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Guidance on the design for structural robustness

Support to the implementation, harmonization and further development of the Eurocodes

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Contents

Abstract

This JRC Technical Report presents scientific and technical background information to introduce the different aspects involved in providing robustness of structures. It is intended to bring together references and ongoing work on the subject as well as stimulate debate. It presents background information, state-of-the-art references and discusses provisions in available guidelines. As such, it serves as a basis for further work to achieve a harmonized European view on the consideration of robustness in the design, execution and assessment of structures. The report focusses on new-build construction, although the underlying principles also apply to existing structures.

The report introduces the general principles of structural robustness, including concepts and terminology, hazards and damage scenarios as well as assessment of the consequences of failure. An overview is provided of current standardization and design guidelines in Europe as well as outside of Europe. Strengths and weaknesses in current provisions are discussed. State-of-the-art information is collected covering alternative design strategies, approaches and considerations. Specific information on strategies to improve robustness is outlined, including the importance of allowing for ageing and deterioration, and aspects related to multi-hazard design. Whilst robustness as a design principle covers a range of extreme design events, including seismic and fire, differences in design approaches for such exposures are also important to recognize. State-of-the-art research information is referenced where available. Finally, a series of novel proposals for robustness provisions is provided encompassing more detailed technical guidance concerning the tying force strategy, the alternative load path strategy, etc. are proposed to encourage discussion.

Foreword

The construction ecosystem is of strategic importance to the European Union (EU), as it delivers the buildings and infrastructures needed by the rest of the economy and society, having a direct impact on the safety of persons and the quality of citizens' life. The construction ecosystem includes activities carried out during the whole lifecycle of buildings and infrastructures, namely design, construction, maintenance, refurbishment and demolition. The industrial construction ecosystem employs around 25 million people in the EU and provides an added value of EUR 1 158 billion (9.6% of the EU total)[1,2,3].

The construction ecosystem is a key element for the implementation of the European Single Market and many other important EU strategies and initiatives. The European Green Deal (COM(2019) 640 final) aims to achieve climate neutrality for Europe by 2050, and relies on numerous initiatives, noteworthy:

— the New Circular Economy Action Plan (COM(2020) 98 final) and the New Industrial Strategy for Europe (COM(2020) 102 final) intending to accelerate the transition of the EU industry to a sustainable model based on the principles of circular economy;

— the revision (COM(2022) 144 final) of the Construction Products Regulation (Regulation (EU) No 305/2011) aiming to enable the construction ecosystem's contribution to meeting climate and sustainability goals and embrace the digital transformation of the built environment;

— the New EU Strategy on Adaptation to Climate Change (COM (2021) 82 final) supported by the recent Commission Communication on managing climate risks (COM(2024) 91 final) that reinforces the need to address climate change concerns to guarantee the resilience and sustainability of built structures and infrastructures and to ensure regular science-based risk assessments;

— the first European Climate Risk Assessment (EUCRA) report which highlights the importance of EU policies for the built environment, including updating construction standards and related European datasets.

Furthermore and recognizing that the EU's ambitions towards a climate neutral, resilient and circular economy cannot be delivered without leveraging the European standardization system, the European Commission presented a new Standardization Strategy (COM(2022) 31 final). The strategy spots standards as "*the silent foundation of the EU Single Market and global competitiveness*".

The EU has put in place a comprehensive legislative and regulatory framework for the construction sector, including European standards (EN). Within this framework, the Eurocodes are a series of 10 European standards, EN 1990 to EN 1999, providing common technical rules for the design of buildings and other civil engineering works. In fact, the Commission Communication on managing climate risks directly mentions the Eurocodes, highlighting the role of building and infrastructure standards in integrating climate adaptation and resilience.

The Commission Recommendation 2003/887/EC on the implementation and use of the Eurocodes for construction works and structural construction products recommends undertaking research to facilitate the integration into the Eurocodes of the latest developments in scientific and technological knowledge. In this context, the so-called second generation of the Eurocodes is under development under Mandate M/515 and expected to be available by 2026. The second generation Eurocodes incorporates improvements to the existing standards and extends their scope by embracing new

[1] Commission staff working document: Scenarios for a transition pathway for a resilient, greener and more digital construction ecosystem (https://ec.europa.eu/docsroom/documents/47996)

[2] Council of the EU, Press release 30 June 2023, https://www.consilium.europa.eu/en/press/press-releases/2023/06/30/council-adopts-position-on-the-construction-products-regulation/

[3] Transition Pathway For Construction, European Commission, DG GROW, https://ec.europa.eu/docsroom/documents/53854

methods, new materials, and new regulatory and market requirements, including considerations for climate change impact on structural design.

This background document is published as a part of the JRC Report Series "Support to the implementation, harmonization and further development of the Eurocodes" and presents **background information, the state of the art and critical assessments relating to technical guidance for the assessment of structural robustness in the design of new structures. This document was developed by CEN/TC 250 Working Group (WG) 6 on Robustness** with the participation of the **JRC**. The purpose of this working group is to bring together the different national views and approaches concerning structural robustness and to develop a broadly accepted and coherent set of harmonised European technical rules for structural robustness.

This JRC Report presents scientific and technical background intended to provide a clear view of the different aspects involved when considering structural robustness and to stimulate debate. As such, it serves as a basis for further work to achieve a harmonized European view on the treatment of structural robustness.

We hope that this report will provide a sound and helpful basis for discussions about the topic of structural robustness, in particular concerning standardization work.

The report is available to download from the "Eurocodes: Building the future" website (http://eurocodes.jrc.ec.europa.eu).

The authors have sought to present useful and consistent information in this report. However, users of the information contained in this report must assess if such information is suitable for their purposes.

Ispra, 2024

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Report Series “Support to the implementation, harmonization and further development of the Eurocodes”

In the light of the Commission Recommendation of 11 December 2003, DG JRC is collaborating with DG GROW and CEN/TC 250 “Structural Eurocodes”, and is publishing the Report Series “Support to the implementation, harmonization and further development of the Eurocodes”. This Report Series includes, at present, the following types of reports:

1. **Science for policy documents**, conveying the implications of scientific and technical evidence for a policymaking process;
2. **Technical documents**, facilitating the implementation and use of the Eurocodes and containing information and practical examples (Worked Examples) on the use of the Eurocodes and covering the design of structures or its parts (e.g. the technical reports containing the practical examples presented in the workshop on the Eurocodes with worked examples organized by the JRC);
3. **Pre-normative documents**, resulting from the works of the CEN/TC 250 and containing background information and/or the first draft of proposed normative parts. These documents can be then converted to CEN technical specifications;
4. **Background documents**, providing approved background information on the current Eurocode part. The publication of the document is at the request of the relevant CEN/TC 250 Sub-Committee;
5. **Scientific/Technical information documents**, containing additional, non-contradictory information on the current Eurocode part, which may facilitate its implementation and use, or preliminary results from pre-normative work and other studies, which may be used in future revisions and further developments of the standards. The authors are various stakeholders involved in the Eurocodes process and the publication of these documents is authorized by the relevant CEN/TC 250 Sub-Committee or Working Group.

Editorial work for this Report Series is **performed by the JRC** together with partners and stakeholders, when appropriate. The publication of the reports types 3, 4 and 5 is made after approval for publication by CEN/TC 250, or the relevant Sub-Committee or Working Group.

The publication of these reports by the JRC serves the purpose of implementation, further harmonization and development of the Eurocodes. However, it is noted that neither the Commission nor CEN are obliged to follow or endorse any recommendation or result included in these reports in the European legislation or standardisation processes.

The reports are available to download from the website “Eurocodes: Building the future” (http://eurocodes.jrc.ec.europa.eu).

Acknowledgements

This report has been prepared to support the development of future European guidance for the assessment of structural robustness under the shield of CEN/TC 250. Both CEN/TC 250 and JRC acknowledge the substantial contribution of the many international experts of CEN/TC 250/WG 6 and others, who have supported the works with their essential input and reviews. In particular, the tremendous work from the writing panel of this JRC report is recognized. Finally, the contribution of Artur Pinto and Silvia Dimova, formerly at JRC Unit Safety & Security of Buildings (now Built Environment), is gratefully acknowledged.

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1 Introduction and scope

1.1 Introduction

Structural failures, especially involving progressive collapse, such as Ronan Point, tragically bring to light the importance of robustness in design – both to the structural engineering community and society at large. Failures can be caused by extreme or exceptional loading and damaging events, though also under standard, to be expected, loading scenarios and structural deterioration processes, and can often be deemed disproportionate. National regulations across Europe and the world and accompanying design standards outline requirements for buildings to be designed to resist disproportionate failure. These provisions generally require structural elements to be tied together, adding redundant structural members, which have sufficient strength and stiffness to resist, etc.

Over the past decades, some accidental or deliberate exceptional events (such as explosions, impact, unforeseen material degradation, etc.) reveal the vulnerability that structures can have to localized damage. The importance of buildings to resist progressive collapse remains one of the contemporary design aims of the structural engineering community. It remains a developing field, with most research undertaken over the last 40 years, and as such brings a need to regularly review and adjust codes of practice and guidelines to keep pace with knowledge.

This JRC Report presents scientific and technical background information to introduce the different aspects involved in providing robustness to structures. It is intended to bring together references and ongoing work on the subject as well as stimulate debate. It presents background information, state-of-the-art references and discusses provisions in available guidelines. As such, it serves as a basis for further work to achieve a harmonized European view on the consideration of robustness in the design, execution and assessment of structures. The report focuses on new-build construction, although the underlying principles also apply to existing structures.

The report consists of six main chapters.

- Chapter 1: General introduction, scope and relevant terms and definitions
- Chapter 2: General principles related to structural robustness.
- Chapter 3: An overview is provided of current standardization and design guidelines in Europe and outside Europe, together with some background information, when available. Strengths and weaknesses in current provisions are discussed.
- Chapter 4: New state-of-the-art information is collected covering alternative design strategies, approaches and considerations. Specific information on strategies to improve robustness is outlined, including the importance of allowing for ageing and deterioration and aspects related to multi-hazard design. Whilst robustness as a design principle covers a range of extreme design events, including seismic and fire, differences in design approaches for such exposures are also important to recognize.
- Chapter 5: Based on more recent developments, state-of-the-art research information is provided concerning quantitative measures and calculation methods for robustness. An overview of performance indicators for structural robustness is provided as well as an overview of the structural analysis methods with their abilities and limitations.
- Chapter 6: A series of novel proposals for robustness provisions is provided based on the work performed by the M/515 mandated work of project team 2 of CEN/TC250/WG6 encompassing more detailed technical guidance concerning the tying force strategy, the alternative load path strategy, etc.

1.2 Scope

The topic of structural robustness is complex and subject to National regulatory provisions, across Europe. In fact, every country has specific regulations about robustness of structures. There are many local practices throughout Europe to achieve robustness. The complexity and diversity of the topic are illustrated by these approaches, ranging from conceptual principles to be considered in the design phase, to prescriptive rules and quantitative design measures that are available in specialist literature. Furthermore, robustness strategies and provisions differ depending on the materials and structural typologies and forms used. As such, finding a harmonized view is a difficult challenge for the standardization work in the coming years. The report provides a broad view of the topic, definitions and practices as well as universal general principles. This work brings together key information to enable the debate that will support a common view as the standardization for robustness is progressed.

The information contained in the report is intended for a broad range of users, encompassing designers, decision-makers, third-party control organizations, experts, etc. As such, not all parts of the report might be of equal importance to a particular user. Depending on the needs of several users, the reader is provided with references and supporting information, aligned with complementary and more detailed provisions where relevant.

Although the authors have attempted to provide a broad overview of the available state-of-the-art knowledge in the field, the intention has not been to provide a complete catalogue, but rather to provide sufficient information to understand the different aspects involved and guide readers to more detailed information if they wish to learn more about the topic. In particular, relevant background information was collected and assistance was given in relation to some of the principles underlying the Eurocode approach to robustness. Further, potentially relevant information that could be considered for the further evolution of the Eurocode in relation to robustness is highlighted. However, it should be acknowledged that the topic of structural robustness is a fast-paced and evolving research field where research is still ongoing; this is highlighted in the report.

Finally, it is important to note that the information presented here is background information for users seeking a more thorough understanding and appreciation of ongoing work in this field. First and foremost designers must consult the Eurocodes and relevant National provisions for their designs and stakeholder interactions. It is hoped that the compilation of this material will support those addressing this most important design consideration and allow them to arrive at informed decisions, enabling them to gain an efficient and swift overview of relevant references.

1.3 Terms and definitions

1.3.1 Context

Set of all relevant circumstances within which engineering decisions are made.

1.3.2 Hazard

Exceptionally unusual and severe threat, e.g. a possible abnormal action or environmental influence, insufficient strength or stiffness, or excessive detrimental deviation from intended dimensions (ISO 2394, 2015).

1.3.3 Hazardous scenario

Series of situations, transient in time, that a system might happen to undergo, and which may endanger the system itself, people, and the environment (ISO 2394, 2015).

1.3.4 Event

Occurrence, or change, of a particular set of circumstances.

1.3.5 Hazardous event

Occurrence of a hazard or a hazardous scenario.

1.3.6 Exposure

Set of different events that could act on the constituents of the system with potential consequences for the considered system.

1.3.7 Load path

Integral of all elements of the system affected by action effects, from the point of application of the action to the boundaries of the system.

1.3.8 Damage

Unfavourable change in the condition of a system that can affect the performance of the latter.

1.3.9 Failure mode

Description of how failures propagate within the structural system.

1.3.10 Consequences

Adverse outcome of a hazardous event. Consequences can be defined to be restricted to the performance of the structural system or as having a wide scope, e.g. including human, economic and environment qualities.

1.3.11 Progressive collapse

Characteristic of a failure mode where an initially localised failure event leads to a sequence of follow-up failure events in a domino style.

1.3.12 Disproportionate collapse

Characteristic of a failure mode where there is a distinct disproportion the immediate damage following a triggering event and the follow-up failure events.

NOTE: It would be desirable that a discrimination between proportionate and disproportionate collapse could be made on the basis of a range of area or percentage of members failed, but such quantitative proposal are currently not readily available in literature or standardization documents.

1.3.13 Fragility

System characteristic expressing its structural performance, typically in terms of probability of exceedance of a certain limit state conditional to the occurrence of a specific intensity measure.

1.3.14 Vulnerability

Describes the degree of susceptibility of a structural system to attain a particular level of consequences, for a given hazardous event.

1.3.15 Damage tolerance

Ability of a structural system to sustain a given level of damage while maintaining equilibrium with the applied loads.

1.3.16 Continuity

Continuous connection of members of a structural system.

1.3.17 Ductility

Ability of a structural system to sustain the applied loads by dissipating plastic energy.

1.3.18 Integrity

Condition of a structural system to enable force transfer among members in case of accidental events.

1.3.19 Uncertainties

State of deficient information, e.g. related to the understanding, or knowledge of, an event, its consequence, or likelihood.

1.3.20 Probability

Mathematical expression of the degree of confidence in a prediction.

1.3.21 Reliability

Probabilistic measure of the ability of a structural system to fulfil specific design requirements. Reliability is commonly expressed as the complement of the probability of failure.

1.3.22 Structural safety

Quality of a structural system, referring to the strength, stability and integrity of a structure to withstand the hazards to which it is likely to be exposed during its life-time.

1.3.23 Risk

A measure of the combination (usually the product) of the probability or frequency of occurrence of a defined hazard and the magnitude of the consequences of the occurrence.

1.3.24 Redundancy

The ability of the system to redistribute among its members the load which can no longer be sustained by some damaged and/or deteriorated elements

1.3.25 Foreseen event

An event to which the structure can be subjected and of which the nature, the magnitude, and the probability of occurrence during the construction or use of the structure, can be defined.

1.3.26 Unforeseen event

An event to which the structure can be subjected, but which occurrence cannot be defined, until it materializes.

References

ISO. (2015). ISO2394 (Fourth edition) - General principles on reliability for structures. International standardization Organization.

2 Main principles on structural robustness

2.1 Concept of robustness and definitions

2.1.1 Introduction

During the last few decades, increasing attention has been focused on the concepts of structural robustness, disproportionate failure and progressive collapse. This is due to the continued occurrence of cases in which the structures are involved in disproportionate collapse with respect to an initial localized structural damage, generated by an accidental action, like explosions and impacts, or underestimated events, like deterioration processes. It is noted that the field of robustness only deals with unforeseen events and loading. Other accidental events are being dealt with by application of Eurocodes.

The first iconic failure in this field was the partial collapse in 1968 of the Ronan Point building in London after a relatively small gas explosion led to the progressive failure of its entire corner (Figure 2-1). The building was a 22-storey residential building, constructed in 1966, assembling precast concrete panels together without a structural frame system. Many connections relied on friction only and each floor was supported by the load-bearing walls directly beneath it. In May 1968, a gas explosion on the 18th floor caused the failure of an external precast concrete panel creating a progressive collapse (chain reaction) upward and then downward due to dynamic effects until the entire southeast corner of the structure collapsed. Moreover, when the building was dismantled, signs of poor workmanship was found (Pearson & Delatte 2005).

The collapse of the Ronan Point building may be attributed to the lack of structural redundancy and robustness of the system (no alternative load paths could be activated following the removal of a single component). A relatively small initial damage ended with a disproportionate effect, namely the collapse of the building corner.

Another failure event which increased the attention of the concept of disproportionate collapse and structural robustness concerns the Alfred Murrah Federal Building (Oklahoma City, Oklahoma, USA). The federal structure was a 9-storey reinforced concrete frame building with shear walls. The stability of the system was governed by a transfer girder at the third-floor level and the lower storey columns supporting it (Corley et al. 1998). In 1995, a truck bomb explosion outside the building resulted in the failure of three main columns supporting the transfer girder. Almost half of the building collapsed (Figure 2-2). Records indicated an extremely well designed and detailed structure. The terrorist attack resulted in the loss of three lower columns and a portion of the surrounding floors as direct consequences of the explosion. But a progressive collapse extended the damage disproportionately, far beyond the structural failure directly caused by the bomb explosion.

Other building collapse events, including the attacks to the twin towers of the World Trade Center and Pentagon Building in 2001, emphasize the need for additional considerations in structural design codes regarding the concept of disproportionate collapse and robust structures (Carper & Smilowitz 2006). The Pentagon Building is a five-storey reinforced concrete building subdivided into five segmented concentric rings separated by expansion joints (Figure 2-3). During the terrorist attack in September 2001, an aircraft hit the building and the damage involved only the impact area thanks to the structural compartmentalisation of the building. Similar results occurred to the Charles de Gaulle Airport (Paris, France). In 2004, a portion of the concrete shell roof of the airport collapsed (Figure 2-4).

Progressive and disproportionate failures are not limited to buildings. They are also recognised as a major concern for the collapse of bridges (Starossek 2008, National Transportation Safety Board 1984). An example is the progressive failure of the Heang-Ju Bridge (Seoul, South Korea), a continuous prestressed concrete girder bridge. In 1992, after the failure of a temporary pier in the main span, the damage propagated through ten adjacent spans (Figure 2-5). To this event the continuous prestressing tendons in the deck played a significant and disastrous role by creating a chain between the spans. The opposite situation occurred in 1975, when the discontinuity of the prestressing tendons between adjacent spans of the Tasman Bridge (Hobart, Australia) avoided a progressive collapse (chain reaction) when a bulk carrier collided against two pylons. The two closest spans to the impact area were the only damaged structural elements (Figure 2-5). More recent bridge collapse events, like the I-35W Mississippi River Bridge failure (Mississippi, Minnesota, USA) in 2007 and the Morandi Bridge event (Genoa, Italy) in 2018, increased the attention on bridge design, execution and maintenance regarding disproportionate collapse, structural integrity and robustness. The Morandi bridge event also highlights the importance of considering the importance of specific robustness provision for isostatic structures.

Figure 2-1. Collapse of Ronan Point Building, London, UK (1968) [4]

Figure 2-2. Collapse of the Alfred Murrah Federal Building, Oklahoma City, USA (1995) [5]

[4] Source: https://www.architecture.com/explore-architecture/inside-the-riba-collections/people-in-high-places, https://failedarchitecture.com/the-downfall-of-british-modernist-architecture/ and https://failedarchitecture.com/the-downfall-of-british-modernist-architecture/

[5] Source: https://en.wikipedia.org/wiki/Alfred_P._Murrah_Federal_Building and https://www.kpbs.org/news/arts-culture/2017/02/02/american-experience-oklahoma-city

Figure 2-3. Terrorist attack - The Pentagon Building, Washington D.C., USA (2001) [6]

Figure 2-4. Collapse of Terminal 2E of the Charles de Gaulle Airport, Paris, France (2004) [7]

Figure 2-5. Collapse of (a) Heang-Ju Bridge, Seoul, South Korea (1992) and (b) and Tasman Bridge, Hobart, Australia (1975) [8]

(a) (b)

Optimisation techniques in contemporary design and the and the speed of the execution phase without adequate quality control may lead to unsatisfactory levels of structural redundancy and robustness, thereby increasing the risk of disproportionate collapse when accidental or unforeseen events occur.

[6] Source: https://wjla.com/news/september-11th-20th-anniversary/gallery/how-the-pentagon-building-saved-lives-on-911-terrorist-attacks-collapse-crashing-september-11?photo=10 and https://en.wikipedia.org/wiki/File:Aerial_view_of_the_Pentagon_during_rescue_operations_post-September_11_attack.JPEG

[7] Source: https://engineers-channel.blogspot.com/p/charles-de-gaulle-airport-terminal-2e.html

[8] Source: https://en.wikipedia.org/wiki/Haengju_Bridge and https://en.wikipedia.org/wiki/Tasman_Bridge_disaster

In structural design codes and standards, the term robustness has been initially used to indicate the ability of a system to resist damage under extreme loads. Afterwards, since the early 1980s, British codes have included requirements to prevent the progressive spreading of an initial local failure leading to a disproportionate collapse.

However, in structural design, the concept of robust structures is still an issue of controversy. In fact, despite the fact that procedures aimed to identify weak links within structures have been reported in literature (Lu et al. 1999, Agarwal et al. 2003) and efforts have been made either to propose design strategies to prevent progressive and disproportionate collapse (Starossek & Wolff 2005, Ellingwood & Dusenberry 2005, Haberland & Starossek 2010, André & Faber 2019) or quantify robustness (Baker et al. 2008, André et al. 2015), there are no well-established and generally accepted criteria for a consistent definition and a quantitative measure of structural robustness (Starossek & Haberland 2011, André 2020). Recently, advances in robustness quantification have been accomplished for deteriorating structural systems (Biondini & Restelli 2008, Biondini 2009, Biondini & Frangopol 2014).

An important issue related to structural robustness is its perception, leading to significant differences in its definition and quantification in structural codes and scientific publications. Moreover, design strategies to avoid progressive collapse in literature and partially covered by design standards do not, in general, distinguish between the concepts of collapse resistance, structural redundancy, and structural robustness.

A selection of structural robustness definitions proposed in the last decades is reported in Table 2-1.

Table 2-1. Definitions related to the concept of structural robustness

Reference	**Structural Robustness Definition**
GSA, 2003	The ability of a structure or structural components to resist damage without premature and/or brittle failure due to events like explosions, impacts, fire or consequences of human error, due to its vigorous strength and toughness.
Starossek & Haberland, 2005	The term robustness is defined as insensitivity to local failure, with "insensitivity" and "local failure" being quantified by the applicable design criteria. Robustness is a property of the structure alone and independent of the loading.
Val & Val, 2006	The ability of a structure to absorb the effect of an accidental event without suffering damage disproportionate to the event that caused it. […] ability of the structure to withstand local damage without disproportionate collapse.
CEN - Eurocode 1. Part 1-7, 2006	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.
Bontempi et al., 2007	The robustness of a structure, intended as its ability not to suffer disproportionate damage as a result of limited initial failure, is an intrinsic requirement, inherent to the structural system organization.
Vrouwenvelder, 2008	The notion of robustness is that a structure should not be too sensitive to local damage, whatever the source of damage […].

Agarwal & England, 2008	Robustness is the ability of a structure to avoid disproportionate consequences in relation to the initial damage.
Biondini & Restelli 2008	Structural robustness can be viewed as the ability of the system to suffer an amount of damage not disproportionate with respect to the causes of the damage itself.
Narasimhan & Faber, 2009	A structure shall not be damaged by events like fire, explosions or consequences of human errors, deterioration effects etc. to an extent disproportionate to the severeness of the triggering event.
Gulvanessian et al., 2012	Property of structures that enables them to withstand unforeseen or unusual circumstances without unacceptable levels of consequences or intolerable risks.
fib Model Code, 2013	Robustness is a specific aspect of structural safety that refers to the ability of a system subject to accidental or exceptional loadings (such as fire, explosions, impact or consequences of human errors) to sustain local damage to some structural components without experiencing a disproportionate degree of overall distress or collapse.
André et al., 2015	Measure of the predisposition of a structural system to loss of global equilibrium, as a result of a failure event for a given hazard scenario.

Considering these definitions, structural robustness is most often associated with the ability of the system to avoid structural consequences that are disproportionate with respect to the extent of the triggering initial damage.

In general, robustness evaluations should not be restricted to cases involving only accidental actions, like explosions or impacts. Other cases are relevant, for example involving the continuous damage over the structures lifetime. The effects of ageing and deterioration process on civil engineering structures can lead over time to unsatisfactory structural performances disproportionate with respect to the corresponding damage (Zhu & Frangopol 2012). Although EN1990 stipulates that structures should be designed, executed and maintained adequately by qualified and experienced people and controlled by appropriate quality management, the occurrence of errors and negligence during the design, execution and operation phases (which could lead to significant and disastrous consequences) can never be fully excluded and for such situations robustness provisions can limit the consequences resulting from such errors or negligence.

However, for most structures, design in accordance with the Eurocodes is assumed to provide an adequate level of robustness without the need for any additional design measures to enhance structural robustness.[9]

2.1.2 Disproportionate and progressive collapse

Disproportionate and progressive collapse denote two different aspects of the evolution of structural damage: the concept of disproportionality between cause and effect and the concept of failure progression.

Disproportionate collapse refers to a significant disproportion between the initial damage event and the ensuing consequences, for example following the collapse of a major part or even the whole of a structure. This is simply a subjective judgment made about observations of the consequences following the occurrence of an initial damage event. Instead, progressive collapse refers to the collapse evolution, describing how, after a local

[9] Formulation in accordance to Note 2 in Clause 4.4(1) in FprEN1990:2022

initial event, failure may propagate to members other than the ones directly affected by the initial damage, leading to a chain reaction between elements. A collapse may be progressive horizontally (e.g. failure in succession of adjacent structural bays) or progressive vertically (e.g. failure in succession of columns supporting a certain number of floors) or a combination of both.

In general, a progressive collapse can result in a disproportionate collapse if the successive failures involve a large part of the structure with respect to the triggering initial event. On the other hand, a disproportionate collapse can be immediate or progressive. Therefore, a collapse may be progressive but not necessarily disproportionate in its extents, for instance if the damage is arrested after the propagation through a limited number of structural bays. On the other hand, a collapse may be disproportionate but not necessarily progressive. A selection of disproportionate and progressive collapse definitions proposed in literature is listed in Table **2-2** and Table **2-3**, respectively.

The concept of structural robustness is related to disproportionate effects of damage with respect to the initial causes or amount of the damage itself. However, the relationship between initial and final damage, which may be different due to a disproportionate and/or progressive damage process, is not sufficient to quantify structural robustness. The amount of damage has to be compared with the corresponding consequences. If damage does not lead to disproportionate consequences, the system is considered robust.

Table 2-2. Definitions related to the concept of disproportionate collapse

Reference	**Disproportionate Collapse Definition**
Agarwal & England, 2008	Disproportionate collapse results from small damage or a minor action leading to the collapse of a relatively large part of the structure.
Starossek & Haberland, 2010	A collapse that is characterized by a pronounced disproportion between a relatively minor event and the ensuing collapse of a major part or the whole of a structure.

Table 2-3. Definitions related to the concept of progressive collapse

Reference	**Progressive Collapse Definition**
ASCE, 2016	Progressive collapse is defined as the spread of an initial local failure from element to element resulting, eventually, in the collapse of an entire structure or a disproportionately large part of it.
Ellingwood, 2006	A progressive collapse initiates as a result of local structural damage and develops, in a chain reaction mechanism, into a failure that is disproportionate to the initiating local damage.
Canisius et al., 2007	Progressive collapse, where the initial failure of one or more components results in a series of subsequent failures of components not directly affected by the original action is a mode of failure that can give rise to disproportionate failure.

2.1.3 Structural robustness and collapse resistance

Structural robustness of a damaged system is a concept useful to prevent disproportionate structural collapse. Robustness refers to the ability of a structure to resist initiating events without exhibiting disproportionate performance reduction with respect to the causes or amount of damage. Generally, in a robust structure, no failures disproportionate to the initial damage will occur. Instead, the concept of collapse resistance can be regarded as the insensitivity of a structure to abnormal collapse events (Haberland & Starossek 2009, André et al. 2015).

For practical assessment, in case one wants to verify the ability of the structure to withstand the initiating event without the need to quantify and define the maximum extent of the progressive damage, a simplified and conservative verification of structural robustness could take basis in the verification of equilibrium of the considered part of the structure after the damage associated to the initiating event.

2.1.4 Structural robustness, redundancy, and static indeterminacy

The terms structural robustness, redundancy and static indeterminacy are often used as synonymous. However, they denote different properties of the structural system.

Robustness is related to the ability of the system to suffer a loss of performance not disproportionate with respect to the causes or amount of damage. Structural redundancy can instead be defined as the ability of the system to redistribute among its member the load which can no longer be sustained by other damaged and/or deteriorated elements. It may be affected by several factors, such as structural topology, member sizes, material properties, applied load and load sequences, among others (Frangopol & Curley 1987, Frangopol & Klisinski 1989, Frangopol & Nakib 1991, Ghosn et al. 2010).

The additional effort affecting the remaining system components after a local failure may reduce the structural performance and therefore lead to an unsatisfactory robustness level. Damage propagation mechanisms under ageing and structural deterioration may also involve disproportionate effects and alternative load redistribution paths, altering structural redundancy and structural robustness (Okasha & Frangopol 2010, Biondini & Frangopol 2017). Alternative load paths may enhance structural redundancy. However, increasing structural redundancy does not necessarily lead to increase structural robustness (Biondini 2009, André 2020). In fact, the redistribution of internal actions on the damaged system may promote the evolution of damage reducing robustness. The collapse of the Nanfang'ao Bridge (Taiwan) in 2019, may be related to lack of structural robustness. The 21-year-old tied-arch steel bridge failed after the collapse of a single supporting cable. The redistribution of internal forces after the initial damage resulted in a progressive cables collapse.

Redundancy is usually associated with the degree of static indeterminacy. However, the degree of static indeterminacy is not a consistent measure for structural redundancy and increasing the degree of static indeterminacy does not necessarily lead to increase structural redundancy. Examples can be found in Biondini et al. (2008).

2.1.5 Structural robustness, vulnerability, and structural integrity

The term "structural vulnerability" is often used in literature to express different concepts. Vulnerability can be adopted to describe the sensitivity of the performance of a structure to damage events. Therefore, in this case, the term "vulnerability" is used to express the susceptibility of a component (or a system) to some external action. In this approach, vulnerability is a property of the system since it examines the effect of potential hazards triggered by unexpected events or unforeseen load scenarios. For instance, an earthquake happening in a site "A" may produce very different consequences from the same earthquake event occurring in a different site "B". The infrastructure in place "B" may be more vulnerable because of its form, technical design, execution and condition properties. A structure can be made collapse resistant by reducing the vulnerability of its members by protection techniques or increasing the resistance of the structural elements.

Sometimes, the term "vulnerability" has also been adopted in terms of the sensitivity of damage leading to disproportionate consequences (Agarwal & Blockley 2007). In this approach, a structure is vulnerable if small damages lead to disproportionate failures.

Within this definition it is clear how vulnerability is considered as antagonistic to structural robustness.

The broad distinction however is related to consider the vulnerability as the conditional potential damage to a system or a state condition as a result of a hazard event. Therefore, in the first case, vulnerability is simply the susceptibility to damage; in the second case, it expresses the idea of the susceptibility in the sense that a small amount of damage can lead to disproportionate consequences. However, the damage-sensitivity definition is recommended to have a clear distinction between structural robustness and vulnerability of a system.

Moreover, damage-sensitivity may also be associated with different levels of structural integrity, that are associated with the severity of a potential structural failure with respect to its consequences. In fact, the global collapse of a structural system is considered more important than the local collapse of a single member or a portion of the system.

2.1.6 Requirements to measure structural robustness

In order to measure the amount of robustness of a structural system, some requirements must be satisfied:

- The system and the design objectives and requirements must be clearly defined, including robustness criteria and measures;
- The intended limit state functions of a system have to be identified and formulated, since structural robustness provides relevant information with respect to certain desirable system performance objectives;
- The hazardous events which may affect the structure have to be identified and the corresponding structural damages have to be computed, at both the element and system levels;
- The overall structural consequences of damaging scenarios have to be analysed with regard to the mentioned limit state functions and compared with the corresponding amount of damage. Two basic ingredients are hence necessary to be compared for robustness evaluation: the global damage affecting the system and the corresponding structural consequences;
- Proper consideration of the uncertainties affecting several input parameters, as a function of the level of accuracy required.

Therefore, robustness evaluation is associated with specific structural performance indicators, limit state conditions, and damage scenarios.

2.1.7 Concept of robustness in various fields

In general terms, without focusing on a specific scientific field, robustness is related to the sensitivity of a qualitative or quantitative features of a system with respect to changes in system state due to unexpected disturbances.

Robustness is therefore associated with certain performance characteristics influenced by some perturbations on the system. In order to define a robust system it is fundamental that the clarification of both the considered *system performance* and *system perturbation* of interest be provided. The comparison between these two features of the system is necessary for robustness evaluation.

Robustness interpretations are used in engineering as well as in other fields such as biology, statistics, control theory, among others. A selection of definitions of robustness in different scientific areas is reported in the following list:

- Structural Engineering: ability/property of a system to avoid a structural performance variation (*system performance*) disproportionately larger with respect to the corresponding damage (*system perturbation*).

- Software Engineering: ability of the software (*system performance*) to react appropriately to abnormal circumstances (*system perturbation*), i.e. circumstances outside of specifications (Meyer 1997).
- Statistics: a robust statistical technique is insensitive against small deviations in the assumptions (*system perturbation*) (Huber 1996).
- Ecosystems: ability of a system (*system performance*) to maintain functions even with changes in internal structure or external environment (*system perturbation*) (Callaway et al. 2000).
- Design optimization: a robust solution of an optimization problem under uncertainty is the one that has the best performance (*system performance*) against the worst contingency that may arise (*system perturbation*) (Kouvelis and Yu 1997).

2.2 Hazards and damage scenarios

In the field of structural robustness, a hazard is identified as a serious threat to the integrity of a structure and the safety of people (Canisius 2011). Instead, an exposure scenario is a set of hazards that possibly affect the structural response during construction and lifetime.

The characteristics of some hazardous events (such as the exact time of occurrence, its intensity and spatial distribution) are unknown to a designer and therefore design codes typically specify rules concerning unidentified actions, ranging from prescriptive rules (e.g. tying of elements or protecting vulnerable components) to notional hazardous scenarios to which a structure should be designed against (e.g. notional removal of element(s)). In addition, design codes (e.g. Eurocode) account for explicit design procedures and requirements for the identified accidental actions such as earthquake, fire, impact and internal gas explosions.

2.2.1 Type of hazards

The possible hazards that may play a role in the safety assessment of structural systems can be classified based on the nature of the event itself (Canisius 2011). Three categories are identified:

1. Type of hazardous events given by natural events and unintentional anthropogenic hazards, e.g. earthquake and wind actions. They can be combined with time-dependent effects from ageing and deterioration processes.
2. Security related events, such as vandalism and malicious attacks, intentionally man-made. Due to their high unpredictably, it is in general, convenient to adopt appropriate measures to prevent their occurrence and limit the possible damage propagation over the entire system, instead of designing a structure to be able to support these actions (which may lead to very strictly requirements).
3. Human errors and negligence during the design, execution and operation phases. Progressive increasing attention has been focused on this type of hazard after the iconic failure event of the Ronan Point Building, 1968 (Ellingwood 1987). An efficient strategy is to implement quality control procedures during the entire life of the structures, from the preliminary design process to the demolition phase. Even simple periodic visual inspections may significantly increase the knowledge of the structural condition over time and can be used to plan appropriate maintenance and repair interventions.

Some (non-limitative) hazards corresponding to these three categories are:

1. Gas explosion; dust explosion; fire; impact by vehicles, aircrafts, ships …; overloading; earthquake; landslide; mining subsidence, tornado or typhoon/hurricanes/cyclone; avalanche; rockfall; high groundwater; flood; storm surge; volcanic eruption; environmental attack; tsunami; …
2. Bomb explosion; fire; vandalism; …

3. Design or assessment error; material error; construction error; user error; lack of maintenance; …

Moreover, in particular cases, some hazards are a direct consequence of previous hazard, i.e. cascading effects, leading to more serious effects. For example:

— Tsunami after an earthquake;

— Fire after an earthquake;

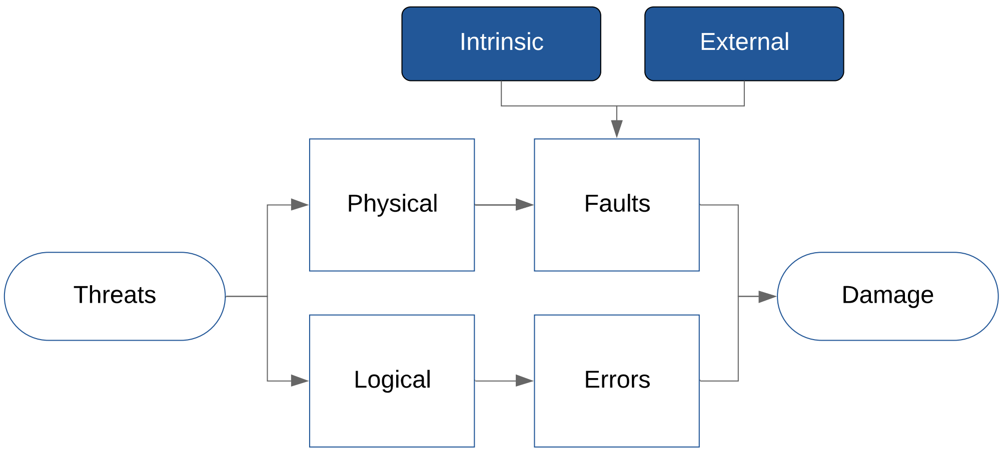
— Fire following a gas explosion or bomb blast;

— Accelerated material deterioration following damage from accidental action on the structure.

According to Bontempi et al. (2007), hazards can be divided into physical and logical threats, as shown in Figure 2-6. Physical threats are all the possible hazards that may create a damage or failure to the structure, further sub-divided into external hazards (e.g. extreme accidental and environmental actions) and intrinsic hazards that include all the undetected defects of the structure. Error threats include design, execution and operation errors. Another critical hazard not always considered, that can be classified in the logical group, is lack of maintenance (or even incorrect maintenance) which may generate disproportionate consequences with respect to the triggering damage over the structural lifetime.

Several hazards that could affect a structure are listed in Table 2-4, partially derived from Starossek & Haberland (2012). It should be noted that in relation to the term 'errors' in Figure 2-6, this should be interpreted that robustness serves as a last resource against adverse events resulting from human errors.

Figure 2-6. Classification of threats



Source: Bontempi et al., 2007 (redrafted)

Table 2-4. Possible hazardous events

Faults	External	Man-made	Impact, explosion, fire, excessive loading, vandalism, terrorist attack
		Environmental	Earthquake, landslide, extreme wind action, tornado, heavy snow load, scour, rock fall, volcano eruption

	Intrinsic	Lack of strength, cracks, ageing and deterioration
Errors		Design, material, construction and usage errors, lack or wrong maintenance

The characteristics of exposures and hazards are very different, depending on the specific type and time and space dependencies. Accidents, explosions and technical failures are generally events occurring suddenly. Floods and fire storms are usually relatively slower hazards, while deterioration processes and climate change are much slower.

The hazards to be considered in relation to structural robustness assessment can be specified either by a relevant authority or, where not specified, on a project-specific basis by relevant parties.[10]

2.2.2 Continuous damage and life-cycle robustness

The concepts of structural robustness, disproportionate failure and progressive collapse are generally associated to damage suddenly caused by accidental and extreme actions, such as explosions or impacts. However, damage may also arise gradually in time due to ageing and deterioration processes (Ellingwood 2005, Biondini & Frangopol 2016). The detrimental effects of these phenomena may lead to unsatisfactory structural performance under service loadings. Moreover, depending on the damage propagation mechanism, such kinds of damage could also involve disproportionate consequences. Deterioration processes may also interact over time with damage induced by other natural or man-made hazards. For instance, these processes may become very relevant for ageing bridges exposed to high levels of traffic loadings and seismic actions.

During the past decades, significant attention has been devoted, in different countries, to the condition rating of existing structures and infrastructures (Biondini & Frangopol 2018). The economic impact of ageing and deterioration processes on these systems is enormously high, particularly for bridges. A proper modelling of the structural system over its entire life-cycle by taking into account the effects of deterioration processes, time variant loadings, maintenance and repair interventions is therefore fundamental. In this context, robustness needs to be ensured over the entire service life.

Deterioration processes caused by environmental aggressiveness are generally very complex. Their behaviour over time depends on the damage mechanisms, the type of materials and the structures considered. For instance, considering traditional civil engineering materials, the lifetime of steel elements may be affected by corrosion and fatigue. For reinforced concrete structures, chemical processes associated with carbonation, sulfate and chloride attacks, reinforcing steel corrosion, alkali-silica reactions, freeze/thaw thermal cycles process and mechanical processes such as cracking, abrasion, erosion and fatigue may seriously affect the life-cycle performance of the structures (Biondini & Frangopol 2019). Moreover, several damage mechanisms might be simultaneously active, but progressing at different rates, including the sequence of occurrence.

The evolution over time of the deterioration processes needs to be described by suitable time-variant models. However, a mathematical description of the physical mechanism underlying the deterioration process is often not available. In such cases, empirical models can be successfully adopted (Ellingwood 2005).

[10] Formulation as included in FprEN1990:2022

2.2.3 Damage modelling

An analytical description of damage processes may be seen as too complex to be adopted or not always suitable to be incorporated in a robustness evaluation. However, effective models can often be established for practical applications by directly assuming the structural damage as a progressive deterioration of the mechanical and geometrical properties at material, member and system levels. To this purpose, the amount of deterioration can be defined by means of damage indices $\delta \in [0;1]$ associated with prescribed patterns of deterioration, with $\delta = 0$ for the undamaged state and $\delta = 1$ the complete damaged state (Frangopol & Curley 1987, Biondini et al. 2008).

The mathematical expression of the time-variant damage index δ depends on the damage mechanism considered and the level of complexity of the analysis. Several mechanisms, including uniform corrosion in steel structures or crushing, cracking, abrasion and erosion in concrete structures, can be effectively represented at the member level by a progressive reduction of the effective resistant area of the member cross-section.

As an example, for steel members with hollow circular cross-sections having internal and external radius r_i and r_e, respectively, and damage along the external layer of uniform thickness Δr, the amount of damage can be specified by means of the following damage index:

$$\delta = \frac{\Delta \mathrm{r}}{r_e - r_i} \tag{2-1}$$

In this way, proper correlation laws may be introduced to define the variation of the geometrical properties of the cross-section, such as area $A = A(\delta)$ and inertia moment $I = I(\delta)$, as a function of the damage index δ.

Different patterns are needed when localized damage occurs. As an example, for corrosion of bars in reinforced concrete structures, denoting p the corrosion penetration depth the damage index δ can be defined as:

$$\delta = \frac{p}{D_0} \tag{2-2}$$

where D_0 is the diameter of the undamaged steel bar section. In turn, the percentage loss $\delta_s = \delta_s(\delta) = 1 - A_s/A_{s0}$ of steel resistant area A_s for a corroded bar depends on the corrosion mechanism. In carbonated concrete with limited chloride content, corrosion tends to develop uniformly on the steel bars along an external layer of thickness Δr, with $p = 2\Delta r$ and $\delta_s = \delta(2 - \delta)$. On the other hand, in presence of significant concentration of chlorides, corrosion tends to localize (pitting corrosion), and the relationship $\delta_s = \delta_s(\delta)$ depends on the shape of the pit (Stewart 2009).

A damage index $\delta = \delta(x)$ provides a comprehensive description of the spatial distribution of damage over the structure. However, due to its local nature, it is not useful for global evaluations of structural robustness. A synthetic global measure of damage Δ can be derived at the member level or over all members of the system level by a weighted average over the structural volume V as follows (Biondini 2004):

$$\Delta = \frac{\int_V \; w(\boldsymbol{x})\delta(\boldsymbol{x})\, dV}{\int_V \; w(\boldsymbol{x})\, dV} \tag{2-3}$$

where $\delta = \delta(\boldsymbol{x})$ is a damage index at point $\boldsymbol{x}$ and $w = w(\boldsymbol{x})$ is a suitable weight function. Arithmetic average with constant weights functions $w(x) = w_m = w_0$ can be adopted if there are no portions of material volume playing a specific role in the damage process. Contrary, different weights may be appropriate for different component materials in non-homogeneous systems, such as reinforced concrete structures (Biondini 2009).

This approach can be adopted for modelling and quantifying sudden damage or continuous damage by assuming suitable time-variant formulations of $\delta = \delta(t)$ and $\Delta = \Delta(t)$.

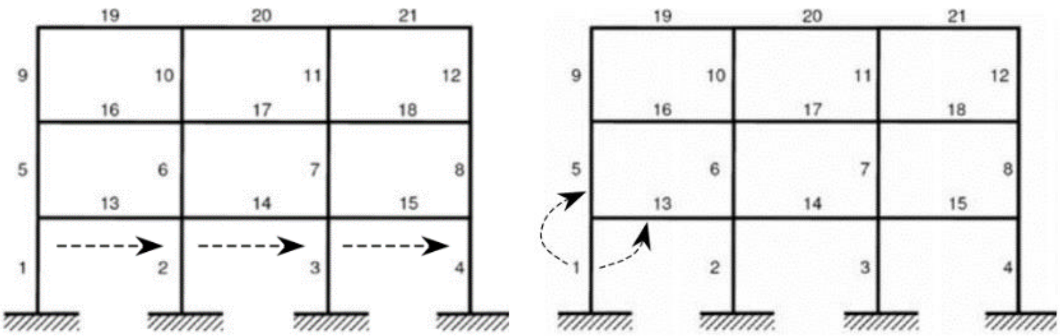
2.2.4 Damage propagation

After failure of one-member, other members may fail leading to a sequence of local failures that propagate throughout the overall system until its collapse is reached. The mechanism of damage propagation is generally related to the causes of the damage itself and also depends on the system configuration. To explain the concept, two alternative propagation mechanisms are considered here, defined as directionality-based and adjacency-based mechanisms (Biondini & Restelli 2008). Other concepts are given in Starossek (2017).

In the directionality-based mechanism, damage propagates along the direction normal to the axis of the first failed member. For example, with reference to the frame system shown in Figure 2-7.a, the damage of member 1 is followed in sequence by the damage of members 2, 3 and 4. The directionality-based mechanism is typical of damage induced by severe loadings, such as explosions or impacts, which generally tend to propagate along the direction of loading.

Figure 2-7. Damage propagation mechanisms: (a) Directionality-based; (b) Adjacency-based.

(a) (b)



Source: Biondini & Restelli, 2008

In the adjacency-based mechanism, damage propagates towards the members directly connected with other members already damaged. For example, with reference to the frame system shown in Figure 2-7.b, the damage of member 1 can be followed by the damage of members 5 and 13. The adjacency-based mechanism is typical of damage induced by aggressive agents, like chlorides, which generally tends to propagate through the structure based on diffusion processes.

Considering the local definition of damage and a certain propagation mechanism, an effective and complete damage scenario can be obtained by adopting a damage sensitive fault-tree analysis (Biondini & Restelli 2008). With this approach, all the possible damage paths, based on the topology of the structure and the propagation mechanism considered, can be represented by branched networks where the level of activation of each nodal connection is properly tuned to account the prescribed amount of local structural damage.

2.2.5 Modelling of hazards

To model natural and man-made hazards, in order to include them into a design process and build a structure able to sustain these kinds of action, a set of parameters, functions and models which describe the event are needed. In this process, there are several sources of aleatory and epistemic uncertainties and all of them need to be properly considered in a probabilistic framework for a realistic modelling.

Malicious attacks to structures, including terrorism, vandalism, or other intentional human actions, are highly unpredictable and unknown. Important indicators of the consequences associated to the occurrence of these events are the number of potential victims and the role in society of the structural facility. It is easy to understand and reasonable to think that a "symbolic" touristic building will have a larger level of exposure to terrorist attacks than a residential building. However, due to the uncertainties involved in this type of hazard, it is generally more efficient to adopt appropriate measures to protect the structure in case they actually do occur (for instance using safety barriers) and to limit the possible damage propagation over the structure.

Although EN1990 stipulates that structures should be designed, executed and maintained adequately by qualified and experienced people and controlled by appropriate quality management, the occurrence of errors and negligence during the design, execution and operation phases (which could lead to significant and disastrous consequences) can never be fully excluded and for such situations robustness provisions can limit the consequences resulting from such errors or negligence.

Potential hazards may be classified according to uncertainties (Vrouwenvelder 2010):

- Known and dealt with: associated risks are accepted with no additional measures or reduced to an acceptable level (may include natural hazards and ordinary loads);
- Known in principle, but unrecognized or ignored: generic design requirements for these actions (as human errors in design, construction and use) are generally provided;
- Unknown or unforeseeable: no specific information is available.

The latter category can be better specified as unforeseen or unforeseeable hazards at the time of design/assessment. The flutter mechanism of the Tacoma Narrow Bridge, for instance, may be considered as unforeseeable at during its design. Past failure events should serve as important lessons to be considered for future design, execution and operation phases.

2.3 Consequences of failure

The consideration of failure consequences is a fundamental task in the evaluation and assessment of robustness of structural systems. A robustness evaluation has to compare the amount of damage and the corresponding structural consequences.

2.3.1 Direct and indirect consequences

Consequences are typically divided into two categories. According to the distance in space and time from the triggering event, direct and indirect consequences may arise (Janssens et al. 2010).

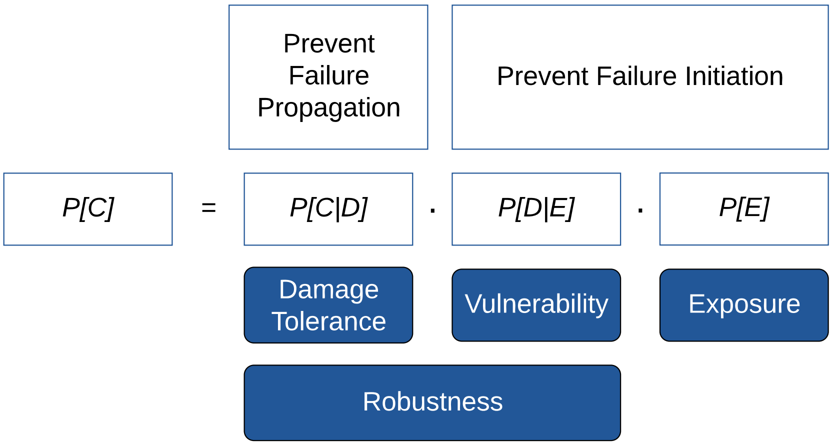
- Direct consequences: consequences associated to damage or failures of the constituents of a system. These are all the possible immediate consequences (not considering loss of system functionality) arising from direct effects of the hazards to the structural system. Depending on how the structure reacts and adapts to its new configuration after damage (e.g. trying to avoid progressive collapse with alternative load paths), indirect consequences may occur.

- Indirect consequences: consequences related to loss of functionality of the system caused by direct consequences. These outcomes are usually indicated as follow-up consequences with respect to initial direct consequences. Indirect consequences can also include environmental, psychological, political and reputational aspects and all the managerial and delay's costs.

In the context of consequences of failure, the concepts of structural robustness and vulnerability play a fundamental role. Structural robustness is the ability of a system to avoid a performance reduction disproportionately larger with respect to the corresponding damage.

In a risk-based context, structural robustness and vulnerability can be associated with collapse and damage probabilities as shown in Figure 2-8. In this Figure, *P[E]* denotes the probability of occurrence of an accidental event *E*, that affects the structure; *P[D|E]* represents the conditional probability of the initial/direct damage *D* due to the event *E*; *P[C|D]* is the conditional probability of a structural collapse *C*, due to damage *D*, and *P[C]* denotes the probability of disproportionate collapse as result of a hazard event.

Figure 2-8. Structural robustness and vulnerability



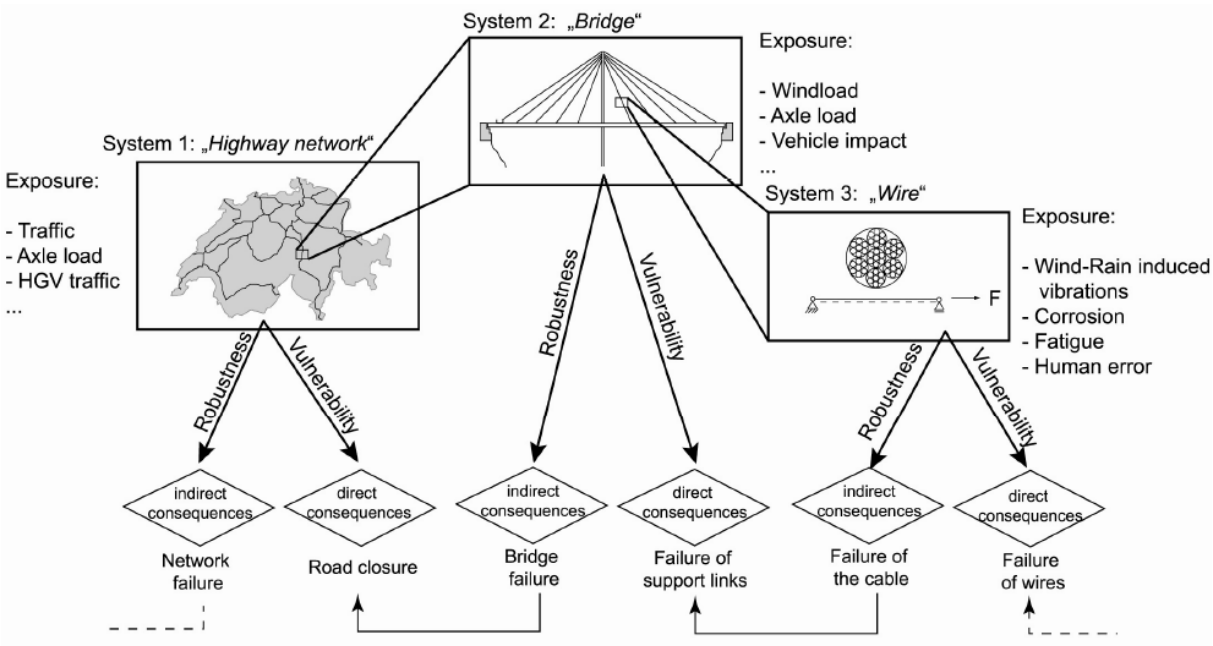
Source: Starossek & Haberland 2012 (redrafted)

Direct consequences may occur under hazardous events and the structural vulnerability plays the role of avoiding initial consequences on the system.

Moreover, direct and indirect consequences also depend on the system definition considered in the analysis. For instance, a bridge can be considered as the main system composed by different structural members that play the role of constituents of the system. In this case, after a hazardous event, several direct and indirect consequences may occur. On the other hand, the same bridge can be seen as a single component of an infrastructural network and the system will be associated to other direct and indirect consequences.

Therefore, consequences from structural failure are also function on the level of detail (i.e. the system definition) in the risk assessment. An example related to a roadway network is presented in Figure 2-9.

Figure 2-9. System definition for a roadway network



Source: Faber et al. 2007

It is worth noting that a different system definition is usually adopted for bridges relative to other types of structures (e.g. buildings), since the former are usually part of a system-of-systems.

2.3.2 Consequences in the Eurocodes

Structures are generally categorised in consequence classes, considering the structural type, occupancy and size. In "Eurocode 0 – Basis of Structural Design", three consequence classes are introduced based on the level of economic, social and environmental consequences. The different classes are identified to suggest appropriate design strategies in order to increase the damage-tolerance of the structural system.

The consequence classes are associated to different accidental design situations, as stated in "Eurocode 1 – Part 1-7: Accidental actions", see the section 3.1.2.2.3 of this report.

The consequence classes proposed by Eurocode are assumed to be dependent on the building type, number of storeys and the floor area. Moreover, Eurocode 1 recommends different strategies to provide an acceptable level of robustness to sustain localised failure without a disproportionate level of collapse, as outlined in 3.1.2.2.3.

Direct and indirect failure consequences depend on the representation of the analysed system and its boundaries. However, consequences may be classified not only based on their distance in time and space with respect to the triggering event. Generally, consequences associated to building and bridge failures are divided into four categories: human, economic, environmental and social consequences (Canisius 2011). Table 2-5 presents a list of possible direct and indirect failure consequences.

Obviously, the definitions of the considered system and its boundaries play an important role in the categorisation of consequences enabling a clear and rational distinction between direct and indirect consequences. For instance, management costs may become direct consequences as result of repairs on damaged portions of a bridge.

An important issue is the comparison of different types of consequences, particularly in the unit of measures used to quantify failure consequences (Faber & Stewart 2003). A monetary value may be associated for example to repair, rescue and clean-up costs. Also consequences belonging to the economical field (e.g. energy and management costs) can be easily quantified through the amount of money used and therefore accounted into a cost-benefit analysis. However, a monetary unit of measure may not be always a suitable choice, in particular when environmental, loss of functionality and reputation consequences and human loss are present. Expressing human injuries and fatalities in terms of monetary values may be problematic for ethical reasons.

Table 2-5. Classification of failure consequences

Type	Consequences
Human	Injuries Fatalities Psychological damage
Economic	Repair of initial damage Replacement/repair of contents Rescue costs Clean up costs Collateral damage to surroundings Loss of functionality Traffic delay/management costs
Environmental	CO2 Emissions Energy use Pollutant releases
Social	Loss of reputation Loss of public confidence

Consequences are also classified according to the item affected:

— Tangible consequences: injuries, fatalities, damage or failure to structural components, physical and functionality losses;

— Intangibles consequences: loss of opportunities and reputation, psychological effects, deferred production, losses in environmental attributes (e.g. pollution).

Consequences of failure can also be classified according to the probability of occurrence *P*:

— Systemic (P=1): construction costs, decommissioning costs, etc.;

— Occasional (P<1): consequences related to frequent accidents;

— Rare (P<<1): consequences of rare accidents.

2.3.3 Factors affecting the consequences of failure

Consequences of failure depend on many factors related to the hazards, properties and function of the structure, time-frame considered and surrounding environment (Chryssanthopoulos et al. 2001, Sørensen 2010). A list of the dominant factors affecting the consequences of failure is given below:

— Nature of the hazard

It is intuitive how the nature, magnitude and the duration of a certain hazard will affect its consequences: the larger the magnitude and duration the larger will be the associated consequences. Moreover, the type/nature of the hazard may lead to additional consequences and risks. A gas explosion will affect the mechanical properties of the system and moreover the nature of the hazard can also generate fumes and toxic pollutants increasing the environmental consequences;

— Properties of the structure

The building typology, its age, size and layout, the choice of materials adopted, the type and quality of construction will influence the consequences of failures. Moreover, vulnerability and robustness are strictly correlated to the properties of the structure;

— Use of the structure:

The average number of people daily present in a building and therefore, the amount of people exposed to the hazard, will affect the consequences of failures (such as injuries and fatalities);

— Location of the structure:

The position of a building exposed to an accidental hazard will influence the consequences of failures for different reasons, such as:

- Pollutants agents may have larger consequences in urban areas than buildings in rural places, potentially increasing the consequences for human health;
- The availability of emergency services may be better in urban areas than in the rural ones. However, the access in the latter areas is likely to be easier and less critical than in crowded places;
- Regarding the cost of repair or reconstruction, remote locations may have higher costs due to increased labour and material costs;
- For bridges, the type of road served by the bridge influences the traffic intensity and therefore the amount of people possibly exposed to a hazard and the associated traffic delay costs.

— Environmental and meteorological conditions:

During and after the exposure hazardous event, environmental and meteorological conditions may influence the consequences of the event itself. The most intuitive example is the quality and condition of air (for instance the wind speed and direction), both during and after the hazardous event, may increase or decrease the environmental consequences (leading, for example, to the dispersion of pollutant agents generated by the hazard event);

— Actual time of the hazardous event:

General structures have different occupancy levels during the day. Work places generally have high occupancy levels during working hours. On the other hand, residential buildings reach high number of people at night. The same behaviour is evident for bridges which have a certain peak of level of usage during the day. These time considerations inevitably lead to periods during the day which the potential human consequences (injuries and fatalities) are larger. Additional daily, weekly, monthly, seasonally variations may influence consequences of failure for certain hazards;

— Time-frame considered:

It is important to specify the time-period adopted for the evaluation. Generally, the time-frame should be chosen based on the consequences' duration. When all the consequences of a certain hazard are completed, the analysis can stop. However, for several hazardous events and its consequences (particularly for the intangible ones) it is difficult to identify a clear duration. Moreover, for bridge failures since their consequences will affect the entire transportation network, the requested period to reconstruct the bridge and offset all the consequences may be very large (years). For these reasons it could be necessary to fix a certain time-frame for the consequence analysis where its value is strictly related to the consequences to be considered.

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3 Overview of current standardization and design guidelines

3.1 Existing national standards, regulations and practices

3.1.1 Development and evolution of current design guidelines

After the collapse at Ronan Point (1968) the first code provisions for structural robustness were introduced in the United Kingdom with the implementation of the Fifth Amendment of the Building Regulations (Minister of Housing and Local Government, 1970). The basic idea of these first code provisions was that minimum levels of structural redundancy must be ensured to provide a minimal level of robustness by member survival. This concept of notional member removal evolved with time into different methods found in contemporary international codes. Current requirements in international design codes are hence an evolution of the guidelines introduced in the Fifth Amendment. Also following the collapse of Ronan Point, research at the National Bureau of Standard (United States) was initiated and a number of technical workshops on progressive collapse were held in the 1970s (Stevens et al., 2011). During this period, engineers expressed their concerns regarding the continued optimisation in structural design and the trend toward speeding execution which may lead to reduced robustness and continuity in the structural system and hence exposing structures to a greater risk of progressive collapse when unexpected loads occur. In 1975, the National Building Code of Canada (National Research Council of Canada, 1975) explicitly implemented provisions to prevent progressive collapse. As pointed out by Ellingwood et al. (2007) the uniqueness of the Canadian code is that it provides specific values on acceptable levels of the likelihood of an extreme event (10^{-4} per year or more) for which a structure should be designed. Later between 1975 and 1995 little progress was made in this research field.

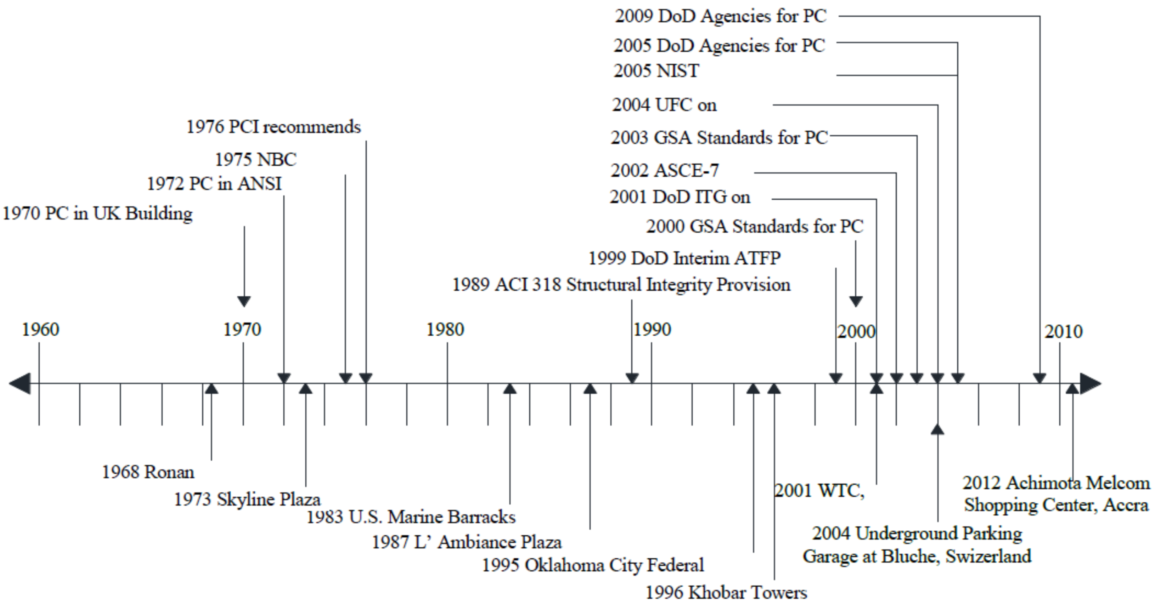
However, failures such as the collapse of the Alfred P. Murrah Federal Building in Oklahoma City and the collapse of the World Trade Centre Towers renewed the interest in progressive collapse design and a number of codes and guidelines (GSA, 2003, 2013) implemented provisions to minimise the likelihood of building collapse. After the bombing of the Alfred P. Murrah Federal Building in downtown Oklahoma City on April 19, 1995, an Interagency Security Committee (ISC) was established which was responsible for developing long-term construction standards for non-military facilities that require blast resistance and other specialised security needs.

After several studies and investigations of different progressive collapse design approaches, the 'alternative path' method was implemented by the General Services Administration (GSA, 2000). This approach requires that additional robustness is provided in a structural frame through the addition of reserve capacity and ductility designed to sustain gravity loads after the loss of a critical load-bearing element. Later in June 1996, the Khobar Towers complex in Saudi Arabia, housing U.S. personnel from the Department of Defense (DoD), was bombed. As a result, the U.S. Congress directed the DoD to develop antiterrorism standards for the construction of military facilities. These standards were developed to reduce the vulnerability of structures on military installations to terrorist attacks and to improve the safety of occupants. In 2001, DoD issued interim design guidance specifically addressing progressive collapse to clarify interim antiterrorism requirements (DoD, 2001). This guidance adopted an alternative load path method to reduce the risk of progressive collapse that was similar to the GSA criteria. DoD updated its antiterrorism standards for buildings in the 2002 publication of Unified Facilities Criteria (UFC) 4-010-01: Minimum Antiterrorism Standards for Buildings (DoD, 2002) that included the requirement to consider progressive collapse. In 2005, DoD developed UFC 4-023-03: Design of Buildings to Resist Progressive Collapse (DoD, 2005) to provide more specific, enforceable guidelines to support compliance with the UFC 4-010-01 requirement. Later the UFC 4-023-03 guideline was updated in 2009 (DoD, 2009).

In other parts of the world, explicit considerations on structural robustness were not included in codes until the beginning of the 21st century. In China, after the triggering event of the 2008 Wenchuan earthquake which resulted in a large number of collapsed buildings, the Code for Anti-Collapse Design of Building Structures was released in 2014 and approved by the Ministry of Housing and Urban-Rural Development of China (China Association for Engineering Construction Standardization (CECS), 2014; Li et al., 2014). This code contains design methods for steel and concrete buildings which align well with other international codes. In Australia, starting in 2016 the current code has introduced general and brief requirements based on the notional member removal and key element design for all building classes (Australian Building Codes Board (ABCB), 2016).

In 2004, the European standard EN 1991-1-7 (CEN, 2006) was completed and received a positive vote by the member states. The code describes the principles and application rules for the assessment of accidental actions on buildings and bridges. The leading design principle of this code is that local damage is acceptable, provided that it will not endanger the structure and that the overall load bearing capacity is maintained during an appropriate length of time to allow necessary emergency measures to be taken (Gulvanessian and Vrouwenvelder, 2006). An overview of well-known progressive collapse events and consecutive developments of design guidelines or standards in time is illustrated in Figure 3-1.

Figure 3-1. Timeline of the main progressive collapse events and the developments of design provisions.



Source: Qian et al., 2016

The close relationship between design code updates and the lessons learnt from damaging events is indicative of the dynamic character of design codes. Design codes are established to be progressively updated in order to enclose new scientific knowledge, state-of-art best practices in design, or lessons learned through empirical evidence. In the particular case of the European standard, the European Commission recently issued a requirement to amend and update the current version of the Eurocode (EC, 2012), being one of the aims of the update the “strengthening of the requirements for robustness” and the “extension of existing rules for robustness”. It is in-line with this requirement that the following Section discusses the existing standardisation, practice, and theory of the robustness provisions.

3.1.2 Existing European provisions for robustness

The following Section provides a comprehensive overview of the state-of-art in the European provisions for robustness up until 2019. Provisions and guidelines included in pre-standards are not included in this overview.

3.1.2.1 Eurocode 0 (2002) – General statements

As a basic requirement for structural robustness, Eurocode EN 1990 states the following the clause in section 2.1:

(4)P - A structure shall be designed and executed in such a way that it will not be damaged by events such as :

- *explosion,*
- *impact, and*
- *the consequences of human errors,*

to an extent ***disproportionate*** *to the original cause.*

NOTE 1 The events to be taken into account are those agreed for an individual project with the client and the relevant authority.

NOTE 2 Further information is given in EN 1991-1-7.

(5)P - Potential damage shall be avoided or limited by appropriate choice of one or more of the following:

- *avoiding, eliminating or reducing the hazards to which the structure can be subjected;*
- *selecting a structural form which has low sensitivity to the hazards considered;*
- *selecting a structural form and design that can survive adequately the accidental removal of an individual member or a limited part of the structure, or the occurrence of acceptable localised damage;*
- *avoiding as far as possible structural systems that can collapse without warning;*
- *tying the structural members together.*

(6) The basic requirements should be met:

- *by the choice of suitable materials,*
- *by appropriate design and detailing, and*
- *by specifying control procedures for design, production, execution, and use relevant to the particular project.*

(7) The provisions of [..] should be interpreted on the basis that due skill and care appropriate to the circumstances is exercised in the design, based on such knowledge and good practice as is generally available at the time that the design of the structure is carried out.

With the basic requirement in clause (4)P the Eurocodes indicate some exposures to consider during the structural robustness assessment and implicitly allow that some local damage/failure may be accepted. Moreover the emphasis is laid on a design and execution which avoid disproportionate consequences and is based on due skill, knowledge and good practice. Clause (5)P provides different strategies which can be applied in the design against disproportionate collapse.

The use of the term disproportionate in clause (4)P makes the design concept clear. However, the term is subjected to the individual interpretation of the designer and does not allow quantifications of what is an acceptable robustness level. However, as indicated by the text of EN 1990, there is a need to interact with the client or relevant authority of a project to discuss the hazardous events which need be considered. Hence the hazardous events which can be considered are not only limited to explosions, impact or human errors. Note that the term '*robustness*' as such is not explicitly defined in EN 1990.

3.1.2.2 Eurocode 1, part 1-7 (2006)

3.1.2.2.1 Scope

In addition to Eurocode EN 1990, EN 1991-1-7 gives the principles and application rules for the assessment of identifiable and unidentifiable accidental actions on buildings and bridges and gives a definition of robustness. Further, as a main design principle, it is stated that local damage is acceptable as long as the structural stability and load-bearing capacity of the building is not endangered for an appropriate period of time to allow necessary emergency procedures to take place, e.g. the safe evacuation and rescue of personnel from the building and its surroundings. Longer periods of survival may be required for buildings used for handling hazardous materials, provision of essential services, or for national security reasons.

3.1.2.2.2 Strategies

Compared to EN 1990, EN 1991-1-7 adds exposures from unidentifiable causes to the list specified in EN 1990. In this regard, strategies applicable for identified accidental actions and for unidentified accidental actions are outlined (Figure 3-2). In EN 1990, an accidental action is defined as:

'*An action, usually of short duration but of significant magnitude that is unlikely to occur on a given structure during the design working life." A note to the definition states: "An accidental action can be expected in many cases to cause severe consequences unless appropriate measures are taken.*'

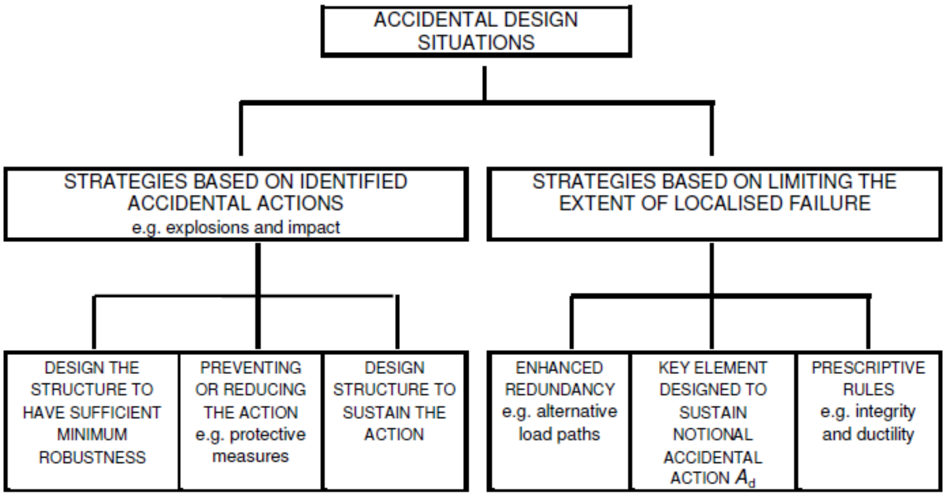
Typical examples of identified accidental actions are fire, explosions, earthquakes, impact, floods, avalanches, landslides and so on. Note that in the Eurocode system, fire and earthquake are dealt with in specific parts of the Eurocodes, supplemented by country-dependent specifications is national annexes, and treated by an alternative limit state formulation. Regarding impact and explosion, some guidelines are given in EN 1991-1-7. Furthermore, for concrete structures in accidental design conditions, EN 1992-1-1 provides some additional guidance.

Once an accidental action is defined, this identified action can be dealt with by classical safety formats (i.e. considering appropriate partial factors and load combinations) and advanced structural analysis. However, it is important to indicate that, for the verification of accidental design situations, no alternative target reliability levels are specified. Next to the strategy of designing the structure to sustain an identified action, EN 1991-1-7 also gives the possibility to prevent or reduce the action and to design the structure to have sufficient minimum robustness. To implement the latter strategy, the following methods can be adopted:

1. Designing certain components of the structure upon which the stability of the structure depends as key elements to increase the likelihood of the structure's survival following an accidental event;
2. Designing structural members, and selecting materials, to have sufficient ductility capable of absorbing significant deformation energy without rupture;
3. Incorporating sufficient redundancy in the structure to facilitate the transfer of actions to alternative load paths following an accidental event.

A note is given with a reference to informative annexes A and C of EN 1991-1-7 and to EN 1992-1999 to find guidance on what is considered as 'sufficient ductility'. However, limited information regarding this topic is available in the so-referred Eurocodes.

Figure 3-2. Distinction strategies for identified and unidentified accidental actions in EN 1991-1-7



Source: CEN, 2006.

3.1.2.2.3 Annex A: Design for consequences of localised failure in buildings

Related to the basic principles for damage mitigation mentioned in Eurocode EN 1990, for unidentified accidental actions, more general robustness requirements (for example prescriptive tying forces) are introduced. As is the case for general design, the objective of these requirements is to reduce the risks to an economically acceptable cost. Risk may be expressed in terms of the probability and the consequences of undesired events. Hence, risk-reducing measures consist of probability reducing measures and consequence reducing measures (Gulvanessian and Vrouwenvelder, 2006). However due to the occurrence of extreme and unexpected events, no design can be made risk free. As a result, localised failure is acceptable to a certain extent as long as the global structural stability is not endangered. To limit the extent of localized failure due to unidentified accidental actions, three strategies are given in EN 1991-1-7 (CEN, 2006) (right hand side of Figure 3-2):

- Designing the structure so that in the event of a localised failure (e.g. failure of a single member) the stability of the whole structure or of a significant part of it would not be endangered;
- Designing key elements, on which the stability of the structure depends, to sustain the effects of a notional accidental action A_d;
- Applying prescriptive design/detailing rules that provide acceptable robustness for the structure (e.g. three-dimensional tying for additional integrity, or a minimum level of ductility of structural members subject to impact).

The design for accidental situations is of particular importance where a collapse may result in consequences in terms of injury to human beings, or may have significant economic, social or environmental consequences. A convenient approach to decide which structures should be designed for accidental actions, is to classify the structures or structural members in categories according to the consequences of an accident. As such, EN 1991-1-7 (CEN , 2006) makes a distinction between the strategies to be applied for unidentified accidental design situations, on the basis of the Consequence Classes defined in EN 1990 (CEN, 2015). EN 1990 classifies structures in three categories based on the consequences of failure, see Table 3-1.

Table 3-1. Consequence Classes according to EN 1990.

Consequence Class (CC)	Description	Examples of buildings and civil engineering works
CC3	High consequence for loss of human life, or economic, social or environmental consequences **very great**	Grandstands, public buildings where consequences of failure are high (e.g. a concert hall)
CC2	Medium consequence for loss of human life, economic, social or environmental consequences **considerable**	Residential and office buildings, public buildings where consequences of failure are medium (e.g. an office building)
CC1	Low consequence for loss of human life, and economic, social or environmental consequences **small or negligible**	Agricultural buildings where people do not normally enter (e.g. storage buildings), greenhouses

Source: CEN, 2015

In annex A of EN 1991-1-7 (CEN, 2006), the table defining the Consequence Classes (CCs) in EN 1990 (CEN, 2015) is further extended. Furthermore, CC2 is subdivided into two subclasses: CC2a (buildings up to 4 storeys) and CC2b (buildings up to 15 storeys). However, this table is not exhaustive and can be adjusted by national annexes. Subsequently, in EN 1991-1-7, the strategy to be adopted for accidental design situations is based on the consequence classes and can be summarized as follows:

- **CC1:** Provided a building has been designed and erected in accordance with the rules given in EN 1990 to EN 1999 for satisfying stability in normal use, no further specific consideration is necessary with regard to accidental actions from unidentified causes.
- **CC2a:** In addition to the recommended strategies for Consequences Class 1, the provision of effective horizontal ties, or effective anchorage of suspended floors to walls should be provided;
- **CC2b:** In addition to the recommended strategies for Consequences Class 1, the provision of:
    - horizontal ties should be provided together with vertical ties in all supporting columns and walls.

      or alternatively,
    - the building should be checked to ensure that upon the notional removal of each supporting column and each beam supporting a column, or any nominal section of load bearing wall, the building remains stable and that any local damage does not exceed a certain limit. Where the notional removal of such columns and sections of walls would result in a damage extent in excess of the agreed limit, or other such limit specified, then such elements should be designed as a **'key element'**;
- **CC3:** A systematic risk assessment of the building should be undertaken taking into account both foreseeable and unforeseeable hazards. An examination of the specific case should be carried out to determine the level of reliability and the depth of the structural analyses required. This may require the use of refined methods such as dynamic analyses and non-linear models. Guidance on the preparation of the risk analysis is given in Annex B of EN 1991-1-7 (CEN, 2006).

3.1.2.2.4 Prescriptive tie rules

Regarding the design of the proposed ties in the strategies presented above, two types of ties are indicated in EN 1991-1-7 (CEN, 2006): horizontal ties and vertical ties. Next a distinction is made between framed structures and load bearing wall structures to determine the design tie force and placing of the ties. In Table 3-2, the different design tie forces given by EN 1991-1-7 are summarised. It is important to note that horizontal floor tying offers a potential resistance mechanism via tensile catenary/membrane action in the event of loss of a column/vertical load bearing member. In this respect, it should be emphasised that the currently prescribed tying forces should be improved for typically expected ductility levels, and that the current rules do not incorporate any ductility considerations. Further information about the different ties specified in EN 1991-1-7 is presented below.

Table 3-2. Design tie forces according to EN 1991-1-7 (CEN, 2006).

Considered tie	**Design value of tensile force**	
	Framed structures	Load bearing wall structures (CC2b)
Peripheral ties	$T_p = 0.4(g_k + \psi\, q_k) \cdot s \cdot L$ $\geq 75kN$	$T_p = F_t$
Internal ties	$T_i = 0.8(g_k + \psi\, q_k) \cdot s \cdot L$ $\geq 75kN$	$T_i = \max\left(\begin{matrix} F_t \\ \frac{F_t(g_k + \psi q_k)}{7.5}\frac{z}{5} \end{matrix}\right)$
Vertical ties	The vertical tie should be capable of carrying an accidental design tensile force equal to the largest design vertical permanent and variable load reaction applied to the column from any one storey.	$T = \max\left(\begin{matrix} \frac{34A}{8000}\left(\frac{H}{t}\right)^2 \\ 100\frac{kN}{m\ of\ wall} \end{matrix}\right)$

s is the spacing of the ties;
L is the span length of the tie;
ψ is the relevant factor in the expression for combination of action effects for the accidental design situation (i.e. ψ_1 or ψ_2 in accordance with expression (6.11b) of EN 1990);
F_t is 60 kN/m or $20 + 4n_s$ kN/m, whichever is less;
n_s is the number of storeys;
z is the minimum of 5 times the clear storey height H or the greatest distance in metres in the direction of the tie, between the centres of the columns or other vertical load bearing members;
A is the cross-section area in mm^2 of the wall measured on plan;
H is the clear storey height of the wall;
t is the thickness of the wall.

Horizontal ties

Framed structures (CC2a and CC2b buildings)

- Horizontal ties should be provided around the perimeter of each floor and roof level and internally in two right angle directions to tie the column and wall elements securely to the structure of the building. The ties should be continuous and be arranged as closely as practicable to the edges of floors and lines of columns and walls. At least 30% of the ties should be located within the close vicinity of the grid lines of the columns and the walls.
- Horizontal ties may comprise rolled steel sections, steel bar reinforcement in concrete slabs, or steel mesh reinforcement and profiled steel sheeting in composite

steel/concrete floors (if directly connected to the steel beams with shear connectors). The ties may consist of a combination of the above types.

- Members used for sustaining actions other than accidental actions may be utilized for the above ties.
- The design strengths of the perimeter and internal ties are given in Table 3-2.

Load bearing wall structures

1. For CC2a buildings:

   Appropriate robustness should be provided by adopting a cellular form of construction designed to facilitate interaction of all components including an appropriate means of anchoring the floor to the walls.

2. For CC2b buildings:

   Continuous horizontal ties should be provided in the floors. These should be internal ties distributed throughout the floors in both orthogonal directions and peripheral ties extending around the perimeter of the floor slabs within a 1.2 m width of the slab. The design strength of the ties can be found in Table 3-2.

Vertical ties

Each column and wall should be tied continuously from the foundation level to the roof level. The design strength for the ties in case of framed and load bearing wall structures can be found in Table 3-2.

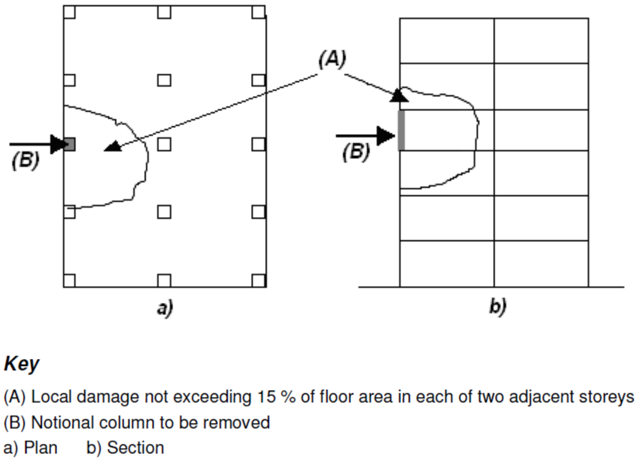
3.1.2.2.5 Notional removal of load bearing elements

This strategy is proposed for CC2b buildings. It consists in checking that upon the notional removal of each load bearing element (one at a time in each storey of the building), the building remains stable and that any local damage does not exceed a certain limit. The application of this method results in checking the ability of a structure to activate alternative load paths in case of loss of a load bearing element. Limited information is provided in EN 1991-1-7 (CEN, 2006) on how to apply this strategy.

3.1.2.2.6 Admissible damage and key element design

In case the second strategy is chosen for a CC2b building, i.e. notional removal of load bearing elements, the stability and damage of the structure should be assessed. Where the notional removal of a load bearing element would result in an extent of damage in excess of the recommended limit of admissible local failure, then such an element should be designed as a 'key element'. This key element is supposed to be designed to withstand a recommended accidental load A_d of 34 kN/m^2 which should be applied in horizontal and vertical directions (in one direction at a time). This recommended value was based on the gas explosion at the collapse of Ronan Point for which the pressure was estimated to be between 14 kPa and 83 kPa (Ellingwood et al., 2007). The recommended limit of admissible local failure in case the notional load bearing element removal is applied, is 15% of the floor area or 100 m^2, whichever is smaller, in each of two adjacent storeys (Figure 3-3). Unfortunately the standard does not provide detailed guidance on how to quantify the extent of the damage. Note that the values above are recommended values and the correct values to be used in a member state can be found in their respective National Annex. Further note that in Figure 3.3 from EN1991-1-7, the term 'collapsed area' instead of 'damage' could be deemed more appropriate.

Figure 3-3. Recommended limit of admissible damage according to EN 1991-1-7.



Source: CEN, 2006

3.1.2.2.7 Annex B: Information on risk assessment

For buildings in consequence class 3, EN 1991-1-7 recommends a formal quantitative risk analysis (CEN, 2006). To perform and execute this risk analysis, guidance can be found in Annex B. The recommended steps for this assessment are:

1. Definition of scope and limitations;
2. Qualitative risk analysis (inventory and description);
3. Quantitative risk analysis (modelling and calculations);
4. Risk evaluation and mitigation measures;
5. Risk communication.

As indicated by Gulvanessian and Vrouwenvelder (2006), the depth and complexity of the risk assessment should be dictated by the problem at hand. Risk analysis in a rigorous form including extensive statistical analyses is used only in special cases. In many cases, a qualitative analysis of risks and envisaged countermeasures should be sufficient.

In case of a formal quantitative risk analysis according to annex B of EN 1991-1-7 (CEN, 2006), following formula is given to evaluate the total risk:

$$R = \sum_{i=1}^{N_H} P[H_i] \sum_{j}^{N_D} \sum_{k=1}^{N_S} P\left[D_j \middle| H_i\right] \cdot P\left[S_k \middle| D_j\right] \cdot C(S_k) \tag{3-1}$$

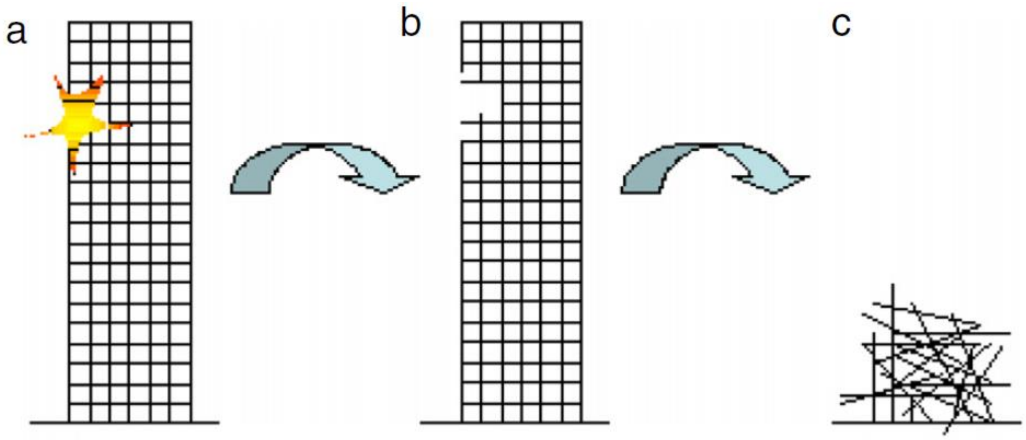
In this equation, it is assumed that the structure is subjected to N_H different hazards, that the hazards may damage the structure in N_D different ways (can be dependent on the considered hazards) and that the performance of the damaged structure can be discretised into N_S adverse states S_k with corresponding consequences $C(S_k)$. $P[H_i]$ is the probability of occurrence (within a reference time interval) of the i^{th} hazard, $P\left[D_j \middle| H_i\right]$ is the conditional probability of the j^{th} damage state of the structure given the i^{th} hazard and $P\left[S_k \middle| D_j\right]$ is the conditional probability of the k^{th} adverse overall structural performance S_k given the i^{th} damage state. Based on this formula, three analysis steps can be distinguished (Figure 3-4):

1. Identification and modelling of relevant accidental hazards. Assessment of the probability of occurrence of different hazards with different intensities;
2. Assessment of damage states to the structure from different hazards. Assessment of the probability of different damage states and corresponding consequences for given hazards;

3. Assessment of the performance of the damaged structure. Assessment of the probability of inadequate performance(s) of the damaged structure together with the corresponding consequence(s).

Figure 3-4. Illustration of the steps in a risk analysis for structures subject to accidental actions according to EN 1991-1-7; (a) hazard, (b) damage, (c) collapse.

Step 1	**Step 2**	**Step 3**
Identification and modeling of relevant accidental hazards	Assessment of damage states to structure from different hazards	Assessment of the performance of the damaged structure



Assessment of the probability of occurrence of different hazards with different intensities	Assessment of the probability of different states of damage and corresponding consequences for given hazards	Assessment of the probability of inadequate performance(s) of the damaged structure together with the corresponding consequence(s)

Source: adapted from CEN, 2006

Next the evaluated risk should be compared to some risk acceptance criteria which are usually left to the member states. In annex B, limited guidance is also given for these criteria. Basically the ALARP is mentioned, which stands for 'as low as reasonably practicable'. In other words, apart from a lower bound for the individual and socially accepted risk levels, an economical optimization is recommended. Based on Equation (3-1), the following strategies are identified to mitigate the risk:

- Reducing the probability of the hazard occurrence (i.e. reducing $P[H_i]$);
- Reducing the probability of significant damage given the hazard (i.e. reducing $P[D_j|H_i]$). This is related to the vulnerability of the structure;
- Reducing the probability of adverse structural performance (i.e. reducing $P[S_k|D_j]$). This can be obtained by providing sufficient redundancy and consequently is related to the robustness of the structure.

3.1.2.3 Material specific rules

Material specific rules are only reflected in EN1992-1-1 for concrete structures. Such specific rules are not reported in the other material oriented Eurocodes.

3.1.2.3.1 EN1992-1-1

For reinforced concrete structures, the Eurocodes give some additional guidelines to design for robustness in Eurocode EN 1992-1-1 (CEN, 2005). In section 9.10 of this Eurocode part, the following is mentioned:

'Structures which are not designed to withstand accidental actions shall have a suitable tying system, to prevent progressive collapse by providing alternative load paths after local damage.'

Note that in this clause, no distinction is made between identified and unidentified accidental actions as is done in EN 1991-1-7 (CEN, 2006). Moreover, the design approach of EN 1991-1-7 (CEN, 2006) based on the categorisation of a structures into Consequence Classes is not mentioned in this section. In section 9.10 of EN 1992-1-1 (CEN, 2005), simple design rules are given to design a tying system incorporating the following ties:

- Peripheral ties at each floor and roof level within 1.2 m from the edge;
- Internal ties at each floor and roof level in two orthogonal directions;
- Horizontal ties for edge columns and walls;
- Vertical ties where required, particularly in panel buildings of 5 storeys or more, continuous from the lowest to the highest level of the structure.

Further it is mentioned that the ties are intended as a minimum reinforcement and not as an additional reinforcement to that required by regular structural analysis. The design values for the respective ties according to EN 1992-1-1 (CEN, 2005) are given in Table 3-3. It should be noted that structural detailing rules, such as minimum reinforcement ratio and minimum anchorage length can also contribute to achieving robustness.

Table 3-3. Design values of ties according to EN 1992-1-1 (CEN, 2005).

Considered tie	Design value of tensile force
Peripheral tie$^{(1)}$	$F_{tie,per} = l \cdot q_1 \leq q_2$
Internal tie$^{(2)}$	$F_{tie,int} = 20\ kN/m$ (floors with screeds) or $F_{tie} = \frac{l_1+l_2}{2}\ q_3 \leq q_4$ (floors without screeds)
Horizontal ties to columns or walls	$f_{tie,fac} = 20\ kN/m$ (façade walls) $F_{tie,col} = 150\ kN$
Vertical tie$^{(3)}$	The vertical ties should be capable of carrying the load in the accidental design situation, acting on the floor above the column/wall accidentally lost.

(1) l is the length of the end span in meters; the recommended value for q_1 is 10 kN/m and for q_2 is 70 kN;
(2) l_1 and l_2 are span lengths in meters on either side of the beam line; the recommended value for q_3 is 20 kN/m and for q_4 is 70 kN;
(3) Other solutions e.g. based on the diaphragm action of remaining wall elements and/or on membrane action in floors, may be used if equilibrium and sufficient deformation capacity can be verified.

Source: CEN, 2005

Note that the values of Table 3-3 are recommend values; the actual values to be used in a member state can be found in their respective National Annex.

3.1.2.4 National Standards

In general, requirements on robustness in national standards are country-specific and it is difficult to provide a general overview. Due to the Ronan Point accident, the British standardisation has the longest tradition in development of the requirements on robustness (COST, 2011).

3.1.2.4.1 British Standards

Since the Ronan Point collapse, the British Standards have taken the lead in stating explicit design provisions against progressive collapse. The British Standards started with the Building (Fifth Amendment) Regulations in 1970 (Minister of Housing and Local Government, 1970) which were applied to all buildings having five or more storeys (including basement storeys). The initial guidelines emphasised general tying of various structural elements of a building together, to provide structural integrity, continuity and redundancy. With the application of ties and ensuring continuity between structural elements the resistance of wall panels subjected to pressure in the event of an explosion is enhanced and the ability to bridge over a lost element is also improved. In addition, load-bearing elements which are vital for the general building stability should be designed as key elements, able to withstand an accidental load, i.e. a pressure of 34 kPa.

Since then, the guidelines in the UK have been evolved to the Approved Document A of the UK Building Regulations, which is generally adopted in the Eurocodes (Eurocode 1, BS EN 1991-1-7:2006). The design requirements are described in Annex A. While Annex A is informative, the UK National Annex effectively makes the annex normative, stating that the 'guidance ... should be used in the absence of specific requirements in BS EN 1992-1-1 to BS EN 1996-1-1 and BS EN 1999-1-1 and their National Annexes.'

3.1.2.4.2 Czech Standards

In the Czech standards, directives are provided for houses constructed of panels. For these constructions, it is required to verify the overall spatial stiffness. The design of the tie reinforcement in the horizontal and vertical joints prescribes a value of 15 kN/m over the width or length of a panel house. Furthermore, reinforcement is required in each joint of vertical and horizontal members by additional or latent ties.

For masonry structures, reinforcing bars at each floor level are required. For multi-storey buildings, recommended construction rules are provided and the total height of the building is restricted (COST, 2011).

3.1.2.4.3 Danish code DS 409

Robustness is introduced in the Danish Code of Practice for Safety of Structures as a general requirement to all structures in order to reduce the sensitivity of the structure with respect to unintentional loads and defects. The Danish code provisions are based on a probabilistic approach and require a step-by-step procedure for all structures where consequences of failure are serious (COST, 2011). Similar to the Eurocodes, the requirements for robustness of structures are related to the consequences of failure of the structure. Only for structures in the highest consequence class CC3, robustness shall be documented (Sorensen-J.D., 2008):

— by demonstrating that those parts of the structure essential for the safety only have little sensitivity with respect to unintentional loads and defects, or;

— by demonstrating a load case with 'removal of a limited part of the structure' or;

— by demonstrating sufficient safety of key elements.

3.1.2.4.1 Italian code and guidelines

The Italian building code (MIT, 2018) is fully consistent with Eurocodes' provisions, recommending that constructions should have an adequate level of structural robustness. This property is therein defined as the ability of the structure to avoid disproportionate damage with respect to the magnitude of hazardous events such as explosions and impact. Depending on the expected occupancy of the construction and consequences of collapse, the Italian building code expects that an adequate level of structural robustness may be achieved through single design strategies or a combination of them. In this respect, a proper conceptual design of the structure, including the selection of a structural shape and typology with low sensitivity to exceptional loads or local damage, is suggested. The possible use of passive or active control systems is recommended as additional measure of risk mitigation.

The Italian code establishes that exceptional loads are those produced by events such as fire, explosion and impact. To achieve an adequate level of robustness, the designer should consider either the hazardous scenarios and exceptional actions that mostly influence the design of the structure or scenarios prescribed by the client, including scenarios associated with local damage and structural deterioration. According to European provisions, structural robustness is assessed under an exceptional load combination. In the case of explosions and impact, the Italian code does not provide analysis methods for assessing the propagation of local damage throughout the structure. It is also noted that the Italian code allows the designer to assume partial safety factors equal to unity when defining design values of material properties for robustness verifications. This provision is amended only in the case of masonry structures, where the partial safety factor of masonry may be set equal to one-half of those provided for gravity load conditions, but this is likely to be seen together with the higher partial factor adopted in the Italian code, resulting in a partial factor higher than 1.

In case of timber structures, the Italian code states that robustness requirements may be met through proper design choices and construction detailing, which should take into account at least the following criteria:

— protection of the structure and its components against humidity;

— use of ductile connections;

— use of composite members with ductile global behaviour;

— limitation of timber portions subjected to tensile stresses perpendicular to fibres.

The Italian code provisions may be effectively used in combination with CNR-DT 214/2018 guidelines issued by the National Research Council of Italy (CNR, 2018). Those guidelines provide detailed formulations for modelling of a variety of exceptional loads, including actions caused by tsunamis, landslides, floods, eruptions, windstorms, detonations, terrorist attacks, sabotage, and human errors in design and construction. A chapter of CNR-DT 214/2018 guidelines deals with the risk of progressive collapse, whereas other chapters provide recommendations for risk mitigation, conceptual design criteria, design methods and detailing rules for several types of structural systems.

3.1.3 Provisions for robustness outside Europe

3.1.3.1 United States

In the United States, the Department of Defense (DoD) researched and then adapted in their Unified Facilities Criteria (UFC) (DoD, 2009) what they considered the best available approaches from different organizations, specifically the British Standards and Eurocodes (CEN, 2006). UFC 4-023-03 employs a combination of direct and indirect design methods and includes specific guidelines for reinforced concrete, structural steel, masonry, cold-formed steel and timber constructions. In the first version of UFC 4-023-03, the specified level of progressive collapse design requirement was based on the Level of Protection (LOP) prescribed for the considered structure. LOP is a military concept and is connected to the asset value of the structure, which in turn depends on the occupancy of the building, its mission and other factors. Consequently, the consequences of a low-probability collapse event are reflected by this concept. In the second version of UFC 4-023-03, the LOP concept was replaced by Occupancy Categories to determine the required level of progressive collapse design to make the document more general and applicable for civilian consensus-based organisations (Stevens et al., 2011). Other improvements of the document were based on further research performed after the 9/11 collapse to make the design requirement more economical and practical.

In the following section, the main concepts of the UFC 4-023-03 guidelines are discussed. For existing and new constructions, the level of progressive collapse design for a structure is correlated to the Occupancy Category (OC) (Table 3-4.). The progressive collapse design requirements employ three design/analysis approaches: Tie Forces (TF), Alternative Path (AP), and Enhanced Local Resistance (ELR). In the following, each design procedure is summarized.

Tie Forces method (TF)

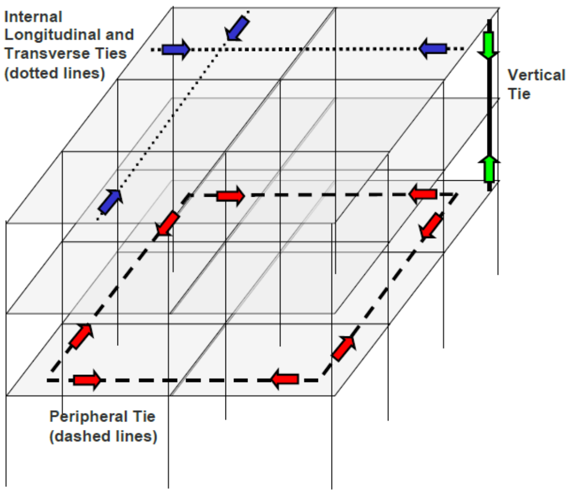
The Tie Forces (TF) procedure prescribes a tensile force strength of the floor or roof system, to allow the transfer of load from the damaged portion of the structure to the undamaged portion. Hence, with this approach, the building is mechanically tied together enhancing continuity and development of alternative load paths. The tie forces can be provided by the existing structural elements that have been designed using conventional design methods to carry the standard loads imposed upon the structure. There are three horizontal ties that must be provided: longitudinal, transverse and peripheral (Figure 3-5). Vertical ties are required in columns and load bearing walls. Structural members such as beams, girders and spandrels, are allowed only to carry the tie forces in case their connections can be shown capable of carrying the required tie force magnitudes while undergoing rotations of 0.20 radians (11.3 degrees).

Table 3-4. Occupancy Categories and Design Requirements.

Occupancy Category	Design Requirement
I	No specific requirements.
II	Option 1: Tie Forces (TF) for the entire structure and Enhanced Local Resistance (ELR) for the corner and penultimate columns or walls at the first storey. **OR** Option 2: Alternative Path (AP) for specified column and wall removal locations.
III	Alternative Path for specified column and wall removal locations and Enhanced Local Resistance (ELR) for all perimeter first storey columns or walls.
IV	Tie Forces and Alternative Path for specified column and wall removal locations and Enhanced Local Resistance for all perimeter first storey columns or walls.

Source: DoD, 2009

Figure 3-5. Tie forces in a frame structure.



Source: DoD, 2009

If all of the structural elements and connections can be shown to provide the required tie strength, then the tie force requirement has been met. If the vertical design tie strength of any structural element or connection is less than the vertical required tie strength, the designer must either: 1) revise the design to meet the tie force requirements or 2) use the Alternative Path method to prove that the structure is capable of bridging over this deficient element. The latter requirement does not avoid the partial collapse of the structure below the column.

To design the ties, a Load and Resistance Factor Design (LRFD) approach is proposed, i.e. the design tie strength is taken as the product of the strength reduction factor ϕ and the nominal tie strength R_n calculated in accordance with the requirements and assumptions of applicable material specific codes:

$$\phi R_n \geq R_u \tag{3-2}$$

where $R_u = \sum \gamma_i Q_i$ is the required tie strength, taking into account the respective load effects Q_i and load factor γ_i. For steel reinforcement in tension in reinforced concrete elements, the strength reduction factor ϕ shall be taken as 0.75. The floor load w_F in kN/m^2 to calculate the tie forces is computed as:

$$w_F = 1.2D + 0.5L \tag{3-3}$$

where D and L are respectively the dead and live load in kN/m^2. For situations with non-uniform load distribution or large concentrated loads over the floor area, additional guidance is given.

The design tie forces for the different types of ties are summarized in Table 3-5. . Further guidance on the positioning and continuity of the ties for different structural systems can be found in UFC 4-023-03 (DoD, 2009).

Table 3-5. Summary design tie forces according to UFC 4-023-03

Tie type	**Required tie strength**
Longitudinal and transverse ties	$F_i = 3w_F L$ [kN/m]
Peripheral ties	Framed and two-way load bearing wall buildings: $F_p = 6w_F\,L + 3W_C$ [kN] One-way load bearing wall buildings: $F_p = 6w_F\,L + 3W_C + 3W_W$ [kN]
Vertical ties	The vertical tie must have a design strength in tension equal to the largest vertical load received by the column or wall from any one storey using the floor load W_F.

(1) The value for the length L depends on the considered system, i.e. framed or load bearing wall buildings;
(2) W_C is 1.2 times the dead load of cladding over the length L;
(3) W_W is the dead load of a wall over a length equal to the clear storey height.

Source: DoD, 2009

Alternative Path method (AP)

When a structural element cannot provide the required tie strength, the designer may use the AP method to determine if the structure can bridge over the deficient element after it has been removed. For Occupancy Categories II Option 2, III and IV, the AP method must be applied for the removal of specific vertical load bearing elements.

The requirement for the AP method is that the structure must be able to bridge over specific vertical load bearing elements which are notionally removed from the structure. If a structure cannot be shown to bridge over a removed element, the engineer must develop suitable or similar re-designs. For alternative load path analyses, the acceptance criteria include the following:

- Strength limits of the members are not exceeded;
- Deformation limits of the members and connections, expressed in terms of deflections and rotations, are not exceeded;
- Spread of damaged members is limited.

To perform the alternative path analyses, a general LRFD philosophy is employed taking into account a load combination for extraordinary events (Table 3-6) and resistance factors to define design strengths. Three analysis procedures can be used, ordered according to increasing complexity: Linear Static (LSP), Non-linear Static (NSP) and Non-linear Dynamic (NDP). Modifications and guidance for each method and material-specific structure acceptance criteria for the analyses are given to accommodate the particular issues associated with progressive collapse. For instance, for the linear static procedure, load increase factors depending on the applied material and structural system are given to account for non-linear effects.

Table 3-6. Load combination for alternative load path analysis according to UFC 4-023-03.

Applied analysis	Load combination after notional member removal
Linear static analysis	Floors above removed column: $G_{LD} = \Omega_{LD}[1.2D + (0.5L \text{ } or \text{ } 0.2L)]$ Floors away from removed column: $G = 1.2D + (0.5L \text{ } or \text{ } 0.2L)$
Non-linear static analysis	Floors above removed column: $G_N = \Omega_N[1.2D + (0.5L \text{ } or \text{ } 0.2L)]$ Floors away from removed column: $G = 1.2D + (0.5L \text{ } or \text{ } 0.2L)$
Non-linear dynamic analysis	Complete structure $G_{ND} = 1.2D + (0.5L \text{ } or \text{ } 0.2 \text{ } S)$

Ω_{LD} and Ω_N are respectively the load increase factors for linear static and non-linear static analysis.

Source: DoD, 2009

Enhanced Local Resistance method (ELR)

The enhanced local resistance is required for some Occupation Categories and has as objective to ensure that a ductile failure mechanism can form when the column or wall is loaded to failure. To meet this objective and hence to reduce the probability and extent of initial damage, the column or wall must not fail in shear prior to the development of the maximum flexural strength. To check the latter requirement, a LRFD approach is used and flexural and shear demands are defined. More information is available in UFC 4-023-03 (DoD, 2009).

3.1.3.2 Canada

The National Building Code of Canada requires structures to be designed for sufficient structural integrity to withstand all effects that may reasonably be expected to occur during the service life. Commentary C on Part 4 advises designers to consider and take measures against severe accidents with probabilities of occurrence of approximately 10^{-4}/per year or more which is distinct from most other national Standards in giving a specific quantified threshold on the likelihood of the extreme event for which structures should be designed. While the concept of placing a quantified threshold on the likelihood of events which are to be considered is in itself valid, quantifying the likelihood of the initiating event if terrorism-related is difficult at best due to the influence by external socio-political factors which fluctuate according to governmental policy and international events (Arup, 2011).

3.1.3.3 Australia and New Zealand

Australian requirements are given as a functional statement with the requirement for the capability of the building to withstand combinations of loads and other actions to which it may reasonably be subjected. Associated performance requirements include resistance at an acceptable level of safety to the most adverse combinations of loads that might result in potential for progressive collapse (Australian/New Zealand Standards, 2002).

AS/NZS 1170.0 2002 Structural design actions – General principles states that all parts of the structure shall be interconnected with ties capable of transmitting 5 percent of the ultimate dead and imposed loads. The supplementary document AS/NZS 1170.0 Supp 1:2002 Structural design actions – General principles – Commentary states that:

'The design should provide alternate load paths so that the damage is absorbed and sufficient local strength to resist failure of critical members so that major collapse is averted. ... Connections ... should be designed to be ductile and have a capacity for large deformation and energy absorption under the effect of abnormal conditions.'

The materials design standards contain implicit consideration of resistance to local collapse by including such provisions such as minimum strength, continuity, and ductility (Arup, 2011).

Bita *et al*., 2019 survey robustness provisions in both the National Building Code of Canada (NBCC), ASCE-7, and Australian and New Zealand AS/NZS 1170.0 2002, and highlight that a common limitation that is found in both is that design provisions for "disproportionate collapse", despite existing, are only partially subjective and not explicit.

3.1.3.4 Hong Kong

The Hong Kong Building Authority uses locally-developed codes of practice for the structural use of steel and concrete. The approach to structural robustness, accidental damage and disproportionate collapse essentially follows the principles and methods adopted in the United Kingdom, although there is little specific reference to robustness in the Hong Kong Building (Construction) Regulations or in Hong Kong Codes of Practice for structural design. The code Structural Use of Steel 2005 issued by the Building Authority gives guidance on the principle of design against disproportionate collapse, requiring elements to be tied together horizontally and vertically, and for the building to be designed to survive the removal of non-key elements by establishing alternative load paths. Key elements which have a critical influence on the overall strength or stability of the structure, should be '*...designed to resist abnormal forces arising from extreme events*.'

The Code of Practice for the Structural Use of Concrete 2004 more closely reflects BS 8110-1:1997, presenting tying requirements consistent with UK practice. The general principle is given that '...a structure should be designed and constructed so that it is inherently robust and not unreasonably susceptible to the effects of accidents or misuse, and disproportionate collapse.' No guidance is given on any requirement for alternative load paths or design of elements critical to the stability of the overall structure (Arup, 2011).

3.1.3.5 International Organization for Standardization

In the international context, a particular mention should be made to the updated ISO 2394 (ISO, 2015). This standard addresses decision-making in design and assessment of structures. ISO2394 - Annex F encloses extensive robustness provisions based on risk-based design that are significantly more in-line with what is identified as state-of-art practice for robustness assessment.

In its risk-based approach the ISO 2394 uses 5 Classes that are comparatively more comprehensive in characterisation of structures than the ones presented in the Eurocodes. These use a more detailed description of class categories, and more important, provide a quantitative indicator of risk based on the expected number of fatalities. For structures up to Class 3, pre-specified robustness provisions are given in the code which involves some of the methods already mentioned, such as the event control method or the alternative path. For Class 4 and 5, risk-based assessment for robustness is provided in Annex F.4. In addition to the reference to progressive or disproportionate collapse, it also uses the term "damage insensitivity" to relate the need for comprehensive robustness provisions.

3.2 Assessment of current design guidelines

Worldwide building standards contain specific provisions for the design against progressive collapse or they provide more general procedures to increase the structural integrity and robustness. Depending on the country, the design guidelines can have a normative/legal or informative status. They all emphasize the need for a good structural layout, redundancy, ductility and continuity in order to design for progressive collapse and/or to avoid disproportional damage. In general, three types of strategies can be identified in the structural guidelines:

- Non-structural strategies such as minimising the exposure to hazards by preventive measures;
- Indirect design methods providing strength, redundancy, continuity and ductility by the use of prescriptive rules;
- Direct design methods which include the alternative path method (ability to bridge over local damage zones) and the specific load resistance or key element method (strengthening vital structural components).

The decision to apply a certain strategy depends on the associated consequences in case of collapse whether using Consequence Classes (i.e. British Standards and Eurocodes) to categorise the structure or by using Occupancy and Risk Categories (UFC 4-023-03). Note that building standards that address progressive collapse, by a direct or an indirect approach, usually contain specific requirements for tying systems as well as requirements to check the structural stability under specific load combinations which take into account structural damage or accidental loads. In the following Sections, the latter is compared for the British Standards, Eurocodes and UFC 4-023-03 guidelines of the American Department of Defence. Note that the British Standards and Eurocodes also allow to perform a complete risk analysis in order to assess the structural robustness of a design.

EN1990 gives general guidance on non-structural measures, by recommending the avoidance, elimination or reduction of the hazards to which the structure can be subjected as a basic requirement for design. However, apart from the case when a full risk assessment approach is required (e.g., EN1991-1-7 Class 3), no more detailed non-structural measures or requirements to be used in design are given. Objectively, non-structural strategies do not increase the resistance of a structure to disproportionate failure. Nonetheless, more detailed measures such as,

- preventive and protective planning in site planning, or consideration of barriers, and
- restrictive planning and operation, with avoidance of non-design-specified activity

are expected to contribute to alleviate the design acceptance criteria to avoid disproportionate collapse (Canisius *et al*. 2011).

3.2.1 Indirect Design methods

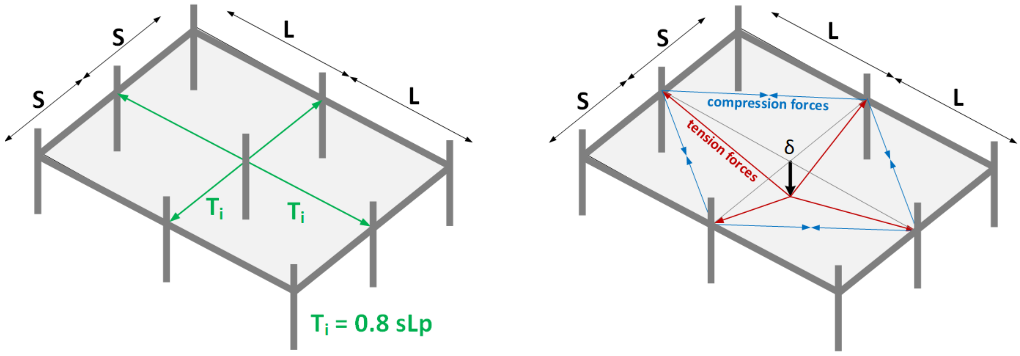
In all considered building standards, some prescriptive tie rules are provided to ensure the integrity of the structure and to allow the development of alternative load paths. In most codes, a distinction is made between horizontal ties (including perimeter ties, internal ties and ties to columns and walls) and vertical ties. However, when comparing the different standards, different design forces can be found for the respective ties. For example, the value for the intensity factor for the longitudinal and transverse ties in UFC 4-023-03 is considerably greater compared to the values for peripheral and internal ties prescribed in EN 1991-1-7, see section 3.1.2.2.4 of this report. Even between the two Eurocode parts EN 1992-1-1 and EN 1991-1-7, conflicting design strengths can be found. Moreover, the British Standards and Eurocodes only indicate some design tie forces whereas the UFC guidelines also indicate some deformation criteria which should ensure the ductility to allow for the activation of the tying system. Next to the design strength of the ties, also the placing of the ties differs between the different codes. Further, UFC 4-023-03 makes a clearer distinction for the prescriptive tie rules between the different applied materials by using material-dependent strength reduction factors.

3.2.1.1 Derivation of formulas for horizontal ties EN 1991-1-7

Although in all considered building standards some prescriptive tie rules are provided to ensure the integrity of the structure, no background documentation can be found in literature related to the derivation of these tie rules. In the following, a possible derivation method for the prescriptive tie rules of the internal horizontal ties for framed structures according to EN 1991-1-7 (CEN, 2006) is illustrated (Vrouwenvelder, 2008b).

In the following derivation, a framed structure is considered for which the span lengths of the bays in both orthogonal directions are equal to s and L respectively (Figure 3-6). Next, it is assumed that a central column is removed in order to activate the horizontal ties, resulting in a central deflection δ. Further it is assumed that the load initially carried by the removed column R is transferred by four internal ties in both orthogonal directions in agreement with EN 1991-1-7 (CEN, 2006) which is recommending to place the internal ties orthogonal in both directions. It is also assumed that the same tensile load is developing in both directions.

Figure 3-6. Justification for the prescriptive tie rules for internal horizontal ties of framed structures according to EN 1991-1-7



Neglecting the ductility and deformation limits of the internal elements, one apparently assumes a central deflection δ of ± 16 % of the sum of both span lengths:

$$\delta = \frac{s + L}{6} \tag{3-4}$$

Next, taking into account the vertical equilibrium in this deformed state (see **Figure 3-7**), the following equations can be derived:

$$R = 4 \cdot T_i \cdot \sin(\alpha) \tag{3-5}$$

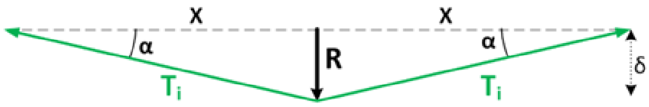
or

$$\sin(\alpha) = \frac{R}{4 \cdot T_i} \approx \frac{\delta}{X} \tag{3-6}$$

where X is the average span length of the bays in both orthogonal directions:

$$X = \frac{s + L}{2} \tag{3-7}$$

Figure 3-7. Vertical equilibrium for the deformed state to derive the prescriptive tie rules for internal horizontal ties of framed structures according to EN 1991-1-7



Taking into account the accidental load combination according to EN 1990 (CEN, 2015), the load initially carried by the removed column R can be assumed as equal to:

$$R = (g_k + \psi \cdot q_k) \cdot s \cdot L \tag{3-8}$$

Finally, combining Equations (II.6), (II.7), (II.8) and (II.9), the internal tie forces T_i can be written as:

$$T_i = 0.78 \cdot R \approx 0.8 \cdot R = 0.8 \cdot (g_k + \psi \cdot q_k) \cdot s \cdot L \tag{3-9}$$

which corresponds to the formula as prescribed by EN 1991-1-7 (CEN, 2006) to design the internal horizontal ties of framed structures. Regarding the derivation discussed in previous paragraph, following remarks should be made:

- As a simplification, the ductility and deformation limits of the elements are neglected in this simplified approach. In case the span lengths s and L are both equal to 6 m, this approach would result in a central deflection of 2 m, which is for instance unrealistic for reinforced concrete elements;
- Considering the vertical equilibrium in a deformed state, large central deflections results in smaller tie forces. As a consequence, the assumption of a large central deflection could result in an unsafe design of the tie forces;
- The assumed angle of rotation is considered to be fixed and unrealistically large.

3.2.2 Direct Design methods

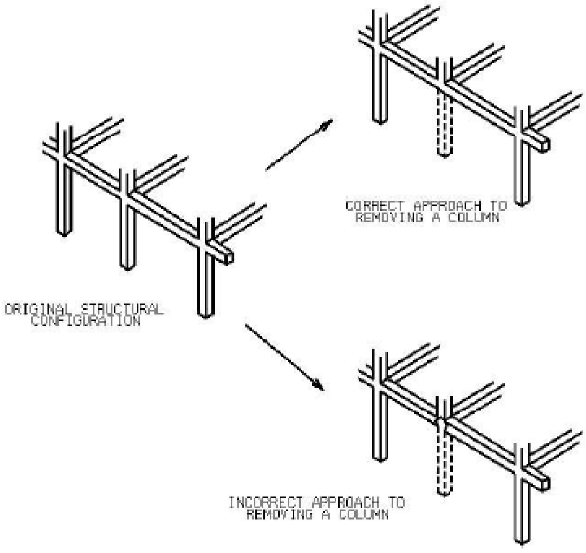
3.2.2.1 Alternative load path method

In case of higher associated failure consequences or in case the prescriptive tie rules from an indirect design method cannot be fulfilled, an alternative load path method is recommended. In each building standard, this alternative load path method is executed by considering the notional removal of a load bearing element and assessing the residual strength or bridging capabilities of the damaged structure. An example of the notional removal for the alternative load path method is given in Figure 3-8, where the removal of a column is investigated accordingly to (GSA, 2013), which is built on the UFC 4-023-03 guidelines. On one hand, in case of the Eurocodes and British Standards, little guidance is given on how to implement this procedure. In these documents, design with the alternative path method is not addressed as a comprehensive measure of design improvement, but instead, it is highly emphasised on the identification of critical elements. Such characteristic is rapidly identified when addressing the differences encountered in the codes in regard to key element design (see for example comparative usage of key element presented in Section 3.2.2.2). André and Faber (2019) highlighted before the strong element-centricity of the present design provisions in the design codes. For the UFC 4-023-03 guidelines, on the other hand, much more guidance is given such as:

- The position of the load bearing elements to be notionally removed;
- The load combinations to be considered to analyse the damaged structure;
- The analysis procedures and assumptions;
- The acceptance criteria for the strength and deformation capacities of the damaged structure, depending on the used material and structural layout.

It should be mentioned that a sudden removal of the load bearing element is considered here, thus allowing for dynamic effects.

Figure 3-8. Alternative Path method and strategy for the notional removal of elements in the structure.



Source: GSA, 2013

In the context of the alternative path method, and to emphasize the review presented, it is of relevance to mention the work of André and Faber (2019) that objectively highlights requirements for the update on the alternative path method implementation in the design codes. For example the consideration of one or more of the following provisions:

- Provision of structural redundancy (with adequate deformation and loading capacity and resistance of elements and connections, including reserves);
- Provision of secondary load carrying mechanisms (provided that elements and their connections are both sufficiently resistant and ductile to allow for additional structural loading and deformations);
- Provision of structural integrity (element continuity and ductility).

3.2.2.2 Key element design

In case the damage acceptance criteria are exceeded in the British Standards and Eurocodes, load bearing elements have to be redesigned as key elements considering a certain accidental load. On the contrary, the key element design method or enhanced local resistance method (Specific Load Resistance method) in the UFC 4-023-03 is applied for higher Occupation Classes to ensure that a ductile failure mechanism can form. Further in case the damage acceptance criteria of UFC 4-023-03 are not fulfilled, the structure should be redesigned. As a consequence of this subtle difference, the analysis procedures of the key element design method differ between these building standards. Little guidance is given in the British Standards and Eurocodes on how to design a load bearing element under the proposed accidental load. In addition, the recommended accidental load of 34 kN/m² appears to be a safe estimation of overpressure associated to a blast loading; the use of such a high value of accidental load is not necessarily appropriate for any structures for which key element design applies.

One of the aspects of key element design that is of relevance to highlight is that it may be suitable and cost-effective for structures with a limited number of identifiable key elements, and of interest when alternative load paths do not exist. However, its performance is highly susceptible to the future scenarios of loads since failure in design cannot be avoided with certainty, and this particularity of key element design should be highlighted and considered in any future key element design provisions.

Another characteristic of the key element design in the current design codes is that it neglects second-order effects that may be of relevance when a structure suffers a localised failure. In particular, being a key element designed for survivability, it will not be uncommon for, in the circumstance of failure of one key element, other neighbour structural members to be badly damaged. Provisions of multi-damage scenarios are of interest in robustness provisions that involve key element design. Whether or not this is considered admissible relates to whether this damage is considered disproportionate.

3.2.3 Systematic Risk assessment

It was highlighted in Section 3.1 that EN 1991-1-7 recommends systematic risk assessment for structures falling in CC3, with a detailed procedure being presented in Annex B. It distinguishes a qualitative and a quantitative risk analysis.

In qualitative risk analysis all hazards and corresponding hazard scenarios should be identified. Identification of hazards and hazard scenarios is a crucial task to a risk analysis. It requires a detailed examination and understanding of the system. For this reason a variety of techniques have been developed to assist the engineer in performing this part of the analysis (e.g. PHA, HAZOP, fault tree, event tree, decision tree, causal networks, etc.).

In the quantitative part of the risk analysis, probabilities should be estimated for all undesired events and their subsequent consequences. The probability estimations are usually at least partly based on judgement and may for that reason differ substantially from actual failure frequencies. If failure can be expressed numerically the risk may be presented as the mathematical expectation of the consequences of an undesired event.

In risk evaluation, acceptance and mitigation, should follow a procedure to decide when a risk is identified, whether mitigating measures should be specified. In acceptance the ALARP (As Low As Reasonable Practicable) principle is used. For mitigation different technique are proposed, such as, hazard control or controlled collapse. A step of reconsideration is enclosed and it involves reviewing the procedure until acceptance is achieved. The final step is then to communicate the results and conclusions to all stakeholders (specifying *e.g.*, analysis, sources, assumptions or further recommendations). EN 1991-1-7 Annex B also provides more detailed analysis of applications to civil engineering structures.

In the context of systematic risk assessment it is of relevance to highlight two particularities: first, this more comprehensive analysis targets a niche of structures, which may generate discussion on the threshold that separates for example the class 2b and 3 and their disassociation with quantified risk (*e.g.,* by comparison, classes of ISO 2394 consider risk–assessment for both top classes, one of which is more representative of the CC2b EN 1991-1-7 Consequence Class); and second, accordingly to (Canisius, et al., 2011), quantitative measures are those that are of interest to achieve comprehensive frameworks (comparable ranking) for decision-making in structural design.

3.3 Flaws and inconsistencies in current standardization

Based on the discussion above, the following shortcomings of the actual building standards or design guidelines can be identified:

- No consensus can be found for the indirect design method using prescriptive tie rules. Neither the design tie force formulas nor the rules for the spacing of the ties are consistent between the different codes;

- Guidance on the detailing of the ties is missing;
- The background of some formulas to calculate the tie forces is not always clear. Some design tie forces of the Eurocodes can be derived from simple equilibrium equations for a deformed state of the structure (see Section 3.2.1.1) but the associated deformations lead to unrealistic deformation demands;
- In the Eurocodes, no specific or quantified requests in terms of deformation capacity and ductility is reported while such properties are identified as a key issues when considering the structural robustness;
- Current indirect design guidelines do not make a clear distinction between different construction methods. Nonetheless different construction methods such as precast constructions will require specific prescriptive tie rules. Specific guidance for the design of precast structures against progressive collapse are given in the *fib* bulletin 63 (2012). However, this bulletin mainly adopts the recommendations according to EN 1991-1-7. Further, an alternative method is presented and illustrated in this bulletin to calculate the prescriptive tie forces for precast structures which assumes unrealistic deformations;
- In the British Standards and the Eurocodes, little guidance is given on how to perform the notional column removal or alternative load path design;
- The accidental key element design load, is based on a domestic gas explosion, but should be applicable for other exposure scenarios as well; it should also enclose considerations on second order effects of failure;
- Little guidance is given on how the accidental key element design load is transferred to the key element (i.e. as a pressure to column or considering tributary area from adjacent walls);
- With regard to the recommended 15% limit of admissible damage, the criteria on how to decide which area is affected are unclear. Also, no background document for the recommended value of 15% is provided and it is not clear whether this value is applicable to all types of structures;
- The guidance to perform a complete risk assessment is insufficient which limits the applicability of this method;
- Design formats presented in British Standards and the Eurocodes still rely to a large extent on individual element safety classifications;
- Analysis of the CC allows to perceive that some ambiguity can be encountered in the classification in design, in particular due to the lack of a quantified measure to separate CC;
- Robustness should be (at minimum) an intrinsic property of the structural system. It is a fact that considering provisions in EN1991-1-7 ensures some level of robustness in design to fundamental actions, nonetheless, there is interest in a more enveloping analysis on whether, and in which situations, further considerations should be enclosed for fundamental actions;
- It seems unlikely to be possible to avoid prescriptive provisions in design codes. In these cases, it is important to accompany robustness provisions with considerations on the assumptions and adequacy to the structure being analysed. Design considerations to apply when using prescriptive provisions can appear in the form of sensitivity analyses with respect to the prescriptive variables and other design assumptions;
- No methods are given to quantify the structural robustness of a design. Hence the framework to obtain a uniform robustness level is missing. Since robustness requirements aim to ensure adequate structural performances at system level, any applicable rules should be based on the performance of structural systems. The importance of performing a rational identification of event scenarios, damages, element failures, collapse and consequences (direct or indirect) is lacking.

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4 State-of-the-art design considerations, approaches and strategies for improving robustness[11]

4.1 General

As explained in the previous Chapters, when designing for robustness, it should be adequately assured that no accidental and/or exceptional events or damage to the structural members would result in disproportional consequences for the structural system, or even total collapse of the whole structure during its lifetime. Therefore, the robustness of the system should be adequately and appropriately considered. Design for robustness should be an integral part of the conceptual design phase of the structure, taking into account different strategies that can be applied. Therefore, it should be considered in the early stages of the design process.

In general, the design of a structure for robustness can involve the – explicit or implicit – identification of:

— Hazards *H;*

— Local (direct) damage *D*;

— Systemic damage *S* (follow-up/indirect damage), encompassing progressive collapse;

— Direct and indirect consequences.

In a risk-based context, the total risk R_{tot} in relation to accidental and/or exceptional events can be based on the following equation (4-1), considering the addition of risk associated to local (direct) damage and systemic (follow-up) damage, enabling to identify the influencing factors. Note that this expression is similar to the previously discussed equation (3-1). However, equation (4-1) is more general than what is currently proposed in EN 1991-1-7.

$$R_{tot} = \sum_i \sum_j C_{\mathrm{dir},ij} P\big[D_j \big| H_i\big] P[H_i] + \sum_i \sum_j \sum_k C_{\mathrm{ind},ijk} P[S_k | D_j \cap H_i] P[D_j | H_i] P[H_i] \tag{4-1}$$

where:

— $P[H_i]$ the probability of occurrence of hazard H_i

— $P[D_j|H_i]$ the probability of (direct) damage D_j conditional on hazard H_i

— $P[S_k|D_j \cap H_i]$ the probability of systemic damage S_k conditional on the damage D_j and hazard H_i

— C_{dir} the direct consequences

— C_{ind} the indirect consequences

Considering the expression (4-1), the following strategies for reducing the risk of disproportionate collapse can be distinguished, which will be discussed more elaborately in the next Section:

— Reducing one or more of the probabilities $P[H_i]$ of the occurrence of hazards;

— Reducing one or more of the probabilities $P[D_j|H_i]$ of (direct) damage;

— Reducing one or more of the probabilities $P[S_k|D_j \cap H_i]$ of systemic damage;

— Reducing direct C_{dir} and indirect C_{ind} consequences.

[11] Except for section 4.2, this chapter was partly drafted in coordination with fib TG3.1 developing the draft text proposals for the fib Model Code 2020, as well as the additional information provided by WG6.PT1 and WG6.PT2 of CEN/TC250/WG6.

4.2 Design strategies to prevent disproportionate collapse

First of all, it is appropriate to underline that the following strategies can be adopted in the design and execution process to prevent or reduce the progressive collapse of the system and/or to increase local safety. However, they do not necessary increase the robustness of the system.

The design strategies aimed to prevent disproportionate collapse have been largely investigated in literature (Ellingwood & Leyendecker 1978, Gross & McGuire 1983, Dusenberry & Juneja 2002) and they can be classified based on the acceptable level of probability of occurrence.

— Design strategies which prevent the occurrence of abnormal and dangerous events (i.e. influencing $P[H_i]$ in equation (4-1)). This strategy aims at reducing the exposure of the structure to hazards. A typical approach is the event control strategy which decreases the exposure of a structure by reducing the probability of occurrence and/or the intensity of abnormal and hazardous events. Safety barriers against impact or base isolators to limit the intensity of a seismic action are examples of such solutions. Event control is a non-structural measure which does not involve structural robustness.

— Design strategies which prevent the occurrence of an initial damage (i.e. influencing $P[D_j|H_i]$ in equation (4-1)). This strategy aims at improving the local component behaviour providing a sufficient local resistance in consequence of the occurrence of abnormal events. The vulnerability of the system can also be decreased by a specific-local-resistance method (e.g. detailing) on critical elements. In this design procedure, aimed to prevent disproportionate collapse, the concept of structural robustness is involved. However, the key element design approach does not necessary enhance the robustness of the system. The possible failure of critical/key elements may promote progressive and disproportionate collapses leading to unsatisfactory robustness levels.

— Design strategies which prevent disproportionate spreading of damage (i.e. influencing $P[S_k\,|D_j \cap H_i]$ in equation (4-1)). This strategy is aimed at the global system behaviour so that the spread of the initial local damage remains limited. The alternative load path approach redistributes the forces originally carried by the failed members in the damaged system trying to avoid a progressive failure. However, if the local damage produces an overloading in the remaining structure, the alternative load path procedure may promote progressive collapse and reduce the robustness of the system. The existence of alternative load paths enhances the structural redundancy. An alternative is the structural compartmentalization approach through which the spreading of failure is prevented or limited by isolating the initially damaged portion of the system.

Methods to prevent progressive and disproportionate collapse can also be divided into direct and indirect design approaches, as discussed in Section 4.5. Direct design methods, like the local resistance approach and alternative load path method, try to verify explicitly the collapse resistance of the structure when subjected to hazard scenarios through specific structural analysis. On the other hand, indirect design methods try to guarantee implicitly the collapse resistance through prescriptive design rules and recommendations.

4.3 Design considerations

A structure should be designed to have an adequate level of robustness so that, during its design service life it will not be damaged by adverse and unforeseen events, such as the failure or collapse of a component or part of a structure, to an extent disproportionate to the original cause (prEN 1990:2020). To this aim, masses, stiffness and member capacities should ideally be uniformly distributed both in plan and in elevation. Vertical members, like columns and walls, should ideally run (without interruption or strong capacity reduction) from the foundations to the top of the building. Also, in order to obtain the maximum capacity in terms of ductility of diaphragm elements, similar span values should be adopted along the longitudinal and transversal direction of the in-plan view of the building. Design for robustness should also consider all possible actions a structure might be exposed to, and the severity level of the consequences due to failure of a structural member or of a limited part of the structure. These topics are further discussed in the Sections below.

4.3.1 Classification of accidental actions

The actions considered for designing structures for robustness are either identified accidental actions (associated to identified accidental events) or accidental actions associated with unidentified hazardous events. These can then be further classified as in Table 4-1. While this is a comprehensive list, what is to be considered has to be decided between the engineer and the client/owner, guided also by applicable regulations.

Table 4-1. Classification of identified accidental actions and accidental actions associated with unidentified hazardous events.

Identified accidental actions		**Accidental actions associated with unidentified hazardous events**	
Sub-class	Sub-sub-class	Sub-class	Sub-sub-class
Impact	Internal impact	Actions	Horizontal action, applied on limited extent of structure (single component)
	External impact (Ground Level)		Horizontal action, applied on large extent of structure
	External impact (Above Ground Level)		Vertical action, applied on a limited extent (single component) of structure
Explosion	Internal Explosion - Deflagration		Vertical action, applied on large extent of structure
	Internal Explosion – Detonation		Simultaneous vertical and horizontal action On a limited extent (to reflect an explosion, internal or external) or a large extent, for example, to reflect a ground movement
		Ageing and deterioration	Indirect action – global or local extent (as a result of a deterioration process to a larger extent than what was accounted for at the design)
	External Explosion – Detonation	Defects and Errors	Indirect action – local extent (from a defect in construction or error in design)

4.3.2 Consequence Classes

The design considerations for robustness should be based on the Consequence Class (CC) of the structure. Consequence Classes categorise the severity level of the (direct and indirect) consequences due to failure of a structural member or of a limited part of the entire structure. Consequence Classes are typically defined based on the type of structure, profile of occupancy and functionality.

For each design requirement, recommended for each CC, detailing rules should be coherent with the basic assumptions of the adopted design method. Detailing rules should ensure ductile failure modes of structural members, characterised by the formation of plastic hinges in beams/joints or yield lines in continuous slabs, taking into account possible deterioration in view of durability. Therefore, a ductile failure mode should be activated prior to a brittle failure mode, such as shear failure modes in beams and slabs or punching shear failure modes in slabs. Adequate transversal reinforcement in beams and slabs or adequate slab thickness have to be designed to obtain this hierarchy of failure mode in structural members.

The qualitative definition for CC in case of robustness can – when considering the difference in severity of possible follow-up consequences – take basis in the CC categorization as applied for reliability differentiation in EN1990, see Consequence Classes CC1 (Low), CC2 (Medium) and CC3 (High) in Chapter 3. Structures complying with the criteria for more than one CC should be designed considering the most severe CC. The different CC are presented in Table 4-2 alongside the definition of the Design Approaches in terms of analysis and design methods.

According to the work of WG6.PT1 four Design Approaches are provided: DA-1, DA-2a, DA-2b and DA-3, where the level of complexity of the Design Approaches increase from DA-1 to DA-3. These can be linked to the CC1, CC2 and CC3 classes. The specificities of each design approach are elaborated further in Section 4.4.

— In DA-1, no particular robustness provisions are required if the structure is designed according to the partial factor design philosophy specified in the Eurocodes for identified non-accidental hazardous events (i.e. persistent, transient and seismic design situations in the EN 1990 terminology) and the consequences of a potential collapse of the structure are acceptable taking into account the hazardous events (i.e. also accidental and unidentified hazardous events for which the structure is not designed explicitly – good practice design and detailing rules may provide an unknown robustness level with respect to these two hazardous events).

— DA-2a and DA-2b apply to similar groups of structures but exhibiting different levels of magnitude of consequences for a given design scenario. Therefore, different levels of sophistication of the analysis and design methods should be applied to DA-2a and DA-2b. A possible discrimination between the two design approaches can relate to:

  *) DA-2a involving a design according to prescriptive rules solely (enabling the possible activation of horizontal ties and ensuring sufficient anchorage of the suspended floors to the wall), whereas DA-2b requires in addition also vertical tying.

  *) The discrimination between the two design approaches can be as such that approach DA-2a involves a simpler analysis and design methods than the approach DA-2b. For example, linear elastic models may still be appropriate in DA-2a to design for robustness with respect to all possible design situations (involving identified and unidentified hazardous events), whereas in DA-2b non-linear analyses are deemed necessary, together with an explicit consideration of the structural performance with respect to relevant design situations (involving identified and unidentified hazardous events) by structural analysis and also of the subsequent assessment of the adequacy of the achieved robustness level.

— In DA-3, the use of risk-based methods for the verification of robustness are considered necessary due to the significant magnitudes of consequences for a given design failure mode scenario. Reliability-based methods may also be used, where appropriate.

Table 4-2. Robustness design approaches.

Robustness design approaches		
ID	**Analysis method**	**Design method**
DA-1	No particular provisions are necessary with respect to robustness.	
DA-2a	Analyses can be based on simplified models of loads and structural behaviour.	Design for robustness using direct and/or indirect design methods.
DA-2b	Simplified models still possible. Dynamic and/or non-linear analysis models may become relevant. Analyses may however be based on simplified models of loads and structural behaviour.	
DA-3	Risk-based or reliability-based design for robustness.	

In general, CCs deal with the severity of the consequences given an adverse event occurrence and they have been used traditionally in design codes to define the target reliability level implicit in the structural design (see EN1990). In cases where the consequences are not negligible and the triggering adverse event is uncertain or unknown, it may be necessary in design codes to specify supplementary design provisions, namely design for robustness as they become more important. Moreover, particular design considerations may be required for existing structures.

4.4 Design approaches

4.4.1 Definition of design scenarios

The existing general approach to the design of robust buildings can be either deterministic or, as allowed for by the design equations of codes, semi-probabilistic. Commonly the risks are considered implicitly and approximately by the use of various classifications of structures. However, on certain occasions, the involved risks are considered explicitly when designing for robustness, for example for buildings in the Consequence Class 3 of Eurocodes.

Design for robustness consists of the identification and assessment of design scenarios. Herein, each design scenario relating to a set of events or conditions occurring during the construction or lifetime of the structure leading to a state of the system for which the effect of disproportionate consequences should be assessed. Design scenarios can be identified based on specified accidental actions, notional damage or loads and should be discussed with the relevant stakeholders.

In case of design for identified hazards, the design scenarios consider prescribed actions, such as accidental actions, and/or condition states and the resulting effect on the structure. The accidental actions and events to be assumed in the design can differ substantially from project to project. Therefore, it is difficult to specify the design scenarios in a standardised manner, in particular when threat-specific design is used.

On the other hand, when possible threats cannot be specified, they are expressed by notional actions or notional damage. Herein, notional damage is typically specified as cases of initial local failure. Possible threats also include known threats that cannot be quantified and are to be discussed an agreed upon between the relevant parties.

4.4.2 Design for specified accidental actions

When designing a structure with specific threats in mind, it is required to identify and quantify all abnormal events that could possibly affect the structure and the resulting actions on it. In general cases, such input data are usually incomplete and imprecise because some threats are unforeseen, combined with the issue that their magnitude is difficult to predict. Therefore, the application of a direct design method against (identified) accidental actions may in some cases require to be complemented by elements of threat-unspecific design (via direct or indirect design methods), in particular by the assumption of notional damage. In cases where threat-specific scenarios are simplified by threat-unspecific scenarios, attention should be paid to the definition of the design scenarios, which should not deviate from the likely damage scenarios the structure would be subjected to by the occurrence of identified accidental actions. For example, if the identified accidental action is the impact of a vehicle on two columns, it is not adequate in general to design the structure for the case where only one column is notionally removed. Note that robustness design for identified accidental actions should preferably consider direct design methods and threat-specific scenarios.

In general, a structure should be designed considering the worst case scenario from both threat-specific and threat-unspecific accidental actions. The European design code EN1991-1-7 provides guidelines for the modelling of impact loading and explosions occurring inside buildings, which are accidental actions with high destructive potential. Furthermore, several types of impact on buildings and bridges are considered. Reference values and locations of impact forces are provided, that are modelled as either equivalent static loads or dynamic loads depending on the Consequence Class of the structure. In case of explosions, EN1991-1-7 takes into account both pressures directly acting on structural members and pressures transferred to structural members from non-structural members.

When structures are checked for specific accidental actions and/or to resist local damage, load combination rules should reflect the low probability of concurrence of the accidental action and the design live loads, i.e. partial factors which are lower than for the case of ultimate limit state verifications.

4.4.3 Design for actions from unspecified threats

4.4.3.1 Notional damage scenarios

The analysis of the structural system under a notional damage scenario should be performed considering the accidental load combination. Of course, when the performance objective that no initial damage shall occur is specified, notional damage cannot be specified as a hazard scenario and only the design methods based on event control, protection and local resistance are applicable. Notional damage scenarios can include both notional removal scenarios and notional deterioration scenarios.

4.4.3.1.1 Notional removal scenarios

Notional removal scenarios, consist of notionally removing structural elements and consequently checking the structure for disproportionate consequences, e.g. applying an alternative load path design or the identification of key elements. Preliminary structural analyses may be used to identify the critical elements to consider to be notionally removed. These scenarios are usually considered to be related to the failure of a connection or structural member due to an unidentified hazard.

Structural elements to be notionally removed can include one or several columns, one or several more panels or a nominal wall length and any other elements judged vital to the structural performance. These members are typically, but not exclusively, perimeter columns and/or load bearing walls between the ground and first levels. Alternatively, these may also include interior load bearing elements in vulnerable locations.

The analysis of the structural system under a notional removal scenario is commonly executed through a static analysis. In cases where dynamic behaviour is dominating the structural response (e.g. referring to the structural response of the intact system), the analysis should consider dynamic effects in a simplified (e.g. energy-balance based) or extensive way.

4.4.3.1.2 Notional deterioration scenarios

In case of notional deterioration scenarios, the geometrical and/or material properties of one or more structural elements are notionally reduced and the structure is checked for disproportionate consequences.

4.4.3.2 Notional loads

Notional loads are generally specified as a uniformly distributed equivalent static load. A value often referred to in standards for a notional load for buildings is 34 kN/m^2. This nominal value was recommended after the Ronan Point (UK) accident, triggered by a gas explosion. The value should however be treated with caution in case the design scenario considered is considerably different, e.g. in case of the design of a key element, especially in case where the failure of the key element relates to large consequences. Moreover, design for notional loads may be completely inefficient if the effects of unidentified actions/influences are not covered by such notional loads.

These loads should be applied in the most unfavourable direction for the element under consideration.

4.5 Strategies for improving robustness

4.5.1 General

Design strategies for robustness can generally be divided into direct design methods and indirect design methods. The direct design methods explicitly aim to limit the effect of local failure. They require structural analyses in order to evaluate the performance of the structure for a certain damage scenario and can start from an alternative load path strategy and/or a consequence reduction strategy. In case the previous approaches do not lead to an adequate level of robustness considering reasonable investments, the design could also be focused on reducing the probability of failure of critical elements by means of event control strategy and/or the specific load resistance strategy. The indirect design methods, on the other hand, do not explicitly consider the ability of a structure to sustain an abnormal load effect, but aim to enhance the robustness implicitly, e.g. through the use of prescriptive horizontal and vertical tie reinforcement.

The adoption of a particular strategy for designing a given structure for robustness may lead to a conceptual solution with structural features which may be beneficial for some hazard scenarios but detrimental for others, depending on the structural system, the abnormal triggering-event, the magnitude and location of the initial failure or the type of collapse. Therefore, in many cases, an appropriate way for economically meeting all structural safety and robustness requirements should be based on a combination of different design strategies as appropriate. For this purpose, a pragmatic approach can be adopted (Tanner & Hingorani 2019):

- Adoption of continuous structural systems with a ductile behaviour;
- For hazard independent scenarios:
    - Provision of either alternative load paths via prescriptive tying forces for indirect design methods, or, notional scenarios that simulate failure of selected key members for direct design methods;
- For hazard dependent scenarios:

- Provision of alternative load paths preferably making use of direct design methods via explicit analyses, or, alternatively via prescriptive tying forces for indirect design methods.;

— If alternative load paths are provided and verified for the case of structural key member failure where the associated collapsed area A_{col} can be deemed negligible, then human safety criteria can be neglected;

— If predefined collapse mechanisms (fuses) are inbuilt and verified, or in the case of non-redundant structural systems (e.g. statically determinate structures) the collapsed area can be readily established then human safety criteria should be applied for design or assessment of key element.

— It should be noted that also horizontal stiffness measures can have a beneficial effect on the robustness of structures, e.g. as the result of bracings, moment-resisting frames, infill masonry walls, etc.

4.5.2 Alternative load path strategy

The alternative load path strategy explicitly considers the resistance to progressive collapse (i.e. 'indirect' or 'follow-up' failure) when the level of damage is specified. The strategy thus examines the situation that one or more structural elements have been damaged and have no more load bearing capacity, and focuses on reducing the probability $P[S|D\cap H]$. For the remaining part of the structure, it is required that, for a specified short period of time, the structure withstands the associated actions with an acceptable probability of failure. An effective application of the alternative load path strategy typically consists of providing sufficient redundancy/integrity, durability and ductility.

In frame type structures, the resisting mechanisms which can minimize the risk of progressive collapse can include:

— Bending of the beam where the column has failed; this mechanism is generally ineffective and seldom adopted since the beams have to be over-dimensioned;

— Vierendeel behaviour of the frame over the failed column;

— Arch effect of the beams where the column has failed; this mechanism is effective in case the neighbouring structure is sufficiently stiff to limit horizontal displacements;

— Catenary/tensile membrane behaviour of beams/slabs, bridging the damaged column by means of large rotations and displacements;

— Contribution of non-structural elements. For buildings, these can be as external infill walls and partitions.

The transition of bending action in beams and slabs to tensile membrane (or catenary) action is commonly considered to be a building's last line of defence against progressive collapse. If the designer relies on one of these resisting mechanisms for robustness, these should be demonstrated through the analyses and design considerations.

4.5.3 Consequence reduction strategy

The consequence reduction strategy aims to limit unacceptable (disproportionate) consequences (C_{ind} and/or $P[S|D\cap H]$) associated with damages D. The consequence reducing measures should be selected on the basis of their risk reduction and the related costs necessary to achieve it. These measures can include:

— Structural segmentation/compartmentalization, through which horizontal progression of collapse can be limited effectively by dividing the structure into independent structural systems by means of so-called 'structural fuses'. The type and location of structural fuses should be appropriately chosen and suitably designed, detailed, executed and verified. Structural fuses are particularly effective in large, low buildings but less effective for tall buildings. In case of the latter, compartmentalization generally involves the installation of strong floors intermittently over the height of the building.

— Changing the context of the structure. This can be related to architectural and structural variables such as the shape and static scheme of the building, organisational variables such as relocation of key business operations to more protected and/or robust parts of the building, Self-rescue and rescue by others and backup facilities.

— Other non-structural mitigation measures, which can include the organisation of an efficient emergency response including fire-fighting teams, police, rescue teams, nearby hospitals, exercises, feedback of experience.

Segmentation offers an alternative strategy where the spreading of failure following initial damage is prevented or limited by isolating the failing part of a structure from the remaining structure by so-called segment borders (Starossek, 2007; Starossek & Haberland, 2012).

The most common form of segmentation relies on weak segment borders, allowing failure of a specific segment without progression of failure to adjacent segments. In this mode, segmentation acts as a fuse (Starossek & Haberland, 2012), where the extent of a segment that is allowed to collapse would need to be determined by the design engineer in consultation with the client and/or local authority, depending on the importance and type of the structure.

Another form of segmentation relies on very strong segment borders which would be designed to arrest an incipient collapse (Starossek & Haberland, 2012). In this mode, segmentation can offer an alternate load path, typically with resistance to local damage achieved at small deformations, or it can arrest the collapse of part of the structure.

4.5.4 Event control strategy

The event control strategy consists of preventing the occurrence of the hazard, or, limiting the occurrence rate of the hazard to an acceptable level. This requires that the hazard (or spectrum of hazards) is identified. The strategy focuses on reducing the probability $P[H]$ and does not increase the intrinsic resistance of a structure to unacceptable damage. Measures associated with event control can include:

— Changes in the building site or access to it, for example through avoiding high risk areas;

— Restricting the use of the structure. This might consist of using certain areas for specific utilizations or prohibiting the storage or transport of explosives or other hazardous material sources;

— Installation of early-warning systems for the hazard under consideration, through active monitoring of wind, gas or fire development;

— Installation of passive measures, such as barriers to prevent vehicle collision, dykes or fire insulation;

— Quality management to prevent human errors;

— Maintenance.

4.5.5 Key element design strategy

The key element design strategy aims at preventing or limiting the local damage (often referred to as 'direct damage') caused by a certain hazard, such as blast pressures. Therefore, the key element design strategy is a hazard specific approach focussing on reducing the probability $P[D|H]$.

Key elements are structural elements and/or connections which are essential to the resistance of the structure. These include structural members on the lower floor levels that are closest to exterior vehicle threats, piers of continuous bridges and cables in cable supported structures. Measures associated with specific load resistance might include increasing the element resistance and providing appropriate durability, increasing the element stiffness, and using active or passive isolation techniques such as base isolation of the structure.

Failure of a key element typically results in significant consequences since, in its absence, the structure is usually unable to develop adequate alternative load paths, unless a combined design strategy is applied. For example, the key element design strategy can be supplemented with providing sufficient structural ductility. Herein structural ductility can be achieved through providing ductility at the level of the system, the elements and the material. Material ductility can be achieved by material strain-hardening and/or by material deformation capacity, while ductile detailing can be achieved by using continuous bottom reinforcement over supports, confinement at joints and adequate ties to allow for load transfer.

4.5.6 Prescriptive rules

The application of prescriptive design rules is an indirect design method. These design rules are not performance based and are assumed to enhance the robustness implicitly. The most common prescriptive rules relate to providing horizontal and/or vertical tension ties. More specifically, in framed structures, common prescriptive rules consist in providing horizontal and/or vertical ties and for load bearing wall systems in providing effective anchorage of floors and roofs to walls. Lastly, prescriptive rules with a more explicit nature could be used based on analytical, numerical, empirical models.

4.6 Design for robustness against ageing and deterioration

Structural robustness should be evaluated not only with reference to accidental and abnormal loadings leading to sudden damage, such as explosions or impacts (Ellingwood 2006), but also considering continuous damage associated with the effects of ageing and deterioration processes. In fact, deterioration processes, such as corrosion in steel or reinforced concrete structures, may lead over time to unsatisfactory structural performance under service loadings and involve disproportionate effects and alternative load redistribution paths (Biondini & Restelli 2008, Biondini 2009, Okasha & Frangopol 2010, Zhu & Frangopol 2012). Moreover, the lack of maintenance and repair activities may also exacerbate these effects and lead over time to systems with insufficient robustness. These effects are particularly relevant for buildings and bridges exposed to corrosion and other kind of environmental damage. Notable events of bridge collapses due to the environmental aggressiveness and related phenomena, such as corrosion and fatigue, include for example the Silver Bridge in 1967 (ASCE 1968), and the Mianus River Bridge in 1983 (NTSB 1983). It is hence necessary to consider and ensure structural robustness over the entire system life-cycle. Quality control procedures, including visual inspections, non-destructive tests and diagnostic activities carried out over the structural lifetime to plan maintenance and repair interventions, may significantly reduce the probability of initiation and propagation of damage associated with ageing and deterioration.

Design guidelines and standards on structural robustness against ageing and deterioration are not available. The definition of damage scenarios involving deterioration processes is a critical task depending on the system location and exposure. Ageing and deterioration effects can be identified based on specific material damage phenomena (e.g. concrete carbonation, steel corrosion and fatigue, among others) or notional damage scenarios (*fib*, 2006). Deterioration processes are generally complex phenomena but they can be described using empirical models where geometrical and mechanical properties of the ageing system are properly reduced in time (Ellingwood 2005). Notional damage scenario can be also considered by means of prescribed patterns of structural deterioration applied at cross-sectional, member, and/or system level (Biondini & Restelli 2008).

In general, the design of robust structures based on very strong key members, i.e. playing a disproportionate role in the structural system, should be carefully considered during the conceptual design phase, accounting for deterioration effects. Adequate solutions should be adopted to properly protect the most important members against occurrence of damage. Moreover, the degree of static indeterminacy should be adequately selected in relation to the expected amount of damage, since an increase in the degree of static indeterminacy does not necessarily lead to an increase of structural robustness (Biondini et al. 2009, Biondini & Frangopol 2014).

The effects of maintenance and repairs activities in terms of structural robustness have to be considered and planned during the design phase focusing on preventing the occurrence and the spreading of initial damage. Structural health monitoring can also be used to capture the occurrence and the evolution of damage processes. Time-variant robustness measures incorporating information from structural health monitoring are hence necessary to plan eventual repair interventions and maintenance actions to protect, improve and/or restore the lifetime system performance.

Finally, it is worth noting that progressive decay of structural performance under ageing and deterioration is generally affected over time by significant uncertainties related to material properties, damage mechanisms, and decision-making processes associated with maintenance and repair policies. A proper modelling of all these uncertainties is therefore of essence to design and maintain robust systems when severe ageing and deterioration effects are expected (Biondini & Frangopol 2016).

4.7 Multi-hazard design considerations

In many cases, structural design should allow multiple performance objectives to be met against different hazards, which implies the activation of strongly different behavioural modes and resisting mechanisms to reach target safety levels under several loading conditions (see e.g. Section 4.4). In this respect, current Eurocodes do not define target safety at structural system level because target reliability is associated with the most critical local failure mode due to the most unfavourable load combination. This means that structural safety at system level is only verified for the envelope of effects of all scenarios. It is also noted that common design practice looks at different scenarios and local failure modes independently, so the cumulative effect of several hazardous scenarios on the safety level is usually ignored. Therefore, more refined design/assessment procedures could be based upon a multi-hazard design framework, as emphasised by some papers (see e.g.: Li et al., 2011; Adam et al., 2018) and mentioned in some building codes (see e.g. EN1991-1-7 Annex B). In such a context, designers are thus requested to find solutions that allow a satisfactory performance of the structure with respect to different criteria, either in case of new constructions or when designing retrofit interventions for existing constructions. For instance, Corley (2004) and Hayes et al. (2005) assessed the effects of alternative seismic design and strengthening of the Murrah Federal Building on its progressive collapse resistance, respectively. Those studies pointed out an influence of seismic resistance on structural robustness. Hence, it is important to check at least different performances (seismic resistance, fire resistance, robustness, etc.) independently because designing for a specific performance can positively influence the design for another performance. Design solutions (either explicit or implicit through prescriptive rules) to accommodate a certain design criterion might not be suitable for another criterion (e.g. catenary/membrane action, rotation capacity without necessarily maintaining moment resistance, etc.).

Provided that the structure is definitely designed or retrofitted to sustain gravity loads in order to meet performance objectives under serviceability and ultimate conditions, there may be several instances where both other loads and structural robustness must be considered. Indeed, multi-hazard conditions may arise from the need to design, assess or retrofit the structure against earthquake actions and identified/unidentified extreme hazards such as fire, explosions, impact, and damage to a single element or a limited sub-system.

Multi-hazard design could be related to

(1) either individual hazards or
(2) interacting hazards, including cascade events such as fire after blast, landslide or tsunami after an earthquake, or vehicle impact after riverine/coastal flood. The

scenarios to be considered have then to be agreed for a specific project. It is important to notice that this is currently not common practice, although the consideration of such combined hazards might be important to consider in the design.

In case of interacting hazards, classical single-hazard-oriented design methods may not meet performance requirements, resulting in huge difficulties for actual implementation of design/retrofit solutions in engineering practice. In addition, interaction with environmental hazards involving continuous damage associated with ageing and structural deterioration should be carefully considered over the system life-cycle since they can exacerbate disproportionate damage effects induced by extreme events and favour damage propagation and progressive collapse (Biondini et al. 2014). The following sub-sections focus on two recurrent multi-hazard conditions where structural robustness must be ensured together with earthquake or fire resistance.

4.8 Seismic design versus robustness

When the structure must be designed or retrofitted to withstand both earthquakes and other extreme events, the engineer is requested to find a multi-hazard solution accounting for the main differences between the effects of earthquake ground motion and those of, for instance, heavy damage to a single structural component (e.g. a corner column of a framed structure). The differences between seismic and robustness designs include, but are not limited to, the following:

3. Identification and modelling of hazards and their action on the structure (impossible to do for unforeseeable events);
4. Modelling of gravity loads (combination, dynamic amplification, etc.);
5. Roles of large deformations and floor system response;
6. Definition of performance objectives; and
7. Response of structural components and systems to earthquake ground motion and local abnormal actions.

Seismic ground motion involves the whole base of the structure, inducing forces (especially in the horizontal direction) and deformations throughout the structural system with some potential concentrations of demand, particularly in case of irregularity in plan and/or elevation. In most cases, extreme events such as impact or blast strike a limited portion of the structure, which in turn may activate a partial or global response (especially in the vertical direction) after that local damage occurs. This latter occurrence is responsible of a possible dynamic amplification of gravity loads in those parts of the structure that mostly contribute to activating the progressive collapse resistance. The response of structural components and systems to extreme events is often characterised by large strain rates and, more importantly, very large deformations of structural members and floor systems that produce additional resisting mechanisms (e.g. arch and catenary action in beams, membrane action in floors). This is not usually observed, and hence neglected, in nonlinear response of structures subjected to seismic actions.

It should be noted that, as a matter of principle, the identification and definition of structural systems (frame systems, wall systems, dual systems, large panel systems) within building codes reflects a single-hazard rationale. For instance, Eurocode 8 (EN1998-1:2004) provides a classification of structural systems with respect to their earthquake resistance, whereas no definition is given for the case of accidental loss of vertical components or, more in general, towards progressive collapse resistance and robustness.

One of the most interesting points of discussion in the literature is whether and how earthquake resistance may produce suitable levels of robustness (Hayes et al., 2005; Pekau & Cui, 2006; Gurley, 2008; Tsai & Lin, 2008; Parisi & Augenti, 2012; Livingston et al., 2015; Lin et al., 2016). On one hand, ductility requirements are expected to improve both seismic performance and structural robustness. By contrast, possible conflicts between seismic and robustness designs may originate from capacity design criteria, particularly the strong-column/weak-beam (SCWB) design rule according to the hierarchy resistance criterion in current seismic design provisions. This could lead to systems that cannot be able to provide the required performances for robustness. Nevertheless, at least two alternative strategies can be implemented to improve both earthquake resistance and robustness: (i) to increase the ultimate bending moment of beams and, consequently, that of columns; and (ii) to provide weak beams with a sufficient overstrength by catenary action, considering the key role of ultimate deformation of reinforcing steel and reinforcement bond in beam-column joints or adjacent beams (Yu & Tan, 2014). There is no doubt that increasing beam strength may violate the SCWB capacity design rule. Conversely, weak columns may produce soft-storey mechanisms under horizontal actions, resulting in a high probability of pancake collapse unless falling of upper storeys is arrested during their impact on lower floors (see e.g. Lalkovski & Starossek, 2016). It is worth noting that capacity design criteria are usually assumed as time-invariant in seismic design. However, the system ductility and hierarchy of member strengths – and hence the energy-dissipating failure mode claimed for a capacity design of the structure according to the SCWB rule – may change over time depending on the environmental exposure of the structure (Biondini & Frangopol 2008), possibly shifting from a typical 'beam sway' to a 'column sway' collapse mechanism (Biondini et al. 2011). This highlights the importance of a proper combination of seismic and environmental hazards in the evaluation of the life-cycle seismic performance and structural robustness of deteriorating systems.

In the case of steel structures, Park & Kim (2010) carried out a pushdown-based fragility analysis to assess the progressive collapse potential of frames with different types of connections. Xu & Ellingwood (2011) investigated the robustness of seismically designed pre-Northridge steel moment-resisting framed buildings, which did not develop a significant catenary action because of a high probability of connection failure.

Dealing with reinforced concrete structures, Brunesi et al. (2015) characterised the progressive collapse fragility of European, low-rise, framed buildings designed according to Eurocodes through incremental dynamic analysis. Both structures designed to gravity loads only and structures designed for earthquake resistance were investigated, indicating a significant impact of seismic design criteria and detailing on robustness. That study was further remarked by a huge amount of pushdown analyses (Brunesi & Parisi, 2017), highlighting that seismic design according to Eurocode 8 (EN1998-1:2004) produced a significant increase in vertical load capacity, ranging between 50% and 80%. Those results confirmed previous numerical analyses that showed higher ultimate load factor of earthquake-resistant buildings, with a mean overload factor greater than unity in case of single-column-loss scenarios (Parisi & Augenti, 2012). Nonetheless, the case-study building designed for earthquake resistance revealed an insufficient robustness under the loss of a single column in a building façade with few bays. Accordingly, Li & Sasani (2015) found that special frames designed according to American codes do not necessarily perform better than their ordinary (i.e. non-seismic) counterparts in resisting progressive collapse. This shows that the positive or negative impacts of earthquake-resistant design on robustness are still a matter of research before general conclusions can be drawn. Hence, seismic design not necessarily leads to an acceptable design for robustness.

Experimental tests have also been carried out to provide further evidence on this issue. Sadek et al. (2011) and Lew et al. (2013) tested cast-in-place reinforced concrete sub-assemblages, which consisted of two span beams, two exterior columns, and a central column that was pulled down. Two types of specimen designed for different seismic categories, namely, intermediate and special moment frames, were experimentally studied. Yu and Tan (2013) tested two specimens with different seismic detailing. That experimental programme was extended to four specimens with non-seismic and seismic detailing (Yu & Tan, 2017) to evaluate the different behaviour of gravity-load-designed and earthquake-resistant structures.

Both numerical and experimental studies on the assessment of structural resistance against both earthquake actions and progressive collapse can promote innovation in design and construction. Recent studies have evaluated the effectiveness of novel solutions in that direction, which can further stimulate other studies aimed at reaching general conclusions on the interaction between seismic and progressive collapse designs (Kim et al., 2011; Feng et al., 2017; Lin et al., 2019a, 2019b; Lu et al., 2019; Quiel et al., 2019).

Based on static and dynamic nonlinear analyses, Kim et al. (2011) found that the combined use of rotational friction dampers and high-strength tendons can significantly enhance both the seismic performance and progressive collapse resistance of existing structures.

Feng et al. (2017) investigated new RC frame structures, proposing a novel kinked rebar configuration to simultaneously improve earthquake resistance and robustness. The location of kinked rebar in beams was optimized so that the progressive collapse resistance was improved. From a physical standpoint, a kinked rebar has greater deformability as it can be gradually straightened under tension. This produces a stair-stepped tensile behaviour with a short elastic branch, followed by a plastic branch with very low hardening, a third branch with very high hardening, and a final perfectly-plastic branch till failure. Two yielding points are then identified at low and high stress/strain levels. Nonetheless, the structural behaviour may be affected by several drawbacks, such as (i) the initial bending capacity of the RC cross section lower than that associated with traditional steel reinforcement, and (ii) possible shear failure when kinked rebar is located within the shear span of beams.

Lin et al. (2019a, 2019b) remarked that considering seismic and progressive collapse designs individually may produce an undesirable performance of the structure as well as waste of construction materials. Those researchers proposed a novel design solution for precast RC frame structures, which was validated via cyclic and progressive collapse tests, the latter reproducing a middle-column removal scenario. Specifically, the case-study structure was a multi-storey frame with precast RC beams and columns, which were connected to each other by means of unbonded post-tensioning (PT) tendons, energy dissipating steel angles, and shear plates. The proposed design solution was able to provide the frame with important capacity features, such as large rotational capacity of beams, slight damage, self-centring, and ease of repair.

Precast RC frame structures were also investigated by Quiel et al. (2019) who proposed two variants of non-emulative beam-column connection for progressive collapse resistance. The study focused on a ten-storey building with perimeter special moment frames, which were subjected to ground-floor column removal. The proposed beam-column connections consist of unbonded, high-strength steel PT bars, which pass through ducts in the column and are anchored to the beams via bearing plates. PT bars act as structural fuses after yielding, then maximising ductility. After that the design solution was validated through full-scale pushdown testing, experimental results were incorporated into a structural model of the frame system that was analysed under column removal. The outcomes of nonlinear dynamic analysis showed that the structural system can arrest progressive collapse under a single-column loss scenario.

Lu et al. (2019) proposed another design solution for composite steel-concrete frames consisting of concrete-filled steel tube columns and steel I-beams, prestressed steel strands, replaceable energy-dissipating components, and shear panels. The strands can develop a self-centring capacity, hence minimising residual deformations and maximising repairability. The energy-dissipating components were made of steel angles and rib stiffeners. Based on experimental tests and finite element simulations, the proposed design solution allowed the composite frame to develop better seismic and progressive collapse performances compared to the traditional frame. Regarding the earthquake resistance, the innovative frame demonstrated smaller residual deformations. At the same time, the progressive collapse resistance was also improved through catenary action, increasing rotational capacity of beam-column connections.

Although few studies have investigated the multi-hazard design for seismic resistance and structural robustness, their outcomes clearly indicate some interesting chances to meet multiple performance objectives, allowing the structure to develop different behavioural modes depending on the type of actions they are subjected to. This promotes technological innovation in the field that is aimed at increasing structural safety, resilience, and sustainability.

Note that in the Eurocode system, seismic aspects are dealt with in EN 1998, supplemented by country-dependent specifications is national annexes and treated by alternative limit state formulations.

4.9 Fire design versus robustness

The fire action induces two main phenomena in the affected structure:

— Modification of the mechanical properties of the materials exposed to elevated temperatures; and

— Redistribution of the existing internal stresses and development of new internal forces, for example due to thermal expansion.

Obviously, these phenomena have to be accounted for when considering the fire design of a structure. The consideration of the second one requires ensuring that the structures and, in particular, key structural elements are able to sustain these extra forces.

When a fire develops in a structure, the temperature of the affected structural members is increasing, inducing an elongation of the latter; so the development of axial compression loads in these members is generally observed (when these members are axially restrained at their extremities). Also, while the fire is developing, the material mechanical properties are modified; in particular, their elastic strength and their young modulus are decreasing. These modifications lead to the development of plastic zones in the structure but may also provide these plastic zones with an additional deformation capacity in comparison to what would be available at room temperature, leading to significant deformations and displacements, in particular in the horizontal structural elements (i.e. the beams and/or the slabs). Accordingly, even if the effect of the member elongation is governing at the beginning of the fire with the evolution of axial compression loads, these axial loads start to decrease at a certain time, i.e. when significant displacement appear, these can become axial membrane tensile loads if the fire action and the associated elevation of temperature are sufficient.

So, in most cases, a fire design requires ensuring that structural horizontal members are able to sustain axial compression and tension loads which is also a requirement regularly met when designing for robustness.

As an example, recent researches in field of fire design resulted in the development of design recommendations to ensure the ability of a composite floor to develop membrane forces and so to resist to fire actions (Vassart & Zhao, 2013). It seems obvious that the application of these recommendations can be of help when considering, for instance, the possibility of activating tying forces or when applying a column loss scenario in the framework of a design for robustness. However, to date, it has not yet been demonstrated that these rules are sufficient.

In addition, for redundant structures the fire performance cannot be evaluated by considering the evolution of the thermal-induced damage at the local level – i.e. at the cross-sectional or member level as addressed by design codes – but needs to be investigated at the system level by taking into account the actual role played by the static scheme in the time-variant stress redistribution process. With this regard, the margin of safety may strongly depend on the prescribed fire scenario, and the most critical scenario may be not associated with the maximum thermal load. Moreover, for a prescribed fire scenario the structural performance is depending on both mechanical and thermal loading history, and the most damaged structural configuration at the end of fire exposition is generally not the most critical one (Biondini & Nero 2011). These aspects are crucial to properly estimate thermo-mechanical damage effects and related consequences under fire and establish suitable design rules to ensure structural robustness against fire events and other interacting hazards.

Another approach which is also used in fire design is the compartmentation (i) to limit the propagation of a fire but also (ii) to limit the propagation of the damages of a localized fire. This last point requires to design the rest of the structure, i.e. the part of the structure not directly affected by the localised fire, to sustain the additional forces associated to the fire action and, in particular, the axial forces developing in the horizontal structural elements affected by the fire. Again, the fact that specific parts of a structure are designed to resist such loads can be seen as an added value when considering the limitation of the propagation of a local damage in the framework of a design for robustness.

Considering these different points, it appears clearly that some structural requirements associated to a fire design are concomitant with some associated to a design for robustness. However, at this stage, it has not yet been demonstrated that the level of requirements coincides, i.e. the fact of satisfying the requirements for fire design is sufficient to ensure an appropriate level of robustness.

In a recent European project (Demonceau et al., 2013), it has been demonstrated that the fact of considering both structural requirements, i.e. the ones from the fire design and the ones from the design for robustness, in a combined design approach allows to guarantee the requested level of safety while limiting the needs, for instance, in terms of fire protection without any extra costs. However, this project was only covering a specific structure typology, i.e. car parks, and further developments in this field are still required.

With respect to interacting hazards, in (Demonceau et al., 2013) a genuine example was also presented of a coupled multi-hazard fire/robustness situation, where the local damage is initiated by a localised fire, which can be treated within an extended robustness assessment framework (Fang et al., 2013).

Note that in the Eurocode system, fire aspects are dealt with in parts 1-2 of EN1991 and the difference material Eurocodes, supplemented by country-dependent specifications is national annexes, and treated by alternative limit state formulations.

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5 Quantitative measures of structural robustness

Structural robustness evaluations should provide objective quantifications to establish design verification criteria and ranking design solution alternatives. Furthermore, a quantitative measure of system robustness should allow to prioritize maintenance and repair interventions on existing structures. However, despite the significant efforts made to develop robustness quantification criteria and procedures considering sudden and continuous damage under time-dependent exposure scenarios, there are no widely accepted criteria in literature for a definition and quantitative measure of robustness. Furthermore, civil engineering standards and design codes do not provide methodologies or specifications for robustness quantification. Moreover, several metrics proposed in literature are formulated on the basis of concepts that may be related to structural robustness, like risk, vulnerability, redundancy, residual strength, and damage tolerance, but that in general do not provide robustness measures. However, at the basis of the proposed approaches lies the same general idea: the comparison of the intact and damaged structure by means of structural performance indicators. This procedure can be formulated in a deterministic or probabilistic approach, accounting for uncertainties and consequences including risk quantification. Moreover, robustness evaluations should not be restricted to accidental actions and abnormal loadings. Ageing and deterioration processes, the effects of design, construction and maintenance errors may also lead to disproportionate effects.

In the following, several robustness indicators under both linear elastic and nonlinear behaviour are briefly introduced. Subsequently, structural analysis methods to be used for quantification of performance indicators under damage are presented. Finally, criteria for robustness quantification are formulated.

5.1 Structural performance indicators for robustness assessment

Structural robustness is associated with the ability of the system to avoid consequences that are disproportionate with respect to the extent of the triggering damaging event. According to this definition, both the loss of performance when damage occurs and the amount of damage needs to be considered to achieve meaningful robustness evaluations. Performance indicators formulated as the ratio of a performance parameter of the intact and damaged system are hence used as state variables (Frangopol & Curley 1987, Biondini & Restelli 2008).

The selection of suitable performance indicators represents a critical task and should be based on both the limit state condition and exposure scenario to be investigated. Moreover, in order to effectively describe the effects of damage on the system, structural performance indicators should be able to capture the role played by the damaged members and identify failure conditions and damage propagation.

Strength and ductility, as well as other performance indicators of nonlinear behaviour, may be used in robustness evaluations associated with damage induced by severe loadings, such as explosions or impacts. However, performance indicators of the serviceability conditions under linear behaviour, such as elastic stiffness and first yielding, may become of major importance in life-cycle robustness evaluations associated with aging of structures. In addition, it has been noted that the assumption of linear behaviour can be successfully used in design of robust structures (Powell 2009).

Robustness indicators can be formulated in deterministic terms. However, civil engineering systems are usually characterized by significant uncertainties related to structural modelling, exposure scenario, loading conditions, and failure consequences. Probability-based or reliability-based formulations can be adopted if uncertainties are incorporated in the robustness indicators. Moreover, risk-based robustness indicators can be formulated based on a suitable quantification of the failure consequences.

The robustness indicators presented in the following can effectively be used to relatively compare the system robustness with respect to different damage scenarios and exposures.

However, such indicators need to be complemented with the amount of damage to quantify in absolute terms the disproportion of damage effects and structural robustness.

5.1.1 Deterministic indicators

The effectiveness of several dimensionless performance indicators $0 \leq \rho \leq 1$ related to the structural behaviour of linear systems in evaluating structural robustness is investigated in Biondini & Restelli (2008). Indicators associated with the properties of the structural system only, such as determinant, trace, and condition number of the overall stiffness matrix and first natural vibration period, and indicators depending also on the loading scenario, including stored energy and displacements, are considered. It is found that determinant, trace, and condition number of the stiffness matrix are not suitable to effectively describe the effects of selected damage scenarios on the structural performance and the following indicators are recommended:

$$\rho_T = \frac{T_{n0}}{T_{n1}} \qquad T_n = 2\pi\sqrt{\max_i \vartheta_i(\mathbf{K}^{-1}\mathbf{M})} \tag{5-1}$$

$$\rho_s = \frac{s_0}{s_1} \qquad s = \| \mathbf{s} \| = \| \mathbf{K}^{-1}\mathbf{f} \| \tag{5-2}$$

$$\rho_\Phi = \frac{\Phi_0}{\Phi_1} \qquad \Phi = \frac{1}{2}\mathbf{s}^T\mathbf{K}\mathbf{s} = \frac{1}{2}\mathbf{s}^T\mathbf{f} \tag{5-3}$$

where T_n is the first natural vibration period associated with the stiffness matrix **K** and mass matrix **M**, $\vartheta_i(\mathbf{A})$ denotes the i^{th} eigenvalue of a square matrix **A**, **s** is a displacement vector, **f** is a load vector, Φ is the stored energy, $\| \cdot \|$ denotes the Euclidean scalar norm, and the subscripts "0" and "1" refer to the intact and damaged states, respectively.

Starossek & Haberland (2011) also proposed a stiffness-based robustness indicator based on the minimum determinant of the stiffness matrix after removing a single structural component or connection j, i.e. for the worst-case damage scenario, as follows:

$$R_S = \min_j \frac{\det(\mathbf{K}_j)}{\det(\mathbf{K}_0)} \tag{5-4}$$

Also in this case, the authors reported that the expressiveness of a stiffness-based robustness indicator is not sufficient and recommend the use of the following energy-based indicator:

$$R_e = 1 - \max_j \frac{E_{r,j}}{E_{f,k}} \tag{5-5}$$

where $E_{r,j}$ is the energy released during initial failure of structural element j and contributing to damaging a subsequently affected element k, and $E_{f,k}$ is the energy required for the failure of the subsequentially element k.

For robustness evaluations it is also of interest to define indicators able to simultaneously account for the structural performance of both the intact and damaged system. To this purpose, vectors of nodal forces equivalent to the effects of damage, defined as backward or forward pseudo-loads, are also considered in Biondini & Restelli (2008). The concept of pseudo-load is qualitatively explained in Figure 5-1 based on the linear equilibrium equations of both the intact and damaged systems (Figure 5-1.a):

$$\mathbf{K}_0\mathbf{s}_0 = \mathbf{f}_0 \qquad \mathbf{K}_1\mathbf{s}_1 = \mathbf{f}_1 \tag{5-6}$$

The displacement vector of the intact system $\mathbf{s}_0$ can be related to the displacement vector of the damaged system $\mathbf{s}_1$ and related stored energy variation $\Delta\Phi_1$ as follows:

$$\mathbf{s}_0 = \mathbf{s}_1 + \mathbf{K}_1^{-1}\hat{\mathbf{f}}_1 = \mathbf{K}_1^{-1}(\mathbf{f}_1 + \hat{\mathbf{f}}_1) \tag{5-7a}$$

$$\hat{\mathbf{f}}_1 = (\mathbf{K}_1 - \mathbf{K}_0)\mathbf{s}_0 - (\mathbf{f}_1 - \mathbf{f}_0) = \Delta\mathbf{K}\mathbf{s}_0 - \Delta\mathbf{f} \tag{5-7b}$$

$$\Delta\Phi_0 = \Phi_0 - \hat{\Phi}_1 = \frac{1}{2}\mathbf{s}_0^T\mathbf{f}_0 - \frac{1}{2}\mathbf{s}_0^T(\mathbf{f}_1 + \hat{\mathbf{f}}_1) = -\frac{1}{2}\mathbf{s}_0^T(\hat{\mathbf{f}}_1 + \Delta\mathbf{f}) \tag{5-7c}$$

where $\hat{\mathbf{f}}_1$ is a vector of nodal forces equivalent to the effects of repair, or backward pseudo-load vector, and $\hat{\Phi}_1$ is the stored energy associated with the damaged system after the application of the backward pseudo-loads (area $\mathrm{OP_0\hat{P}_1}$ in Figure 5-1.a for the case $\Delta\mathbf{f}=\mathbf{0}$).

The vector $\hat{\mathbf{f}}_1$ represents the additional nodal forces that must be applied to the damaged system to achieve the nodal displacements of the intact system (Figure 5-1.b).

In a dual way, the displacement vector of the damaged system $\mathbf{s}_1$ can be related to the displacement vector of the intact system $\mathbf{s}_0$ and related stored energy variation $\Delta\Phi_0$ as follows:

$$\mathbf{s}_1 = \mathbf{s}_0 + \mathbf{K}_0^{-1}\hat{\mathbf{f}}_0 = \mathbf{K}_0^{-1}(\mathbf{f}_0 + \hat{\mathbf{f}}_0) \tag{5-8a}$$

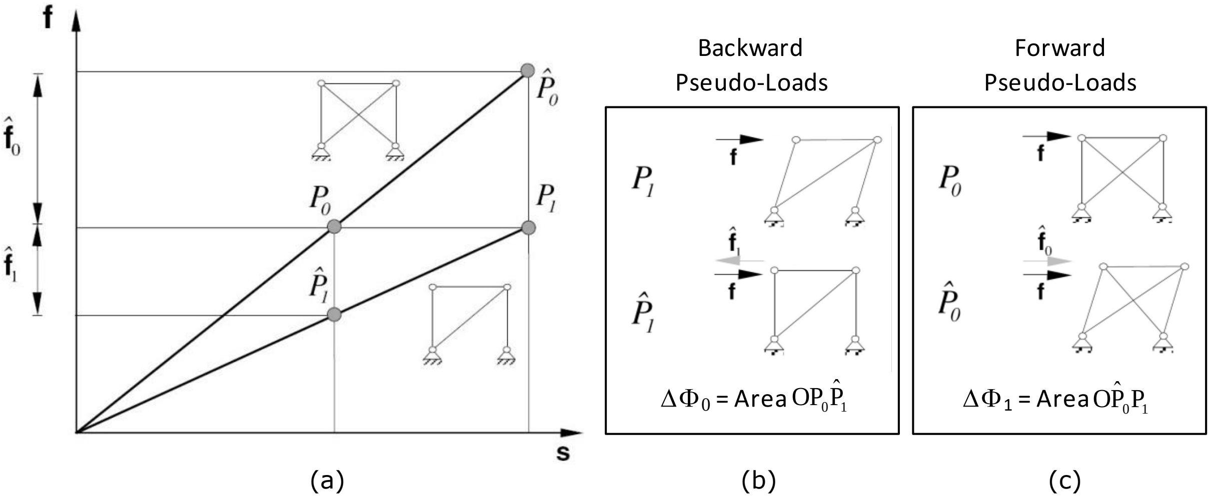
$$\hat{\mathbf{f}}_0 = -(\mathbf{K}_1 - \mathbf{K}_0)\mathbf{s}_1 + (\mathbf{f}_1 - \mathbf{f}_0) = -\Delta\mathbf{K}\mathbf{s}_1 + \Delta\mathbf{f} \tag{5-8b}$$

$$\Delta\Phi_1 = \hat{\Phi}_0 - \Phi_1 = \frac{1}{2}\mathbf{s}_1^T(\mathbf{f}_0 + \hat{\mathbf{f}}_0) - \frac{1}{2}\mathbf{s}_1^T\mathbf{f}_1 = \frac{1}{2}\mathbf{s}_1^T(\hat{\mathbf{f}}_0 - \Delta\mathbf{f}) \tag{5-8c}$$

where $\hat{\mathbf{f}}_0$ is a vector of nodal forces equivalent to the effects of damage, or forward pseudo-load vector, and $\hat{\Phi}_0$ is the stored energy associated with the intact system after the application of the forward pseudo-loads (area $\mathrm{O\hat{P}_0P_1}$ in Figure 5-1.a for the case $\Delta\mathbf{f}=\mathbf{0}$).

The vector $\hat{\mathbf{f}}_0$ represents the additional nodal forces that must be applied to the intact system to achieve the nodal displacements of the damaged system (Figure 5-1.c).

Figure 5-1. Force $\mathbf{f}=\mathbf{f}_0=\mathbf{f}_1$ versus displacement $\mathbf{s}$ of a truss system in the intact state and after elimination of one member. (a) Force-displacement diagrams. (b) Backward pseudo-loads (effects of repair). (c) Forward pseudo-loads (effects of damage).



(a) (b) (c)

Source: Biondini & Restelli, 2008.

It is found that backward pseudo-loads exhibit little sensitivity to damage, particularly under extensive damage. Contrary, forward pseudo-loads can effectively be used for robustness evaluations and the use of the following robustness indicator is preferred:

$$\rho_1 = 1 - \frac{\Delta\Phi_1}{\hat{\Phi}_0} \tag{5-9}$$

Indicators associated with first failure and structural collapse have been also investigated by several authors. The effect of damage on structural performance is quantified in terms of structural redundancy by Frangopol and Curley (1987) based on residual load-carrying capacity as follows:

$$R_L = \frac{L_{intact}}{L_{intact} - L_{damaged}} \tag{5-10}$$

where L_{intact} and $L_{damaged}$ are the collapse loads for the intact and damaged system, respectively. This measure is effective to investigate the reserve of load capacity after damage occurs and the importance of individual structural components.

Ghosn & Moses (1998) formulated a redundancy factor for highway bridges considering the load factors associated with ultimate (u), serviceability (s), and damage (d) limit states. The system reserve factor R_{LF} is defined as follows:

$$R_{u,s,d} = \frac{LF_{u,s,d}}{LF_1} \tag{5-11}$$

where $LF_{u,s,d}$ is the load factor which exceeds a specific limit state $(\mathrm{u,s,d})$ and LF_1 is the load multiplier of failed member. A unit value of the reserve ratio corresponds to a non-redundant system with respect to the failure of the analysed member.

Wisniewski et al. (2006) developed a similar method to evaluate robustness of railway bridges. According to this approach, structural robustness is defined as the ability of the system to continue carrying loads after a member failure and can be quantified using redundancy ratios which compare system and member capacities at the serviceability or ultimate limit state.

Along similar research lines, Maes et al. (2006) proposes the use of a Reserve Strength Ratio (RSR) associated with the load-carrying capacity evaluated in the damaged (RSR_i) and undamaged (RSR_0) state as follows:

$$R_1 = \min_i \frac{RSR_i}{RSR_0} \tag{5-12}$$

where minimization allows to capture the worst case scenario associated with damage of the *i*-th individual structural component.

Indicators associated with first failure and sequential failures up to structural collapse can be found also in Biondini & Frangopol (2014, 2017). Denoting $\lambda \geq 0$ a scalar load multiplier, the limit states associated to the occurrence of a series of sequential failures k=1,2,… can be identified by the corresponding failure load multiplier λ_k. The ability of the system to redistribute the load after the failure $k=i$ up to the failure $k=j$ depends on the reserve load carrying capacity associated to the failure load multipliers $\lambda_i=\lambda_i$ and $\lambda_j=\lambda_j$ and the following quantity can be assumed as a measure of redundancy between subsequent failures:

$$\Lambda_{ij} = \frac{\lambda_j - \lambda_i}{\lambda_j} \tag{5-13}$$

This factor can assume values in the range [0;1]. It is zero when there is no reserve of load capacity between the failures *i* and *j* ($\lambda_i=\lambda_j$), and tends to unity when the failure load capacity λ_i is negligible with respect to λ_j ($\lambda_i<<\lambda_j$). It is worth noting that this definition incorporates the classical measure of redundancy associated with the ability of the system to redistribute the load after the occurrence of the first local failure, reached for $\lambda_i=\lambda_1$, up to structural collapse, reached for a collapse load multiplier $\lambda_j=\lambda_c$:

$$\Lambda = \frac{\lambda_c - \lambda_1}{\lambda_c} \tag{5-14}$$

This redundancy measure is further generalized for deteriorating systems to introduce the concept of failure times and elapsed times between subsequent failures (Biondini 2012).

Progressive collapse of systems is studied by analogy with fast fracture in metals by Smith (2006). The energy released in structural component-loss damage scenarios is compared with the energy absorbed by the damaged members, like in metal cracks propagation. If the energy released is greater than the energy adsorbed, progressive collapse will occur. The proposed methodology also allows to identify critical sequence of damaged members by sequentially removing damaged elements and solving a minimisation process on the damage energy.

André et al. (2015) point out some limitations in the approach proposed by Smith (2006) and proposes an energy-based robustness indicator defined as follows:

$$I_R(A_L|H) = \frac{D_{uc}-D_{1^{st}failure}}{D_c-D_{1^{st}failure}} \tag{5-15}$$

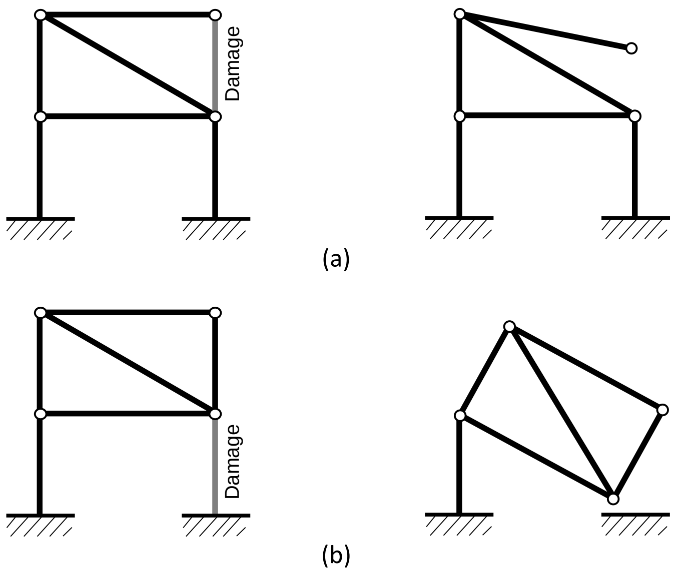
where A_L is the leading action, H is a set of hazard scenarios and $D_{1^{st}failure}$, D_{uc}, and D_c, are the damage energies associated respectively with first failure, "unavoidable collapse" state, and the structural collapse. This indicator allows to capture the role of damage propagation on the value of "unavoidable collapse" energy state and how the latter compares with the potential maximum energy state of the system.

Robustness assessment should involve a quantification of the direct and indirect failure consequences (which is inherently done e.g. in case of applying a risk-based robustness indicator). To this purpose it is important to consider that structural collapse may have different consequences. For example, the global collapse of a whole structural system should be considered more important than the local collapse of a single member or a portion of the structure (Figure 5-2). Despite the evaluation of failure consequences is a challenging task and a reliable estimation is often not straightforward, a relatively simple importance measure of structural failure could be provided by the following structural integrity index (Biondini & Restelli 2008):

$$\rho_V = \frac{V_1}{V_0} \tag{5-16}$$

where V_1 is the portion of structural volume V_0 which remains intact after damage. Failed members involved in a collapse mechanism can be identified based on the eigenvectors $\mathbf{s}_i$ of the stiffness matrix $\mathbf{K}$ associated with the eigenvalues $\vartheta_i(\mathbf{K})=0$.

Figure 5-2. Types of failure (adapted from Biondini & Restelli 2008): (a) local and (b) global collapse.



Source: Biondini & Restelli, 2008.

5.1.2 Probability-based and reliability-based indicators

The robustness indicators previously introduced can be formulated in a probabilistic context to account for the uncertainties related to structural model, exposure scenario, loading conditions, and failure consequences, among others. However, indicators explicitly related to quantification of probability of failure or reliability index have been proposed to directly formulate robustness indicators in probabilistic terms.

Frangopol & Curley (1987) proposed a redundancy measure based on the reliability index β computed with respect to serviceability or ultimate limit state conditions as follows:

$$R_\beta = \frac{\beta_{intact}}{\beta_{intact} - \beta_{damaged}} \tag{5-17}$$

where β_{intact} and $\beta_{damaged}$ are the reliability indices of the intact and damaged systems, respectively.

A measure of the effects of damage evaluated in probabilistic terms was proposed by Lind (1995) based on the concept of structural vulnerability, defined as the insensitivity of the system to damage, as follows:

$$V = \frac{P(r_d)}{P(r_0)} \tag{5-18}$$

where $P(r)$ is the probability of failure in the damaged (r_d) and intact (r_0) state.

Maes et al. (2006) proposes a similar approach by a minimisation aimed at identifying the worst case scenario associated with damage of the *i*-th individual structural component:

$$R_P = \min_i \frac{P_{s0}}{P_{si}} \tag{5-19}$$

where P_{s0} and P_{si} are the probabilities of failure in the intact and damaged states, respectively.

5.1.3 Risk-based indicators

Risk-based robustness assessment incorporates the quantification of failure consequences. Baker et al. (2008) developed a risk-based approach where consequences are separated into two contributions: Direct consequences associated to the damage of elements directly affected by the hazardous event, and indirect consequences associated with the subsequent partial or total system failure. Risk is computed by the product of probability of occurrence of disproportionate collapse and corresponding consequences. The quantification of consequences may also include both monetary and human losses. Considering that a lower amount of indirect risk indicates a more robust system, the following indicator is proposed:

$$I_R = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \tag{5-20}$$

where R_{Dir} and R_{Ind} are the direct and indirect risks, respectively, associated with a specific exposure scenario. If the system failure does not involve indirect consequences, then $I_R = 1$. On the other hand, $I_R = 0$ if direct consequences are negligible compared with indirect consequences. This approach can be extended to account for multiple exposure scenarios:

$$I_R = \frac{\sum_i R_{Dir}}{\sum_i R_{Dir} + \sum_j R_{Ind}} \tag{5-21}$$

where the risks for each scenario *i* need to be computed.

5.2 Structural analysis methods for robustness assessment

5.2.1 General

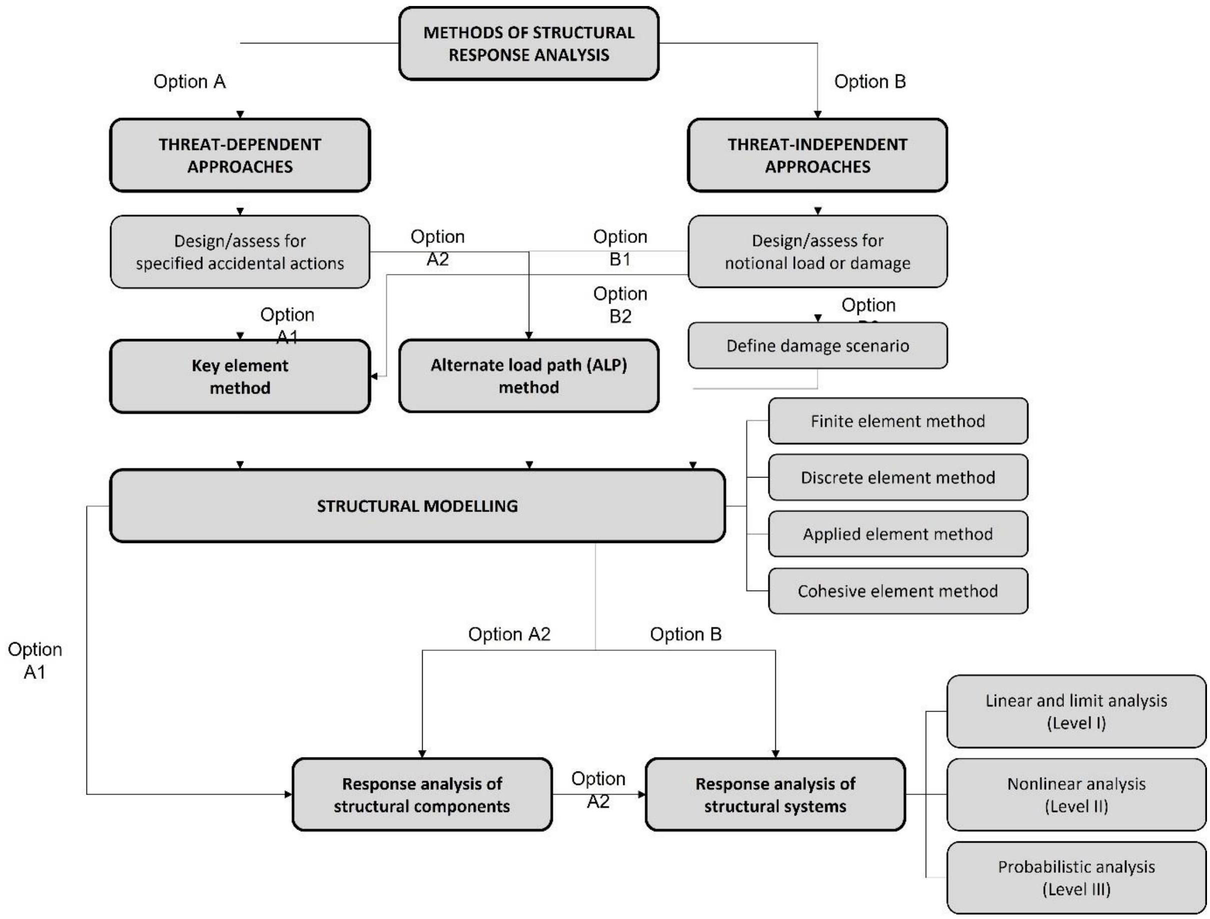
Quantification of progressive collapse resistance and robustness relies upon the analysis of structural response to certain design scenarios, which should be defined according to building codes, authorities, and stakeholders. In line with Sect. 4.4, each scenario consists of either single or multiple damaging events/conditions that may occur in the structure's lifetime, producing the need to assess possible disproportionate consequences. In threat-dependent design/assessment approaches, scenarios are delineated in the form of specified accidental actions (e.g. fire, blast, impact or notional loads) and/or deterioration processes (i.e. corrosion, fatigue, among others). These threats can be modelled according to technical documents (e.g. building codes, standards, guidelines, and reports), scientific literature, or ad-hoc investigations committed for the structure under study (as in the case of, e.g., critical infrastructure). In threat-independent approaches, scenarios are defined in terms of notional damage applied to single or a few components of the structure to assess robustness and/or investigate progressive collapse (Biondini and Restelli, 2008; Parisi and Scalvenzi, 2020). The latter is thus evaluated as the insensitivity of the structure to such scenarios (see e.g. Sect. 2.1.3). Both threat-dependent and threat-independent approaches are usually included in the general category of direct design methods, which are aimed at explicitly evaluating the performance of the structural system to redistribute loads in case of local damage and to avoid disproportionate collapse.

Several codes and guidelines include two types of direct design/assessment approaches, namely the key element method and alternative load path (ALP) method (see e.g. Sects. 3.1, 3.2, 4.5.2 and 4.5.5). The former method focuses on the capacity of single components to withstand abnormal loads, which for instance is a major situation for peripheral columns of frame buildings due to their high exposure. Hence, the implementation of the key element method requires modelling and analysis of selected structural components. By contrast, the ALP method is based on the response analysis of the structure under either specified abnormal actions or notional damage applied to some components (e.g. Parisi and Augenti, 2012).

Regardless of the approach selected for robustness quantification and/or progressive collapse analysis, the numerical analysis of the structure under damage scenarios is a key issue to address and is discussed in the following sub-sections. In this respect, it should be noted that robustness quantification is different from progressive collapse analysis. Robustness analysis is aimed at comparing the loss of performance with the amount of damage to evaluate the ability to avoid disproportionate consequences under prescribed damage scenarios. Progressive collapse analysis investigates how damage will propapate leading to structural collapse. Linear and limit analysis methods allow to investigate the system performance in the elastic stage and at collapse and can be effectively used for robustness quantification (Biondini & Restelli 2008, Biondini 2009, Powell 2009). Conversely, progressive collapse analysis usually requires nonlinear analysis methods because it is aimed at evaluating the propagation of damage throughout the structure. Based on such considerations, Sects. 5.2.4, 5.2.5 and 5.2.6 describe different methods of structural analysis according to an increasing level of complexity, moving from Level I methods (involving linear and limit analyses), through Level II methods (including nonlinear analysis procedures), and to Level III methods (i.e. probabilistic analyses). Such an organization of this section is in line with most of building codes and guidelines at both national and international levels, which allow several methods for response analysis of structural systems, i.e. linear or nonlinear, static or dynamic analyses. In some cases, easier calculation methods for structural modelling and response analysis can be used, i.e., avoiding more complex computations through numerical approaches.

Figure 5-3 shows a general overview of the most typical methods for structural response analysis and robustness quantification under either specified accidental actions (threat-dependent approaches; option A) or notional damage (threat-independent approaches; option B). In case the structure is designed or assessed for specified accidental actions, one can choose between key element design (option A1) or ALP analysis (option A2). Conversely, design and assessment for notional damage can be followed by the application of the ALP method (option B1) or directly lead to a basic framework for structural analysis (option B2). Whichever option is selected, the structural engineer has to choose appropriate methods for structural analysis. Sect. 5.2.2 briefly describe the following methods: finite element method; discrete element method; applied element method; or cohesive element method. If option A1 is selected, then numerical modelling focuses on single components and can be carried out according to several computational strategies, as discussed in Sect. 5.2.3. If option A2 is selected, the ALP method starts with numerical analysis of single structural components, then moving to the analysis of the pre-damaged structural system via one of the methods described in Sects. 5.2.4, 5.2.5 and 5.2.6. In case of design/assessment for notional damage (option B), the response analysis of the structural system is directly performed without analysing individual components.

Figure 5-3. Overview of structural analysis methods for progressive collapse analysis and robustness quantification.



5.2.2 Numerical modelling strategies

The progress in computer technology and structural software has given rise to a number of computational strategies that allow researchers and practitioners to assess complex phenomena involving material nonlinearities, fragmentation, impact, large deformations, and dynamics. Nonetheless, the level of sophistication should be accurately managed depending on the type of structure, design scenarios and their potential consequences. For instance, in progressive collapse simulation, combining the outputs of linear static response analysis and rigid-plastic (or limit equilibrium) analysis can provide a computationally efficient support to robustness quantification, according to indicators presented in Sect. 5.1. By contrast, more advanced, nonlinear static/dynamic analysis methods – which require high computational expertise – are needed particularly when more complex cases (e.g. the assessment of existing structures) must be investigated.

According to recent literature reviews (El-Tawil et al., 2014; Adam et al., 2018; Kunnath et al., 2018), there are several numerical modelling strategies that have been successfully validated through experimental tests at different structural scales (i.e. component, sub-system, system). This allows realistic simulations of structural behaviour, saving cost and time for further experimental testing.

Most of modelling strategies are based on the finite element method (FEM), which is implemented with various degrees of approximation and complexity. FEM allows several 2D and 3D structural models to be developed, such as micro-models, macro-models, and hybrid models. It is also noted that FEM allows almost any type of structural response analysis, i.e. linear or nonlinear, static or dynamic, implicit or explicit, depending on the expected accuracy level. Micro-modelling approaches are feasible for relatively small-scale problems related to single structural components or sub-systems, whereas macro-models are used to analyse whole structures. In the framework of micro-models, solid (brick) elements can be used for detailed geometric representations and structural simulations where different materials (e.g. concrete, steel rebar, welds, bolts) and their mutual interaction (e.g. steel-concrete bond) is explicitly considered. Studies based on micro-modelling FEM procedures were carried out by, amongst others, the following researchers: Sadek et al. (2011) and Guo et al. (2013) for steel and composite structures; Shi et al. (2011), Li et al. (2016) and Pham et al. (2017) for cast-in-place RC structures; and Elsanadedy et al. (2017) for precast RC structures. Macro-models consist of beam/shell elements, so their lower computational cost allows the progressive collapse analysis of entire structures under either specified abnormal actions and notional scenarios (e.g. Izzuddin et al., 2008; Fu et al., 2011; Parisi and Augenti, 2012; Kazemi-Moghaddam and Sasani, 2015; Lu et al., 2017; Feng et al., 2021). In that context, beam-type elements are normally used to model columns and beams, sometimes according to fibre-based modelling approaches (e.g. Brunesi and Nascimbene, 2014; Mucedero et al., 2020). Effective beam finite element models have been also proposed to incorporate the effects of aging and deterioration processes, such as reinforcement corrosion in concrete structures under static and seismic loadings (Biondini et al. 2013, Biondini & Vergani 2015). Shell elements are utilised to simulate floor slabs and thin-walled steel components. The relative simplicity of macro-models makes them the most used computational technique in engineering computations for progressive collapse analysis and robustness assessment. Nonetheless, higher complexity levels may arise from numerical simulation of beam-column and slab-beam joints, particularly in existing RC frame structures not designed for earthquake resistance (see e.g. Kunnath et al., 2018), precast RC frame structures (e.g. Ravasini et al., 2021), as well as steel and composite structures. Those parts of structures mobilise the interaction between large-deformation resisting mechanisms such as catenary action of beams, membrane action of slabs, and axial-shear-flexure interaction. Beam-column macro-models consisting of spring and rigid elements that are connected to beam/shell elements were proposed by, e.g., Bao et al. (2008), Sadek et al. (2008), Liu et al. (2010), Khandelwal and El-Tawil (2011), Jahromi et al. (2013), and Sun et al. (2015). In some studies, different types of finite elements were combined with each other to capture both local and global behavioural features of structures, delineating hybrid (or multi-scale) models (e.g. Alashker et al., 2011, El Hajj Diab et al., 2021, 2022). Such models significantly reduce the number of elements and computational cost, even though kinematic compatibility between different elements is a critical issue to take into account.

In recent years, progressive collapse simulations have been carried out using the discrete element method (DEM), which can be also combined with FEM to obtain accurate results in a computationally efficient manner. DEM was originally developed to model granular and discontinuous materials particularly in rock mechanics, but it was recently extended to masonry structures and other types of engineering facilities. The interaction between discrete elements is modelled through contact relationships, allowing the dynamic behaviour of the material at the macro-scale to be derived from normal, tangent, rolling and twisting motion of the interacting elements. The computational cost of DEM-based numerical models is usually high, especially in case of structural sub-systems and systems (e.g. Pekau and Cui, 2006; Masoero et al., 2010; Gu et al., 2014), but the use of discrete elements coupled with finite elements can allow successful simulations accounting for material fragmentation and impact (e.g. Lu et al., 2009).

Besides FEM and DEM, some advanced software packages have been developed by implementing the applied element method (AEM) originally proposed by Tagel-Din and Meguro (2000). The use of AEM is rapidly increasing for high-fidelity, progressive collapse simulations of structural sub-systems and entire structures. This numerical method is based on the modelling of the structure as an assembly of relatively small elements, which are connected to each other at their contact (boundary) points through normal and shear springs. The presence of discontinuities between applied elements allows realistic simulations of cracking, separation, and collision of structural components up to collapse. Latest developments in high-performance computing nowadays allow a moderate level of computational cost for progressive collapse simulation, with successful application to both structural sub-assemblages and entire structures (e.g. Dinu et al., 2016; Salem and Helmy, 2014; Salem et al., 2016; Khalil, 2012).

Another modelling strategy for progressive collapse analysis makes use of the cohesive element method (CEM), which is widely effective for fracture mechanics problems. CEM essentially smears a finite, potential damage zone by means of a zero-thickness nonlinear element, while modelling undamaged parts via linear elastic elements. Nonlinear cohesive elements are thus used to lump nonlinear behaviour of structural components where damage is expected to occur during progressive collapse (Keyvani and Sasani, 2015). Le and Xue (2014) used CEM to investigate the progressive collapse resistance of a thirty-storey RC building under several column removal scenarios. The same researchers developed a simplified method based on a two-scale model (Xue and Le, 2016a), which was experimentally validated and applied to a ten-storey building (Xue and Le, 2016b).

Finally, in a context of design for robustness, several simplified analytical methods capable of predicting the ultimate load and the collapse mode following column removal were proposed in the literature. Due to their simplicity, analytical methods are particularly suitable for conducting parametric analyses. In the case of multi-story RC frame with RC flat slabs, the analytical method proposed by Martinelli et al. (2022) provides the ultimate bearing capacity and the failure modes of a large variety of flat slabs. The analytical method provides, in the form of design nomographs, the maximum load in accidental design situation.

It should be emphasized that in relation to the application of numerical modelling strategies further investigations are required in relation to the use of partial factors, the selection of characteristic and representative values and load combinations.

5.2.3 Response of single components to abnormal loads

Structural robustness and progressive collapse should be investigated at the system level. However, to this purpose, the response of single structural components to abnormal loads has to be carefully modelled and analysed. This consideration applies to both key element design and threat-dependent ALP analysis, where critical structural members (i.e. those components the failure of which may activate progressive collapse) are analysed under either specified or notional actions. Therefore, as outlined in several studies (e.g. Parisi and Augenti, 2012), a local response analysis is carried out using either a linear or nonlinear, static or dynamic analysis. Particularly under impulsive loading (associated with, e.g., blast or impact), each single structural member can be assumed to be doubly-fixed at its ends and can be ideally taken out from the entire structure because the latter does not have sufficient time to develop global vibration modes, damping and inertia forces. In line of principle, this assumption is met if the duration of loading is significantly lower than the fundamental vibration period of the structure. Strain-rate effects can be explicitly considered in dynamic analysis, provided that the structural software allows the user to model strain-rate-dependent constitutive behaviour of materials. Otherwise, strain-rate effects (particularly sensitivity of material strengths) can be indirectly incorporated in the response analysis (be it static or dynamic) through dynamic increase factors (e.g. CEB, 1988; Malvar, 1999; Malvar and Ross, 1999). Even though various studies considered strain-rate effects in subsequent response analysis of the residual structure, i.e. after damage/removal of single/few members (e.g. Jayasooriya et al., 2012), they do not usually influence the robustness of the structure.

If linear static analysis of a single structural member is carried out, strength demand in terms of bending moment and shear force is simply compared to capacity. In this regard, shear-flexure interaction should be taken in due consideration particularly in existing structures not designed for earthquake resistance. Using nonlinear incremental static analysis, the structural member is assumed to reach collapse if a plastic mechanism occurs under the specified/notional load.

Linear or nonlinear dynamic analysis can be performed on one of the following capacity models of the structural component (e.g. column, beam, slab, wall):

(1) a continuum model developed according to FEM (most used method) or other numerical strategies presented in Sect. 5.2.2, or

(2) a single-degree-of-freedom (SDOF) system, which is a model recognised in guidelines (e.g. ASCE, 1997, 2008; UFC 3-340-02, 2008) and widely investigated in several studies (Nassr et al. 2012; Urgessa and Maji 2010; Wang et al. 2012).

In case (1), a doubly-fixed beam element with constant bending stiffness, distributed mass, and zero or very low damping is widely used (e.g. Parisi and Augenti, 2012). Such analysis then provides the motion of the dynamic system in terms of displacement, velocity, and acceleration time histories, as well as strength and deformation demands to be compared with capacity. In case of linear dynamic analysis, closed-form solutions are readily available in the literature to predict the effects of different input time histories. Thus, even linear dynamic analysis of continuum models can be sometimes sufficient to assess the performance of single structural components, avoiding complex modelling procedures and computations. This may be the case of, e.g., blast loading on building columns, which can be basically analysed under triangular impulse uniformly distributed over the column height, with peak overpressure and positive duration predicted through simplified analytical models available in the literature (Parisi and Augenti, 2012). Otherwise, nonlinear dynamic analysis of SDOF systems is a computationally efficient tool (e.g. Dragos and Wu, 2013, 2015), but it may be affected by some limitations such as the accurate prediction of shear and mixed failure modes of RC columns (e.g. Shi et al., 2008).

Based on such considerations, several researchers proposed pressure–impulse diagrams to assess performance and damage of single structural members under different assumptions regarding dynamic loading and structural behaviour (e.g. Fallah and Louca, 2007; Krauthammer et al., 2008; Shi et al., 2008; Park and Krauthammer, 2011; Ding et al., 2013). The impulse is defined as the integral of pressure over the duration of the pressure-time history. The pressure–impulse diagram is an iso-damage capacity curve of the structural member defined by the combinations of pressure and impulse that produce the same level of damage. Structural damage can be quantified through a scalar or vector-valued measure, such as the percentage loss of axial load-bearing capacity of a column (e.g. 20%, 50%, 80%). This highlights the need to define performance limit states corresponding to increasing levels of structural damage. Therefore, the pressure–impulse diagram is the boundary line of an iso-damage safety domain, meaning that dynamic loads with pressure-impulse combinations falling below the pressure–impulse diagram do not cause failure, and hence the prescribed level of damage considered in the performance assessment. Pressure–impulse diagrams allow all types of failures to be considered, so they are widely and easily used in research and engineering practice. Latest developments have also produced probabilistic pressure–impulse diagrams for their use in performance-based design and assessment of structural members (e.g. Parisi, 2015), as well as in quantitative risk analysis of structures for robustness quantification.

Dealing with EC8-conforming RC frame structures subjected to blast scenarios, Parisi and Augenti (2012) found that static, dynamic, and pressure–impulse analyses produce the same safe/failed tagging of columns, delineating the same local damage scenarios for subsequent global analysis of the structure. By contrast, static analysis produced too conservative outcomes for buildings designed only to gravity loads, hence significantly overestimating the number of failed elements under the same loading conditions.

The following sub-sections deal with the response of structural systems, which can be analysed under either specified abnormal loads or notional scenarios. This is because robustness assessment is not necessarily related to abnormal loads and structural collapse, allowing the use of Level I methods in case of robustness quantification (see Sect. 5.2.4). More sophisticated methods of structural response analysis can be applied to progressive collapse analysis of structures, as described in Sects. 5.2.5 and 5.2.6.

It should be noted that in case of key element design, pushdown type analysis will not necessarily highlight failures where load reversal occurs, e.g. in case of gas explosions.

Table 5-1 outlines the main advantages and drawbacks of the different methods, which will be described in detail in the next sub-sections. That table particularly focuses on the ease/complexity of structural modelling and computational cost, associated with lacking/implicit/explicit consideration of material nonlinearities, second-order effects, and dynamic effects (e.g. load amplification, inertia forces, damping forces).

Table 5-1. Some advantages and drawbacks of different methods of structural response analysis

Method	**Advantages**	**Drawbacks**
Linear static analysis	Ease of structural modelling Lowest computational cost Simple safety checking	Implicit consideration of dynamic effects through a dynamic amplification factor Lacking consideration of both material and geometric nonlinearities Performance limit states to be properly defined to ensure representativeness for robustness quantification
Linear dynamic analysis	Consideration of dynamic effects (including dynamic load amplification, inertia forces, and damping forces) Moderate complexity of time-history analysis Moderate simplicity of safety checking	Lacking consideration of both material and geometric nonlinearities Potential incorrect evaluation of dynamic effects in case of structures with large inelastic deformations Higher computational cost for large/complex structures Performance limit states to be properly defined to ensure conservative safety checks
Limit analysis	Ease of structural modelling Moderate computational cost Simplified modelling of material nonlinearities Effective prediction of collapse mechanisms	Lacking consideration of second-order effects, to be included through specific procedures Threshold load factor to be properly defined considering dynamic loading conditions
Nonlinear static analysis	Moderate ease of structural modelling Moderate computational cost	Inelastic capacity of structural members to be properly defined

	Consideration of material and geometric nonlinearities Satisfactory prediction of collapse mechanisms	Dynamic amplification factor to define based on the expected level of inelastic deformations
Nonlinear dynamic analysis	Consideration of material and geometric nonlinearities Explicit evaluation of dynamic effects Realistic prediction of nonlinear dynamic response and collapse mechanisms	Sophistication of structural modelling (including damping and inertia masses) High computational cost
Probabilistic analysis	Explicit consideration of uncertainties Rational assessment of progressive collapse resistance and robustness Possible use of analysis output in decision-making for disaster risk mitigation	Highest computational cost

5.2.4 Linear and limit analysis methods for structural systems (Level I)

The structural response to different hazardous scenarios can be investigated through either linear or limit analysis methods, which are computationally efficient and easy to use in engineering practice. These methods can effectively be used for robustness quantification. Indeed, response of single components plays a role when assessing progressive collapse against abnormal loads, but it is not required in other hazardous scenarios.

Linear analysis procedures are based upon the assumption of linear elastic behaviour of materials, provided that their mechanical strength is properly scaled down to a design value via partial safety factors that account for both material and model uncertainties. Structural response can be evaluated either statically or dynamically, in the former case using load amplification factors to define equivalent static loads. This approach can extremely useful to compare the structural response to different damage scenarios and, therefore, to quantify structural robustness (Powell 2009).

Linear static analysis (LSA) procedures require the assumption of dynamic amplification factors for gravity loads, the definition of which can be a critical issue depending on the expected level of ductility demand on structural components and the availability of specific formulations for the structure under consideration. In robustness quantification and progressive collapse analysis, dynamic amplification factors are applied to gravity loads in structural components (e.g. frame members, floor systems) that are directly involved in the dynamic response of the structure to damage. In the frequent case of buildings for which robustness to notional removal of single or few structural components (e.g. columns, walls) has to be evaluated, dynamic amplification factors are applied to increase the intensity of gravity loads in floor areas above the removed components. This is because inertia masses in those floor areas are accelerated by the sudden failure of those structural elements, and the amount of vertical acceleration those masses are subjected to depends on the level of inelasticity that develops within the structure (particularly in beams and floor systems). Accordingly, Brunesi and Parisi (2017) derived a set of regression models for both gravity-load designed and earthquake-resistance RC frame buildings designed according to Eurocodes, because no formulation is still available in those codes for European structures. The same applies to other structural types, as pointed out by Mucedero et al. (2021).

As pointed out by Marjanishvili (2004), LSA is the simplest method of structural response analysis and it usually leads to conservative predictions of strength demand on structural elements. The advantages of LSA include relative simplicity of structural modelling, quick calculations for structural response analysis, and ease of safety checking, the latter involving force-based verifications of structural components such as beams in frame structures. By contrast, LSA has several disadvantages that include the lacking consideration of both material and geometric nonlinearities, as well as dynamic effects such as inertia forces and damping. Therefore, LSA can be a very useful tool for simple structures with predictable behaviour, while calling for careful use on complex and large structures that should be evaluated with nonlinear analysis methods.

Linear dynamic analysis (LDA) allows consideration of dynamic effects, while material and geometric nonlinearity effects can be indirectly considered. LDA is typically carried out when assessing structural robustness under sudden loss of major load-bearing elements, which produces dynamic motion of the structural system. More specifically, a time-history analysis of the structure is performed, for instance using direct integration methods for solving the equations of motion. LDA is deemed more accurate than its LSA counterpart because it accounts for dynamic amplification, inertia forces, and damping forces, while ensuring a moderate complexity of calculations. In addition to the lacking consideration of nonlinearity effects in the modelling and analysis of the structure, LDA has the disadvantage of being more time consuming for large structures. It is also noted that dynamic amplification and both inertia and damping forces may be incorrectly evaluated for structures that develop large inelastic deformations. Force-based safety checks are performed, as observed in the case of LSA. Maximum strength demands (in terms of internal forces such as bending moments and shear forces in frame members) are evaluated over the entire duration of the structural analysis. Performance evaluation criteria can be assumed to be rather conservative for structures with nearly elastic behaviour, while they could become non-conservative in the case of structures that are expected to experience large inelastic deformations. Marjanishvili and Agnew (2006) showed that dynamic analysis procedures not only yield more accurate response predictions but allow ease of use in progressive collapse assessment. Nonetheless, performance limit states for linear analysis should be properly defined to avoid unconservative safety checks and robustness quantification. Otherwise, a structure that meets performance criteria in linear analysis may exceed performance limits in nonlinear dynamic analysis.

In addition to linear elastic analysis methods, the structural behaviour can be evaluated by means of limit analysis (LA) that makes use of rigid-plastic models. This modelling strategy is relatively simple to use in several software packages, even allowing hand calculations for relatively simple structures. LEA can consist of plastic analysis (e.g.: Watwood, 1979; Corotis and Nafday, 1990), which searches for the load factor on the applied loads for which the following requirements are met: equilibrium equations are satisfied (static condition), and a sufficient number of plastic hinges are formed in the structure to activate a partial or total collapse mechanism (kinematic condition). For instance, in the case of a frame structure, nonlinear behaviour of the structure is lumped at the ends of each frame member, which is assumed to develop its ultimate bending moment accounting for its interaction with axial and shear forces. It could be shown that LA turns out to be a linear optimization programming problem, the objective of which is to minimise the load factor (e.g. Grierson and Gladwell, 1971). This optimization problem can be solved through a simplex algorithm, assuming independent mechanisms. In the case of a frame structure, these collapse mechanisms can be soft-storey mechanisms, beam mechanisms, or joint mechanisms. Whilst in static loading conditions the structure is assumed to collapse when the load factor is less than or equal to 1, in dynamic loading conditions the threshold load factor can be conservatively set to 2. Ruth et al. (2006) found that the actual load factor associated with structural instability under gravity loads is between 1 and 2. Limit analysis can easily incorporate the effects of aging and deterioration processes and be applied to assess the time-variant load capacity of deteriorating systems (Biondini & Frangopol 2008). Second-order internal forces that can prevent the activation of a collapse mechanism can be taken into account in LA as shown e.g. by Park and Gamble (2000) for arching in axially restrained reinforced concrete elements and by Sawczuk and Winnicki (1965) or Herraiz and Vogel (2016) for tensile membrane action in reinforced concrete slabs.

5.2.5 Nonlinear analysis methods for structural systems (Level II)

If a progressive collapse analysis of the structure has to be performed, a nonlinear analysis method is required. In such a context, the capacity model of the structural system should be based on realistic assumptions regarding, e.g., rotational capacity of plastic hinges or ultimate strains of individual materials depending on whether a lumped or spread plasticity approach is used. This motivates the upper-bound values prescribed by some building codes and guidelines for rotational capacity of plastic hinges in different types of structures (e.g. steel frames, concrete frames). It is evident that too large capacity values may result in unrealistic, non-conservative design/assessment solutions for progressive collapse resistance, such as: (i) small tying systems associated with too large rotational capacity values assigned to beams (in indirect design methods); and (ii) very low values of dynamic amplification factors for gravity loads on floor areas above locally damaged/removed elements (in direct design methods based on static response analysis). Another remark involves the role of strain rate effects, which do not typically play a key role when assessing the progressive collapse resistance of the structure, as opposed to the case of single components subjected to impulsive loading (e.g. impact, bomb detonation). Lastly, it should be emphasized that not all detailed material properties are specified in commonly available product specifications.

Nonlinear incremental static (pushdown) analysis, such as the one proposed by Izzuddin et al. (2008) in their ductility-based robustness assessment methodology, was recognised to be a powerful tool and was applied to several types of structures (e.g. Vlassis et al., 2008; Khandelwal and E-Tawil, 2011; Mucedero et al., 2021). De Biagi et al. (2017) developed a pushdown-based method where structural robustness can be evaluated under progressive damage to structural members of RC structures. This allows effects of degradation to be incorporated in robustness assessment. Based on pushdown analysis, the robustness of the structure subjected to local damage can be quantified starting from the ultimate load factor of gravity loads. It is recalled that static analysis procedures require the assumption of dynamic amplification factors for gravity loads, as noted in Sect. 5.2.4. Alternatively, in order to avoid the use of dynamic amplification factors or dynamic analyses, the energy-based method was developed, which allows to derive a dynamic capacity curve from a static (pushdown) analysis (Xu and Ellingwood, 2011).

As discussed by, e.g., Arup (2011) and Byfield et al. (2014), nonlinear time history analysis has the highest level of complexity, but it explicitly visualises all features of the structural behaviour. Implicit solution algorithms in nonlinear time history analysis are generally affected by convergence issues that can be usually overcome by explicit algorithms, resulting in more robust and accurate computations. Brunesi et al. (2015) proposed and implemented incremental dynamic analysis (IDA) for robustness assessment of structures, where inertia masses associated with gravity loads are gradually increased to assess whether progressive collapse occurs or not. To that aim and based on IDA, Parisi et al. (2019) defined a set of performance limit states for progressive collapse analysis, which could be considered in ALP methods for robustness assessment of RC structures. Depending on the performance level under consideration (from slight damage to near collapse), structure response in the range of large deformations is sensitive to different properties (e.g. concrete compressive strength, steel yield strength, beam span length, ultimate/fracture steel strain). This highlights that the accuracy level of capacity modelling should be defined on the basis of the target performance to assess.

5.2.6 Probabilistic analysis methods for structural systems (Level III)

The use of different nonlinear analysis methods, design provisions (e.g. seismic or non-seismic design criteria and detailing), and capacity models (e.g. 2D or 3D) can produce different predictions on the same structure. This applies to the response analysis of both structural components and entire structures, resulting in different quantifications of element resistance and system robustness.

The structure can be analysed accounting for different sources of uncertainty, such as those involving material properties, geometric parameters, loads, capacity modelling, exposure scenario and deterioration processes. In that context, the progressive collapse resistance and robustness of a structure can be quantified through simulation procedures such as the Monte Carlo method. This allows the structural engineer to assess the probability of failure, which should not be greater than a target probability. In threat-independent scenarios (see e.g. Parisi, 2015), the collapse fragility of the structure can be decomposed in the fragility of single components (i.e. the conditional probability of component failure given hazard) and the progressive collapse fragility of the structural system (i.e. the conditional probability of system failure given component damage). Comparing the failure probability of the structural system to that of single components allows robustness quantification through the computation of probability-based measures (see Sect. 5.1.2). Furthermore, (conditional) risk-based measures can be efficiently calculated using efficient simulation procedures such as Latin Hypercube sampling, as e.g. shown in (Droogné et al., 2018) where the structure is decomposed in two parts to further reduce the computational cost of such analyses.

Hence, uncertainties affect structural response at both component and system scales. Parisi et al. (2015) developed fragility surfaces of RC columns subjected to blast loading, using a pressure–impulse formulation proposed by Shi et al. (2008). Each fragility surface provides the conditional probability of exceeding a prescribed level of damage (e.g. 20% loss of axial load-bearing capacity) given the peak overpressure and impulse of blast load. Uncertainty in material strengths, column dimensions, reinforcement ratios, and blast capacity model were modelled and propagated. Horizontal sections of fragility surfaces at different probability levels produced performance-based pressure–impulse diagrams for their use in research and engineering practice. Both fragility surfaces and pressure–impulse diagrams demonstrated that RC columns of earthquake-resistant buildings have high resistance to blast loading. Fragility surfaces of RC columns were more recently used to evaluate the risk-targeted safety distance of RC frame buildings from natural-gas pipelines (Russo and Parisi, 2016) and assess potential damage due to hydrogen pipeline explosions (Russo et al., 2019).

Brunesi et al. (2015) derived fragility functions for European, low-rise, RC framed buildings representative of both gravity-load and seismically designed structures according to Eurocodes. Fragility functions corresponding to multiple damage states indicated a significant influence of seismic design/detailing on robustness of RC buildings. Structural robustness was thus probabilistically evaluated through fragility functions, which provide the conditional probability of exceeding a damage level given the magnitude of gravity loads. The inclusion of secondary beams in the capacity model of the structure also played a significant role, notably increasing the building robustness because of the increased capability of redistributing loads from areas directly involved in progressive collapse mechanisms.

The influence of uncertainties in robustness quantification has been investigated also in Biondini and Frangopol (2014), where a simulation-based probabilistic analysis is applied to time-variant robustness of concrete structures under reinforcement corrosion.

Botte et al. (2021) determined the sensitivity of the development of arching or catenary actions in RC elements and showed that variables which are usually not explicitly taken into account by traditional design methods, such as the axial restraint stiffness and ultimate reinforcement strain have a significant influence on the resistance of such elements.

Such studies further remarked that the accuracy level of capacity modelling and response analysis methods has significant impact on safety assessment and robustness quantification. It is also emphasised that effects of degradation may also notably affect the output of fragility analysis, but this point needs to be investigated to provide detailed information and to draw some conclusions.

5.3 Damage-based robustness quantification

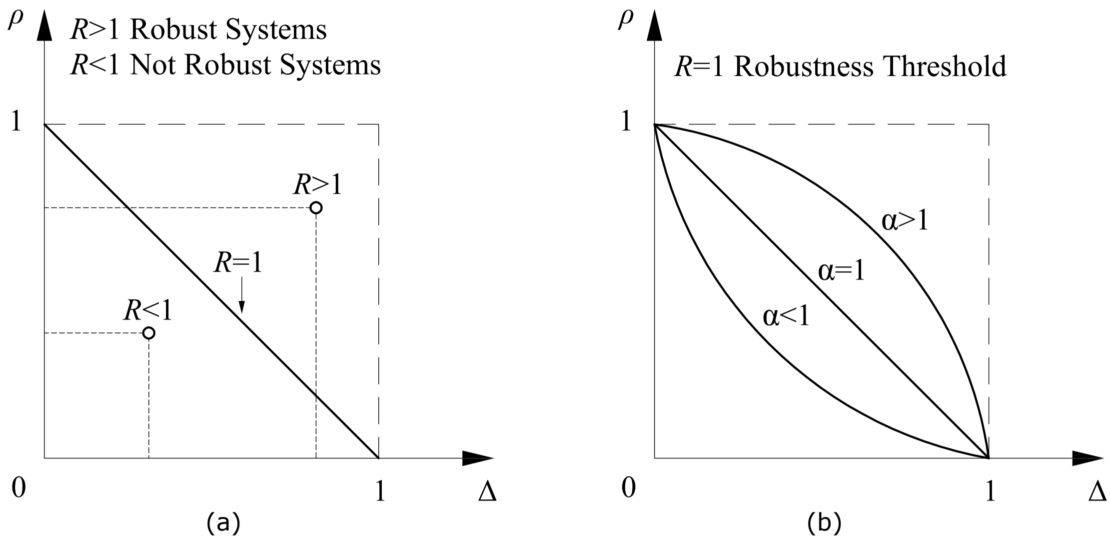
Several robustness indicators have been presented that express the variation of system performance induced by initial damage compared with the intact system in terms of risk, vulnerability, redundancy, residual strength, and damage tolerance. These robustness indicators can effectively be used to compare in a relative way the system robustness with respect to different damage scenarios and exposures. However, it is worth noting that such indicators are, in general, not sufficient to enable comparisons between different structural systems or ranking of different design alternatives. To this purpose, the residual performance indicator $0 \leq \rho \leq 1$ associated with the damage scenario should be related with the amount of damage $0 \leq \Delta \leq 1$ to provide a functional $\rho = \rho(\Delta)$. Performance indicators ρ can be referred to serviceability or ultimate limit states and the damage index Δ should be computed taking into account the spatial distribution of damage and the role of different component materials in non-homogeneous systems, as already discussed. Depending on the purpose of the robustness analysis, this procedure can be applied using a deterministic approach or taking uncertainties and consequences into account with probability-based, reliability-based, and risk-based approaches.

The following criterion has been proposed in Biondini (2009) to quantify structural robustness:

$$R(\rho, \Delta) = \rho^{\alpha} + \Delta^{\alpha} \geq 1 \qquad (5\text{-}22)$$

where $R = R(\rho, \Delta)$ is a robustness factor, and α is a shape parameter of the boundary $R(\rho, \Delta) = 1$. The structural system is robust when the criterion is satisfied ($R > 1$), and not robust otherwise ($R < 1$). This concept is illustrated in Figure 5-4.a for the case $\alpha = 1$. As shown in Figure 5-4.b, the value of the parameter α can be properly selected according to the acceptable level of damage susceptibility for the structure under investigation. A value $\alpha = 1$, which indicates a proportionality between acceptable loss of performance and damage, should be appropriate in most cases. Values $\alpha < 1$ should be avoided since allow for disproportionate damage effects. On the other hand, values $\alpha > 1$ can be required for structures of strategic importance.

Figure 5-4. Robustness factor $R = R(\rho, \Delta)$. (a) Performance ρ vs damage Δ state diagram ($\alpha = 1$); (b) Role of the shape parameter α on the robustness threshold $R = 1$.
(Biondini and Frangopol, 2014)



A proper value of the parameter α can be selected based on a threshold value of the area $A = A(\alpha) \in [0,1]$ lying under the curve $R = 1$ (Di Silvestri et al. 2014):

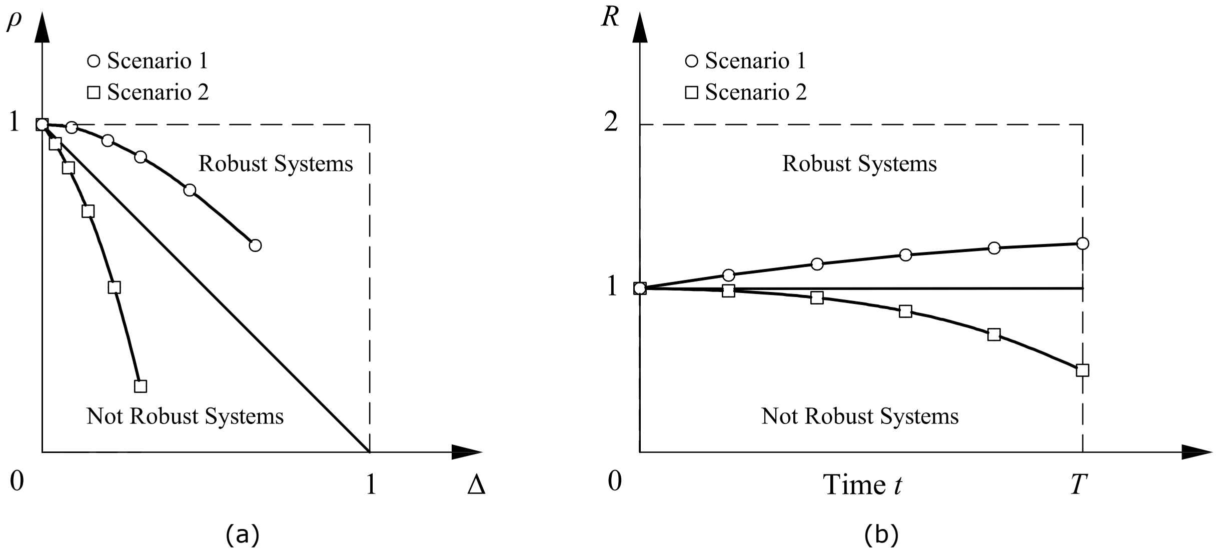
$$A(\alpha) = \left\{ \int_0^1 \rho(\alpha, \Delta) d\Delta \mid R = 1 \right\} = \int_0^1 (1 - \Delta^\alpha)^{\frac{1}{\alpha}} d\Delta \qquad (5\text{-}23)$$

which leads to $A = 0$ for $\alpha = 0$, $A = 0.5$ for $\alpha = 1$, and $A = 1$ for $\alpha = \infty$.

The importance factor α emphasizes that the robustness measure should depend not only on system properties and damage mechanisms, but also on the importance of the system. This approach can be effectively used to compare the robustness associated to different structural systems and rank design alternatives. Moreover, it can accommodate damage scenarios with sudden damage or continuous damage and life-cycle formulations based on time-variant state variables, i.e. time-variant damage index $\Delta = \Delta(t)$, performance index $\rho = \rho(t)$, and robustness factor $R = R(t)$ (Biondini 2009, Biondini & Frangopol 2014).

An illustrative example is qualitatively presented in Figure 5-5 for two different damage scenarios in terms of functions $\rho = \rho(\Delta)$ (Figure 5-5(a)) and time-variant robustness $R = R(t)$ for α=1 (Figure 5-5(b)). In this example, scenario #2 with lower damage rate is more detrimental to robustness compared to scenario #1. Using this approach, the robustness factor may also provide useful information to plan repair interventions and maintenance actions to ensure long-term robustness.

Figure 5-5. Comparison of time-variant robustness over a structural lifetime *T* for two damage scenarios (indicators at equal time intervals t=T/5): (a) performance functions $\rho = \rho(\Delta)$, and (b) robustness factor profiles $R = R(t)$ for $\alpha = 1$.



Damage-based integral measures of structural robustness, averaging the robustness factor over damage extension or time intervals, are also proposed in literature (Starossek & Haberland 2011, Cavaco et al. 2018). However, it is worth noting that the relationship R=$R(\rho,\Delta)$ is time-variant and nonlinear and the robustness criterion $R(t)\geq 1$ needs to be verified at discrete points in time over the structural life-cycle. In fact, integral measures of robustness based on the following formulation:

$$R' = \int_0^1 \rho(\Delta) d\Delta \qquad (5\text{-}24)$$

should be avoided since they provide only average indications over the lifetime and are not able to describe the actual level of structural robustness. Examples can be found in Starossek & Haberland (2011).

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6 Novel proposals for robustness provisions

The aim of this Chapter is to discuss specific practical strategies for the assessment and realisation of structural robustness, with particular focus on the avoidance of disproportionate collapse under local damage scenarios in building and bridge structures. The presented material draws on the main recommendations made by project team WG6.T2 working under mandate M515 of CEN/TC 250. The Chapter presents a consideration of both prescriptive and performance based approaches. A novel treatment of tying forces is presented and discussed in the context of concrete, steel, composite, timber and aluminium structures. Detailed examples, drawn from the reports of the project team WG6.T2 are presented to illustrate application of the proposed methodology.

6.1 Building Structures

This section deals with the assessment of multi-storey building structures, with particular emphasis on a local damage scenario consisting of the sudden loss of a vertical load bearing member. Besides offering a practical test of structural robustness, enabling the direct comparison of candidate structural designs, this scenario was previously shown to offer an upper bound on the response of the building structure in comparison with a scenario in which the column is damaged by blast loading (Gudmundsson & Izzuddin, 2010).

The two most commonly applied strategies related to robustness design and assessment are i) tying force methods, and ii) alternative load path methods. An overview of these strategies, including recent developments, is provided in the following sub-sections.

Focus is given to horizontal tying, which is applicable to the loss of vertical load bearing members, including columns and walls. While the additional contribution of intact walls and infill panels is currently considered in the tying force method only simplistically via a reduction factor, this can be considered effectively using the alternate load path method.

6.1.1 Horizontal Tying Force Strategy

Horizontal tying is recognised in the most recent UFC design code (DoD, 2009) as a means of bridging over a lost vertical load bearing member. Although this intent is not explicitly stated in the Eurocodes, EC0 (EN 1990: CEN, 2005) and EC1 (EN 1991-1-7: CEN, 2006), the consideration of tying under robustness requirements implies an intent to minimize local damage and to avoid progressive collapse under local damage scenarios. It should also be noted that the tying rules for frame structures originate from the tying rules for large panel structures. Within this context, there is a significant discrepancy between the intensity factors used in the UFC and those used in the current Eurocodes, where a typical intensity factor for tying via beams is 3.0 and 0.8 for the UFC and Eurocodes, respectively. While the UFC has taken a fixed level of rotational ductility of 0.2 rad as a realistic value for typical forms of building construction, the tying force requirements in the Eurocodes neglect ductility considerations completely, with the intensity factor of 0.8 necessitating more than 3 times the rotational ductility assumed in the UFC code, which is grossly unrealistic.

As part of mandate M515 for project team WG6.T2 to enhance the robustness requirements for the next generation of the Eurocodes, a new rational tying force method has been proposed by Izzuddin (Izzuddin & Sio, 2022). While this tying force method is based, as a rational approach, on similar principles as the performance-based alternative load path method, it is presented within a simplified framework that is much more practical for robustness assessment and design, making it a suitable replacement for the current prescriptive tying force requirements in the Eurocodes. Moreover, greater prescription can be imposed on the new tying force method, such as the prescription of a specific rotational ductility depending on the construction form and material, thus enabling more simplification in the application to robustness assessment and design practice.

The new method for horizontal tying (Izzuddin & Sio, 2022) is cast within a simplified framework, with the explicitly stated aim that horizontal tying is intended to offer a safe bridging catenary/membrane mechanism under a double-span condition in the event of sudden loss of a column/vertical load bearing member. Importantly, it allows for variable ductility levels, realistic representation of beams and slabs, and dynamic effects; moreover, consideration is given to the strength and stiffness requirements from the surrounding structure to support the horizontal tying forces and the redistributed gravity loads.

The new approach is formulated in such a way so as to allow the superposition of different types of load – including floor distributed load, line load and point load – and different sources of tying within a single floor system – including uniformly distributed reinforcement in one or two orthogonal directions and tying along a line such as via a beam. The general formulation is given by (Izzuddin & Sio, 2022):

$$T \geq \eta\, \rho \left(\frac{i_f}{\bar{\alpha}} \right) P, \quad \bar{\alpha} = \frac{\alpha}{0.2}, \quad (\alpha \text{ in rad}) \tag{6-1}$$

where P is the total equivalent load obtained as a superposition from all loads applied to the double-span beam/floor system, and T is the total equivalent tying force obtained as a superposition from all active tying forces within the beam/floor system. The remaining parameters enhance the treatment considered in existing robustness design codes (EN 1991-1-7, 2006; DoD, 2009) as follows:

— i_f is a tying force intensity factor that depends on the system under consideration. For reference, this is currently taken as 0.8 in Eurocode 1 (EN 1991-1-7, 2006) and as 3.0 in UFC (DoD, 2009), with the latter widely acknowledged to be more realistic for typical levels of ductility;

— $\bar{\alpha} = \alpha/0.2$ is introduced in the proposed approach to allow for different levels of chord rotation ductility α (rad) in different materials and forms of construction;

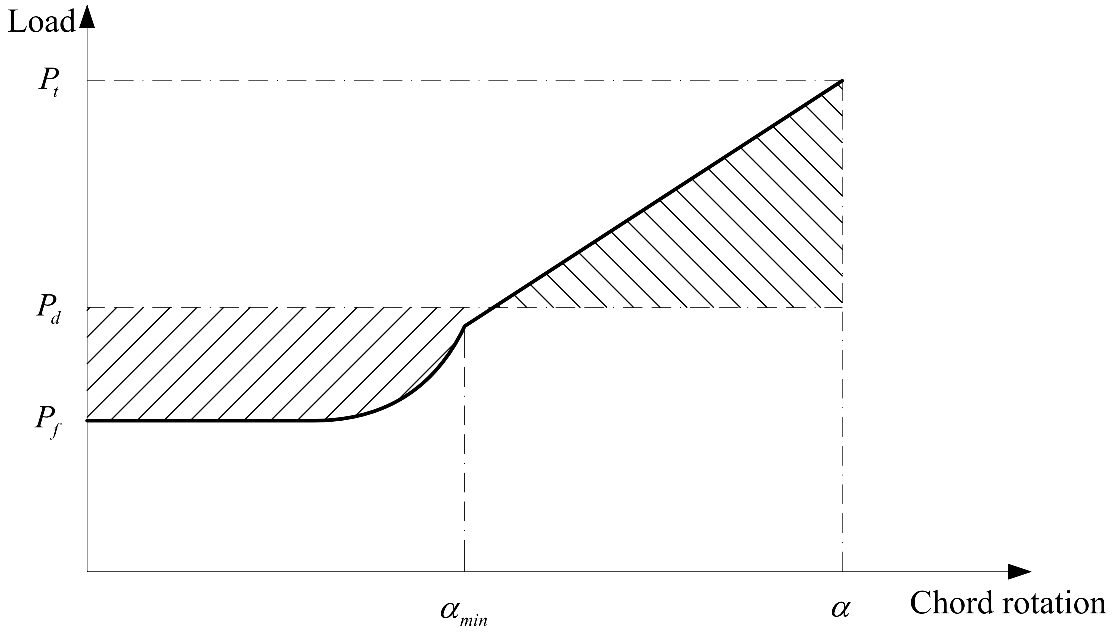
— η is a dynamic amplification factor allowing for the influence of 'sudden' column/load bearing member loss; and

— ρ is a reduction factor that allows for such effects as strain-hardening, contributions of infill walls/infill panels and interaction between tying and flexural actions (taken conservatively as 1). Further research is needed to establish suitable reduction factors for such effects, with progress already made on representing the interaction between tying and flexural actions (Izzuddin, 2022).

The above Equation (6-1) provides the basis for checking the adequacy of the provided equivalent tying force T (as assembled from different active components within the affected beam/floor system) to resist the equivalent load P (as assembled from different loads applied to the system) for the maximum normalised rotational ductility $\bar{\alpha}$ of the system. This clearly requires knowledge of the rotational ductility α, which typically depends on the construction material and structural form. Table 1 presents the parameters i_f, T and P, as required in Equation (6-1), for selected systems subject to uniformly distributed loading, specifically i) tying via beam (symmetric mode), ii) two-way tying via floor slab (with distributed slab reinforcement), and iii) one-way tying via floor slab (with distributed slab reinforcement, assuming torsional edge restrains). Full details, including other types of loading and sources of tying, can be found in (Izzuddin & Sio, 2022).

For consistency with alternative load path methods, dynamic amplification is introduced in the proposed tying force method through factor η, which establishes an equivalent amplification of the loading under sudden loss of a column or vertical load bearing member. The development of load resistance with tying is typically linear as a function of the chord rotation after the attainment of the minimum chord rotation α_{min}, as illustrated in Figure 6-1. In the absence of information on other resistance mechanisms before the development of tensile catenary/membrane action, the most realistic value for the dynamic amplification factor is therefore η = 2. Notwithstanding, a refinement of η can be obtained if the nature of the response preceding the attainment of full tensile catenary/membrane action is known. The most typical case relates to flexural action at lower levels of deflection/chord rotation, as illustrated in Figure 6-1, where it may be assumed that the maximum flexural resistance P_f is achieved at relatively small deflections. Using energy balance principles (Izzuddin et al., 2008; Izzuddin, 2010), a refined dynamic amplification factor can be readily obtained according to an explicit expression (Izzuddin & Sio, 2022).

Figure 6-1. Development of tying and flexural resistance

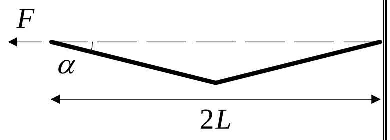
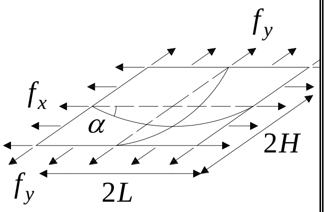
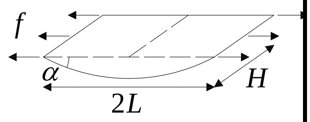
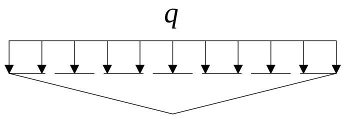
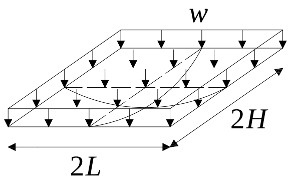
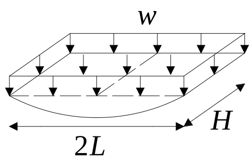


Source: Izzuddin & Sio, 2022

An important factor affecting the development of horizontal tying forces following the loss of a vertical load bearing member is the planar restraint offered by the structure surrounding the affected floor system, both in terms of stiffness and strength. While the planar strength has to be checked against the ultimate tying force capacity, the planar stiffness depends on the rotational capacity of the floor system, with a lower planar stiffness demanding a greater rotational capacity before the development of catenary/membrane action. For a given planar stiffness from the surrounding structure, a minimum chord rotation α_{min} is established in the new method (Izzuddin & Sio, 2022) for different structural tying systems. Provided that the rotational capacity α exceeds α_{min}, full catenary/membrane action can be developed; otherwise, other resistance mechanisms (e.g. flexural/compressive arching action) should be considered.

Another important factor relating to the interaction with the surrounding structure is the dynamic amplification of the redistributed gravity loading. In this respect, a static redistribution is unsafe for a sudden vertical member loss, while a redistribution based on the dynamic load amplification factor η used for the deformation demands can be grossly conservative, due to difference between the distributions of load and inertia forces. A more realistic amplification of the redistributed gravity loading to the surrounding structure can be obtained accounting for the difference, as given in Table 6.1 for the selected cases, with full details covering other cases presented in (Izzuddin & Sio, 2022).

Table 6-1. Tying parameters and redistributed load amplification for selected 1D/2D systems

<table>
<tr><td></td><td>Tying via beam</td><td>Two-way tying via floor</td><td>One-way tying via floor</td></tr>
<tr><td></td><td>
</td><td>
</td><td>
</td></tr>
<tr><td>Intensity factor: i_f</td><td>2.5</td><td>3.125</td><td>3.125</td></tr>
<tr><td>Equivalent tying force: T</td><td>F</td><td>$f_x H + f_y L\left(\frac{L}{H}\right)$</td><td>$\frac{f\,H}{2}$</td></tr>
<tr><td>Loading</td><td>
</td><td>
</td><td>
</td></tr>
<tr><td>Equivalent load: P</td><td>qL</td><td>wLH</td><td>$\frac{wLH}{2}$</td></tr>
<tr><td>Redistributed load amplification</td><td>$0.25+0.75\eta$</td><td>$0.3056+0.6944\eta$</td><td>$0.3056+0.6944\eta$</td></tr>
</table>

Source: Izzuddin & Sio, 2022

6.1.2 Application of different structural Systems/ Materials

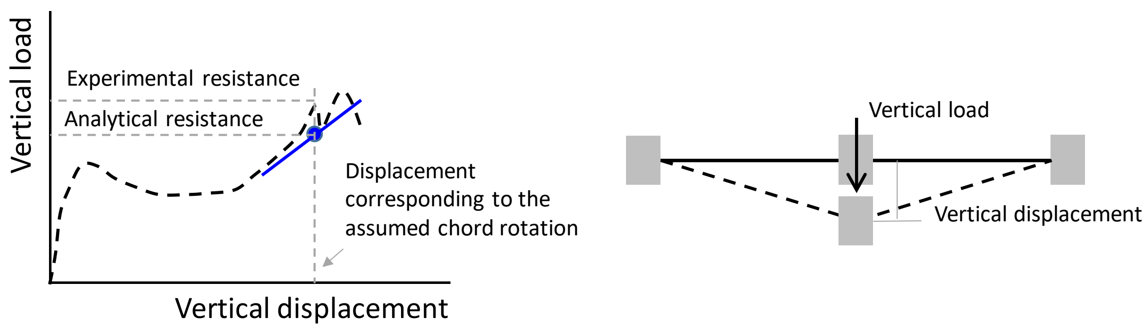
6.1.2.1 Concrete structures

Several applications of tying systems are available in the scientific literature for reinforced concrete frames. Distributed or concentrated tying systems can be adopted to avoid disproportionate collapse under local damage scenarios. In Belletti et al. (2019) a database of experimental tests on peripheral ties in reinforced concrete frames sub-assemblies is illustrated, while in the report by CEN/TC250/WG6 Project Team WG6.T2 "Experimental Database and Validation of Analytical Models for Tie Force and Alternative Load Path Methods for RC Structures" (Belletti et al. 2021) a more complete database, which includes also experimental tests on internal two-way tying systems concentrated in beam and peripheral and internal tying systems distributed in slabs, is reported. The results collected in the database allow to evaluate the rotational ductility of the investigated members in order to provide useful information for the application of the tie force method in the engineering practice.

The capacity of tying systems is experimentally observed by simulating the *Column Loss Scenario* with static or dynamic tests. The column loss scenario can be applied in different positions with respect to the building plan – such as interior, peripheral and corner locations. Most of the experimental tests have been carried out on structural sub-assemblies of RC frames, by analysing concentrated tying systems; few experimental tests on full- scale RC buildings have been conducted. Few experimental tests on the dependency of robustness performance on the position in-elevation of the column loss scenario in multi-story buildings are available. Experimental tests on RC sub-assemblies deal with the tie's capacity dependency on brittle and ductile failure modes - in beams, columns, beam-to-columns joints and slabs -, compressive membrane actions - which enhances the flexural and shear resistance -, tensile membrane action - which activate the catenary stage -, tie anchorage and tie continuity, ductility and dynamic effects. Experimental tests on full- scale structures allows to appreciate the tie's capacity dependency on boundary conditions, activation of Vierendeel actions and internal forces redistribution. No data are available on experimental test on precast structures with tying systems and pinned connections. Few experimental investigations on tying systems in precast moment resisting frames are available. Therefore, extension of the concepts applied to cast-in-situ concrete structures may be carefully applied to precast structures. For the robustness verification of precast structures, experimental tests are highly recommended to evaluate the effective tying resistance and the equilibrium and compatibility conditions to be respected. The compatibility requirements are particularly important in the case of precast structures realised with dried connections.

The tie method proposed by Izzuddin & Sio (2022) has been validated in Belletti et al. (2021) and Ravasini at al. (2021) by comparing the analytical results with the experimental results obtained from the tests reported in the database. The rotational ductility has been obtained from the average value of the rotational ductility observed from experimental test. It must be noted that for distributed tying systems or for coupled tying systems - with concentrated tying reinforcement in the beam and distributed tying reinforcement in the slab, the population of the tested specimens is not representative for a reliable estimation of the ductility level and that further experimental results are required. The ratio between the experimental resistance and the resistance calculated according with the Tying force method has been evaluated for each analysed specimen, Figure 6-2. These latter resistances are evaluated in correspondence of the assumed rotational ductility. The total equivalent tying force is calculated as a function of the tying reinforcement area and the yield strength of steel. The average value, the standard deviation and the Coefficient of Variation (CoV) of this ratio have been evaluated for each group of sub-assemblies (1D-beam, 2D-beam, beam-slab and flat-slab assemblies).

Figure 6-2. Experimental and analytical tie resistance at the catenary stage evaluated at the assumed ductility level



The main hypotheses assumed for the validation and the main results are reported in the following:

— The tie resistance is calculated by adopting the yield strength of steel;

— 1D-beam sub-assembly: a chord rotation equal to equal to 0.2 rad is assumed for beams characterised by a ductile failure and plastic hinges formations at beam ends. The average value of the ratio between the experimental and the analytical resistance is equal to 1.02; the standard deviation is equal to 0.3 and the coefficient of variation is equal to 29.4%;

— For beams characterised by values of span-to-depth ratio lower than 9, the failure mode is usually governed by brittle mechanisms and lower ductility levels must be adopted. Indeed, the flexural resistance enhanced by compressive arch action results higher than the resistance at the catenary stage;

— 2D-beam sub-assembly: the capacity of a tying system composed by two intersecting beams is obtained by adopting the principle of superposition by adding the contribution of each individual beam. The maximum displacement at failure is achieved in correspondence of the minimum ductility of the two intersecting beams;

— 2D-beam-slab sub-assembly: the capacity of the tying system composed by 2D-beams and the slab is obtained by adopting the principle of superposition by adding the contribution of each individual beam and the contribution of the slab. In the case of two intersecting beams and slab sub-assembly of frame systems, if the non-linear response is governed by ductile flexural mechanisms a chord rotation value equal to 0.2 may be assumed. If the non-linear response is governed by shear failure or punching failure a lower ductility level must be assumed. It must be noted that the population of the tested specimens available in the database is not representative for a reliable estimation of the ductility level and that further experimental results are required. The average value of the ratio between the experimental and the analytical resistance is equal to 1.37; the standard deviation is equal to 0.14 and the coefficient of variation is equal to 10%;

— In continuous flat slabs, supported by columns, the non-linear response is usually governed by the punching shear. The maximum ductility must be carefully evaluated. Integrity reinforcement can enhance the post-punching capacity both in term of resistance and ductility; in general, the maximum chord rotation should be lower than or equal to 0.1 rad. It must be noted that the population of the tested specimens available in the database is not representative for a reliable estimation of the ductility level and that further experimental results are required.

— Tie method in precast frame systems has been rarely applied. Few numerical data on the coupled effect of pinned beam-to-columns connections and concentrated ties are reported in literature (Ravasini et al. 2021a). Few experimental investigations on tying systems in precast moment resisting frames are available.

In the Report of the Project Team WG6.T2 some examples of application of the tie force method to RC Systems Under Interior Column Loss are provided. In particular, the cases of concentrated ties and distributed ties have been analysed for frames having an in-plane distance between columns equal to 8.1m. The tying reinforcement adopted in the example is assumed continuous and well anchorated.

In the first example, a precast moment resisting frame is analysed. The tying reinforcement - concentrated along the beam supporting the floor slab and constituted of 3 Ø24 + 4 Ø26 at the top and 5 Ø24 at the bottom longitudinal continuous bars – allows to verify the capacity of the structure for a distributed load, applied to the beam and evaluated in the accidental load combination, equal to 63.34 kN/m. Of course, additional verifications, not reported in the example are required to avoid brittle failures occurring before the formation of the catenary mechanism. Therefore, appropriate resistance and compatibility controls of the connections between the topping slab and the hollow-core slabs and of the connections between the transversal beam and the hollow-core slab have to be carried out. Furthermore, appropriate check of the hollow-core slabs resistance and of shear resistance of the beams are required.

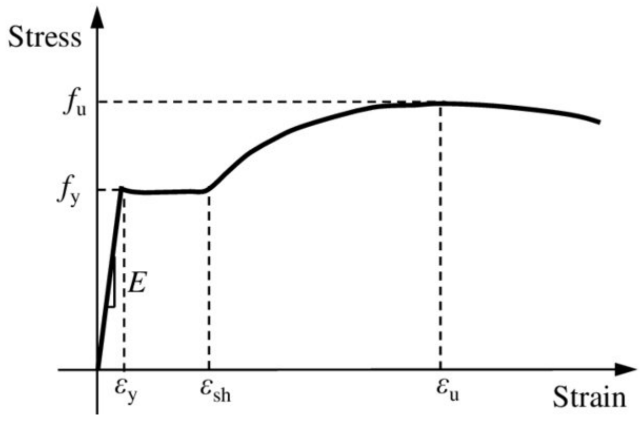
In the second example, the RC frame is realised by cast in situ members and the tying system is distributed in the slab. The total equivalent tying force is calculated as a function of the continuous bottom (sagging) reinforcement (Ø14/250) and the continuous top (hogging) reinforcement (Ø16/300). Since the contribution of the slab has been considered under applied surface load, a chord rotation ductility of α = 0.1 rad is assumed. It results that the slab offers a safe bridging membrane mechanism under the event of sudden loss of the interior column when a distributed load, corresponding to the accidental load calculation, equal to 10.2 kN/m^2 is applied. Of course, additional verifications, not reported in the example are required to avoid brittle failures occurring before the formation of the catenary mechanism. Therefore, an appropriate check of the punching and post-punching resistances is required.

6.1.2.2 Steel and Composite structures

The properties and behaviour of rolled steel sections are well known and thoroughly investigated by various extensive (European) studies (EUR 28906, 2017). Consequently, the results of this type of studies are only valid for steel grades and production methods that comply with the various European product standards. Steel members can be recognised as ductile under the various load types if instability effects can be avoided by proper means. It is also known that mostly a lack of robustness can be mainly attributed to the joints which turn out to be critical (Knoll & Vogel, 2009) or the zones immediately adjacent. In that way, the proposed Tying Force Strategy (TFS) out of section 6.6.1 perfectly match the behaviour of steel elements as in the underlying model; all plasticity is concentrated at the locations of the nodes, which in general coincides with the location of the assemblies. This system property can be even more explored so that with relatively simple numerical aids, a complete robustness assessment can be obtained (Molkens, 2021).

So-called mild steels with low to modest yield strength exhibit an increase of the strength (stress) with increasing deformation (strain), Figure 6-3. This property is used in regular steel design where the following criteria should be met: $A \times f_y < A_{net} \times f_u$ with A = the surface area of a member without holes, f_y (or $f_{y,nom}$) = the yield strength, A_{net} = reduced surface area with bolt holes and f_t (or $f_{u,nom}$) the tensile ultimate strength. Similar criteria can be stated for connections designed for higher strength than one of the beam elements.

Figure 6-3. Engineering stress-strain relationship for steel (Yun & Gardner, 2017)



Source: Yun & Gardner, 2017

Out of analysis of the data from some experiments (Dinu, Marginean, & Dubina, 2017) and (Demonceau et al., 2010)), even in combination with large deformations, the tested connection can still transfer some moments. Favourable effects of strain hardening and possible combinations of moment and arch- or catenary effects can be implemented by the ρ factor out of the proposed Eq. (6-1). A European valorisation project involving 14 European partners coordinated by the University of Liège is ongoing and will be released in 2022. Outcomes of this project will be of high importance to address the behaviour of steel and composite structures and, in particular, to ensure a ductile behaviour and the possibility of activating alternative load paths (Demonceau et al, 2022). For the time being, designers can use data out of (UFC 4-023-03, 2016) with the proposed chord rotation α =

0,20 rad, besides concrete also valid for steel and composite. For cold-formed steel components, it should be proven that the system can carry the required longitudinal, transverse, and peripheral tie strength before applying this chord rotation.

In some instances, partial buckling of the member zones adjacent to connections may be permissible provided that the changed load arrangement (pure tension in case of full catenary action instead of shear, normal force, and moments) can be safely transmitted to the rest of the structure. Ductile failure modes of the joints are, under any circumstance, obligatory when designing for robustness except in case of full-strength joints. In practice, this means that the failure of bolts, welds or other fasteners should be avoided (Demonceau et al, 2022).

The design of full-strength joints should account for overstrength, which is also typical for mild steels, the data out of prEN 1998 can be used, see Table 6-2. One way to achieve it, based on the same philosophy as for strain-hardening, is the following: the nominal or code specified values should be used to determine the resistance of the joint. An assessment should be carried out to check if the beam capacity based on the mean values including an overstrength or randomness coefficient γ_{rm} is lower than the resistance of the joint. The designer should be aware that this will lead to rather massive and mostly non-economical solutions.

Table 6-2. Recommended value of γ_{rm}

Steel Grade	γ_{rm}
S235	1,45
S275	1,35
S355	1,25
S460	1,20

Source: prEN1998, not yet published

Significant research activities have been conducted in the past decades to investigate the behaviour of steel and composite framed structures subjected to sudden column removal in the framework of RFCS (Research Fund for Coal and Steel) projects involving different European research institutions. The main outcome coming from these projects are reflected in (Demonceau et al, 2022).

Information about robustness related items of structures made of stainless steels is even more rare to find. However, high ductility of these steels (strains of 20 up to more than 50 % are usual) associated to substantial strain hardening leads to the conclusion that a design following the rules for carbon steel will result in a (over) conservative design. Proper methods considering a Ramberg-Osgood constitutive behaviour law like the continuous strength method out of (SCI, 2017) can enhance material use and reduce related costs.

As stated before, it is also possible to adapt the design from a conceptual point of view to avoid possible problems; some sound engineering principles can be followed:

— To promote the activation of ductile structural components in the structure and, accordingly, to avoid brittle failure modes, with a specific attention to be paid to the welds (the use of full strength welds is recommended);

— To ensure links between the elements using the tying approach according to the proposed TFS;

— Optimise the design making use of strain-hardening and combined actions if this can be supported by tests or reliable data.

Some application examples can be found in the Report of the Project Team WG6.T2, starting with a continuous steel beam of a 2D frame. In a second example, the spacers typically used to limit buckling length and avoid lateral-torsional buckling are also activated. In a third simulation, the reinforcement mesh of the compression layer is used to provide additional contribution for robustness. Under normal conditions, there is no contribution of the compression layer (except superimposed dead load), but in the accidental situation, the topping behaves as a membrane and no requirements are set to the steel structure. The fourth example makes use of all components, topping, beams and spacers. For that reason, all these components should satisfy robustness requirements. In the case of existing structures, an approach whereby new additional elements serves as robustness provisions can be the most straightforward approach to obtain or enhance robustness. In some extend this is reflected in example three.

6.1.2.3 Timber structures

The tie method proposed by Izzuddin & Sio (2022) has been validated in Martinelli and & Izzuddin (2022) as regards the timber post-and-beam structural typology, the only one at the moment for which some experimental data on rotational ductility under column loss scenario is available.

The newly proposed tying force formulation given in Eq. (6-1), depends on the parameter $\bar{\alpha} = \alpha/0.2$ that accounts different levels of chord rotation ductility α (rad) in different materials and forms of construction.

With reference to post-and-beam timber structures, the rotational ductility is strongly influenced by the type of connection between beams and columns. Although the rotational ductility data available in the literature are rather limited and refer to a single structural typology of timber constructions (i.e. post-and-beam structures), it is still considered important to analyse here the data presented in the literature. Based on the experimental data reported by Masaeli et al., (2020) and by Lyu et al., (2020), the ultimate rotations for the different types of beam-column connectors are reviewed in this section.

Three types of commercially available beam-to-column connectors, currently used in mass timber buildings, were investigated by Lyu et al., (2020). A description of the four connectors is also given in the report by CEN/TC250/WG6 Project Team WG6.T2 “Considerations on the rotational ductility in timber post-and-beam structures” (Martinelli 2021). These connectors were designed as shear connectors, not moment resisting ones, assuming that the horizontal stability of the structure is ensured by shear walls and cross-bracing elements instead of frame actions. The three types of beam-to-column connectors were not specifically designed for robustness. The scaling effects of these three types of connectors on the moment and shear responses were investigated in Masaeli et al., (2020) where full-scale and ¼ scale connectors were compared. The collapse response of the three types of connectors under a quasi-static column removal scenario (i.e. push down test) was investigated in Lyu et al., (2020) adopting a ¼ scale prototype.

A fourth connector was proposed by the same research group (Lyu et al., 2020) and designed to resist the loss of a column through catenary action. The performance of all four types of connectors was studied by Lyu et al., (2020). All connectors were designed based on the specification given in EN 1995-1-1 (2004) to sustain the same factored design shear force of a representative building. The factored design shear force of the beam-to-column connectors under medium-term actions was equal to 183 kN for the full-scale connectors.

Table 6.3 reports the ultimate rotations for connectors Types 1-3 obtained from bending tests at full-scale and at ¼ scale. Data reported in Table 6-3 are in good agreement with those obtained from push-down tests at ¼ scale (Table 6-4). All commercial connectors (Types 1-3) provided enough rotation for the catenary action to either develop or start developing. Only connector Type 1 shows a limited rotation ductility that prevents a full development of catenary action during the push-down test. The other types of connectors (Types 2-4) show a good rotational ductility that allow a full development of catenary action during the push-down tests. The connector Type 4 (double plate connector) proposed by Lyu et al., (2020) show a higher capacities and higher rotational ductility compared to the other connectors. Connector Type 4 represents a potential solution to improve the robustness of post-and-beam timber buildings.

Table 6-3. Ultimate rotations for connectors Types 1-3 from bending tests at full-scale and at ¼ scale

	Type 1	**Type 2**	**Type 3**
α (rad) Full-scale test	0.12	0.087	0.16
α (rad) ¼ scale test	0.070	0.23	0.16

Source: Masaeli et al., 2020

Table 6-4. Ultimate rotations for connectors Types 1-4 from push-down tests at ¼ scale

	Type 1	**Type 2**	**Type 3**	**Type 4-1**	**Type 4-2**	**Type 4-3**
α (rad) ¼ scale test	0.07	0.17	0.15	0.21	0.21	0.17

Source: Lyu et al., 2020

In the background report of the Project Team WG6.T2 (Martinelli 2021) a preliminary validation of the tie force method to post-and-beam timber systems under interior column loss is provided. In the Report of the Project Team WG6.T2 one example of application of the tie force method to post-and-beam timber structure is given.

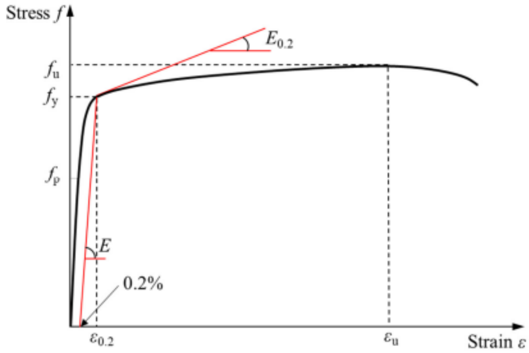
In the example, a 4 storeys 5 × 5-bays post-and-beam timber structure is considered (§6.3.3). The structure is used as offices (building category B according to BS EN 1991-1-1, 2002) and falls in consequence class 2a according to BS EN 1991-1-7 (2006). Span lengths along x- and y-direction are equals to 8 and 6 m, respectively, while the inter-story height is equal to 3.6 m. Continuous multi-storey glued laminated columns are adopted. Glulam is used for both primary and periphery beams, labelled as B1 and B2, and for continuous multi-story column labelled as C1. CLT slabs 295 mm thick are adopted.

6.1.2.4 Aluminium Structures

The design of aluminium structures for robustness can be executed in a very similar way as for steel. The designer should be, however, aware about:

- Only a few grades are suitable for structural works. The in Europe, usable wrought and cast alloys can be found in Tables 3.1a and 3.1b of CEN (2007).
- The material is strain hardening with a Ramberg-Osgood constitutive relationship, see Figure 6-4.

Figure 6-4. Engineering Typical engineering stress-strain curve for aluminium alloys



Source: Yun et al., 2021

6.1.3 Alternative load path strategy

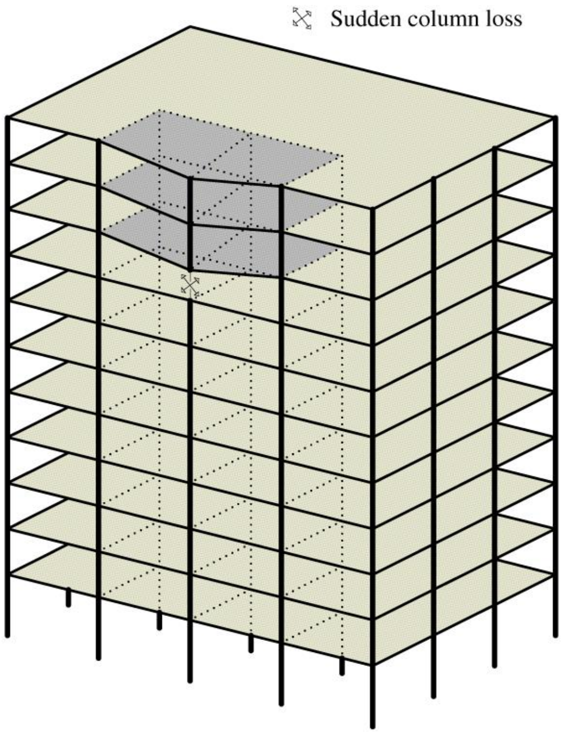
The alternative load path methods offer a more performance-based approach to the robustness assessment of structures compared to tying methods, allowing for other resistance mechanisms such as flexural and compressive actions as well as for a more accurate representation of the interaction between the affected part of the structure and the surrounding parts. Focus is given here to the application of alternative load path methods to multi-storey building structures subject to sudden column loss as a local damage scenario.

A simplified multi-level robustness assessment framework was proposed by Izzuddin et al. (2008) for multi-storey buildings subject to sudden column loss, which can be applied at various levels of structural idealisation, and which combines the influence of redundancy, ductility, energy absorption and dynamic effects into a single measure of robustness, namely the pseudo-static capacity. To illustrate the multi-level aspect of this method, with reference to the building structure subject to sudden column loss as shown in Figure 6-5, it is the ability of the upper floors to sustain the ensuing dynamic deformations that determines whether disproportionate (progressive) collapse can be avoided. Accordingly, alternative reduced models may be considered at various levels of structural idealisation, as illustrated in Figure 6-6, depending on the regularity of the building in terms structure and loading (Izzuddin et al., 2008). At the first level of model reduction, consideration may be given to the affected bay of the multi-storey building (Figure 6-6a), with appropriate boundary conditions to represent the interaction with the surrounding structure. If it is established that the surrounding columns can resist the redistributed load, further model reduction may consider only the floors above the lost column where deformation is concentrated (Figure 6-6). If additionally the affected floors are identical in terms of structure and loading, the axial force in the columns immediately above the lost column becomes negligible, and a reduced model consisting of a single floor system may be considered (Figure 6-6c). Finally, ignoring planar effects within the floor slab, individual beams may be considered at the lowest level of model reduction (Figure 6-6d), subject to appropriate proportions of the gravity load.

Once the extent of the structural model is defined, it is used to establish the ability of the locally damaged structure to resist the applied gravity loading, allowing for dynamic effects, without further failure. Consideration should be given to the factored gravity loading and to the ensuing dynamic effects due to sudden local damage which can be assessed by means of static or dynamic analysis methods. In all cases, the robustness limit state should be concerned with the ability of the locally damaged structure to maintain integrity through the explicit consideration of redundancy, energy absorption capacity and ductility limits under dynamic conditions (Izzuddin et al., 2008). This can be based on ensuring that no component in the structure outside the locally damaged region exceeds its deformation or strength limit, as appropriate. Allowance can also be made for successive component failures (Izzuddin, 2010), provided these do not lead to collapse outside the locally damaged region due to sufficient residual dynamic strength in the surrounding structure. While some types of structure may be assessed for robustness using linear analysis methods, large inelastic deformations would be allowed and expected in typical forms of construction, such as steel-framed and reinforced concrete buildings, which necessitate the use of nonlinear analysis. Moreover, while nonlinear dynamic analysis offers the most accurate representation of the structural response under sudden column loss, focus is given here to nonlinear static methods combined with simplified dynamic assessment as a more practical approach for application in robustness design and assessment practice.

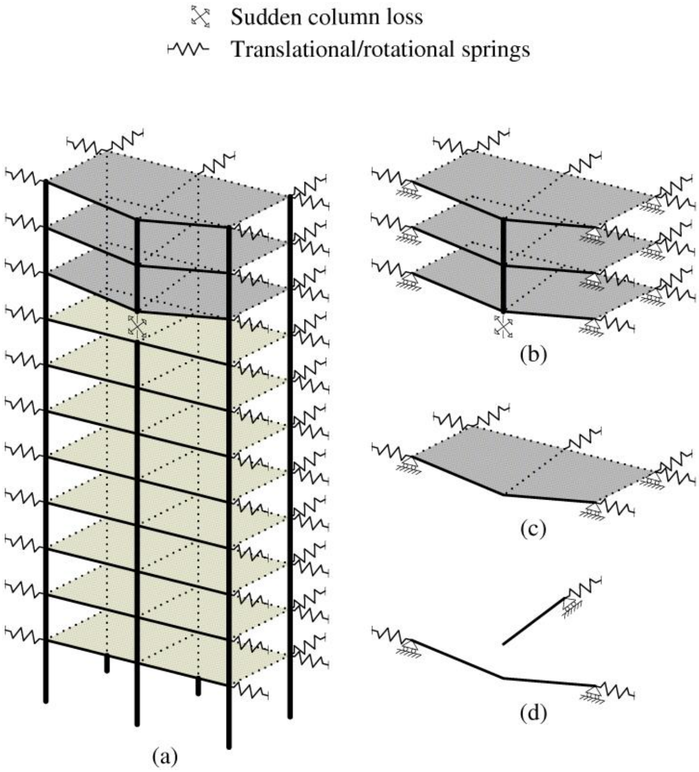
Nonlinear structural analysis offers major advantages compared to linear analysis, particularly for structures in which components fail by fracture well after the development of significant plastic deformation. In this type of analysis, both geometric and material nonlinearity should typically be considered, allowing for the modelling of large displacements, structural instability and inelastic material response (e.g. yielding of steel, cracking of concrete). Depending on the sophistication of the adopted models, nonlinear phenomena preceding component failure/structural collapse can be represented to various degrees of accuracy, including the flexural strength, compressive arching action, tensile catenary/membrane action, diaphragm action of slabs/walls, etc. In this respect, the use of nonlinear analysis in robustness assessment allows the consideration of the structure at relatively large deformations thus exceeding conventional strength-based norms, provided the deformation capacities at which components fail by fracture are not exceeded. Accordingly, the robustness limit state requires the structure subject to a sudden local damage scenario to exhibit component deformations that are within their respective deformation capacity at fracture, collectively defining a so-called ductility limit on the maximum dynamic deformations of the locally damaged structure (Izzuddin et al., 2008).

Figure 6-5. Multi-storey building subject to sudden column loss



Source: Izzuddin et al., 2008

Figure 6-6. Sub-structural levels for robustness assessment



Source: Izzuddin et al., 2008

There are numerous types of elements that can be used for nonlinear structural analysis, the review of which is outside the current scope. Some indicative references on nonlinear analysis of structural components are as follows:

— 1D beam-column elements of the fibre-type can be used to model steel (Izzuddin & Elnashai, 1993) and reinforced concrete (RC) (Izzuddin & Lloyd Smith, 2000) frames with reasonable accuracy.

— 2D shell elements can be used to model RC and composite floor slabs (Izzuddin et al., 2004), allowing for the development of membrane action, while an approximate grillage representation of the floor slabs can also be used for a conservative assessment of robustness (Zolghadr Jahromi et al., 2013).

— Masonry infill panels can be modelled at different levels of sophistication, including as equivalent 1D struts (Farazman et al., 2013), simplified macro-elements (Minga et al., 2020), and sophisticated 3D meso-scale models (Xavier et al., 2015). Previous work has shown that the incorporation of infill panels in the structural model can lead through vertical diaphragm action to a significant enhancement of robustness, with reduced requirements on the deformation limit of the structure.

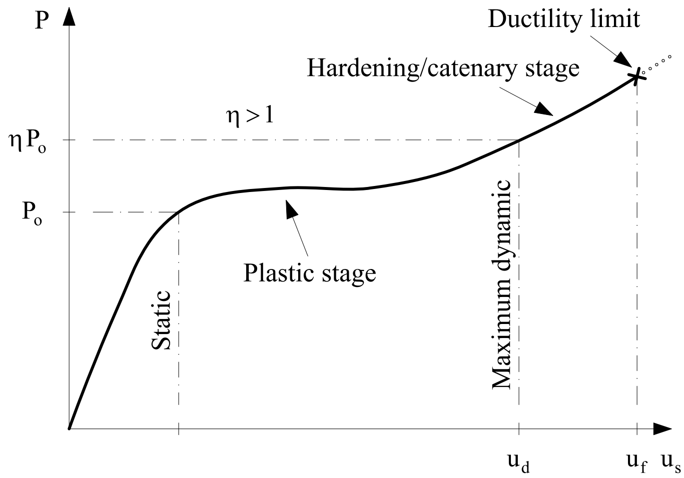
— Discrete elements may be used to model RC concrete joints (Favvata et al., 2008) and steel connections (Fang et al., 2013), including in the latter case component-based methods (Steenhuis et al., 1998) that are available in EC3. It is important to note that, in application to robustness scenarios, the joint models must capture the interaction between bending and axial actions not only in terms of the force-deformation relationships but also in terms of the assessment of the deformation capacity; this goes beyond the specific requirements of joint ductility under seismic action, where the axial action in the joints is typically neglected.

For nonlinear analysis models which do not account for the component response following the initiation of fracture failure, the robustness assessment can be based conservatively on the initiation of first-component failure. It is also worth highlighting that nonlinear analysis models of different sophistication may be applied at the various levels of structural idealisation highlighted before, where lower-level models are often sufficient for a preliminary conservative assessment of structural robustness (Izzuddin et al., 2008; Zolghadr Jahromi, 2013).

Focusing on nonlinear static analysis methods for robustness assessment of building structures subject to sudden column loss, this involves a pushdown scenario utilising gravity loading, leading to a static load-deflection response of the locally damaged structure, with a typical example shown in Figure 6-7. Such a pushdown nonlinear static response can be generated for the full structure or substructures, as discussed previously, making use of detailed nonlinear finite element models or simplified mechanics-based models (Izzuddin et al., 2008), though it is important to highlight the potential inaccuracy of the latter models and their typical inability to represent well the interaction with the surrounding structure.

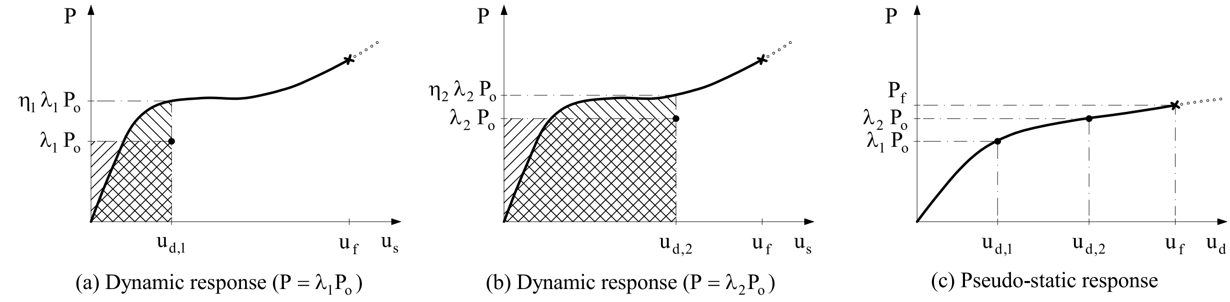
The nonlinear static pushdown response can be used to recover the maximum dynamic deformation, resulting from the sudden nature of column loss, as that corresponding to an amplified static load with a factor η, as illustrated in Figure 6-5. This dynamic amplification factor does not need to be assumed, but it can be easily recovered using a simplified dynamic assessment approach based on an energy balance concept (Izzuddin et al., 2008), which assumes that the pushdown response is governed by a dominant deformation mode, consequently leading to a close similarity between the effects of sudden local damage and loading that is applied suddenly to the locally damaged structure (Izzuddin, 2010). The simplified dynamic assessment approach is illustrated in Figure 6-6, where for any level of applied gravity loading ($\lambda_i P_0$), the corresponding dynamic displacement ($u_{d,i}$) can be obtained from the nonlinear static pushdown response by equating the work done by the suddenly applied load (rectangular hatched area) to the internal energy absorbed by the structure (hatched area under nonlinear static curve). The check for the robustness limit state at a specific level of gravity loading ($\lambda_i P_0$) would then consider whether the dynamic displacement ($u_{d,i}$) is within the ductility limit of the locally damaged structure, or alternatively whether all the components of this structure are within their respective deformation capacity.

Figure 6-7. Nonlinear static pushdown response



Source: Izzuddin, 2010

Figure 6-8. Simplified dynamic assessment and pseudo-static response (Izzuddin et al., 2008)



Source: Izzuddin et al., 2008

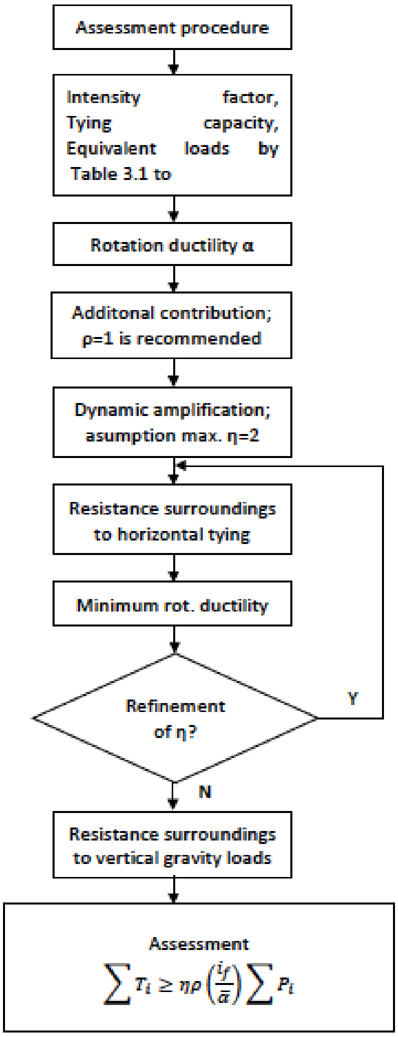
The simplified dynamic assessment approach provides a simple transformation of the nonlinear static pushdown response to a maximum dynamic response, so-called pseudo-static response (Figure 6-8c), as it inherits similar characteristics to the static response, including the assembly of resistance contributions from different parts of the locally damaged structure (Izzuddin et al., 2008). This transformation implies that the pseudo-static resistance at a specific level of dynamic deformation is the average of the static resistance up to this level of displacement. This approach does not require the explicit determination of the dynamic amplification factor, though this can be easily recovered as the ratio of the static resistance to the pseudo-static resistance at a specific level of displacement.

Unlike nonlinear dynamic analysis, the use of nonlinear static analysis with the simplified dynamic (pseudo-static) assessment enables the consideration of robustness at different levels of gravity loading without the need for re-analysis, as the relevant margin of the applied loading ($\lambda_i P_0$) to the resistance (P_f) at the robustness limit state is already available from the pseudo-static response (Figure 6-8c). However, besides the assumption of a dominant deformation mode, the applicability of the standard pseudo-static assessment to successive component failures requires the same mode to persist after component failure(s). Further information on the treatment of successive component failures in robustness assessment using the energy balance concept can be found in the work of Izzuddin (2012).

6.1.4 Examples of the application of the novel proposals

The following examples are reproduced from the final report of PT WG6.T2 on Robustness Rules in the Material Related Eurocode Parts. The calculation scheme employed for the newly developed tying force method, which represents an assessment procedure, is as illustrated in Figure 6-9.

Figure 6-9. Performance indicators



6.1.4.1 *Reinforced Concrete Structure Example*

A frame is realised by precast members. The plan view is depicted in Figure 6-10. The dimensions of beams and columns are:

— Beams: height equal to 60 cm and width equal to 60 cm;

— Columns: height equal to 80 cm and width equal to 80 cm.

The diaphragm is realised by hollow-core slab and by a topping slab 5 cm thick. Continuous beams support the diaphragm.

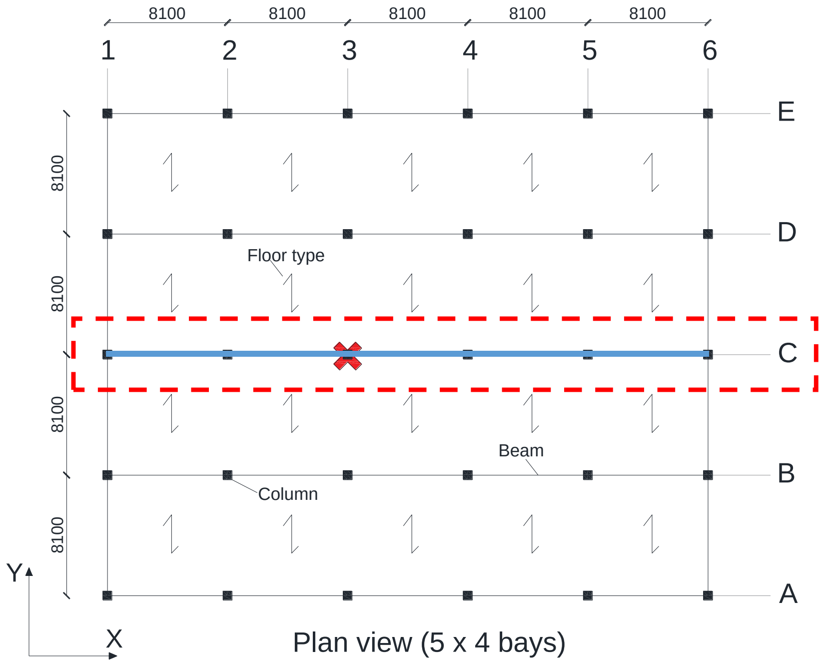
In this example it is assumed that the contribution of the topping slab, to the tie force assessment, is neglected. Therefore, the tie reinforcement is concentrated in the beam supporting the hollow-core slabs, which behave as double-span beams. The continuous longitudinal reinforcements, which is used as tie reinforcement in transversal beams are:

— Top reinforcement: 3 Ø24 + 4 Ø 26

— Bottom reinforcement: 5 Ø24

— A_s = 5739.92 mm^2

Figure 6-10. Plan view of the emulative precast RC frame under interior column loss



For both dead and live loads, the values are reported in Table 6-5.

Table 6-5. Applied loads

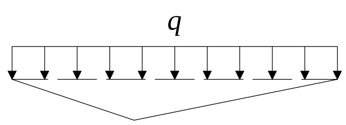
Load types	**Magnitude**	**Units**
DL: beams	8.83	kN/m
DL: walls	0.80	kN/m^2
DL: false floor	0.50	kN/m^2
DL: equipment	0.50	kN/m^2
DL: hollow-core slab	2.78	kN/m^2
DL: topping	1.25	kN/m^2
LL: Live Load	3.00	kN/m^2

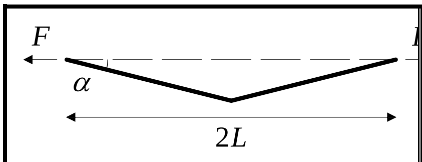
The loads, reported in Table 6-5, are used for the calculation of accidental load combination. The uniformly distributed load q, acting on the transversal beam, is given as:

$$q = 8.83 + 8.1 \cdot (0.8 + 0.5 + 0.5 + 2.78 + 1.25) + 0.3 \cdot 8.1 \cdot 3 = 63.34 \text{ kN/m} \qquad (6\text{-}2)$$

The calculation of the equivalent tying force T and load P are reported in Table 6-6.

Table 6-6. Tying parameters for double-span beams

	Equal spans	

		
Intensity factor: i_f	2.5	
Equivalent tying force: T	F	$T = A_s \cdot f_{yk} = 5739.92 \cdot 450 = 2582.96\ kN$
Equivalent load: P	qL	$P = q \cdot L = 63.34 \cdot 8.1 = 513.07\ kN$

Tie capacity verification

As a first demonstration of the tie force verification, the dynamic amplification factor and the reduction factor are taken as $\eta = 2$ and $\rho = 1$, respectively.

The normalized chord rotation $\bar{\alpha}$ is evaluated by assuming a chord rotation ductility equal to $\alpha = 0.2$ rad:

$$\bar{\alpha} = \frac{\alpha}{0.2} = \frac{0.2}{0.2} = 1 \tag{6-3}$$

The adequacy of tying force is evaluated in the following:

$$T = 2582.96\ kN \geq 2 \cdot 1 \cdot \left(\frac{2.5}{1}\right) 513.05 = 2565.25\ kN \tag{6-4}$$

Therefore, for the assumed level of chord rotation ductility and applied vertical loads, the transversal beam offers a safe bridging catenary mechanism under the event of sudden loss of the interior column.

Appropriate checks of the connections between the topping slab and the hollow-core slabs and of the connections between the transversal beam and the hollow-core slab are required. An appropriate check of the hollow-core slabs capacity is required. An appropriate check of the shear capacity of the beam is required.

Strength and stiffness requirements of surrounding structure

The surrounding structure must provide the necessary strength to resist the tying forces induced in any double-span beams and the horizontal floor system. The total pull-in displacement u must be limited to the following value:

$$u \leq \frac{L_1}{2}\left(\alpha - \frac{d_{eff}}{L_1}\right)^2 \left(\frac{L_1 + L_2}{L_2}\right) - \delta \tag{6-5}$$

For the evaluation of the displacement u the horizontal force provided by the tie, equal to F=2582.96 kN, is applied to the topping slab, 5 cm thick, by neglecting the stiffness contribution provided by the frame and by assuming a reduced value of the modulus of elasticity for concrete in order to take into account for cracking effects, see Figure 6-11. Therefore, the lateral displacement has been calculated by accounting for the in-plane shear and flexural deformation of the topping RC slab.

The lateral displacement to the left u_L and to the right u_R of the double-span beam results respectively equal to u_L =5.88 mm and u_R =2.35 mm.

The total displacement results equal to:

$$u = u_R + u_L = 8.23\ mm \tag{6-6}$$

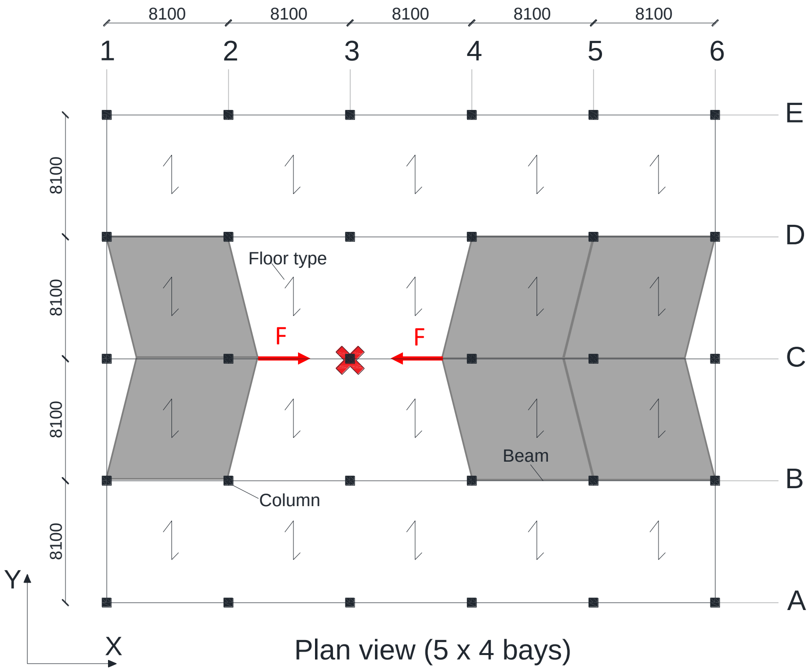
The elastic extension of the double-span beam under *F*, here calculated by neglecting tension stiffening effects, is given in the following:

$$\delta = 2 \cdot L \cdot \varepsilon_y = 2 \cdot 8100 \cdot \frac{450}{200000} = 36.45\ mm \tag{6-7}$$

The verification of the stiffness of the surrounding structures is provided in the following, being d_{eff} equal to 495 mm:

$$u = 8.23\ mm \leq \frac{8100}{2}\left(0.2 - \frac{495}{8100}\right)^2 \left(\frac{8100 + 8100}{8100}\right) - 36.45 = 119.8\ mm \tag{6-8}$$

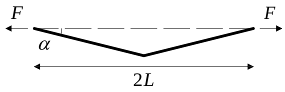
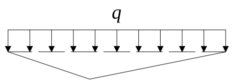
Figure 6-11. Lateral stiffness provided by the topping slab of 5 cm



Resistance to redistributed vertical gravity loads

The use of the previous full amplification, η, would be conservative, as this assumes inertia forces to have the same spatial distribution as the actual gravity loading. Instead, the amplification factors can be established for the dominant load according to its type and the system under consideration.

Table 6-7. Amplification of redistributed vertical gravity loading for double-span beams

	Equal spans 
Redistributed gravity load amplification	
	$0.25 + 0.75 \cdot \eta = 0.25 + 0.75 \cdot 2 = 1.75$

Minimum rotational ductility for activation of tying

The minimum level of ductility α_{min} required for the activation of tensile catenary action is given in the following:

$$\alpha_{min} = \frac{d_{eff}}{L_1} + \sqrt{\frac{2(u+\delta)}{L_1 + L_2}\left(\frac{L_2}{L_1}\right)} \tag{6-9}$$

The calculation yields:

$$\alpha_{min} = \frac{495}{8100} + \sqrt{\frac{2(8.24 + 36.45)}{2 \cdot 8100} \cdot 1} = 0.135\, rad \tag{6-10}$$

with *u* and *δ* as previously defined.

Since $\alpha > \alpha_{min}$, the use of the tie method is justified.

6.1.4.2 Steel Structure Example

A frame is realised by steel members. The static system is as shown in Figure 6-12.

Load assumptions

Self-weight: g_k = 0.56 kN/m (assume an IPE360 section 57.1 kg/m A-M catalogue, width 170 mm and height 360 mm)

Dead load: p_k = 1.5+8.1/2·(0.8+0.5+1.25+2.78+0.5) = 25.11 kN/m (façade + partition walls, false floor, topping, hollow cores and false ceiling with equipment)

Live load B: q_k = 8.1/2·3 = 12.15 kN/m (office building EN 1991-1-1; Ψ_0 = 0.70, Ψ_1 = 0.50 and Ψ_2 = 0.30)

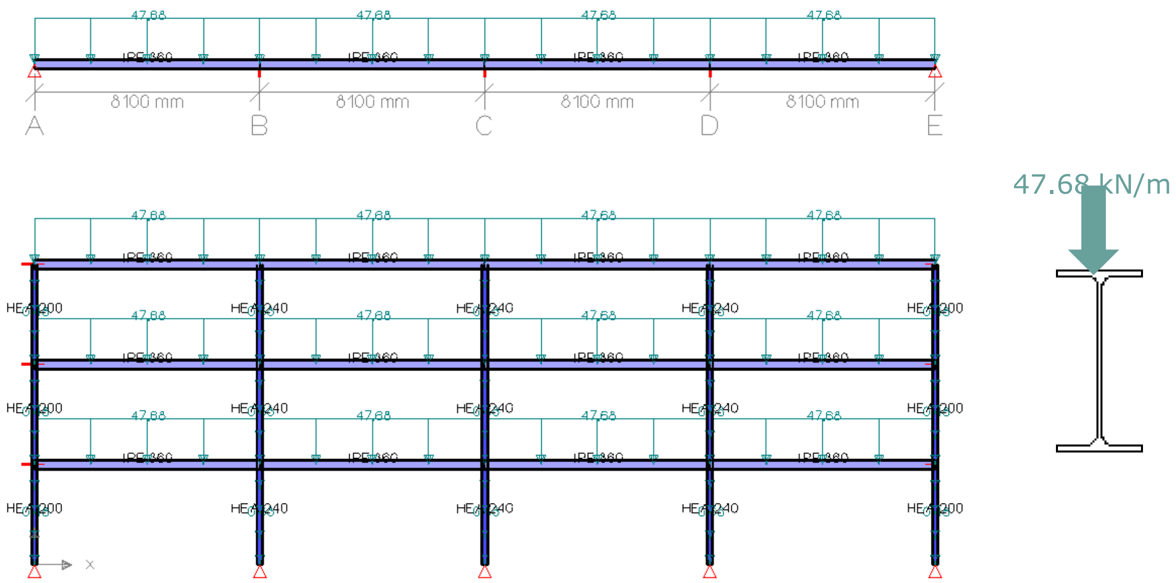
Service load: q = 0.56+25.11+12.15 = 37.82 kN/m (Eq. 6.14 EN 1990)

Design load: q_d = MAX[(1.35·(0.56+25.11)+1.5·0.7·12.15); (1.35·0.85·(0.56+25.11)+1.5·12.15)] = 47.68 kN/m
(CC2 with ξ = 0.85, Eq. 6.10 (a) + (b) EN 1990)

Sketch

The beam can be part of a frame, only the beam will be studied. Storey height 3.6 m.

Figure 6-12. Static system



Commentary

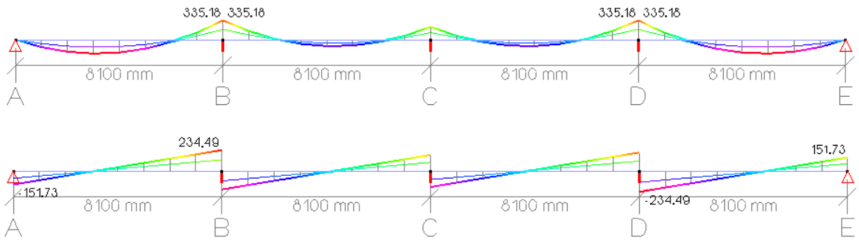
The beam is laterally restrained to prevent out of plane and lateral torsional buckling. Further lateral restraints have to be provided at all plastic hinge locations. Effects of foundation settlements, horizontal (wind) loads, normal forces and more unfavourable combination of mobile loads are disregarded. Elastic displacements are limited till L/250 in the rare combination and an absolute value of 10 mm for the mobile loads. Horizontal displacements at the ends of the frame are blocked by stiff concrete cores or wind bracings.

Determination of internal forces and moments under normal conditions of use

Most practical design starts from an elastic distribution of forces by the aid of commercial (ordinary) software tools or even analytical expressions. This explains the reason why in the following a global elastic analysis is followed.

Elastic global analysis under NC

The moment and shear force redistribution of a four-span beam, equally loaded gives:



$M_{AB} = 0.077 \cdot q_d \cdot L^2$

$M_B = -0.107 \cdot q_d \cdot L^2$

$M_{BC} = 0.036 \cdot q_d \cdot L^2$

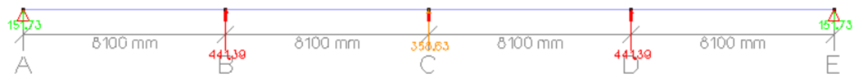
$M_C = -0.071 \cdot q_d \cdot L^2$

$V_{BA} = -0.607 \cdot q_d \cdot L$

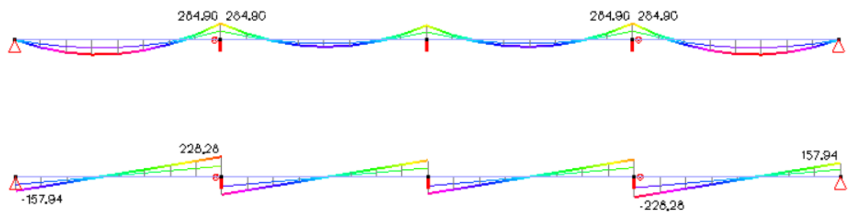
$V_{BC} = 0.536 \cdot q_d \cdot L$

$V_{CB} = -0.464 \cdot q_d \cdot L$

Reactions in ULS:



For class 1 and 2 section a limited (15%) redistribution of forces is allowed according to clause 5.4.1.(4) from EN 1993-1-1. The internal forces and moments in the frame remain in equilibrium with the applied loads, or:



$M^*_{AB} = 0.084 \cdot q_d \cdot L^2$

$M^*_B = -0.091 \cdot q_d \cdot L^2$

$M^*_{BC} = 0.044 \cdot q_d \cdot L^2$

$M^*_C = -0.071 \cdot q_d \cdot L^2$

$V^*_{BA} = -0.591 \cdot q_d \cdot L$

$V^*_{BC} = 0.520 \cdot q_d \cdot L$

$V^*_{CB} = -0.480 \cdot q_d \cdot L$

Reactions in ULS:

Design of cross section under NC

Steel grade S355 EN10025 or $\varepsilon = 0.81$, section IPE360; $t_f = 12.7$ mm, $t_w = 8$ mm and $r = 18$ mm

Flange: $c/t_f = ((170-8-2 \cdot 18)/2)/12.7 = 4.96 < 9\varepsilon = 7.29 \rightarrow$ class 1 (6-11)

Web: $c/t_w = (360-2 \cdot 12.7-2 \cdot 18)/8 = 37.33 < 72\varepsilon = 58.32 \rightarrow$ class 1 (6-12)

Shear

$$V_{Ed,max} = 0.607 \cdot 47.68 \cdot 8.1 = 234.4 \text{ kN} < V_{Rd,IPE360} = \frac{A_{vz;IPE360} \cdot f_{yk}}{\gamma_{M0}\sqrt{3}} = \frac{2976 \cdot 355}{1 \cdot \sqrt{3}} = 609.96 \text{ kN} \quad (6\text{-}13)$$

To avoid shear buckling:

$$h_w/t_w < 72 \cdot \varepsilon/\eta \text{ or } (360-2 \cdot 12.7)/8 = 41.83 < 72 \cdot 0.81/1.2 = 48.6 \quad (6\text{-}14)$$

$\rightarrow$ fulfilled

Bending moment

$V_{Ed,max} < 50\%\ V_{Rd,IPE360}$ (6-15)

$\rightarrow$ no interaction with bending moment

$M_{Ed,max} = 0.107 \cdot 47.68 \cdot 8.1^2 = 334.726$ kNm and/or (6-16)

$M^*_{Ed,max} = 0.091 \cdot 47.68 \cdot 8.1^2 = 284.674$ kNm (6-17)

$M_{Rd,IPE360} \geq \frac{W_{pl;IPE360} \cdot f_{yk}}{\gamma_{M0}} = \frac{1019 \cdot 355}{1} = 361.745 \text{ kNm} > M_{Ed,max}$ (6-18)

$\rightarrow$ both solutions are possible

Determination of internal forces and moments due to a column loss scenario

Accidental load:

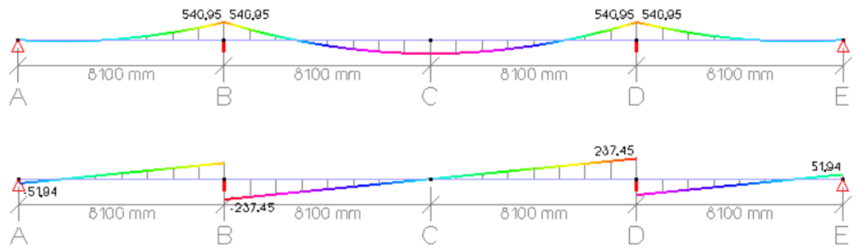
$q_{acc} = 0.56 + 25.11 + 0.3 \cdot 12.15 = 29.31$ kN/m (6-19)

(Eq. 6.11 EN 1990 with the use of Ψ_2)

Dynamic amplification is here neglected and will be treated subsequently.

Elastic global analysis under CL without dynamic amplification

The moment and shear force redistribution of a four-span beam, equally loaded gives:



$M_{AB} = 0.024 \cdot q_{acc} \cdot L^2$

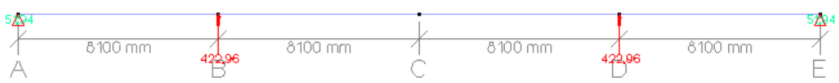
$M_B = -0.281 \cdot q_{acc} \cdot L^2$

$M_C = 0.219 \cdot q_{acc} \cdot L^2$

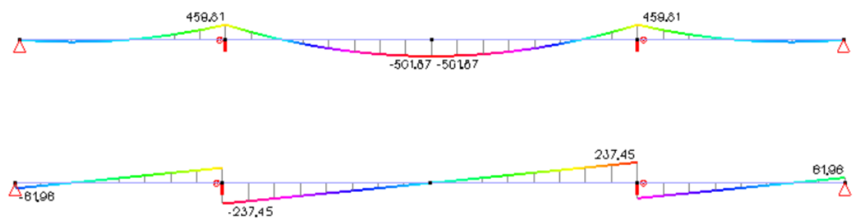
$V_{BA} = -0.781 \cdot q_{acc} \cdot L$

$V_{BC} = 1.000 \cdot q_{acc} \cdot L$

Reactions in the accidental combination give for this example lower values as in the normal ULS design situation:



With limited (15%) redistribution of forces and equilibrium with the applied loads, this becomes:



$M^*_{AB} = 0.034 \cdot q_{acc} \cdot L^2$

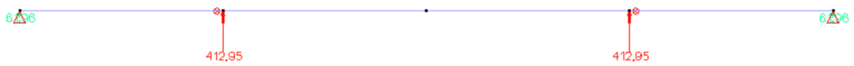
$M^*_B = -0.239 \cdot q_{acc} \cdot L^2$

$M^*_C = 0.261 \cdot q_{acc} \cdot L^2$

$V^*_{BA} = -0.739 \cdot q_{acc} \cdot L$

$V^*_{BC} = 1.000 \cdot q_{acc} \cdot L$

Reactions in the accidental combination give for this example lower values as in the normal ULS design situation:



Remark; plastic global analysis means $M_{B,pl} = M_{C,pl} = (q_{acc} \cdot (2L)^2/8)/2 = 0.250 \cdot q_d \cdot L^2$, $V_{BA,pl} = -0.750 \cdot q_d \cdot L$ and $V_{BC} = 1.000 \cdot q_d \cdot L$ → continue with this optimal values as the section is class 1.

Verification of cross section under CL without dynamic amplification

Shear

$$V_{Acc,pl} = 1 \cdot 29.31 \cdot 8.1 = 237.452 \text{ kN} < V_{Rd,IPE360} = 609.96 \text{ kN} \tag{6-20}$$

To avoid shear buckling:

$$h_w/t_w < 72 \cdot \varepsilon/\eta \text{ or } (360-2 \cdot 12.7)/8 = 41.83 < 72 \cdot 0.81/1.2 = 48.6 \tag{6-21}$$

→ fulfilled

$V_{Ed,max} = 234.42$ kN > 10% $V_{Rd,IPE360} = 60.996$ kN and plastic global analysis → web stiffeners should be provided within a distance along the member of h/2 from the plastic hinge location, where h is the height of the cross section at this locations.

Bending moment

$$V_{Ed,max} < 50\% \; V_{Rd,IPE360} \tag{6-22}$$

→ no interaction with bending moment

$$M_{Acc,pl} = 0.250 \cdot 29.31 \cdot 8.1^2 = 480.757 \text{ kNm} \tag{6-23}$$

$M_{Rd,IPE360} = 361.745 \text{ kNm} < 480.757 \text{ kNm}$

→ plastic hinges are forming which gives cause for the formation of a mechanism. Application of the proposed tying force method can offer a solution to fulfil robustness requirements.

Tying force requirement due to a column loss scenario via double-span beams

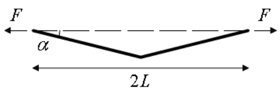
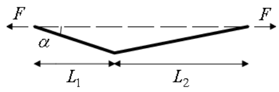
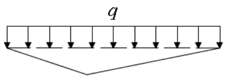
General

The general formulation is given by Eq. (1). The maximum equivalent force coming from the single beam IPE360 (S355 and 7273 mm²) is based on the tensile yield strength or:

$$T = F = f_y \cdot A_{IPE360} = 355 \cdot 7273 = 2581.92 \text{ kN} \tag{6-24}$$

According to **Table 6-8**. with equal spans the intensity factor $i_f = 2.5$ and the equivalent load $P = q \cdot L$ or $P = 29.31 \cdot 8.1 = 237.411$ kN

Table 6-8. Tying parameters for double span beams.

	Equal spans 	Unequal spans ($L_1 \le L_2$) 
Intensity factor: i_f	2.5	$5.0\left(\frac{L_2}{L_1+L_2}\right)$
Equivalent tying force: T	F	F
Equivalent load: P		
	qL	$q\left(\frac{L_1+L_2}{2}\right)$

Without further knowledge about the connections between beams and columns a safe assumption of α should be respected, see proposition in clause 3.6.3.2; α = 0.15 rad and $\bar{\alpha}$ = 0.15/0.20 = 0.75.

At this moment in the assessment procedure this factor η should be taken equally to 2.

The possible reduction factor ρ should be taken equally to 1 as there is no proven strain-hardening and interaction between tying and flexural action at this moment.

Substituting the relevant values in Eq. (A.3-1) gives:

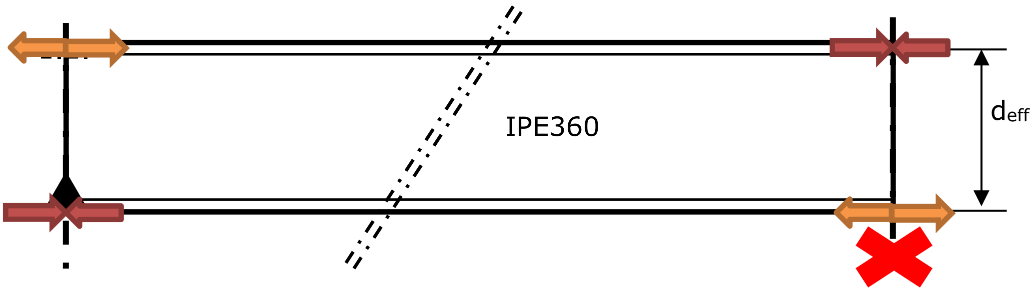
$$T = 2581.92\ kN \geq 2 \cdot 1 \cdot \left(\frac{2.5}{0.75}\right) \cdot 237.41 = 1582.73\ kN \qquad (6\text{-}25)$$

Resistance surrounding structure to horizontal tying

The resistance of the surrounding structure should be checked is based on the ultimate strength of the ties subject to a corresponding material factor. Steel S355 do have a specified yield strength of 355 N/mm² and ultimate strength of 510 N/mm². To be complete no reduction according to the thickness of the flange or web is plied, but is allowed.

$$F_u = 510 \cdot 7273 = 3709.23 \text{ kN} \qquad (6\text{-}26)$$

The vertical distance between the effective compressive centres of rotation <u>before displacement</u> = effective depth d_{eff} of the section. For an IPE section this is the distance between the centroids of the flanges or in this example 360-12.7 = 347.3 mm



The elastic extension δ of the beam under the axial <u>load</u> F_u is equal to

$$F_u/(EA/L) = 3709.23 \cdot 10^3/(210000 \cdot 7273/8100) = 19.7 \text{ mm} \qquad (6\text{-}27)$$

Maximum displacement of the surrounding structure:

$$u \leq \frac{L_1}{2}\left(\alpha - \frac{d_{eff}}{L_1}\right)^2\left(\frac{L_1 + L_2}{L_2}\right) - \delta \tag{6-28}$$

$$u \leq \frac{8100}{2}\left(0.15 - \frac{347.3}{8100}\right)^2\left(\frac{16200}{8100}\right) - 19.7 = 73\,mm \tag{6-29}$$

The concrete cores or wind bracings at the ends of the frame should limit the horizontal displacements in the accidental load case to 73 mm to allow the development of catenary action.

Assume at each end a concrete core (C30/37) with a second moment of area = 5.024 m^4 (4x2,4 m^2 outer dimensions and walls of 20 cm), bending around the strong axes with at 3 levels the load F or by approximation the displacement of a cantilever beam with a height of 3·3.6 = 10.8 m with a distributed load of 3709.23/3.6 = 1030.3 kN/m gives 10.6 mm horizontal displacement = $\underline{u_{core}}$.

Between the cores and starting point of the catenary action there is still 8.1 m steel beam which will be also subjected to the force F. The elongation of this part u_{beam} can be calculated as before:

$$u_{x,beam} = \frac{F}{E_s A / L} = \frac{3709230 \cdot 8100}{210000 \cdot 7273} = 19.7\,mm \tag{6-30}$$

The total displacement u_{eff} = $\underline{u_{core}}$ + $\underline{u_{beam}}$ = 10.6+19.7 = 30.3 mm smaller as 73 mm or criterion fulfilled.

Minimum rotational ductility for activation of tying

The minimum rotational ductility becomes with previous values substituted:

$$\alpha_{min} = \frac{d_{eff}}{L_1} + \sqrt{\frac{2(u_{eff} + \delta)}{L_1 + L_2}\left(\frac{L_2}{L_1}\right)} = \frac{347.3}{8100} + \sqrt{\frac{2(34 + 19.7)}{16200}\left(\frac{8100}{8100}\right)} = 0.124\,rad \tag{6-31}$$

The assumed α = 0.15 rad attends to be bigger as α_{min} or the condition to develop catenary action is fulfilled.

Dynamic amplification

Validation of the conservative approach η = 2 can be made by the application of the energy balance principles can be used as follows:

Proportion of the load resisted by flexural action $\lambda_f = P_f/P$ = part of the load that can be taken by flexural action<u>, even with a plastic redistribution</u> $M_{Rd,IPE360}$ = 361.745 kNm = $P_f 2L/8$ or P_f = 178.64 kN which leads to P_f/P = 178.64/237.41 = 0.75. Note; $M_{Rd,pl}/M_{Acc,pl}$ = 361.745/480.757 = 0.75.

Proportion of the load <u>that can be resisted</u> by tying action under static conditions with the chosen profile; T = 2581 kN and P_t = 2581·0.75/(1·2.5) = 774.57 kN. The proportion of the load that can be resisted by tying action under static conditions will be $\lambda_t = P_t/P$ = 774.57/237.41 = 3.26.

Relative load ratio $\lambda = \lambda_f/\lambda_t$ = 0.75/3.26 = 0.23

Verification is needed if α_{min} = 0.124 rad attends to be bigger or smaller as $\lambda\alpha$ = 0.231·0.15 = 0.035 rad. It is bigger ($\alpha_{min} > \lambda\alpha$) so the dynamic amplification factor becomes:

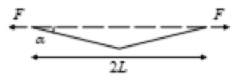
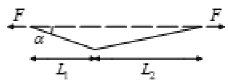
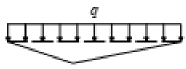
$$\eta = \frac{2}{1+\lambda^2-\frac{1}{3}\left(\frac{\alpha_{min}}{\alpha}-\lambda\right)^2} = \frac{2}{1+0.231^2-\frac{1}{3}\left(\frac{0.124}{0.15}-0.231\right)^2} = 2.14 \leq 2\,? \tag{6-32}$$

With a value higher as the theoretical maximum of 2, the assumed value of 2 will be kept.

Resistance surrounding structure to redistributed vertical gravity loads

Amplification of the gravity loading must be considered to the surrounding structure, especially the neighbouring columns. Arising from the sudden loss of a specific column/load bearing member. From Table 6-9 an amplification factor 0.25+0.75·1.955 = 1.716 can be derived.

Table 6-9 – Amplification of redistributed vertical gravity loading for double span beams.

	Equal spans 	Unequal spans ($L_1 \leq L_2$) 
Redistributed gravity load amplification		
	$0.25 + 0.75\eta$	

Previously the design load of these columns in ULS was equally to (0.607+0.536)·q_d·L = 441.434 kN (elastic analysis without redistribution). In the accidental case this became (0.75+1.00)·q_{acc}·L = 408.38 kN (plastic) in combination with the amplification this becomes however: (0.75+(0.25+0.75·2)·1.00)·q_{acc}·L = 583.40 kN. If the column was optimal designed (Unity Check of 1) in ULS, it will be overloaded by a factor of 1.32! → Additional check of the column and foundation capacity is needed.

Assessment

Global analysis

It is inevitable that the deformations will be concentrated in the joint, and hence we accept them as the weakest link; what is important though is to know their deformation capacity. As the capacity of the section is sufficiently large, the connections must be verified to be able to develop a rotation of at least 0.15 rad and to resist a tying force of at least (or the capacity of the section):

$$T \geq 1582.74 \text{ kN} \tag{6-33}$$

The resistance of the surroundings submitted to horizontal tying was also proofed.

Details

Web stiffeners should be provided within a distance along the member of 360/2 = 180 mm from the plastic hinge location.

Where the cross-section of the member varies along their length (i.e. openings in beams), the following additional criteria should be satisfied:

— Adjacent to plastic hinge locations, the thickness or section of the web should not be reduced for a distance each way along the member from the plastic hinge location of at least 2d, where d is the clear depth of the web at the plastic hinge location see clause 5.6 of EN 1993-1-1.

— Adjacent to plastic hinge locations, the compression flange should be Class 1 for a distance each way along the member from the plastic hinge location of not less than the greater of:

    - 2d, where d is as defined in a) just above.
    - the distance to the adjacent point at which the moment in the member has fallen to 0,8 times the plastic moment resistance at the point concerned.
- Elsewhere in the member the compression flange should be class 1 or class 2 and the web should be class 1, class 2 or class 3.

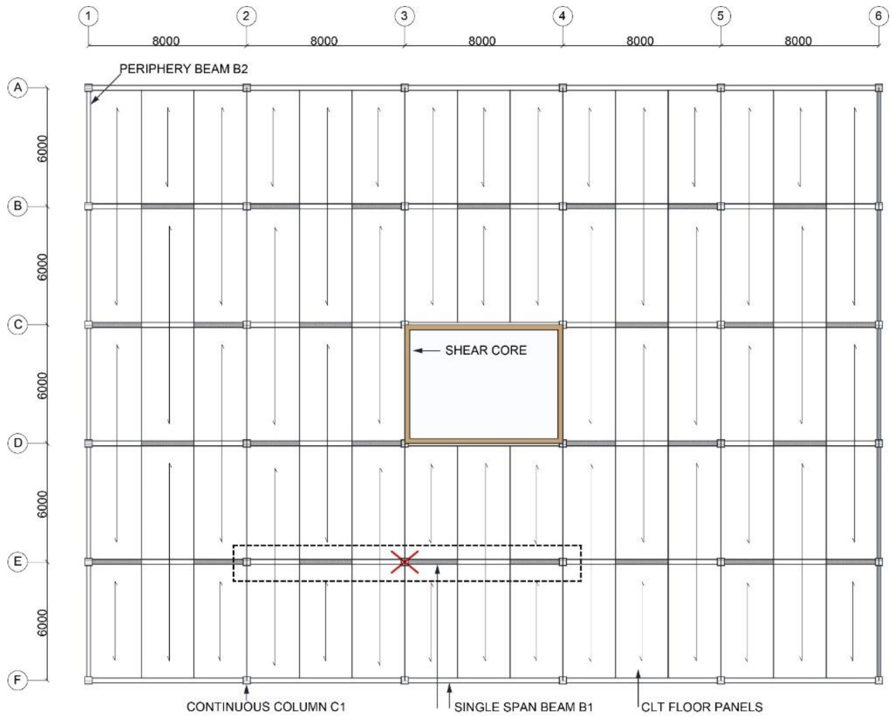
It should be proofed that the connections with the columns can still resist to a tensile force equal to T = 1582.74 kN after yielding due to bending.

Measures should be taken that no other failure mechanism (even not due to overstrength) can cause a failure than those verified in the assessment procedure.

6.1.4.3 Timber Structure Example

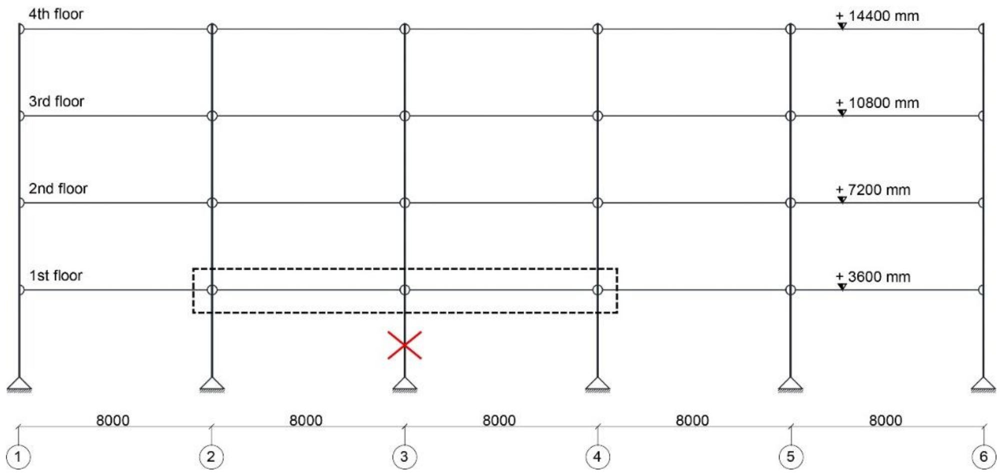
In this example, a 4 storeys 5 × 5-bays post-and-beam timber structure is considered. Plan and elevation views are shown in Figure 6.13 and Figure 6.14, respectively. The structure is used as offices (building category B according to BS EN 1991-1-1, 2002) and falls in consequence class 2a according to BS EN 1991-1-7 (2006). Span lengths along x- and y-direction are equals to 8 and 6 m, respectively, while the inter-story height is equal to 3.6 m. Continuous multi-storey glued laminated columns are adopted. Glulam is used for both primary and periphery beams, labelled as B1 and B2 in Figure 6-13, respectively. CLT slabs 295 mm thick are adopted. Red cross in Figure 6.13 and Figure 6.14 indicates the column suddenly loss, while the dashed rectangle indicates the substructure under consideration.

Figure 6-13. Schematic plan view of the post-and-beam timber building (adapted from Lyu et al., 2020; dimensions in mm) (Martinelli & Izzuddin, 2022)



Source: Martinelli & Izzuddin (2022)

Figure 6-14. Schematic elevation view of the post-and-beam timber building (dimensions in mm)



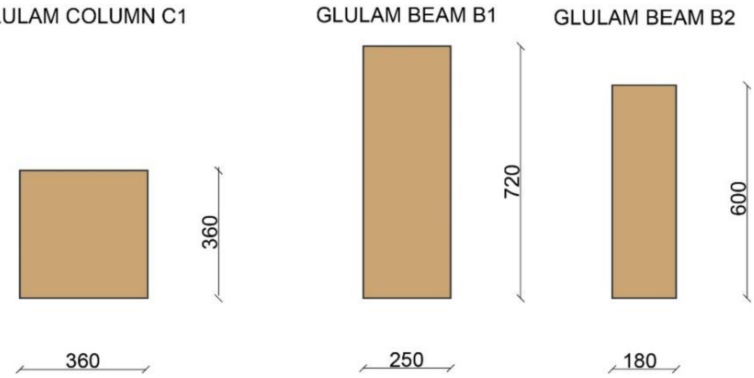
Source: Martinelli & Izzuddin (2022)

With reference to primary beam B1, the beam self-weight, the self-weight of CLT panels, the superimposed dead load and the live load are equal to:

- Self-weight: g_k = 0.72 kN/m (assume a homogeneous glulam beam 250x720 mm of class GL28h with $\rho_k = 410$ kg/m^3)
- Dead load (self-weight of CLT panels): pk = 6.0×1.45 = 8.7 kN/m (assume a 295 mm thickness $\rho_k = 500$ kg/m^3);
- Superimposed Dead load: pk = 6.0×1.0 = 6.0 kN/m;
- Live load B: qk = 6.0×3 = 18.0 kN/m (office building EN 1991-1-1; Ψ0 = 0.70, Ψ1 = 0.50 and Ψ2 = 0.30)

Figure 6-5 shows the cross-sections of the main structural elements C1, B1 and B2. A GL28h class was selected for the glulam structural elements having a characteristic density $\rho_k = 410$ kg/m3, and values of mean and fifth percentile elastic modulus equal to $E_{0,mean} = 12600\ \mathrm{MPa}$ $E_{0,05} = 10200\ \mathrm{MPa}$. CLT panels have a density $\rho_k = 500$ kg/m3.

Figure 6-15. Cross-sections of beams and columns (dimensions in mm)



Source: Martinelli & Izzuddin (2022)

A service class 1 is assumed (EN 1995-1-1, 2004) providing a deformation factor $k_{def} = 0.6$ and modification factors (k_{mod}) for permanent, medium-term and instantaneous load duration class equals to $k_{mod} = 0.6$, $k_{mod} = 0.8$, and $k_{mod} = 1.1$, respectively. Material safety factors for fundamental and accidental load combinations are set equal to $\gamma_M = 1.25$ and $\gamma_M = 1.0$, respectively. Strength values for permanent ($k_{mod} = 0.6$; $\gamma_M = 1.25$), transient ($k_{mod} = 0.8$; $\gamma_M = 1.25$) and accidental ($k_{mod} = 1.1$; $\gamma_M = 1.0$) design situations are listed in Table 6-10.

Determination of internal forces and moment under normal condition of use

Most practical design starts from an elastic distribution of forces by the aid of commercial (ordinary) software tools or even analytical expressions. Reason why in the following a global elastic analysis is followed.

Ultimate limit state

Load combination I

The design load is calculated as (Eq. 6.10 (a) + (b) EN 1990):

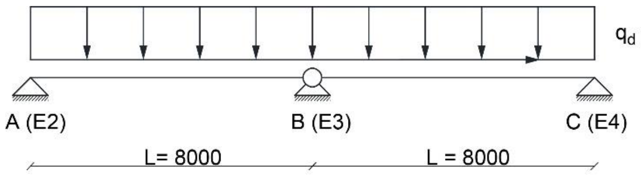
$$q_{d,I} = max\begin{cases} 1.35 \times (0.72 + 8.7 + 6.0) + 1.5 \times 0.7 \times 18 = 39.7\ \mathrm{kN/m} \\ 0.85 \times 1.35 \times (0.72 + 8.7 + 6.0) + 1.5 \times 18 = 44.7\ \mathrm{kN/m} \end{cases} = 44.7\ \mathrm{kN/m} \qquad (6\text{-}34)$$

Table 6-10. Strength values for permanent ($k_{mod} = 0.6$; $\gamma_M = 1.25$), transient ($k_{mod} = 0.8$; $\gamma_M = 1.25$) and accidental ($k_{mod} = 1.1$; $\gamma_M = 1.0$) design situations

Characteristic values			**Design values**			
				$k_{mod} = 0.6$; $\gamma_M = 1.25$	$k_{mod} = 0.8$; $\gamma_M = 1.25$	$k_{mod} = 1.1$; $\gamma_M = 1.0$
$f_{m,k}$ (MPa)	28.0		$f_{m,d}$ (MPa)	13.44	17.92	30.8
$f_{t,0,k}$ (MPa)	19.5		$f_{t,0,d}$(MPa)	9.36	12.48	21.45
$f_{t,90,k}$ (MPa)	0.45		$f_{t,90,d}$ (MPa)	0.216	0.288	0.495
$f_{c,0,k}$ (MPa)	26.5		$f_{c,0,d}$ (MPa)	12.72	16.96	29.15
$f_{c,90,k}$ (MPa)	3.0		$f_{c,90,d}$ (MPa)	1.44	1.92	3.3
$f_{v,k}$ (MPa)	3.2		$f_{v,d}$ (MPa)	1.536	2.048	3.52

Figure 6-16 shows the loading scheme for the glued laminated beam under study.

Figure 6-16. Loading scheme for the single-span beam B1 (dimensions in mm)



Source: Martinelli & Izzuddin (2022)

The shear force at the supports, the maximum bending moment at mid-span and the reactions are reported below:

Shear forces:

$$V_{AB} = V_{BC} = +\frac{1}{2}q \cdot L;\ V_{BA} = V_{CB} = -\frac{1}{2}q \cdot L; \tag{6-35}$$

Bending moments:

$$M_{max}^{+} = \frac{1}{8}q \cdot L^2 \tag{6-36}$$

Reactions:

$$R_A = R_C = \frac{1}{2}q \cdot L;\ R_B = q \cdot L \tag{6-37}$$

The maximum bending moment at mid-span is equal to: $M_{d,I} = 0.125 \cdot 44.7 \cdot 8^2 = 357.6\ \mathrm{kNm}$

The shear force at the support is equal to: $V_{d,I} = 0.5 \cdot 44.7 \cdot 8 = 178.8\ \mathrm{kN}$

Since the load combination includes actions belonging to different duration classes, it will be necessary to choose the value of k_{mod} which corresponds to the actions of shorter duration; for this load combination the value for the medium-term duration must therefore be used: $k_{mod,I} = 0.8$

Load combination II

In this load combination only the dead load are considered:

$$q_{d,II} = 1.35 \times (0.72 + 8.7 + 6.0) + 0 \times 18 = 20.8\ \mathrm{kN/m} \tag{6-38}$$

The maximum bending moment at mid-span is equal to:

$$M_{d,II} = 0.125 \cdot 20.8 \cdot 8^2 = 166.4\ \mathrm{kNm} \tag{6-39}$$

The shear force at the support is equal to:

$$V_{d,II} = 0.5 \cdot 20.8 \cdot 8 = 83.2\ \mathrm{kN} \tag{6-40}$$

In this case, only permanent loads are acting, thus the value $k_{mod,II} = 0.6$ is adopted

Verification of failure conditions of the timber beam and connectors

The most severe load combination for both the bending and shear checks is the one that include both dead and live loads (Load combinations I). Bending and shear checks are satisfied but omitted here for sake of brevity. Readers can refer to the Report of the Project Team WG6.T2 for the complete checks.

The factored design shear force of the beam-to-column connectors under medium-term actions ($V_{d,I}$) is 178.8 kN for the building under study.

Service limit states

The service load is equal to (Eq. 6.14 EN 1990):

$$q = 0.72+14.7+18.0 = 33.4\ \mathrm{kN/m} \tag{6-41}$$

Deflection checks are also satisfied but omitted here for sake of brevity. The complete derivation is reported in the Report of the Project Team WG6.T2.

Column loss scenario

The typical failure modes for which a design strategy for robustness is applicable are:

1. failure/loss of the column only (the beam-column node remains intact): in this case, robustness exploits the coupled behaviour between:
   (a) residual tensile strength of the beam-column connection which allows the activation of a catenary behaviour of the beam itself (second order effects);
   (b) membrane resistance of the floor (taking care to ensure adequate resistance of the connections in the presence of this state of stress);
2. failure of the beam-column node: in this case, the robustness is guaranteed exclusively by the membrane behaviour of the floor.

In this example, the loss of the column only is assumed (the beam-column node remains intact) and the robustness relies on the catenary action of the beam (point 1a)).

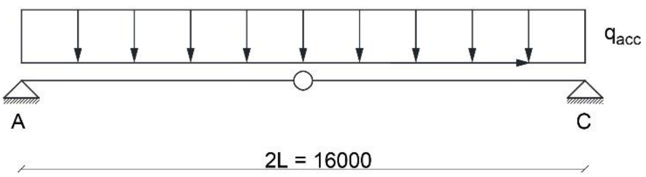
The accidental load combination (Eq. 6.11 EN 1990 with the use of ψ_2) is equal to:

$$q_{acc} = 15.42 + 0.3 \times 18 = 20.8\ \mathrm{kN/m} \tag{6-42}$$

Elastic global analysis under Column Loss

Figure 6-17 shows the loading scheme for the glued laminated beam under study in case of a central column loss (column E3 in6-14). In this case, the substructure shown in Figure 6-17 is unstable and can carry the load only in a deformed configuration.

Figure 6-17. Loading scheme for the beam B1 under central column loss (dimensions in mm)



Source: Martinelli & Izzuddin (2022)

The reactions without dynamic amplification due to the sudden column loss are equal to:

$$R_A = R_C = \frac{1}{2} q \cdot 2L = 166.4 \text{ kN} \tag{6-43}$$

Application of prescriptive tying force method can offer a solution to fulfil robustness requirements.

Tying force requirement due to a column loss scenario via double-span beams

Equivalent load and intensity factor

Table 6-1 provides tying parameters for the double-span beam under study:

Intensity factor: $i_f = 2.5$

Equivalent load: $P = q \cdot L = 20.8 \cdot 8 = 166.4 \text{ kN}$

Equivalent tying force: $T = F$

Rotation ductility

In post-and-beam timber buildings, the lateral stability is provided by either shear core, bracing members, or both. As a consequence, the beam-to-column connections are only designed to carry the shear loads. Nevertheless, it does not imply that these connectors are pinned and cannot resist an applied moment. In the case of a column loss scenario, bending may be applied to these connections. Moreover, shear connectors are in timber buildings mostly made by screws or nails, those components are mainly designed in bending. With this as background, the behaviour can be in a certain way described as ductile. Shear connectors are thus assumed in this example.

Based on the experimental data of Lyu et al. (2020), a beam rotation of 8° is assumed that corresponds to α = 0.14 rad and $\bar{\alpha}$ = 0.14/0.20 = 0.70.

Reduction factor

CLT floor panels which could remain intact after element removal may contribute to the load distribution after column removal. The contribution of the CLT floor panel can be accounted in two ways:

- The first option includes directly the contribution of the CLT panels by using the general formulation expressed in Eq. (1), where the tying via beam and the tying via floor system are both considered and superimposed.
- The second option includes indirectly the contribution of the CLT panels by the aid of the ρ-factor.

In this example the option b) has been selected and a value of ρ lower than one could be assumed. Nevertheless, given the limited knowledge of the CLT effect, a conservative approach is followed and a value of ρ equal to 1 is chosen.

Dynamic amplification

A conservative approach is adopted in this example and a dynamic amplification factor $\eta = 2$ has been used.

Assessment

Surrounding structure

The maximum displacement of the surrounding structure can be evaluated with Eq. (3-2) of Report of the Project Team WG6.T2 with $L_1 = L_2 = L$:

$$u \leq \frac{L}{2}\left(\alpha - \frac{d_{eff}}{L}\right)^2 \left(\frac{2L}{L}\right) - \delta \tag{6-44}$$

where d_{eff} is the vertical distance between the effective compressive centres of rotation (pivots) for the end and internal hinges under bending action. In the example under study, the plastic hinges are in the connections. The evaluation of d_{eff} strictly depends on the type of connections. Assuming a shear connectors at a first approximation d_{eff} can be evaluated as: $d_{eff} = 0.8 \cdot h = 576\ \mathrm{mm}$

The axial force F that produces the elastic extension δ of the beam is equal to:

$$F = \eta \cdot \rho \cdot \left(\frac{i_f}{\bar{\alpha}}\right) \cdot\ P = 2 \cdot 1 \cdot \left(\frac{2.5}{0.70}\right) \cdot\ 166.4 = 1188.6\ \mathrm{kN} \tag{6-45}$$

The elastic extension δ of the beam is equal to:

$$\delta = \frac{F}{EA}L = \frac{1188.6 \times 10^3}{12600 \times (250 \times 720)} 8000 = 4.2\ \mathrm{mm} \tag{6-46}$$

$\delta = \frac{F}{EA}L = \frac{1188.6\times10^3}{12600\times(250\times720)}8000 = 4.2\ \mathrm{mm}$

The maximum displacement of the surrounding structure is equal to:

$$u \leq \frac{8000}{2}\left(0.14 - \frac{576}{8000}\right)^2 \left(\frac{16000}{8000}\right) - 4.2 = 32.8\ \mathrm{mm} \tag{6-47}$$

As general remark, a core on one side may results not sufficient; the combined axial displacements from both end of the tie must be considered. These displacements may be assessed considering the joint details and diaphragm action from the CLT panels. In this example, we assume that there is enough stiffness so that the combined axial displacements under *F* are less than ~33 mm.

Timber beam

The capacity of the timber beam must be verified to satisfy the following expression:

$$T \geq \eta\ \rho\ \left(\frac{i_f}{\bar{\alpha}}\right)\ P \tag{6-48}$$

The maximum tensile force in the beam is equal to:

$$T = f_{t,0,d} \cdot A = 21.45 \cdot (250 \times 720) = 3861 \text{ kN} \tag{6-49}$$

In case of a sudden column loss, the tying force requirement in the timber beam reads as:

$$T = 3861 \text{ kN} \geq \; 2 \cdot 1 \cdot \left(\frac{2.5}{0.70}\right) \cdot 166.4 = 1188.6 \text{ kN} \;\rightarrow \text{fulfilled} \tag{6-50}$$

Timber column

Amplification of the gravity loading arising from the sudden loss of a specific column/load bearing member must be considered to the surrounding structure, especially the neighbouring columns. Table 3.4 of Report of the Project Team WG6.T2 provides an amplification factor 0.25+0.75×2 = 1.75.

Previously the design load of internal column E2 (see plan view in Figure 6-13) in ULS was equal to:

$$N_{d,E2} = (0.5 + 0.5) \cdot q_d \cdot L = 1 \cdot 44.7 \cdot 8 = 357.6 \text{ kN} \tag{6-51}$$

having adopted an elastic analysis. In the accidental load condition this becomes:

$$N_{acc,E2} = 0.5 \cdot q_{acc} \cdot L + 1.75 \cdot 1.0 \cdot q_{acc} \cdot L = 0.5 \cdot 20.8 \cdot 8 + 1.75 \cdot 1.0 \cdot 20.8 \cdot 8 = 374.4 \text{ kN} \tag{6-52}$$

It is worth noting that in the above expression only the double span contribution needs to be amplified by the factor 1.75. The column is overloaded by a factor of 1.05 thus meaning that additional check of the column and foundation capacity are needed.

Connectors

The factored design shear force of the beam-to-column connectors under medium-term actions was calculated previously and was equal to:

$$R_{A,d} = 178.8 \text{ kN} \tag{6-53}$$

In case of column loss, the shear force at the beam-to-column connectors taking into consideration an amplification factor of 1.75 is equal to:

$$R_{A,acc} = 1.75 \cdot \frac{1}{2} q_{acc} \cdot 2L = 1.75 \cdot 0.5 \cdot 20.8 \cdot 16 = 291.2 \text{ kN} \tag{6-54}$$

The connectors is overloaded in shear by a factor of 1.62.

Moreover, it should be proved that the connectors can resist, in addition to the shear force, to a tensile force larger than:

$$T_{conn} \geq 2 \cdot 1 \cdot \left(\frac{2.5}{0.70}\right) \cdot 166.4 = 1188.6 \text{ kN} \tag{6-55}$$

The axial capacity of the timber beam is in general much higher than the axial capacity of the connectors. For this reason, the axial tensile bearing capacity of the connector probably represents the most severe check condition for the system. In a column loss scenario, compared to an ordinary condition, there must be additional connectors to absorb the horizontal forces. Since the column is not able to transfer tension forces perpendicular to the fibres, a good strategy is represented by connectors that pass through the column and then fixed into the beam.

The last equation can be considered a conservative approach in design the connector under a column loss scenario for the following reasons:

— The largest dynamic amplification factor $\eta = 2$ was assumed in the example;

— The potential tying via the CLT floor system was completely ignored; the maximum value of the reduction factor $\rho = 1$ was assumed in the example;

In the example under study, a beam rotation of 8° (α = 0.14 rad) was assumed based on the data of commercially available connectors reported by Lyu *et al.,* (2020) not specifically designed to resist the loss of a column through catenary action.

6.2 Bridge Structures

Agarwal et al. (2012) discuss 20 structural failures, paying particular attention to structural robustness. Faulty design and construction practices determined the collapse of an under construction cantilever steel bridge (Quebec Bridge) in 1907 in Canada. The Almo Bridge, a tubular steel arch bridge, built in 1960 in Sweden, collapsed in 1980 following a ship impact. A 3-span composite concrete-steel road bridge, built in 1961 in Czech Republic, collapsed in 2008 when under reconstruction, due to local overloading of temporary supports. The Haeng-Ju Grand Bridge in Seoul, Korea, is another example of bridge failure during construction. Eleven spans of the continuous pre-stressed concrete girder bridge, collapsed in 1992 due to the failure of a temporary pier. The overall conclusion is that the type/shape of the structure as well as the (design and construction) management practices determine the structural robustness and by default the bridge safety. General considerations with respect to structural robustness of bridges can be found in Starossek (2009), advocating the consideration of design strategies that are related to segmentation or the prevention of local failure.

Wisniewski et al. (2006) formulate a simplistic and efficient deterministic approach to evaluate the load carrying capacity of and existing railway bridge from a robustness perspective. The method comprises two steps: (1) analysis of an individual member that is critical to the structural integrity of the bridge and (2) consider the system response, following the failure of a member, to check the functionality, ultimate and damage condition limit states in order to assess the safety of the system. Step 2 presumes to quantify the robustness of the structure, as a function of redundancy ratio factors (r_f for serviceability; r_u and r_d for ultimate and damaged condition limit states). A total redundancy factor, Φ_{red}, can then be estimated and used to assess the overall safety of the bridge. The case study refers to the Brunna Bridge, built in 1969, a reinforced concrete frame bridge, stretching over four spans of various lengths. The U-shape girder, skewed at approximately 50°, supports a single railway track. The approach does not consider the effects of the skewness and the properties of concrete and steel are considered to be time invariant. The analysis is performed for the bridge being in two condition states: undamaged and damaged. The redundancy of the system is found to be greater than 1 for both condition states. This confirms the safety of the bridge, despite member failure. The authors pointed out the high potential of this methodology if included in the maintenance policies, expanding them from safety assessment at member level to system level.

Cavaco (2009) proposes a deterministic measure to quantify the robustness of two reinforced concrete footbridges subjected to corrosion. Robustness was assumed to be the attribute of a structure used to quantify the level of structural performance at various levels of damage. The analysis was performed for two simply supported beams, 14 m long and 2 m wide. A concentrated load was applied to each at mid-span. A slab was adopted for one structure and an I-beam for the other. The load carrying capacity was evaluated for different levels or depths of corrosion. The results obtained from the analysis were illustrated as functions of normalized structural performance and normalized damage. The area under the curves was assumed to represents the robustness index.

Björnsson and Thelandersson (2010) assume a probabilistic measure to determine the structural robustness of a bridge in case of a train collision caused by a derailment. The response of a multi span post-tensioned reinforced concrete bridge to such an incident was examined. The 172 m bridge located in Malmo, Sweden, stretches over several railway tracks and a four-lane highway. The probability of a derailment occurrence was computed using statistical railway accidents. The hard-impact model (EN 1991-1-7: CEN, 2006) was employed to determine the force of impact resulting from such an event. The probability of a support failure was computed given the train impact. The probability of deck failure was estimated following support collapse. The results indicate that two of the three supports investigated have a substantial risk of failure following a train impact. The findings suggested also that the deck was not design adequately to resist the failure of one of the supports.

Olmati et al. (2013) aim to estimate the robustness of truss bridges from the structure's response to impact load. This method examines the effect of an individual member failure on the overall load carrying capacity of the structure. Therefore, a coefficient known as *member consequence factor* (C_f) was introduced in the structural robustness assessment. Values of 1 for the C_f classify the element as a primary structural element, which means that the failure of the element could cause the total collapse of the structure. Whereas, a value close to 0 of the C_f distinguishes the member as a secondary element of the structure and its failure does not impact the general stability of the structure. The equation adopted to calculate the robustness index on the basis of damage scenario, $R^{scenario} = 100 - C^{scenario}_f$, was updated from the equation proposed in (Nafday, 2011). High values of C_f (up to 100 %) suggests that the member collapse would almost certainly result in the entire system collapsing. On the other hand, values of the C_f on the other side of the spectrum were assumed to indicate a good structural robustness. The approach was tested on small theoretical structures as well as on an existing bridge. The investigation focused on the I-35W Minneapolis steel truss bridge which was built in the early 1960s and collapsed in 2007. Different members of one of the two trusses were removed from the 2D model to simulate damage. The structural robustness was then measured from the C_f for two cases: one in its original form of the truss and the second for an improved form of the truss. The same damage scenarios were considered in both situations. A low robustness index was found from investigating the original structure.

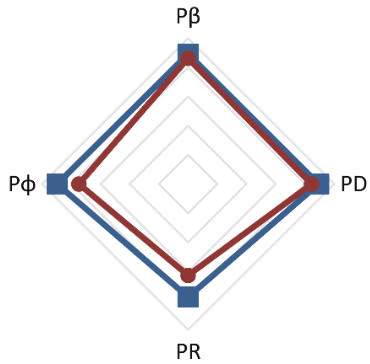
Biondini and Frangopol (2015) employ deterministic and probabilistic approaches to evaluate the life-cycle robustness of a reinforced concrete bridge pier with a box cross-section. The actual damage state of the pier and performance level at different time instants over the structural lifetime were considered in the evaluation. To conform with the principle that the robustness index is the difference between an undamaged and damaged system as per (Frangopol and Curley, 1987, Biondini and Frangopol, 2008), a robustness factor was introduced. This factor was defined as a function of the performance index and the related damage index. It was assumed that the condition of structural robustness was met for robustness factor greater than or equal to 1; otherwise, the system is weak, if the robustness factor is less than 1. The importance of the structure (temporary, ordinary or strategic) is considered to impact the robustness measure and it is integrated in the approach through the *importance factor*. The outcomes of this analysis showed that the probabilistic approach validated the deterministic findings, but emphasised that *"the effects of uncertainty tend to increase over time periods when the susceptibility to damage increases and robustness decreases"*. Moreover, the evaluation of life-cycle robustness of existing structures has the potential to give details on the deterioration impact on the overall performance of a structure, considering the damage state and performance level at the time of the assessment.

Moreira et al. (2016) adopt deterministic measures (such as load capacity and extent of damage) to quantify the robustness index of a masonry arch railway bridge. They adopted the robustness approach proposed in (Cavaco, 2009, Cavaco et al., 2013) to evaluate the ultimate load carrying capacity of the Calharda Viaduct, built in 1882 in Portugal. Damage scenarios such as longitudinal cracking, transversal cracking, spalled masonry arch voussoirs and masonry deterioration and fatigue were considered in the analysis. For different damage scenario with various percentage of damage – 0%, 10%, 25%, 50% and

100% damage – the robustness index was found to be close to 1, which identifies a robust bridge.

Guimarães et al. (2017) suggest a reliability-based robustness assessment model for bridge safety evaluation. Reliability (P_β), damage tolerance (P_D), redundancy (P_R) and ductility (P_Φ) were the robustness indicators evaluated in their research in order to determine the structural performance. The model was validated against a reinforced concrete clamped beam. Different percentages of corrosion (25% and 40 %) were introduced to define the damage scenarios considered. For each damage scenario, the values of the performance indicators were obtained and displayed in a quadratic diagram, with side length of 1, as shown in Figure 6-18. The quadrilateral surface area denoted by the performance indicators was the normalized robustness index. It was observed that each indicator may be quantified for its effect. The robustness index was found to be decreased with the decrease of ductility and redundancy.

Figure 6-18. Performance indicators



Source: Guimarães et al., 2017

Finally, various authors consider the important case of loss of support of bridge structures. Shoa et al. (2021) focus on the case of cable breaks in cable supported structures, Olmati et al. (2012) consider extreme loads resulting in the loss of supporting elements in steel bridge structures whilst Björnsson and Thelandersson (2010) consider robustness evaluation of a multi-span concrete bridge crossing multiple rail tracks in the case of train derailment at the supports. Clearly an important scenario to consider in the analysis of bridge structure robustness the loss of support, its probability and associated consequences should be carefully considered in robustness evaluation of bridge structures.

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