# 6 Earthquakes and robustness

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Some of the properties sought in seismic design of buildings are also considered fundamental to guarantee robustness of structures. Moreover, some key concepts are common to both seismic and robustness design. In fact, both analyses consider events with a very small probability of occurrence, and consequently, a significant level of damage is admissible. As very rare events, in both cases, the actions are extremely hard to quantify. The acceptance of limited damage requires a system based analysis of structures, rather than an element by element methodology, as employed for other load cases.

As for robustness analysis in seismic design, the main objective is to guarantee that the structure survives an earthquake, without extensive damage. In the case of seismic design, this is achieved by guaranteeing the dissipation of energy through plastic hinges distributed in the structure. For this to be possible, some key properties must be assured, in particular ductility and redundancy.

The same properties are fundamental in robustness design, as a structure can only sustain significant damage if capable of distributing stresses to parts of the structure unaffected by the triggering event.

## 6.1 Earthquake design

In order to obtain structures resistant to earthquakes, the following aspects must be considered: structural simplicity; uniformity, symmetry and redundancy; bi-directional resistance and stiffness; torsional resistance and stiffness; diaphragmatic behaviour at the storey level; and, adequate foundations.

A clear and direct path for the transmission of the seismic forces is available in simple structures while uniformity allows the inertial forces created in the distributed masses of the building to be transmitted via short and direct paths. Redundancy allows a more favorable redistribution of action effects and widespread energy dissipation across the entire structure. A basic goal of a seismic design is the establishment of diaphragmatic action of the horizontal load bearing systems and the connection (anchorage of the diaphragms) to the vertical load bearing components (walls or frames) in order to transfer the seismic forces to the most rigid ones and tie the whole building.

The choice of the methods of analysis depends on the structure and the objective of the analysis: linear static analysis (termed the "lateral force" method of analysis in EN 1998-1 (CEN 2004)); modal response spectrum analysis (also termed in practice "linear dynamic); non-linear static analysis (commonly known as "pushover" analysis); and, non-linear dynamic analysis (time-history or response-history analysis).

Most earthquake design codes provide an acceleration response spectrum curve that specifies the design acceleration (which means the horizontal load) based on the natural period of the structure. The basic principle of EN 1998-1 (CEN 2004) is that when the structure presents a ductile behaviour, the design acceleration and the horizontal force imposed to the building is reduced by division by the so called behaviour factor $q$. The behaviour factor $q$ is an approximation of the ratio of the internal forces that the structure would experience if its response was completely elastic, to those that may be considered in the design to ensure a satisfactory response of the structure. The behaviour factor is affected by several parameters such as ductility, overstrength and redundancy reduction factors.

## 6.2 Timber structures under seismic loads

Satisfactory performance of timber buildings, in general, can be partially attributed to the material characteristics of wood itself, and to the lightness and high redundancy of most wood-based structural systems. The lateral redundancy plays an important role in seismic performance of timber structures. A redundant design will almost certainly offer more parallel load paths that can transmit the applied lateral loading on the building down to the foundation. The detailing of connections is very important because the more integrated and interconnected the structure is, the more load distribution possibilities there are. The building's structural integrity is only as good as the weakest link in the load transmission path, and as a consequence, good performance expectations are contingent on appropriate design, quality workmanship and proper maintenance.

For timber structures, EN 1998-1 (CEN 2004) presents upper limit values of the behaviour factor depending on the ductility class, on the structural type (essentially reflecting the greater or lesser redundancy of the structure as a whole) and on the nature of the structural connections (essentially reflecting its ductility and energy dissipation capacity). Semi-rigid and rigid connections are normally associated with the distinction between dissipative and low-dissipative structures, respectively.

EN 1998-1 (CEN 2004) proposes a classification of timber structures in Ductility Class Medium (DCM) and Ductility Class High (DCH) for dissipative structures and Ductility Class Low (DCL) in the case of non-dissipative structures. Besides the general upper limit of $q = 1.5$ for DCL accounting for overstrength, for DCM and DCH the values indicated for $q$ in table 8.1 of EN 1998-1 (CEN 2004) are reproduced in Table 3 with a different arrangement that highlights the influence of the various parameters on the ductility of timber structures (namely the superior behavior of correctly designed and executed nailed connections).

**Table 3:** Maximum values of the behavior factor q for timber structures of DCM and DCH

| Structural type | DCM | DCH |
|---|---|---|
| Wall panels with glued diaphragms connected with nails and bolts | Glued panels<br>$q = 2.0$ | Nailed panels<br>$q = 3.0$ |
| Wall panels with nailed diaphragms connected with nails and bolts | - | Nailed panels<br>$q = 5.0$ |
| Trusses | Doweled and bolted joints<br>$q = 2.0$ | Nailed joints<br>$q = 3.0$ |
| Mixed structures with timber framing and non-load-bearing infills | $q = 2.0$ | - |
| Hyperstatic portal frame with doweled and bolted joints | $\mu \geq 4$<br>$q = 2.5$ | $\mu \geq 6$<br>$q = 4.0$ |

NOTE: μ is the static ductility ratio.

## 6.3 Seismic design and robustness

To analyze the influence of seismic design in the robustness of structures, it is fundamental to define the main strategies to improve robustness. In general, robustness can be improved by reducing the probability of damage, reducing the probability of failure if damage occurs, or by reducing the cost of failure. In the first case, it is paramount to define alternative load paths and to guarantee that: (i) enough resistance exists in these paths to prevent failure; (ii) enough ductility exists to guarantee these paths can be mobilized. If the improvement in robustness is to be achieved through reduction in cost associated with partial failures, then compartmentalization is crucial. In this case, load paths must be cut, in order to limit the extent of failure.

The philosophy of designing to limit the spread of damage, rather than to prevent damage entirely, is different from the traditional approach to designing to withstand dead, live, snow, and wind loads, but is similar to the philosophy adopted in modern earthquake-resistant design (FEMA 2002).

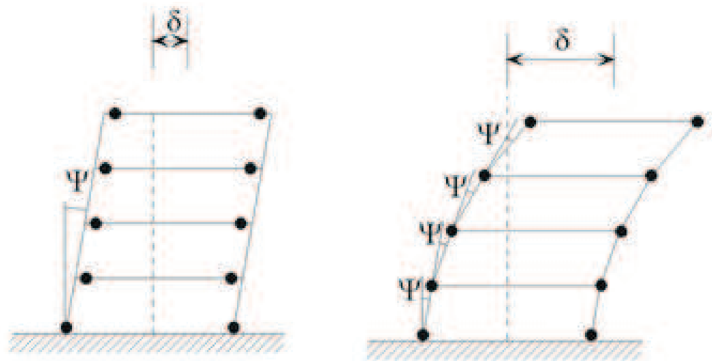
The guiding principles for a good conceptual design for earthquake resistant buildings have a significant influence on the robustness of structures. In fact, structural simplicity, uniformity, symmetry and redundancy are fundamental in the existence of alternate load paths, a key concept in robustness design.

Above all, the seismic design leads to an improvement in ductility and redundancy, as well as ensuring the interconnection of the structure. As a consequence, if a structure is designed according to existing seismic codes, a significant improvement to its resistance in the event of damage might be achieved. On the other hand, the increased redundancy and removal of weak links between elements and parts of the structure will allow damage to propagate through the structure, leading to higher costs in the event of failure.

In the particular case of timber structures, seismic design requires a much closer attention to detailing of connections. This can, indirectly, provide enhanced robustness since a significant number of observed failures are associated with errors in connections between elements.

Lastly, the consideration of earthquakes in some regions has led to significant evolution of engineering practice, leading to significant differences in common practice between countries where earthquakes are likely to occur, if only over long time periods, and those where they are not considered in design. Some of these practices can have a large effect on the robustness of structures, in particular, timber structures.

A clear example of this is the use of strong column – weak beam concept in designing buildings, common for seismic resistance. In seismic areas, columns are usually continuous elements, and beams are connected to column at each span. This situation guarantees that key elements, as the columns, are capable of sustaining additional loads, and failure will occur in the beams. This will limit progressive collapse to a single floor and to a bay. If, on the other hand, strong beams or continuous beam are used, failure will progress from bay to bay, increasing the affected area and, consequently, failure costs.




a. Weak beams b. Strong beams
**Figure 16.** Strong column – weak beam concept

## 6.4 Eurocode 8 and robustness prescriptive rules

At present, few existing codes present significant prescriptive rules to improve robustness of structures. However, there are some general rules identified to have a positive influence on the robustness, namely: (i) selective "overstrength" (strong column/weak beam concept); (ii) redundancy (e.g. by providing alternative paths for loads shed from damaged elements); (iii) ductility of response (e.g. by adopting members and connections that can absorb significant strain energy without rupture or collapse).

Analyzing the EN 1998-1 (CEN 2004) provisions, in particular the ones specific to timber structures, several measures can be pointed out to enhance robustness:

- [8.6(4)] In order to ensure the development of cyclic yielding in the dissipative zones, all other structural members and connections should be designed with sufficient overstrength. This overstrength requirement applies especially to: anchor-ties and any connections to massive sub-elements; and, connections between horizontal diaphragms and lateral load resisting vertical elements;
- [4.2.1.2(5)] The use of evenly distributed structural elements increases redundancy and allows a more favorable redistribution of action effects and widespread energy dissipation across the entire structure;
- [5.2.3.5(1)] A high degree of redundancy accompanied by redistribution capacity should be sought, enabling a more widely spread energy dissipation and an increased total dissipated energy. Consequently structural systems of lower static indeterminacy should be assigned lower behaviour factors;
- [2.2.4.1 (2)P] In order to ensure an overall dissipative and ductile behaviour, brittle failure or the premature formation of unstable mechanisms should be avoided. To this end, where required in the relevant Parts of EN 1998, resort should be made to the capacity design procedure, which is used to obtain the hierarchy of resistance of the various structural components and failure modes necessary for ensuring a suitable plastic mechanism and for avoiding brittle failure modes.

Using the capacity design method, it is possible, by choosing certain modes of deformation, to ensure that brittle elements have the capacity to remain intact, while the inelastic deformations occur in selected ductile elements. These “fuses” or energy absorbers act as dampers to reduce force level in the structure (Thelandersson 2003). In timber structures, the ductility is concentrated in the joints whereas the timber elements must be regarded as behaving elastically. Therefore, a reliable strength prediction of the joint and its components is essential for applying the capacity design and ensuring the required ductility. This is the possible explanation for the absence of EN 1998-1 (CEN 2004) provisions for the capacity design method application to the case of timber structures.

## 6.5 Examples

In this section, several examples of failures are analyzed and the foreseeable influence of considering seismic design on the outcome will be evaluated.

The emphasis will be placed on prescriptions defined for areas of moderate or strong seismic risk, in particular when medium or high ductility is required. Prescriptions focused on areas of low seismic risk, in particular, the use of horizontal loads without any further requirements, have little influence of the structural robustness.

The first example is the Ronan Point Building failure, triggered by a gas explosion. In this pre-fabricated structure, the consequences of the explosion were amplified by poor workmanship and very limited connection between elements. The existence of strong links between elements is a central requirement in seismic design, and, had earthquake loading been considered, a different, more redundant, structure would have been erected. In principle, this would have reduced the impact of the explosion, limiting the indirect costs associated to the incident.

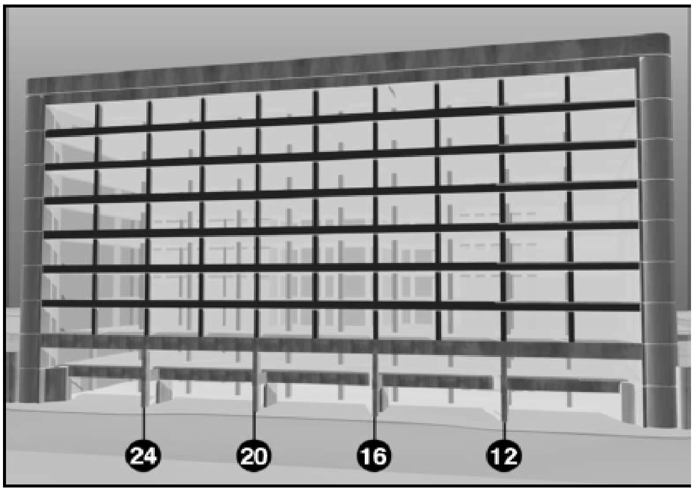



**Figure 17**. Alfred Murrah Federal Building structure

The Alfred Murrah Federal Building collapsed following the explosion of a car bomb parked in the basement. The building had a structural system composed of regular frames, but, at the ground level, the number of columns was reduced, as shown in figure 17. This structural system led to an increase in consequences of the explosion, and could have been avoided, had the building been analyzed in a seismic design perspective. In fact, the soft first story failure is prevented by the seismic design. (Corley et al. 1996) pointed out that more than 50% of the collapsed area would have stood if the structure had been designed with special moment frames found in seismic regions as opposed to the ordinary moment frames used in the building.

In 1993, a car bomb exploded in the parking lot under World Trade Centre building, causing a significant local damage with a cost of $300,000,000. However, the redundant structure, supported by numerous smaller columns, rather than a central nucleus, significantly reduced the consequences of damages, and no important indirect damages resulted from the explosion.

At the beginning of the year 2006, 2$^{nd}$ January, the ice-arena roof in Bad Reichenhall collapsed under the actual snow load (Figure 18). Fifteen people died, thirty were partly heavily injured. The main reasons for the collapse are: (i) use of urea-formaldehyde glue under moist conditions; (ii) mistakes in the static calculation; (iii) non robust construction; and, (iv) lack of maintenance.

**Figure 18.** Bad Reichenhall ice-arena collapse

According to the findings of experts (Winter and Kreuzinger 2008), one of the three main box-girders on the east side failed first. Due to the stiff cross girders, the loads were shifted from the box-girder that failed first to the neighbouring girders. These box-girders, which were already pre-damaged were also overloaded due to which the entire roof collapsed like a zipper. This transversal stiffness is, however, a desirable property under seismic design, and no real advantage could have been obtained from considering earthquake as a load. In fact, as shown in Figure 18 an increase in stiffness of transversal elements can, in fact, led to an increased risk associated with damage.

In the case of the Siemens Arena failure (figure 19), the first consequence of a seismic design would have been the increase of transversal stiffness. This could have caused progressive failure, following the collapse of one truss, leading to large increases in indirect consequences of damage. In fact, the 12 m long purlins between the trusses were only moderately fastened, so that a failure of one truss should not initiate progressive collapse. As all trusses had much lower strength than required by the failure of a neighbour element, it might be fair to conclude that the extent of the collapse was not disproportionate to the cause, as analyzed by (Munch-Andersen 2009). The result of a seismic design could have been an increase in transversal stiffness, which could have caused progressive collapse of the structure.

In these last two cases, the only possible advantage of seismic design would have been the closer attention paid to the detailing of connections, required for the definition of the dissipation zones defined in EN 1998-1 (CEN 2004). In fact, connections played a major role in both incidents, and a more careful design could have avoided the errors.

a) An intact truss is seen to the right

b) Rupture at the critical cross section in the corner connection

**Figure 19.** Siemens Arena roof after the collapse of two trusses (Munch-Andersen 2009).