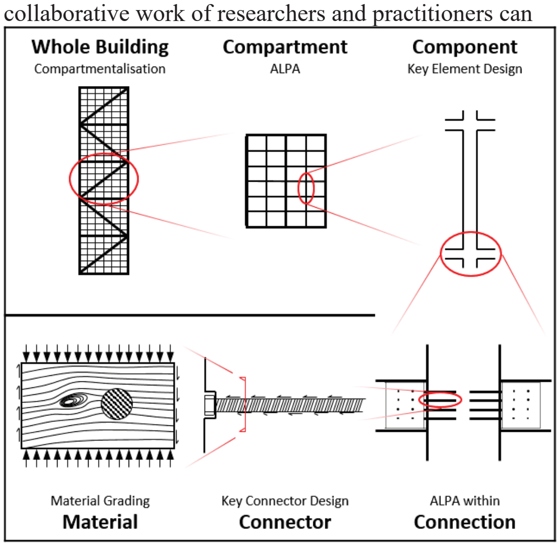

Figure 8: *Robustness can be qualitatively achieved by assessing the load redistributions at different levels of the scale*

lead to a much better understanding of the structural behaviour of new typologies in an implicit fashion, that is, the expert will be able to have a "gut feeling" of a sufficiently robust structure in the exact same way the experienced engineer knows if a beam or column cross section is sufficient in size to serve its purpose without performing any calculations. We may thus be able one day to decide on robustness design improvements based on tabulated design data or using simplified equations, like it is currently done with component design in the building codes.

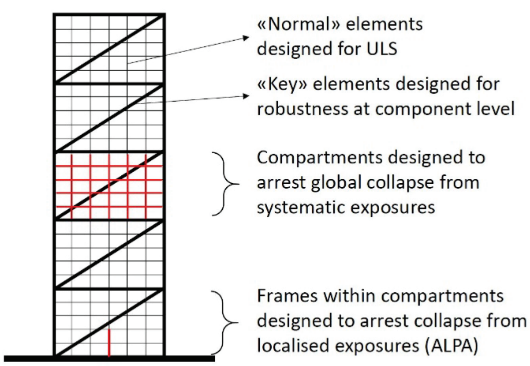

Figure 9: *Proposal of a theoretically "holistically robust" building according to Voulpiotis et al. (2021)* [2]

4.2 RELEVANCE OF BUILDING CODES

As we are aspiring for the building codes to be able to guide practitioners in more detail when it comes to robustness (whether a step-by-step guidance is truly feasible is a debate outside the scope of this paper), we hope to see a shift in the current approach, which offers little more than vague requirements without guidance. The work of Mpidi Bita, et al. (2019) [21] summarises the existing approaches of building codes in various regions (Canada, USA, Europe, Australia/New Zealand) and presents the results from a 171 participant survey. The survey asked practitioners whether they consider robustness in their design and to what extent this is affected by building codes. By looking at the different approaches in concrete, steel, and timber in each of the studied regions, the authors identify key improvements which can be made to existing codes and guidelines, such as the inclusion of specific recommendations for structural robustness, and a performance-based approach, rather than prescriptive requirements.

One of the most important findings of the survey is that robustness is practiced more consistently in regions or materials where the building codes provide more detailed help. With the case study in this paper, we hope to have started a necessary contribution of modelling data, such that timber-specific building codes can adapt and include more detailed guidance on designing robust tall timber buildings.

5 CONCLUSIONS

In this paper we presented an application of the quantification methodology originally presented by Voulpiotis et al. (2021) [2] in the previous world conference (WCTE 2021).

A case study, 15-storey timber building is designed to the Swiss building codes and is run through the quantification methodology using high-fidelity Finite Element Modelling and an adaptive surrogate modelling approach based on Random Forest Classifiers.

The results demonstrate how a sound conceptual design is more important to achieve robustness than localised strength or ductility improvements.

We finally paint a bigger picture of a qualitative-quantitative robustness approach, where the designer considers robustness on different levels of the structural scale, hoping that eventually we will be able to understand robustness intuitively for tall timber buildings as we currently do for simpler structures, such as cast-in-situ reinforced concrete midrise buildings.

The quantification framework and its application are valid for structures of any material and make an important step towards a better understanding of the collapse behaviour – and hence safety – of new structural typologies such as tall timber buildings.

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