COST Action TU0601
Robustness of Structures

STRUCTURAL ROBUSTNESS DESIGN FOR PRACTISING ENGINEERS

Editor

T. D. Gerard Canisius

EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

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Foreword

This report is a publication of the European network COST Action: TU0601 "Robustness of Structures", prepared by the JCSS-COST Task Group on Robustness of Structures.

'COST Action TU0601: Robustness of Structures' (website: www.cost-tu0601.ethz.ch) is a research network established under the aegis of the COST (European Cooperation in Science and Technology) programme. COST (website: http://www.cost.eu/) is an intergovernmental European framework for international cooperation between nationally funded research activities. The main objective of COST Action TU0601 is to provide the basic framework, methods and strategies necessary to ensure that the level of robustness of structural systems is adequate and sufficient in relation to their function and exposure over their lifetime and in accordance with societal preferences on safety of personnel, the environment and the economy.

The Joint Committee on Structural Safety (JCSS; website : http://www.jcss.ethz.ch/) is concerned with fundamental and pre-normative research in the fields of structural reliability, risk analysis and engineering decision making. The JCSS is supported by the CIB (International Council for Research and Innovation in Building and Construction), ECCS (European Convention for Constructional Steelwork), fib (International Federation for Structural Concrete), IABSE (International Association for Bridge and Structural Engineering) and RILEM (International Union of Laboratories and Experts in Construction Materials, Systems and Structures).

The JCSS Task Group was formed following a recommendation made by the participants at the JCSS/IABSE International Workshop on Robustness of Structures, held in November 2005. The driver for this Workshop attended by an international group of leading researchers and practitioners in structural safety was the renewed interest in structural robustness generated by the tragic events of 11 September 2001, commonly called "9/11". The intention of the Task Group was to produce a document that will bring the latest developments in (risk-based) robustness design of structures to the attention of practising engineers and to aid them in designing buildings. With the initiation of the COST Action TU0601, the development of this document was absorbed into this European Union programme.

The JCSS-COST Task Group on Robustness consisted of the following members of the JCSS and other volunteers drawn from outside of it. (The authors of this document are given in bold font.)

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Although "9/11" created renewed significance, it was with the partial collapse of the Ronan Point building in the UK in 1968 that robustness of structures became important in structural design. Since then many documents that deal with the relevant issues have been published, and they include several post-9/11 guidance documents that originated in the UK and the USA. However, still there is a major information gap which has become more important since the advent of Eurocodes. This is because, for its "Class 3" structures that can have high consequences of failure, Eurocodes recommend risk-based robustness design. Owing to this situation, practising engineers frequently seek relevant guidance and other information. It is to aid them, at least partially, that this document was developed.

This document provides information on methods of quantifying, assessing and designing for robustness, based on current international thinking and new knowledge generated by research and development work including that took place within the JCSS and the COST Action TU0601. It also addresses issues such as robustness during construction and effects of quality control and deterioration that are either not covered or not covered in sufficient detail in current Regulations, Codes of Practice and various guidance documents. However, it was not an objective of the Task Group to give prominence or to advise upon issues related to any particular event, such as "9/11", because the aim was only to help designers and decision makers to deal with robustness issues generally. Designers should to take into account actions and events appropriate for an individual project not just on their own, but through discussions with stakeholders such as the client and relevant regulatory authorities.

The authors and the editor of this document gratefully acknowledge members of the Task Group for their various contributions including the sharing of their knowledge and experience, provision of text and commenting on the document. Although some contributors' names do not appear as authors of chapters because their contributions to the particular text were small in volume, they have significantly helped to improve this document in various other ways.

T.D. Gerard Canisius, *Chairman, JCSS-COST Task Group on Robustness*

Michael Faber, *Chairman, COST TU0601 and President, JCSS*

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1. Introduction

T.D. Gerard Canisius

Robustness, which is a property that makes buildings not suffer disproportionate collapse, including progressive collapse, became a major design criterion following the Ronan Point failure in 1968 (Figure 1.1). In the years following the adoption of this new design criterion, there was an absence of similar and significant disproportionate collapses and, consequently, a gradual reduction in related research. However, some research continued into the second millennium, even to the day of the World Trade Centre collapses on September 11, 2001. Most of this work happened in the UK, particularly at the Building Research Establishment (BRE) and, especially following the bombing of the Murrah Building, in the USA. This research work was predominantly, if not solely, confined to the loss of a single load-bearing member due to a single action.

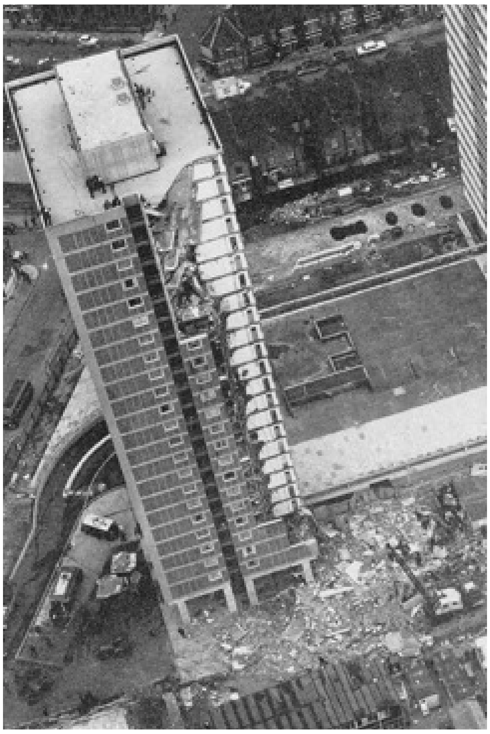


Figure 1.1: Ronan Point Building following partial progressive collapse in 1968. (Photo: UK Crown Copyright)

A significant and worldwide interest in robustness was regenerated by the World Trade Centre incidents, which are commonly called the “9/11” incidents. This renewed interest was unlike any other of the past because the combination of several reasons helped to mark these incidents indelibly in those who experienced, witnessed or heard of them. Among these reasons were the large number of deaths and injuries, the maliciousness of the attacks, the iconic nature of the collapsed buildings, the emotions stirred by the real time worldwide television broadcast of the disaster and public fears that extended to the future. For structural engineers, "9/11" disaster generated further interest because of the following reasons:

- A large number of load bearing members failed on impact of aeroplanes on the two buildings, but none of the buildings collapsed immediately.
- Subsequent fires caused the ultimate collapse of both buildings. (Note: The main fire load here was office furniture and not aeroplane fuel.)

Thus, following '9/11', a combination of human safety and public perception issues made the world think again about the robustness of structures. This new interest made engineers and regulators in various countries to query:

- the adequacy of their national building regulations;
- the adequacy of current knowledge in relation to severe malicious attacks, including impact and explosions, in combination or not with other hazards; and
- the perception of the public on issues related to safety.

It was such issues and concerns that made the International Workshop on Robustness, held at BRE in November 2005, recommend the formation of a Task Group on Robustness with objectives as stated in the Foreword.

1.1 Purpose and Scope of this Document

1.2.1 Objectives

The main objective of this document is to provide practising engineers and relevant stakeholders with state-of-the-art information on robustness issues and help them design structures that are safe in relation to disproportionate failure. Towards this, information is provided on methods of quantifying, assessing and designing for robustness, based on current international thinking and new knowledge generated by research and development work including that took place within the JCSS and the COST Action TU601.

The Task Group's main objective was to address issues that are either unaddressed or not addressed adequately in current Regulations, Codes of Practice and various guidance documents.

A particular objective was to address the relation between robustness and risk and inform engineers on how and why they are inversely related. Another objective was aid designers in the use of risk-based comprehensive concepts for the design of robust buildings as recommended by Eurocodes.

1.2.2 Target Audience

The target audience of this document is wide ranging and includes specialist designers, code developers, regulatory authorities and those involved in research and development activities. However, those who wish to use this document, for their maximum benefit and the correct use of concepts, should preferably be knowledgeable of reliability and risk concepts of structural engineering.

This document is also geared towards providing assistance to CEN TC250 and ISO TC98, when they decide to embark upon developing standards or codes for robustness design of structures.

1.2.3 Scope

The following types of structures are covered by this document.

- Structures that can have common rules or methods for designing for robustness.
- Structures that need special consideration.

Civil engineering structures to which this document is applicable are those referred to by EN1991-1-7:2006 when describing Consequence Classes detailed in Section 2.1 of this document. Accordingly, the definition of robustness used here is that provided in that Eurocode:

> "*Robustness is 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*", as its starting point.

(However, there is an exception when robustness is quantified based on some indices that consider only progressive collapse as described in Chapter 6.)

The principles presented here are valid for any situation involving structures, either on-shore or near-shore. However, in this first edition of the document, **attention is predominantly directed towards buildings.**

Special robustness-related aspects considered within this document are as follows.

- Failure of more than one load-bearing member, by considering the actual hazards
- Assessment and quantification of robustness
- Risk-based decision making
- Methods of providing robustness
- Robustness during construction
- Effects of poor quality and deterioration of materials

The robustness during construction is considered here because of the many reported collapses that had occurred in temporary conditions, even prior to the Ronan Point collapse. However, with them being failures that occurred during the construction of (incomplete) structures, they do not seem to have affected building regulations and codes that deal with structures in service.

The scope of this document is wide as to make it a unique compendium of knowledge potentially not available elsewhere. However, because not all information can be covered in a single document such as this, the reader is also often referred other publications.

1.3 Organisation of the Document

This document consists of ten chapters as follows. The enumerated bullet numbers correspond to the chapter numbers.

1. The first chapter is this introduction to the document.
2. In the second chapter the historic approaches for designing robust structures, current stakeholder requirements and existing practice and regulations are covered.
3. In the third chapter public perception of issues related to robustness and risk acceptance criteria are addressed. The topics covered include issues such as ‘tolerable risk’, risk communication, risk acceptance, and stakeholder participation in decision making.
4. In the fourth chapter attention of the reader is directed towards hazards that can cause disproportionate collapse.
5. In the fifth chapter, various types of consequences of failure of a structure are described. As consequences considered in a risk analysis would depend on the boundary of the system considered, the definition of structural systems is given importance.
6. In the sixth chapter methods of quantifying robustness, that help to study the behaviour of structural systems and their sensitivity to various parameters or decisions, are discussed. In addition, methods for selecting various alternative robustness measures to obtain optimal solutions that maximise performance while reducing costs are discussed.
7. The seventh chapter is used to present more practical information on designing structural systems for robustness.
8. The eighth chapter is dedicated towards the important, but often neglected, aspect of robustness during the construction of structures.
9. The ninth chapter is used to discuss how quality control during construction and post-construction management of deterioration can be used to enhance and/or preserve robustness of structures.
10. The conclusions and recommendations are presented in the tenth chapter.

The information presented in the main chapters is complemented by three appendices that, in the order of presentation, deal with the following topics.

- Appendix A: Terms and Definitions
- Appendix B: Hazard Models
- Appendix C: Robustness in other disciplines.

2. Philosophy and Principles of Robustness

T.D. Gerard. Canisius, Dimitris Diamantidis, Milan Holicky, Jana Markova and Thomas Vogel

The philosophy and principles of robustness are presented in this chapter via descriptions of:

- The history of disproportionate collapse issues and the evolution of relevant regulations.
- Stakeholder requirements that designers should consider, and
- Existing practice and regulations which form the background to the rest of this document.

2.1 History of Disproportionate Collapse Provisions

An overview of the history of the requirements for the mitigation of disproportionate collapse, and progressive collapse, is presented in this section.

The progressive collapse of structures, where the initial failure of one or more load bearing components results in a series of subsequent failures of other components, became a significant safety topic in 1968 after the partial collapse of the Ronan Point multi-storey flats building in London. Although progressive failures, especially of structures during construction, had occurred before the Ronan Point incident, they had not interested engineers and regulators in the way this occupied residential building did. This sudden worldwide interest occurred not only because of the potential that existed for a larger number of fatalities and injuries to have occurred, but also due to the public fears stoked by media reports and the conclusions of the official parliamentary inquiry [Griffith *et al.*, 1968].

The progressive collapses during the construction phase, which too had occurred prior to and subsequent to Ronan Point, have usually been attributed to construction errors. A significant and well-recorded pre-Ronan Point disproportionate collapse that occurred during construction was that of the Officers' Mess at Aldershot in the UK. This too had an official investigation (BRS, 1963) and among its recommendations were:

a) *Where a system of building using prefabricated structural components is extended by use in a new building type, a fundamental re-examination of the system design is necessary. This must include a reconsideration of all design assumptions and, if necessary, a recalculation of the structural design from first principles.*

b) *When novel, or relatively novel building methods are used, the thorough and systematic communication of the designer's intentions to the operative is more than ever essential.*

c) *In systems of construction depending on the assembly of prefabricated structural components, the erection procedure is an essential part of the engineering design.*

These conclusions are applicable now as they were nearly fifty years ago.

An important aspect of the Ronan Point failure was that it happened due to a shortcoming in design knowledge, and this led to a re-evaluation of the Building Regulations and design codes of the time. Although the Ronan Point failure was a progressive collapse, the British regulators pioneered the new design criteria to cover a wider class of failure known as 'disproportionate collapse' of which progressive collapse is a particular example.

Note: *A **disproportionate collapse** need not be progressive, but suffers damage that is disproportionate to the original cause of failure. An example is the collapse of a statically determinate structure from the failure of a single member. In the case of a **progressive collapse**, different members of a statically indeterminate structure fail one after the other as they get overloaded with an accompanying redistribution of load.*

2.1.1 The Ronan Point Building

The partial failure of the 24-storey precast concrete residential flats building occurred on May 16, 1968. In the early morning, a domestic gas explosion within the kitchen of a flat in the eighteenth storey blew out concrete panels forming part of the load-bearing flank wall at a corner of the building. The removal of this part of the load-bearing wall precipitated the collapse of the corner of the block above the eighteenth floor. The weight of this part of the building as it fell caused collapse of the remainder of the south-east corner down to the level of the in-situ concrete podium. The result was a progressive collapse that gave rise to spectacular pictures such as that shown in Figure 1.1

The investigation (Griffith *et al.* 1968) found that there was neither a violation of applicable building standards nor any defect in workmanship in the design (based on the state of the art) and the construction of Ronan Point. *Note:* However, subsequent investigations by others on similar 'tower block' buildings had revealed poor quality construction in other such buildings (see Chapter 9).

2.1.2 Post-Ronan Point Building Regulations

Following the recommendations of the official inquiry in to the Ronan Point failure, the world's first disproportionate collapse regulations came into force in the UK. They were first issued via Circulars of the Ministry of Housing and Local Government (MHLG 1968a and 1968b) and these were aided by documents issued by the Institution of Structural Engineers (IStructE 1968).

The design requirements were first introduced via government circulars, instead of by revising the Building Regulations, because of the urgency created by the many buildings similar to Ronan Point that existed in the country. Implementation via the Regulations, where they are included now, could have been an arduous and time consuming process that involved consultation with stakeholders. It is the Building Regulations of 1972 (HMSO 1972) that gave formal regulatory status to the instructions carried by the Local Government Circulars of 1968 (see Figure 2.1).

(4) A building to which the provisions of this regulation apply shall be so constructed that if any portion of any one structural member (other than a portion which satisfies the conditions specified in paragraph (5)) were to be removed—

(*a*) structural failure consequent on that removal would not occur within any storey other than the storey of which that portion forms part, the storey next above (if any) and the storey next below (if any); and

(*b*) any structural failure would be localised within each such storey.

(5) The conditions referred to in paragraph (4) are that the portion should be capable of sustaining without structural failure the following loads applied simultaneously—

(*a*) the combined dead load, imposed load and wind load;

(*b*) a load of 34 kN/m² applied to that portion from any direction; and

(*c*) the load, if any, which would be directly transmitted to that portion by any immediately adjacent part of the building if that part were subjected to a load of 34 kN/m² applied in the same direction as the load specified in sub-paragraph (*b*).

Deemed-to-satisfy provision for localisation of structural failure

D20.—(1) *In this regulation "storey" and "structural failure" shall have the same respective meanings as in regulation D19(2).*

(2) *The requirements of regulation D19(4)(b) shall be deemed to be satisfied if the area within which structural failure would occur would not exceed 70 m² or 15% of the area of the storey, measured in the horizontal plane, whichever is the less.*

Figure 2.1: An extract from the UK Building Regulations of 1972. (UK Crown Copyright)

The new requirements stated in the above documents were developed in relation to the hazard of internal gas explosion that caused the Ronan Point failure. The overpressure of 34 kN/m^2 (or 5 psi) prescribed for use when designing 'Key Elements' is the most visible evidence of the internal gas explosion related roots of the UK requirements [Canisius, 2006] which were later adopted, or adapted, by other countries as exemplified in the Eurocodes and US standards such as that of the American Society of Civil Engineers (ASCE), General Services Administration (GSA) and the Department of Defense (DOD).

The robustness related requirements currently in force in Europe is the result of further developments that followed much theoretical and experimental studies and professional debate. These include the specification of risk-based design for "Class 3" structures that can have very high consequences of failure.

2.1.3 Design for Situations other than Internal Gas Explosions

The prescriptive detailing methods and the design over pressure of 34 kN/m^2 were originally developed for the design and assessment of wall structures against internal gas explosions. (Note: In the UK, an overpressure of 17 kN/m^2 is used when designing against non-piped gas explosions.) However, these methods are used to design buildings also against other accidental situations. For example, the survival without disproportionate collapse of British buildings during the IRA bombing campaigns in the final decades of the 20th century was considered as a demonstration of the sufficiency of these design and detailing requirements for other situations.

To summarise, for example, the accepted UK practices for general disproportionate collapse design are as follows.

- A building is detailed as given in the Approved Document A of the Building Regulations.
- Where required, components of a building are designed against the uniformly distributed accidental force given in the Approved Document A of the Building Regulations.
- If designed as above, a building is considered as sufficiently safe against other 'accidental' or 'abnormal' loads not explicitly considered in the design, e.g. other explosions and impact. (Note: In reality, bomb explosions are not within the scope of many regulations and building codes, such as the UK Building Regulations and Eurocodes.)
- Buildings with high consequences of failure are to be designed based on a risk assessment.

This British philosophy has been extended and used in the current suite of Eurocodes. According to EN 1991-1-7:2006, a building is to be designed against identified hazards such as the internal gas explosions and impact, as relevant. Then, the building is to be detailed to provide a minimum level of performance against unidentified hazards – which could well be another accidental action that the building was not designed against either in error or because of its (the action's) irrelevance at the time. However, these design requirements are still based on those developed for wall/panel buildings, such as Ronan Point, except that the allowed damage to floors when a load bearing component fails is now 100 m^2 compared to 75 m^2 of the original UK regulations.

A risk-based method can be used for designing against any hazard on any structure. Its pioneering inclusion in the UK Building Regulations was the result of various discussions and consultations in the country. In 2001, an investigation into using 'risk' as the basis for building categorisation provided in the Approved Document A of the UK's building regulations was conducted by the then Department for Transport, Local

Government and the Regions (DTLR,2001). This study, after being further developed by BRE, formed the basis for Table A1 in Annex A of EN 1991-1-7 “Accidental Actions” using an acceptable target risk taken from the CIRIA Report 63 [CIRIA, 1977]. In addition, a probabilistic risk-based method for progressive collapse assessment, developed and implemented by Canisius [2008] for several tall and short large panel system buildings, were used by several Local Authorities to base their management decisions.

2.1.4 Modern Malicious Attacks

The bombing of the Murrah Building in Oklahoma in 1995 [FEMA, 1996] and the aircraft impact and subsequent fires in World Trade Centre buildings in 2001 made nations look anew at their existing disproportionate collapse regulations. More than one single load-bearing member failed in those buildings and, in the case of the latter incidents, two hazards, aircraft impact and fire, materialised one after the other. These seem to point to inadequacies in the existing regulations based on the occurrence of a single hazard and the potential loss of a single member. These major failures resulted in demands for an adequate response from engineers because the public perception was that the involved risks were intolerable.

Since 2001, there have been a large number of new initiatives on disproportionate collapse throughout the world. Most of these have concentrated on bomb explosions and/or fire, the two modern threats to the safety of buildings and their occupants. Besides all these efforts, to date, no building regulation or general codes of practice is known to have specified new requirements for design, with all known publications being ‘best practice’ or codes that remain within the existing frameworks for design. Two publications that considered the potential for more than a single load bearing member failure were by Alexander [2002], who suggested that columns within a certain radius of the location of the potential bomb blast be considered as failing, and Canisius [2006], who discussed how multiple member failures may be considered during a revision of the UK’s Building Regulations.

2.2 Stakeholder requirements

Every building or structure is designed because a client wishes it to be built for some purpose. Therefore, the client is either directly or, via its agent, indirectly involved in specifying the performance requirements for a facility. In addition, because failure of buildings could result in human casualties, governments have introduced Building Regulations as the legal requirements with which any building should comply. These requirements of the Regulations and clients are generally the ‘stakeholder requirements’ addressed in this section.

The requirements present in design codes and standards are ‘good practice’ that aim to deliver the objectives of the government regulations, and they may be supplemented with other performance criteria related to unregulated aspects of design. A building or other structure may also have additional stakeholders requirements such as those related to people affected by its construction, operation and failure. These issues are usually considered during planning or permission applications and there is a duty on a client to take necessary steps to satisfy agreed constraints and reduce/control risks.

The requirements of clients and other stakeholders can be separated into two types as follows.

- The requirements related to the designer, builder and manager of a structure, i.e. the 'professionals', and

- The requirements related to the structure itself.

These two aspects are discussed below.

2.2.1 The Requirements Related to the Professionals

The stakeholders and clients require the professionals to be aware of and consider the following in their design, construction and management of a structure.

- Robustness requirements in relevant regulations and codes, such as:
  - national regulations and standards
  - international standards
  - documents of international organizations

- Principles of verification and design for structural robustness, such as:
  - identification of accidental design situations
  - specification of accidental actions
  - verification of overall structural stiffness
  - verification of vulnerability of structural details
  - quantitative design of horizontal and vertical ties

- Consequences due to insufficient structural robustness, such as:
  - analysis of economic consequences
  - analysis of ecological consequences
  - expected and societal risk

- Fundamental principles of risk assessment (which should be considered), such as:
  - identification structural system(s)
  - identification of hazard scenarios
  - probability analysis
  - quantitative consequence analysis
  - criteria for acceptable risk
  - risk treatment

- Principles of risk optimization (where considered), such as:
  - specification of decisive structural parameters
  - identification of objective function
  - economic measures for fatalities - LQI concept
  - risk criteria based of optimization
  - decisions based on risk optimization.

2.2.2 The Requirements Related to a Structure or Structural System

The basic requirements related to a structure or a structural system are those given in, for example, the European Union's Construction Products Directive (CPD,1989) national regulations such as the UK's Building Regulations and applicable codes and standards.

Robustness requirements in European regulations

Basic requirements on construction works are given in the Construction Product Directive CPD and also in the new Construction Product Requirements (CPR, 2011) (Basic Work Requirements - BWRs). In both documents, the first requirement concerns mechanical stability and resistance. It is stated that "*the construction works shall be designed and built in such a way that the loadings that are liable to act on them during their constructions and use will not lead to any of the following:*

- *collapse of the whole or part of the work*
- *major deformations to an inadmissible degree*
- *damage to other parts of the works or to fittings or installed equipment as a result of major deformation of the load-bearing construction*
- *damage by an event to an extent disproportionate to the original cause."*

The last item above is related to robustness of construction works. It is expected that the new generation of Eurocodes will make reference mainly to the first two BWRs of the CPR and will include extensive provisions for structural robustness.

Robustness requirements in national regulations

A nation's building regulations are the law and hold a pre-eminent position above the codes and standards applicable within the country. Hence, for example, the requirements of the UK's Building Regulations have to be reflected by the codes and standards applicable there.

Robustness requirements in national and international standards

<u>European standards</u>

As a basic requirement the Eurocode for structural design, EN 1990:2002 states that "*a structure shall be designed and executed in such a way that it will not be damaged by events such as:*

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

to an extent disproportionate to the original cause.

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."*

The Eurocode EN 1990:2002 also indicates that levels of reliability relating to structural resistance and serviceability can be achieved by suitable combinations of measures including the degree of robustness (structural integrity). However, the term robustness is not explicitly defined.

As mentioned previously, the Eurocode EN 1991-1-7:2006 defines robustness as "*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*": According to this code, "*a localised failure due to accidental actions may be acceptable, provided it will not endanger the stability of the whole structure, and that the overall load-bearing capacity of the structure is maintained and allows necessary emergency measures to be taken.*"

<u>International standards</u>

The definition of structural robustness (integrity) provided in ISO 2394:1998 was applied during the development of EN 1991-1-7:2006. However, more information on how to achieve the structural robustness is not given in this ISO which is being revised at present by a Task Group that includes several JCSS members.

<u>National standards</u>

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. According to EN 1991-1-7:2006, the key issue is the provision of ties to achieve continuity between adjacent structural members. The development of British standards were driven by the requirements of the local Building Regulations (see above).

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.

The following requirements are listed in Czech standards:

- Directives for houses constructed of panels: requirements for verification of overall spatial stiffness, design of reinforcement of horizontal and vertical joints for 15 kN/m of width or length of a panel house, reinforcement in each joint of vertical and horizontal member by additional or latent ties,

- Design of masonry structures: reinforcing bars at each floor level are required for multi-storey buildings, restriction of the total height of a masonry building, recommended construction rules,

- Regulation 268: 2009 on technical requirements on construction works of the Ministry for Regional Development of the Czech Republic [MRD, 2009]: "*A building shall be designed in such a way that explosion, impact or other overloading will not cause inadequate damage.*"

Documents of international organizations

Some recent documents on structural robustness from international organisations are the following NIST Best Practice Guidelines [NIST, 2006] and the recommendations of the SEI of ASCE [ASCE/SEI, 2010]. Requirements are classified with respect to exposures and failure consequences (notional actions and notional damage). For each class the acceptable extent of collapse and acceptable level of other damage are defined. Design methods and combinations of actions to be taken into account are recommended.

Requirements beyond standards and regulations

In addition to the requirements in regulations and standards, a stakeholder may require robustness of a structure to be based on optimisation concepts, while taking into account the consequences of its failure. Consequences may be expressed in terms of loss of life, injury, economic loss, environmental damage etc.

The objective function for optimisation may be, however, very complex and depend on the type of the structural system, robustness measures, characteristics of failure consequences and probabilities of occurrence and intensities of various hazards. An elementary relationship between the "cost of robustness measures" and "reduction of failure consequences" may be expressed as the following inequality

$$\text{Cost of robustness measures} \quad \leq \quad \text{Reduction of failure consequences} \tag{2.1}$$

If the total cost of robustness measures exceeds the reduction in failure consequences, then the system may be considered as robust but uneconomic. In such a situation, probabilistic methods of risk assessment may be effectively used (Faber, 2006 and Maes *et al.* 2006).

2.3 Existing Approaches on Robustness in European and American Practice

The existing general approach to the design of robust buildings is 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 buildings. However, on certain occasions the involved risks are considered explicitly when designing for robustness, for example with Class 3 buildings of Eurocodes. (Risk-based methodologies that may be used for these buildings are detailed in Chapters 3 to 7 of this document.)

The approaches for robustness design to prevent or reduce the likelihood of disproportionate collapse from two major codes of practice, the Eurocodes and the ASCE standards, are summarised below as additional background to the developments presented in this document.

2.3.1 General

Robustness is commonly understood as the ability of a structural system to withstand events such as explosion, impact or consequences of human errors without being damaged to an extent disproportional to the original cause (ISO 2394:1998 and EN 1990:2002). While ordinary limit states to common types of loads are given, for example, in EN 1991-1-1:2002, the robustness requirements are usually linked to accidental actions (EN 1991-1-7:2006) or other abnormal events. In other words, robustness is also the property of systems that enables them to withstand unforeseen or unusual circumstances without unacceptable levels of consequences or intolerable risks (Gulvanessian *et al.* 2002).

The above mentioned explanation of the term "robustness" indicates that two types of circumstances may cause a failure of a structure or a structural system:

- Extreme but foreseen adverse combinations of actions and material properties. These extreme events include quantifiable abnormal events such as internal gas explosions or impact of vehicles.

- Unforeseen events that may be hardly identified or whose intensity cannot be known in advance, such as bomb explosions, malicious impacts or the effects of unknown errors.

Whereas ordinary structural design is mainly orientated towards the design of structural elements or a structure, the robustness design is concerned also with 'what if' scenarios in relation to component failure. While the main purpose of ordinary design is to avoid failure under foreseen circumstances, the aim of robustness verification is to limit consequences of a local failure due to foreseen and unforeseen circumstances. Here the use of the word 'foreseen' itself is problematic because then it may be asked why a structure was not designed against such a 'foreseen' action. However, a 'foreseen action" should be interpreted as an intensity larger than the design value of that action. Therefore, the term "robustness" should be primarily considered as a property of a structure or structural system, and should not be limited to specific circumstances. A method to define robustness solely as a property of a structure, independent of accidental events, has been proposed by Val [2006].

An important aspect of robustness seems to be the concept of a structure as a system of load bearing members and the way they function together as a 'structural system' (ISO 2394:1998). The considered system may include also non-structural components, in which case the structural system may represent just a subsystem of the whole relevant system. This approach may be particularly useful when consequences of failure are included in a verification of structural robustness and the decision concerning appropriate measures to avoid damage or to ensure that damage is not disproportional to the original cause (Faber, 2006 and Maes *et al.* 2006). Thus, the term "system" can have different meaning depending on the purpose of its use. In general it may be any bounded group of interrelated, interdependent or interacting elements forming an entity that achieves a defined objective in its environment through interaction of its parts. These concepts are further discussed in later chapters of this document.

For a structural system to survive unforeseen events or circumstances, it must possess sufficient reserve capacity to withstand conditions during and after the undesirable event. Therefore, following the event, a robust structural system therefore has to fulfil the inequality given as:

$$\text{Residual capacity} \quad \geq \quad \text{Residual demand} \qquad (2.2)$$

where the word 'residual' refers to the situation after the event, for a considered length of time. *Capacity* usually relates to resistance to forces (i.e. strength), but it may also mean deformability, ductility, stability, weight, or stiffness.

At present the design of structures for robustness is commonly limited to general structural requirements related to:

- Horizontal ties at the level of floor slab.
- Vertical ties between columns and walls at different floors.
- Horizontal to vertical ties between floors and columns or walls.
- Design of 'key elements' against specified forces, when the failure of a component could give rise to disproportionate collapse.

The events to be taken into account during 'key element' design may be those specified by the National Competent Authority and provided in National Annex to EN 1991-1-7:2006 of each CEN Member State. The structural form, size and the consequences of failure of the individual project will also have a bearing on the events to consider.

The existing practices specified within the Eurocodes and the American ASCE codes are discussed in the next two sections.

2.3.2 Robustness in Eurocodes

The basic European document for structural design is the Eurocode EN 1990:2002, according to which sufficient structural reliability can be achieved by suitable measures, including with an appropriate degree of structural robustness. In EN 1991-1-7:2006 [8] two strategies are presented for the accidental design condition, in general.

- The first strategy is based on identified extreme events (internal explosions, impact etc) and includes:
    a) design of the structure to have sufficient robustness
    b) prevention and/or reduction of the intensity of the action (protective measures)
    c) design the structure to sustain the action
- The second strategy is based on the limiting of the extent of local failure, i.e.:
    a) enhanced redundancy (alternative load paths)
    b) key element designed to sustain additional accidental load
    c) prescriptive rules (integrity, ductility)

For these strategies the Eurocode EN 1991-1-7:2006 provides three consequence categories for the design of structures under extraordinary events as shown in Table 2.1.

Consequences class	**Example structures**
Class 1	low rise buildings where only few people are present
Class 2, lower group	most buildings up to 4 storeys
Class 2, upper group	most buildings up to 15 storeys
Class 3	high rise building, grandstands etc.

Table 2.1: Consequence Classes of Eurocode EN 1991-1-7:2006.

The design strategies that consider these consequence classes are expected to lead to adequate robustness of structures such that damage to them is not disproportional to the original action. Thus, a structure is expected to withstand the effects of undefined extraordinary events to a level of safety implied by these measures. Where a failure could occur only a limited time after the activation of a hazard, a code may prescribe the minimum period of time that the structure must stand after the event. For example, in the case of a fire, this requirement can be the time necessary to safely evacuate people from the affected building and its surrounding area, and any potential time emergency services may need to be inside the building. Structures that affect public security require longer survival times than the evacuation time.

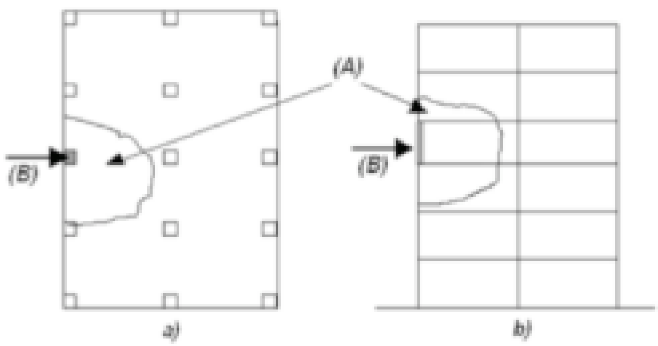
Enhanced redundancy measures

The Eurocode provides some structural measures to achieve robustness in buildings. These measures are mainly active vertical and horizontal ties (traction anchors). For main structural elements, that are designed to be capable of carrying an accidental action, the design verification is to be done using the actions that act on the main element and the adjacent components and their joints. It is thus necessary to consider the entire structure and not single elements in isolation.

The accidental design load according to EN 1990:2002 is to be applied as a concentrated load or a uniformly distributed load (when the accidental action may be considered as quasi-static one).

Key element design

A building should be checked to ensure that upon the notional removal of each supporting column, each beam supporting a column (i.e. a transfer beam/girder), or any nominal section of load-bearing wall, one at a time in each storey, the building remains stable and that any resulting damage does not exceed the limit given in Figure 2.1. Where the loss of a structural member causes more structural damage than allowed, that member should be designed as a Key Element to sustain a load of 34 kN/m^2. For assessing damage, an analytical model of the structure can be used (EN 1991-1-7:2006).



Legend:
a) Floor plan
b) Elevation with vertical section
(A) Local damage less than 15 % of floor area but not more than 100 m^2 simultaneously in two adjacent floors
(B) Column, removed for analysis

Figure 2.1: Recommended limit of acceptable damage

Risk-based design

For structures in Consequence Class 3 (CC3) group, a systematic risk assessment is required under applicable hazards. However, there is no requirements related to risk prescribed in the code. It is the function of the authorities and/or stakeholders such as facility owners and users to prescribe these. Some information on this aspect and examples are available in (ISO2394:1998, Vrouwenvelder *et al.* 2001, Canisius 2008, etc) and in Chapter 3 of this document. General guidance for the planning and execution of risk assessment in the field of buildings and civil engineering structures is given in EN 1991-1-7:2006.

The three steps of the risk analysis can be based on the methodology of EN 1990:2002 as follows.

(a) Assessment of the probability of occurrence of various hazards, including their intensity

(b) Assessment of the probability of various states of damage and of the associated consequences of failure under the considered hazards

(c) Assessment of the probability of further failure of the damaged structure, together with the associated additional consequences of failure.

In the code, measures are also proposed to minimize the risk such as:

a) Prevent occurrence or decrease intensity of the hazard

b) Monitoring of the hazard in order to control it

c) Avoidance of collapse by changing the structural system

d) Overcoming of the hazard by enhanced strength and robustness, availability of alternative load paths by redundancies, and so on

e) Controlled failure of the structure, if the risks to human life is low.

2.3.3 The United States Approach (ASCE 7- 10, 2010)

The ASCE document 7-10 includes a commentary, that provides the user with precautions in design to limit the effects of local collapse. The ASCE recommends design alternatives for multi-storey buildings to make them posses a level of structural integrity similar to that inherent in properly designed conventional frame structures. There are a number of ways to obtain resistance to progressive collapse and in the ASCE 7-10 two ways of design, direct and indirect design, are described.

The *direct design* considers the resistance to progressive collapse *explicitly* during the design process itself. This can be obtained by the *alternative load path method* which allows local failure to occur without major collapse, because the other load path(s) will allow the damage to be 'absorbed'. The structural integrity of a structure may be tested by analysis to ascertain whether alternative paths around hypothetically collapsed regions exist. In addition the Standard recommends the *specific load resistance method*. This method seeks to provide sufficient strength to resist failure from accidents or misuse. This may be provided in regions of high risk since it may be necessary for some elements to have sufficient strength to resist abnormal loads in order for the structure as a whole to develop alternate paths.

The design philosophy necessitates that accidental actions are treated in a special manner with respect to load factors and load combinations.

The *indirect design* considers the resistance of progressive collapse during the design process *implicitly* through the provision of minimum levels of strength, continuity, and ductility. Alternative path studies may be used as guides to develop rules for the minimum levels of these properties needed to apply the indirect design approach to enhance structural integrity. Furthermore the ASCE provides specific recommendations to achieve a resistance to progressive collapse, as described next.

<u>Ties:</u> Provide an integrated system of ties among the principal elements of the structural system. These ties may be designed specifically as components of secondary load-carrying systems, which often must sustain very large deformations during catastrophic events.

<u>Returns on walls:</u> Returns on interior and exterior walls will make them more stable.

<u>Changing direction of span of floor slab:</u> Here, a single span floor can be reinforced also in the perpendicular direction such that, in the case of failure of a load-bearing wall, collapse of the slab can be prevented and the debris loading of other parts of the structure minimised. Often, shrinkage and temperature steel will be enough to enable the slab to span in an additional direction.

<u>Load-bearing interior walls:</u> The interior walls must be capable of carrying enough load to achieve the change of span direction in the floor slabs.

<u>Catenary action of floor slab:</u> Where the slab cannot change span direction, the span will increase if an intermediate supporting wall is removed. In this case, if there is enough reinforcement throughout the slab and enough continuity and restraint, the slab may be capable of carrying the loads by catenary action, though very large deflections will result.

<u>Beam actions of walls:</u> Walls may be assumed to be capable of spanning an opening if sufficient tying steel at the top and the bottom of the walls allows them to act as the web of a beam with the slabs above and below acting as flanges.

<u>Redundant structural system:</u> Provide a secondary load path (e.g., an upper level truss or transfer girder system that allows the lower floors of a multi-storey building to hang from the upper floors in emergency) that allows framing to survive removal of key support elements.

<u>Ductile detailing:</u> Avoid low ductility detailing in elements that might be subject to dynamic loads or very large distortions during localized failures (e.g., consider the implications of shear failures in beams or supported slabs under the influence of building weights falling from above).

<u>Reinforcement:</u> Provide additional reinforcement to resist blast and load reversal when blast loads are considered in design.

<u>Compartmentalization:</u> Consider the use of compartmentalized construction in combination with special moment-resisting frames in the design of new buildings when considering blast protection.

Additional: While not directly adding structural integrity for the prevention of progressive collapse, the use of special, non-frangible glass for fenestration can greatly reduce risk to occupants during exterior blasts. To the extend that non-fragible glass isolates a building's interior from blast shock waves, it can also reduce damage to interior framing elements (e.g., supported floor slabs could be made to be less likely to fail due to uplift forces) for exterior blasts.

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3. Public Perception and Acceptance of Risk in Buildings and Other Structures

Bruce R. Ellingwood

3.1 Introduction

Building structures customarily are designed to withstand loads arising from occupancy and natural environmental events. The normal design process usually provides a degree of structural integrity that is also available to withstand challenges from unforeseen events. Events outside the design envelope, including accidents, misuse, and sabotage or terrorist attack, may precipitate a disproportionate (or progressive) catastrophic collapse. Changes in design and construction practices over the past several decades have removed inherent robustness from many modern structural systems, making them vulnerable to challenges from such events. Social and political events and factors also have led to an increase in abnormal events that may pose a threat to buildings. Finally, public awareness of building safety issues has increased markedly during the past thirty years as a result of media coverage of natural and man-made disasters. Improvements to building practices to enhance robustness and to lessen the likelihood of unacceptable damage from low-probability, high-consequence threats or progressive (disproportionate) collapse now are receiving heightened interest among structural engineers and other building design professionals.[1]

Enhancing building robustness through the design and construction process requires consideration of numerous uncertainties. Some of these uncertainties are inherent (or aleatoric) at the customary scales employed in structural analysis; these would include material strengths, occupant and environmental loads or actions. Other uncertainties are knowledge-based (or epistemic), and arise from limitations in modelling and insufficient databases. These uncertainties give rise to risk. Building risk cannot be eliminated; it must be managed in the public interest through both technical and non-technical means. Risk management often involves difficult choices. Achieving reductions in risk requires additional resources, which must be balanced against competing priorities for those resources and may impact building amenity as well. Concepts and constraints in risk management must be communicated among the project stakeholders – the prospective owner, project developer, architect, engineer, contractor, occupants or tenants (if they can be identified at the project development phase) – and to the building regulatory community and the public at large. Most of these individuals are unfamiliar with concepts of quantitative risk or structural reliability analysis. Accordingly, risk must be measured and communicated in such a way that non-technically trained decision-makers can understand its full dimensions and can devise effective strategies for its management. It is essential for building stakeholders to arrive at a common understanding of how risk is to be measured for any building project, as this will

[1] A special issue of the *Journal of Performance of Constructed Facilities, ASCE*, Vol. 20, No. 4, November 2006, is devoted to mitigating the potential for progressive structural collapse.

determine the performance objectives for that project. The public expects the built environment to be essentially risk-free. The performance of civil infrastructure during recent natural disasters has drawn attention to deficiencies in socio-political approaches to hazard management, but appears to have done little to change these expectations on the part of the public.[2]

The term “risk” is often used interchangeably with “probability” when confronting a potentially hazardous natural or man-made event, and is thought of as the complement of “reliability.” The notion of relative likelihood (expressed as probability or annual frequency) certainly is essential to understanding risk in the everyday sense. But the risk of low-probability events with trivial consequences is negligible, while the risk of events with the same low probabilities but with grave consequences can be judged unacceptable. The annual probabilities or frequencies of events that threaten most buildings and other structures in the built environment are very low by any objective measure. The fact that the probabilities are so small makes communicating risks to project stakeholders and public regulators or decision-makers especially difficult, even if the potential for human or economic losses are substantial, because there is little information against which the risks can be benchmarked. Thus, one often must look beyond probability for a satisfactory metric of risk. To most decision-makers, ranging from engineers and facility managers with professional training to elected officials representing the public at large, it is the consequences (deaths or injuries, direct economic losses, and deferred opportunity losses) that are most important.

To understand how the public perceives and processes information on risk in the built environment and the context in which such risks are accepted, it is necessary to introduce some fundamental concepts of risk analysis and assessment [Ellingwood, 2001; 2007]. Following this introduction, we explore what make a risk “acceptable” and propose several strategies for communicating risks.

3.2 Framework for risk assessment and engineering decision analysis

The concept of risk involves three components: hazard, consequences and context (Elms, 1992). The hazard is an action or state of nature or an action – earthquake, fire, terrorist attack - that has the potential for causing harm. In some instances, the hazard (or spectrum of hazards) can be defined in terms of annual frequency. More often than not, however, it is necessary to envision a scenario (or spectrum of scenarios), without regard to their probability or frequency of occurrence. The occurrence of the hazard has consequences – building damage or collapse, personal injury, economic losses, damage to the environment – which must be measured by an appropriate metric reflective of some value system. Finally, there is the context – individuals or groups at risk and decision-makers concerned with managing the risk

[2] A notable exception is in the seismic risk arena, where the surge in interest in performance-based earthquake engineering following the Northridge Earthquake of 1994 stems from the recognition that providing for life safety through building regulation is necessary but not sufficient to mitigate unacceptable economic losses to the residents or the business community affected by the earthquake.

may have different value systems, view the same risk very differently, and may take different views on how investments in risk reduction must be balanced against available resources.

Quantitative measures of risk are required to achieve (at least) an ordinal ranking of preferences in decision-making. A basic mathematical framework for building risk assessment that can be used for this purpose is provided by the familiar theorem of total probability:

$$P[\text{Loss} > \vartheta] = \Sigma_H \Sigma_{LS} \Sigma_D \, P[\text{Loss} > \vartheta|D] \, P[D|LS] \, P[LS|H] \, P[H] \qquad (3.1)$$

in which P[•] = probability of the event in the brackets. The term P[H] is annual probability of occurrence of hazard H (for rare events, P[H] is numerically equivalent to the annual mean rate of occurrence, λ_H, which is more easily estimated from the data maintained by public agencies); P[LS|H] = conditional probability of a structural limit state (yielding, fracture, instability), given the occurrence of H; P[D|LS] = conditional probability of damage state D (e.g., negligible, minor, moderate, major, severe) arising from structural damage, and P[Loss > ϑ|D] = annual probability (mean frequency) of loss exceeding ϑ, given a particular damage state. If the hazard is defined in terms of a scenario (or set of scenarios), the risk assessment Eq (3.1) becomes,

$$P[\text{Loss} > \vartheta|\text{Scenario}] = \Sigma_{LS} \Sigma_D \, P[\text{Loss} > \vartheta|D] \, P[D|LS] \, P[LS|\text{Scenario}] \qquad (3.2)$$

The parameter ϑ is a loss metric: number of injuries or death, damage costs exceeding a fraction of overall replacement costs, loss of opportunity costs, etc, depending on the objectives of the assessment. Note that it is not possible to obtain an unconditional loss estimate with the scenario approach. This is a potential drawback of the scenario-based approach to risk assessment if the engineering decision process requires the risk estimates to be benchmarked against risks associated with other technological hazards.

Turning now to buildings, bridges and other civil infrastructure, the over-riding concern is with public safety and the event {Loss > ϑ} is replaced with {Life-threatening damage} or {Building collapse}. If each of the distinct hazards challenging a building is represented by an event, H, and if D denotes one of several states of local damage, e.g., {loss of exterior load-bearing wall}, then the probability of structural collapse is (cf Eq (3.1)),

$$P[\text{Collapse}] = \Sigma_H \, \Sigma_D \, P[\text{Collapse}|D] \, P[D|H] \, P[H] \qquad (3.3)$$

in which P[D|H] = probability of local damage given that H occurs, and P[Collapse|D] = probability of collapse, given that hazard and local damage both occur. Many of the current regulatory approaches to progressive collapse mitigation (e.g., UFC, 2009; GSA, 2003) stipulate removal of certain key structural elements (columns or bearing walls) around the perimeter of the building. In that case, the decision metric becomes simply P[Collapse|D][3].

Eqs (3.1) – (3.3) deconstruct the risk analysis into its major constituents and, as an added feature, along disciplinary lines. Reading these equations from right to left conveys the order in which the risk assessment and mitigation process should be approached. Structural engineering has little or no impact on λ_H, which is affected by other means – changing the building site, requiring minimum stand-off distances, installing protective barriers, limiting access to the building, controlling the use of hazardous substances, etc. If λ_H is less than the *de minimis* threshold,[4] the probability of damage or failure due to H is unlikely to contribute to P[Loss > ϑ] or P[Collapse]. That hazard then can safely be ignored. Conversely, if λ_H is one or two orders above the *de minimis* threshold, further investigation of that hazard is warranted. An analysis of competing hazards allows decision-makers to screen out trivial hazards and to devise appropriate risk mitigation strategies for those hazards that lead to unacceptable increases in building failure rates above the *de minimis* level. Clearly, the relative importance of competing hazards may depend on the target risk selected for a building project. For example, suppose that a target risk for a facility has been set at 10^{-6}/yr. If it can be demonstrated that the annual frequency of fire is also 10^{-6}/yr, the fire poses a relatively insignificant threat; conversely, if the annual frequency is 10^{-4}/yr, additional measures to mitigate the risk are warranted. Structural engineering of course directly impacts probabilities P[LS|H], P[D|LS] and P[LS|Scenario]. The conditional probability, P[Loss > ϑ|D], is best determined by the building owner (or manager) and insurance underwriter, as it involves estimation of losses in revenue or business opportunity and the cost required to insure those losses. It should be emphasized that risk mitigation strategies require appropriate attention to *all* terms in Eqs (3.1) - (3.3) to determine the most cost-effective solution for the circumstances or scenario at hand. All sources of uncertainty, from the hazard occurrence to the response of the structural system, must be considered, propagated through the risk analysis framework defined by Eqs (3.1) – (3.3), and displayed clearly to obtain an accurate picture of the risk.

3.3 Measuring Risk

The notion of measuring risk in structural design through probabilities or expected losses has been adopted in general building codes only relatively recently (Ellingwood, 1994). First-generation probability-based limit states design criteria for buildings and other structures, such as *ASCE Standard 7-10* (ASCE, 2010), are based on notional benchmark limit state probabilities (related to “reliability indices” in

[3] In Europe, the collapse area for decision making, when a load bearing member is removed notionally, is limited to a specified value or fraction of the total floor area. See Chapter 2.
[4] The *de minimis* threshold defines the annual frequency below which societies normally do not impose any regulatory requirements for risk management. This level is thought to be approximately 10^{-7} to 10^{-6}/year (Pate-Cornell, 1994).
(This threshold is in relation to common buildings designed with codes and do not involve significant consequences of failure.)

so-called first-order reliability analysis)[5], which were determined through a complex process of calibration to existing engineering practice rather than *ab initio.* The United States *DOE Standard 1020-02* (DOE, 2002) is applicable to a much narrower range of buildings, but takes a similar approach, identifying target performance goals in terms of annual probability for structures, systems and components in different design categories (SDC) exposed to natural hazards such as wind, earthquake and flood. Since the SDCs are identified with progressively more severe consequences of failure (severity of radiological or toxicological effects), the "consequence" element of risk is addressed indirectly by reducing the target probability in order-of-magnitude steps for the more critical SDCs. More sophisticated risk measurements through expected losses – deaths and injuries, direct and indirect economic losses – have been conducted for certain very large or unique projects, where the consequences of failure are catastrophic or where the cost of the project warrants the additional investment in the risk analysis, or where the regulatory authority requires it.[6] The idea of performing such evaluations for general civil infrastructure, however, is quite recent; such evaluations tend to be project-specific and concentrated in the earthquake arena, where the databases necessary to perform the risk assessment are gradually becoming available.

Along with measuring risk it is important to convey some sense of the accuracy (or confidence) in the measurement. If there were no uncertainties in the models of engineered systems or deficiencies in the supporting databases, the risk measure would be a point estimate that reflects purely aleatoric uncertainty. This is seldom the case; the epistemic uncertainties in the assessment of performance of a complex facility subjected to a rare event usually are substantial, and it is essential that they be displayed in the assessment and reflected in any risk-informed decision because they determine the level of confidence that can be placed in the assessment. The impact of epistemic uncertainty on the risk analysis framework, Eqs (3.1 – 3.3) can be visualised as causing the probability models describing hazard, structural capacity, damage and loss to be, themselves, random, with relative frequencies that define the relative plausibility of the models. As a result, the risk estimate (damage probability, expected loss) is characterized by a (Bayesian) frequency distribution, the dispersion of which displays the epistemic uncertainty in the estimate and conveys a sense of the accuracy in the risk assessment[7]. If the epistemic uncertainties in the risk assessment are small, the frequency distribution is centered on the point estimate; conversely, if the epistemic uncertainties are large, the frequency distribution of P[LS] will be broad. One can, of course, compute a mean value of this frequency distribution if a point estimate of risk is required. The mean risk is an unambiguous metric of risk, is a natural choice in performing minimum expected cost (or loss) analyses, and there are decision-theoretic

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[5] The importance of the reliability index, β, for measuring and communicating risk should be noted. It is unlikely that probability-based limit states criteria would have been accepted in the 1980's by professional structural engineers, let alone the building codes, had they been justified on the basis of a benchmark probability of 10^{-5}/yr. The reliability index is more easily understood, and avoids the difficulty that is inherent to explaining such low-probability events.
[6] Risk-informed decision-making has a longer history in design and development of critical facilities such as nuclear power plants, chemical refineries and process facilities, LNG storage facilities, offshore platforms, and similar facilities.
[7] This frequency distribution is analogous to a sampling distribution for a parameter estimate in classical statistics.

justifications for selecting it (McGuire, et al, 2005). Whether to use a point or interval estimate of risk depends on the nature of the decision at hand.

3.4 Risk tolerance and acceptance

As noted above, individuals or decision-makers may have different views on the acceptability of risk and on whether/how investments in risk reduction should be balanced against available resources. Most individuals are risk-averse (implying that they require a substantial benefit in return for accepting marginal increases in risk). Starr's early (1969) study revealed that acceptable risk to individuals (measured by annual frequency of occurrence) may be approximately three orders of magnitude higher for activities that are undertaken voluntarily than for those that are involuntary. Starr later noted that acceptable risk is determined by an individual's *perception* of his/her ability to manage the situation creating the risk. Governments and large corporations tend to be risk-neutral and often base their decisions on minimum expected cost. Recent studies, summarized by Corotis (2003), have indicated that *acceptance* of risk is based more on the *perception* of risk than on the actual probability of occurrence and that any biases in perception shape decisions. While probability of occurrence is a significant component of risk, other factors also are important (Vrijling, et al, 1998). For example, most societies view incidents involving large numbers people differently from incidents involving individuals, as illustrated by public policy toward airline vs. automobile safety. Familiarity and dread also play a role (Slovic, 2000); public attitudes toward the risks posed by commercial nuclear power are a case in point. Reid (2000) has suggested that individuals view risks as negligible if comparable to mortality risk from natural hazards (on the order 10^{-6}/yr) and as unacceptable if comparable to mortality from disease (on the order 10^{-3}/yr in the 30 to 40 age group). Although it is natural to use mortality statistics from disease and accidents to benchmark risk (e.g., automobile traffic fatalities in the United States have been roughly 2 x 10^{-4}/yr for many years), comparisons of annual frequencies must consider differences in exposure and consequences, and attempts to correct for such effects are subjective.

Consideration of acceptable risk for building projects, which traditionally have been regulated by public codes, is a relatively new development. Building codes and design practice aim at delivering structural systems at reasonable cost that meet minimum requirements for occupant and public safety. Despite the growth and acceptance of structural reliability as a decision tool in recent years (Ellingwood, 1994; 2001), the question of what constitutes acceptable risk in the built environment has not been answered definitively. Current building codes appear to be delivering building products that are acceptable from a life-safety point of view. But the threshold of acceptable risk depends on one's point of view. To a building occupant, any risk below the (unknown) threshold is acceptable. To a developer, on the other hand, any risk above the threshold represents wasted cost. Acceptable risk in buildings can be determined only in the context of what is acceptable in other activities, what investment is required to reduce the risk marginally, and what losses might be incurred if the risk were to increase. Unfortunately, the information required to make these trade-offs often is not available, and efforts to acquire the necessary data often are objected to as an unnecessary expense; hence, the reliance on code calibration in first-generation probability-based limit states design. Such calibrations have focused on member rather than system behaviour, and do not address the robustness issue in structural systems exposed to extreme events.

With these considerations in mind, the building project design team must aim for an engineering solution for which the conditional probabilities P(Collapse|D) and P(D|H) are sufficient for P[Collapse] to be at or below the *de minimis* threshold. Equation (3.3) is the basis for the current treatment of load combinations for abnormal loads in Section 2.5 of *ASCE Standard 7-10* (ASCE, 2010). This implies that,

$$P[C|D]\ P[D|H] \approx 10^{-7}/\lambda_H \ \text{(annualised)}^{8} \qquad (3.4)$$

A similar basis exists for the accidental load combinations in *Eurocode 1* (EN1991, 2006), Structural design procedures to meet these probabilistic objectives can be obtained using structural reliability analysis. While the details of the required analysis are outside the scope of this chapter, it should be remarked that recent advances in structural engineering computation have made it feasible for high-visibility building projects (see, e.g., Ellingwood and Dusenberry 2005). It should be noted that Section 1.4 of *ASCE Standard 7-10* (ASCE, 2010) has introduced new requirements for general structural integrity, stipulating that all structures should be provided with a continuous load path, have a complete lateral force-resisting system, and adequately anchored to their supports. These provisions are not risk-informed but simply represent good engineering practice.

3.5 Risk communication

Risk communication requires a continuing dialogue among the members of the project team and other project stakeholders that is aimed at facilitating an understanding of basic issues and enhancing the credibility and acceptance of the results of the risk assessment. A successful risk assessment is one that can be scrutinised independently by a peer reviewer, and that provides an easily understood audit trail leading to key decisions affecting project safety and other performance objectives.

A starting point is in understanding the notion of low-probability, high-consequence events. A criterion must be established to identify the relatively small subset of building projects where the threat of abnormal events is significant and where additional measures might be warranted to provide robustness and general structural integrity. There currently is no agreement in the building community as to precisely what this criterion should be. The development of guidelines should be a high priority because the economic impact of providing additional robustness beyond current code requirements can be substantial, particularly if imposed as a requirement for rehabilitation in order to continue building occupancy. One might expect that the elements of this criterion would include building size, the nature of its anticipated occupancy, and the potential impact of a catastrophic failure on the surrounding community. In the 21st Century, symbolic buildings or buildings housing government, financial or corporate entities are more likely to be at risk from malevolent attack than

[8] In Europe, Class 3 building design should consider risk in a formal sense, including with consequences, instead of only the probability of failure/collapse.

the building inventory as a whole. The hazards and risks posed for such buildings must be considered and those that are not dominant contributors screened out early in the design process. In many cases, this process will lead to a set of hazard scenarios (Garrick, 2004) and the use of Eq (3.2) as a basis for assessing and communicating the building risk. The performance objectives and metrics must be clearly identified and agreed upon, and uncertainty analysis should be a central part of the decision model. Tradeoffs that occur between investment and risk reduction must be treated candidly, and the entire decision process must be made as transparent as possible.

Most of the people who control the funds available to a project for risk mitigation are not expert in probabilistic risk analysis, especially when rare events are involved, and their needs (and the context of the decision process) should be given heavy weight in selecting appropriate metrics for communication risk. To these people, the results of a scenario analysis generally are more understandable than stating that the design-basis event has a 0.01% probability or, equivalently, has a return period of 10,000 years. Moreover, focusing on the probability without considering the consequences explicitly omits an important dimension of the assessment and decision process. It is difficult to see how, considering the diversity of civil infrastructure, it is possible to collapse all the consequences – mortality, direct and indirect economic losses – into a change of one or two orders of magnitude in the target probability. Recent research has made it clear that many building owners in the civil arena want the consequences measured explicitly. Moreover, many decision-makers would like to see the risk estimate accompanied by a statement of confidence, particularly when the probabilities are very small, supporting data are limited, and consequences are severe. Providing decision-makers with a point estimate of risk does not convey a sense of the confidence that the analyst has in his/her assessment of risk. Rather than the statement, "The risk is *p*", they would prefer the statement, "With this analysis, there is 90% confidence that the limit state probability is less than *p*."

3.6 Closure

Proper structural design involves looking beyond the minimum design code requirements. The need for structural robustness and the possibility of disproportionate collapse must be acknowledged explicitly in codes, standards and other regulatory documents. The building design team should take responsibility for documenting that steps have been taken to achieve a measure of robustness that is sufficient that the occurrence of natural or environmental events outside the design envelope, accidents or human malevolence will not precipitate a disproportionate structural collapse or unacceptable human or economic losses. The technical feasibility and effectiveness of specific provisions depend on specific building design practices and construction technologies. It is essential for the client, developer, and owner to be educated at the conceptual project development stage on issues related to structural robustness. The structural engineer must be clear as to what can be achieved at reasonable cost by good engineering practice. The building team must acknowledge that uncertainty in achieving the project performance goals and objectives cannot be eliminated; that risks presented by events outside the customary design envelope cannot be avoided; and that reduction in risk can be achieved through both technical and non-technical measures by additional investment in building robustness.

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4. Hazards

A. (Ton) Vrouwenvelder

The previous chapters showed that an essential part of designing for robustness is the consideration of hazards. The robustness design can be carried out against unidentified hazards and identified hazards. Whether a hazard was considered in a design or not (by intent or in error) would determine whether it is an identified or unidentified one, respectively.

4.1 Types of Hazards

In the context of robustness, a hazard is a serious threat to the integrity of a structure and the safety of people. Hazards may have natural or human origins. (Note: Sometimes, human causes could be behind what may be considered as a 'natural' hazard. For example, loss of support to a structure from a geotechnical movement could have its root cause in underground mining or other activities that give rise to a landslide or changes to the water table.) A list of hazards that may play a role in the design and assessment of buildings and other engineering structures is given in Table 4.1.

The characteristics of a hazardous event (the point in time of the occurrence, its intensity and distribution) that may materialise are usually unknown to a designer or users of a structure. However, in some cases the user or occupant of a structure may receive an early warning before a hazard materialises to affect safety.

Explicit design values and requirements are given in Eurocode for only a limited number of 'accidental' hazards that a structure can experience. These hazards include fire, earthquake, impact, internal gas explosions and dust explosions. The other hazards, the so-called unidentified actions, are addressed for lower consequence class structures (see Table 2.1) by prescribing more or less general measures such the tying of parts of a structure together. In the case of higher consequence Class 3 structures, where a risk assessment is specified, such prescriptive measures can help to provide some level of robustness. However, it is the responsibility of the designer to provide a sufficiently safe structure by using the freedom he has been given by the code.

In Table 4.1 there are three categories of hazards:

- The first category is the type of hazards that are more or less given rise to by nature or general human activities. Natural hazard are those such as strong winds and earthquake. (Unintentional) man-made hazards include explosions. However, the difference between them is hardly relevant for structural design.

- The second category includes the type of (man-made) actions that are deliberate, such as vandalism and malicious attacks. To some extent it may not help to make a structure stronger to resist them, because it could generate more (re)action on the loading side. In such cases, design efforts to limit damage propagation can be more efficient. This type of hazard has become more important after the events of 9 September 2001.

- The third category includes errors and negligence. There is a direct link of this type of hazards with quality control and supervision. These hazards are best controlled by good supervision and quality control during all stages of the life of structure (see Chapter 10) and by including general robustness measures suitable to deal with unidentified actions.

Hazard	Considered in Eurocodes?	Category
Internal gas explosion	X	1
Internal dust explosion	X	1
Internal bomb explosion		1
External bomb explosion		1
Internal fire	X	1
External fire		1
Impact by vehicle	X	1
Impact of aircraft, ships		1
Earthquakes	X	1
Landslide		1
Mining subsidence		1
Tornado and Typhoons/Hurricanes/Cyclones		1
Avalanche		1
Rock fall		1
High groundwater		1
Flood		1
Storm surge		1
Volcanic eruption		1
Environmental attack		1
Tsunami		1
Vandalism		2
Public disorder effects		2
Design or assessment error		3
Material error		3
Construction error		3
User error		3
Lack of maintenance (Deterioration)		3
Errors in communication.		3

Table 4.1: Overview of relevant hazards for structural safety

4.2 Modelling of Hazards

The principles of modelling the three types of hazard mentioned above are discussed below, individually, because the method of modelling is different for each of them.

For most **natural hazards** the following set of models will be needed. (Those for an earthquake selected as an example given within parenthesis):

- the occurrence model (average number of events per year or design life)
- a model to describe (the intensity of) the event (e.g. ground acceleration time-history)
- a model to describe the effect of distance (attenuation law)
- a model to describe the effect of mitigating measures (e.g. base isolation)

In Appendix B some common hazard models for earthquakes, fire, vehicle impact, ship impact and explosions have been detailed. Based on such models the exposure of the structure and corresponding occupants and contents can be estimated. In combination with other loads and events, a complete *hazard scenario* can be formulated as a basis for the estimation of the consequences. Within such an analysis, the effect of mitigating or preventive measurers can be incorporated.

In principle, the modelling of **malicious attack and vandalism** is more difficult. As already stated, the intention of the attacker is destruction and the strength of a structure is the starting point. Past statistics may not be of much use here, except to evaluate what sort of intensity of action is to be expected. Of course, by good design, it is possible to make it more difficult to achieve the intended destruction of the structure. A key word for such good design is robustness. It is possible to have some indications on the likelihood of a malicious attack based on:

- The strategic role of the building in society (energy supply, water supply, etc)
- The possibility of a large number of victims
- The type of building (monuments, embassies, government buildings, 'symbolic' building).

Errors and quality related robustness is best controlled by general measures of robustness because, usually, gross errors could have very significant effects. If their potential occurrence is expected, then the best option is to control them before they materialise. These aspects are further considered in Chapter 9.

5. Consequences of Failure

A. (Ton) Vrouwenvelder

Robustness must be given attention during structural design because of the intolerable consequences that could result otherwise. The consequences, which depend on the boundary of the considered system, and their modelling are discussed in this chapter.

5.1 Introduction

Consequences are possible outcomes of a desired or undesired event and must be considered during a risk assessment. Their extent, such as the amount of human fatalities and injuries, environmental damage and economic losses from an undesirable event, may be expressed either verbally or numerically. When an event is desirable, the consequences are the benefits that may accrue due to its occurrence. Where consequences result from 'hazards', as in the case of robustness design, the event is undesirable. In the case of a repair of a damaged structure, the described event is a desirable one with positive consequences. Only undesirable events are considered in this document.

Some consequences, such as social effects (including fears of the public) and political damage, have not been quantified and are expected to remain so, at least in the foreseeable future. Such unquantifiable consequences are better handled via communication and acceptance of risk as discussed in Chapter 3. This, however, needs prior 'education' of stakeholders to the nature of risk and the possibilities of a situation. The stakeholders' consent to accept the attendant risks, would help to limit the socio-political consequences of an undesirable event.

5.2 Types of Consequences

The consequences to be considered in relation to a structural design or assessment will depend on the considered 'boundary' of the structural system, or the system involving structures. For example, larger the extent of a defined 'system', different and more could be the types of consequences that must be considered. (The definition of systems is considered in Section 5.5.)

A list of undesirable consequences that may be related to a system is given in Table 5.1. These have been divided into human safety, which is regulated by law, business continuity, economic or property, environmental, socio-political consequences. Some of these are quantifiable while the others are not.

Consequence Type	Consequence	Quantifiable?
Human safety	Fatalities	Yes
	Injuries	Yes, but difficult
	By damaging vital facilities (e.g. hospitals), spread of diseases	
	Delayed long term effects	Difficult
	Psychological	No
Economic/Property	Damage to the building/structure	Yes
	Damage to surrounding properties	Yes
	Damage to contents	Yes
Business Continuity	Loss of income	
	Loss of customers	
	Inability provide vital services and/or activities	
	Costs of detours and delays	
	Costs to the economy of a region	
Environmental	Reversible environmental damage.	
	Irreversible environment damage	
	Effect on wildlife	
Social and Political	Loss of reputation	
	Increase of public fears	
	Loss of political support/enforcement of stringent new measures.	
	"Blight"/long-term evacuation	

Table 5.1: Types of Consequences of an Undesirable Event

5.3 Consequence Analysis

The systematic procedure to describe and/or calculate consequences is called Consequence Analysis. Consequences are generally multi-faceted (or multi-dimensional). However, in specific cases they may be simplified and described with a limited number of components such as, for example, human fatalities, property damage, environmental damage and costs of disruption due to the unavailability of a facility.

A consequence analysis should start with a technical and functional description of the system under consideration. Important for this are the type of the structure, its intended use and foreseen activities and the number of people expected to be affected by a failure. The strategic role of the building in the society, such as energy and water supply, transport, economic and industrial activities, governmental activities, medical services, etc. too should be considered in the analysis.

Some consequences are independent of the structural behaviour. For example, many people can die or be injured from a fire in a building due to smoke and radiation effects. In such a case, where there is a lead in time from the initiation of the hazard to feeling its effects, human lives can be saved by providing proper warning systems and adequate escape routes and refuges. There are extensive models to calculate the probability of survival of exposed people during such events.

Sometimes it is thought that, for example, during a fire if the Required Safe Egress Time (RSET) for all occupants is lower than the Available Safe Egress Time (ASET), which is based on fire and smoke development study, then there is no further robustness requirement for the structure. However, this not always correct because consideration should be given to the safety of emergency workers who may still be inside, for example, attempting to extinguish the fire.

If the structural response is of importance, then it is necessary to distinguish between the direct response of the exposed elements and the subsequent behaviour of the rest of the structure. If the direct response of a structural element is ‘inadequate’, then that element is considered as vulnerable. If the failure of vulnerable elements is followed by inadequate behaviour of the remaining part of the structure, then the latter is said to lack sufficient robustness. Both the assessment of direct and indirect structural behaviour may require quite advanced structural analysis which considers, for example, non-linear effects, dynamic effects and temperature effects. The properties of the structural system (e.g. series or parallel arrangements) are expected to play a very important role in providing robustness.

In the case of business continuity, robustness of a structural system may also result from the provision of alternative facilities that could carry out the same functions, even at a reduced efficiency. Whether such duplicate facilities are considered in an analysis depends on the definition of the system used in risk analysis.

5.4 Consequences Classes

Design for accidental situations, under which robustness design falls, needs to be included only for structures the collapse of which, either in part or on whole, may cause particularly large undesirable consequences.

A convenient way to decide whether to design against hazards such as accidental situations is to categorise structures or their structural components according to the *consequences* of an undesirable event (i.e. an ‘accident’). In Eurocode EN 1991-1-7, although the categorisation of structures is based on consequences of failure, they are not quantified and are given only qualitatively as follows:

- Consequences Class 1 Limited consequences
- Consequences Class 2 Medium consequences
- Consequences Class 3 Large consequences

In such a categorisation, less important individual structural members or sub-systems may be placed in a lower safety category than the overall structural system. An example of placing structures in safety categories, obtained from the EN 1991-1-7 is given in Table 2.1 of Chapter 2.

The appropriate robustness measures and the appropriate method of analysis to use for a situation may depend on its safety category, e.g. in the following manner:

- Consequences Class 1: no specific consideration of accidental actions.
- Consequences Class 2: depending on the specific circumstances of the structure in question: a simplified analysis by static equivalent load models for identified accidental loads and/or by applying prescriptive design/detailing rules.
- Consequences Class 3: extensive study of accident scenarios and using scenario approach, risk analysis, dynamic analysis and non-linear analysis, if appropriate.

It is up to a country or a state to decide on the appropriate strategy for the various consequences classes.

5.5 System Representation

As discussed above, the consequences of an undesirable event and the level of risk assessment carried out would depend on how a particular structural system has been defined. The risk assessment of a given system can be facilitated by considering the generic representation illustrated in Figure 5.1 from JCSS (2008). The exposure to hazards is represented by different exposure events that can act on the constituents of the considered system.

The constituents of a system can be considered as its first defence against a hazard. The damage to the system, caused by failures of the constituents, is considered as associated with “direct consequences”. Direct consequences may comprise different attributes of a system, such as monetary losses, loss of lives, and damage to the environment or even just changes to the characteristics of the constituents (see Table 5.1). Depending on the combination of events of constituent failure and the corresponding consequences, follow-up (or “indirect”) consequences may occur. If the structure is robust, these follow up consequences will be small. The opposite is true when a structure is not robust.

Note: In some situations, it is possible for one hazard to be followed by another, resulting in much more serious consequences to a structural system. Some examples of such situation are:

- gas explosion and/or fire following an earthquake
- tsunami following an earthquake
- fire following either a gas explosion or bomb blast
- fire following a tornado or other wind storm
- component deterioration, following damage from an accidental action.

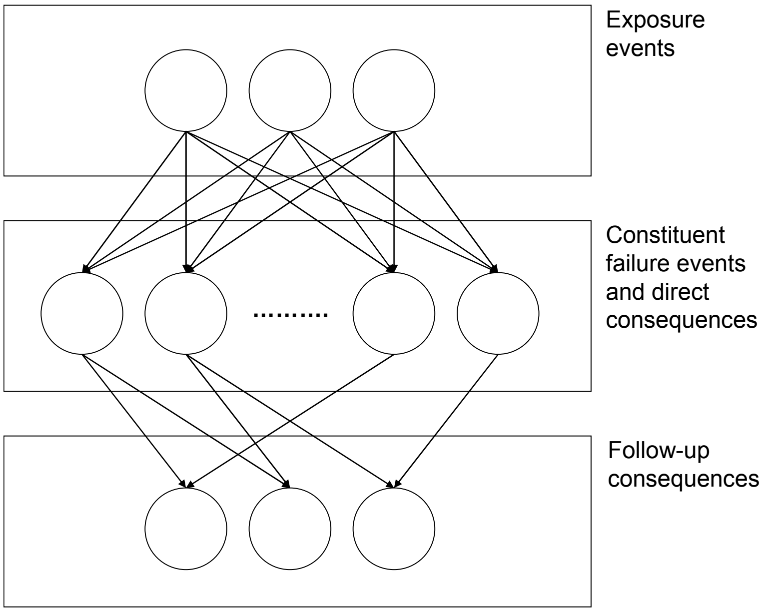

Figure 5.1: Generic system representation in risk assessments. Follow-up consequences are known also as “indirect” consequences (JCSS, 2008).

The consequences could be expressed also by, for example, the sum of monetary losses associated with

a) the constituent failures, and

b) the physical changes of the system as a whole caused by the combined effect of constituent failures.

In expressing consequences in terms of monetary values, it is assumed that the human fatalities and injuries too can be expressed in similar terms although, for some, it is difficult and problematic on an ethical basis.

In the risk assessment of systems, a major role is played by the follow-up consequences, and the modelling of these should be given great emphasis. It should be noted that any constituent in a system can be modelled as a system itself. A system could be a road network with constituents being, for example, bridges. A bridge, in turn, could also be a system with constituent structural members. Depending on the level of detail in the risk assessment, i.e. the system definition, the exposure, constituents and consequences will be different.

An example of the foregoing general statements, applied with respect to a building subject to an explosion at one of the upper storeys is shown in Figure 5.2 from Faber et al. (2004). Here, the direct consequences are defined by the change in behaviour and/or damage suffered by the (directly) exposed components (see ‘Step 2’ of the figure). Based on the level of this response, the behaviour of the other parts of the structure may result in follow-up or indirect consequences (see ‘Step 3’ of the figure).

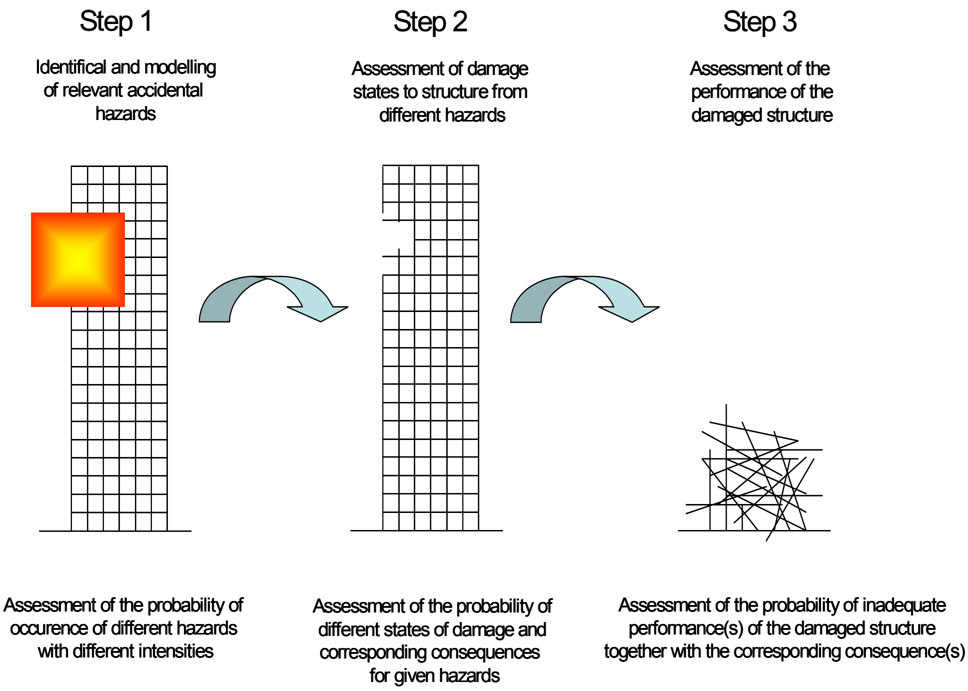

Fig 5.2 Steps in a consequence analysis (Faber et al., 2004).

5.6 Formal Scenario Approach

In formal risk assessments carried out for the purpose of decision-making, a scenario approach can be used as defined by the three steps given below.

Step 1: The modelling of the hazards H_i and exposures

Step 2: The assessment of the direct damage D_j

Step 3: The assessment of follow-up structural behaviour S_k and corresponding total consequences $C(S_k)$.

Given the relevant (conditional) probabilities, the risk related to a structural system may then be written as (see Chapter 3 and JCSS (2008)):

$$R = \sum_{i=1}^{N_H} p(H_i) \sum_{j}^{N_D} \sum_{k=1}^{N_S} p(D_j|H_i)\, p(S_k|D_j)\, C(S_k) \qquad (5.1)$$

In Equation 5.1, it is assumed that there are N_H hazards, N_D ways or levels of direct structural damage (D_j) and N_S types of follow-up behaviour (S_k).

The dimension of the consequences C could be either a monetary unit (often per time unit) or the expected number of casualties when only life safety is the concern. The latter is usually used to assess the societal risk or individual risk, where relevant. As mentioned previously, the relevant consequences depend on the boundary of the system considered in the risk analysis.

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6. Quantification and Decisions on Robustness

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6.1 Introduction

Although qualitative properties of robust structures can be considered as relatively well understood, quantitative evaluation and decisions with respect to robustness is a significant challenge (Canisius et al. 2007). The latter is discussed in this chapter.

There are two primary approaches used to assess robustness in structures:

a) Practical evaluation methods, where the behaviour of a structure under a scenario loading is modelled, and

b) Reliability or risk-based approaches that study, respectively, the reliability or attendant risks related to a system, under a more general description of potential loading scenarios.

The practical evaluation methods are usually feasible to perform within typical structural design situations and their results can be more easily compared with prescribed acceptance criteria. Currently, there is such a method and it is applicable to Class 2–Upper group structures designed by Eurocodes and to similar situations in many non-European codes and regulations. However, with this method only one or a few of the many potential loading situations can be incorporated because the consideration is given only to the loss of a single load bearing component of the structure. That is, the expectation of the codes is only a conditional assessment of consequences due to the loss of a considered component. However, as suggested for example by (Alexander 2004) and (Canisius 2007), this method may be extended to consider the loss of more than one load bearing member.

Reliability-based and risk-based quantification approaches can address the shortcomings of the practical approaches by explicitly considering the many uncertainties associated with an accidental design situation. For example, it is possible to incorporate uncertainties in loading and system properties into the calculations. In this way, instead of dealing with the loss of a deterministic number of components, it is possible to consider the probability of losing more than one load bearing component during a single event. This type of approach has the potential to provide a more comprehensive picture of a system's robustness, but it can be generally too onerous as to be practical in typical design situations.

When considering reliability and risk, the stakeholders have the responsibility and/or the authority to prescribe the required reliability. This is traditionally done by application of the As Low As Reasonably Practicable (ALARP), see Canisius (2008), or more modern approaches such as the marginal life saving costs principle, see e.g. Faber and Maes (2010). The system to be considered should be defined with due consideration to the relevant consequences and the decision alternatives which are to be assessed and optimized. Therefore, the definition of a structural 'system', or a system involving structures, is an important decision to be made before the detailed examination of robustness. For example, the robustness of a bridge structure may be considered either on its own or within a larger system e.g. a roadway network containing several infrastructure components, see Figure 6.1 (JCSS, 2008).

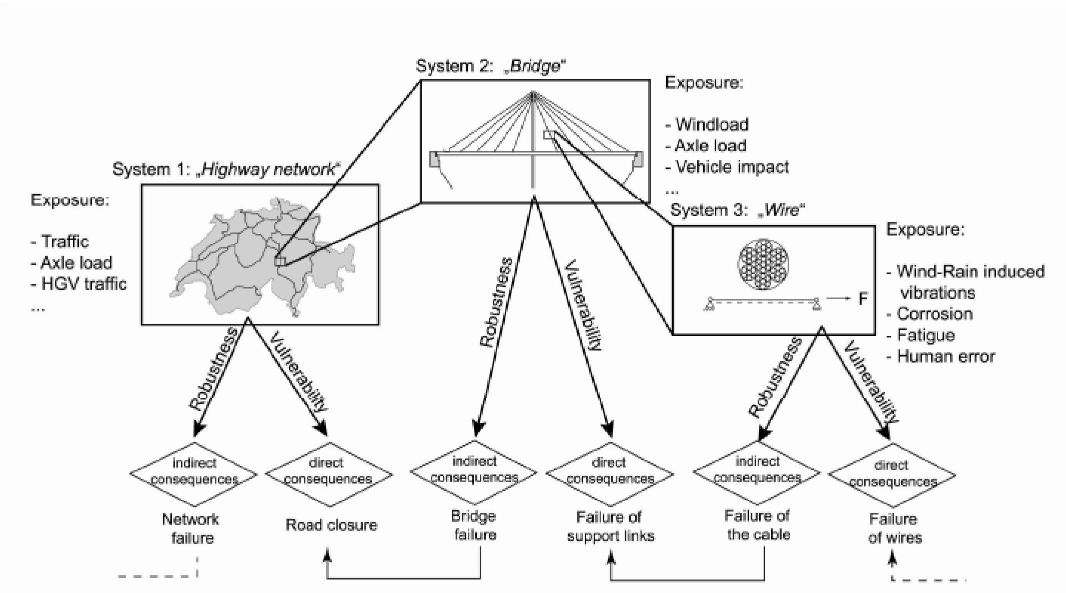

Figure 6.1 Illustration of systems involving structures (JCSS, 2008).

It is difficult, if not impossible at times, to probabilistically model the future loading from events such as sabotage and malicious attacks. They do not depend on natural phenomena but are affected by, for example, the contemporary social, political and military environments. Under these conditions, the probabilities and risks conditional on the loss of components may be used to address the effects of such actions.

6.2 Robustness Quantification in Codes

In accordance with Faber and Narasimhan (2011) the code-based structural design process may be seen as a means of providing efficient rules for the design of the broad range of structures that fall within the range of validity of the codes. The design codes fundamentally aim to provide a basis for the design of structures such that on one hand it provides adequate structural safety with an efficient use of materials and technologies and on the other hand it sufficiently caters to the need for a highly standardised and efficient design process. In doing so, several necessary assumptions and idealisations are introduced in relation to different aspects relevant to the design. Among these, the more important include:

- A representative set of loads and combinations thereof is assumed to envelope the effects of the actual applicable loads.

- The performance of materials is described in terms of a few selected parameters.

- Design values are defined to account for relevant uncertainties associated with loads, materials, models, deterioration phenomena and execution.

- Structural failure modes are normally considered by addressing critically loaded cross sections, components and details.

- Dependencies between different failure modes in a structure are generally ignored.

- When setting criteria for acceptable failure probabilities, the consequences of failure and damage are considered only very crudely.

- Accidental loads and consequence reduction due to pre-warned structural failure are considered only implicitly through the criteria for acceptable failure probabilities.

- Gross errors are assumed to be identified and taken care of by means of quality control procedures.

- Structural deterioration is assumed to remain under control through best practice inspection and maintenance strategies.

On the basis of such idealisations and simplifications, design codes facilitate a relatively simple design verification by means of safety formats such as e.g. the LRFD.

Subject to the simplifications and idealisations listed above, the design formats may be calibrated such that the reliability of structures with respect to the explicitly considered aspects of structural performance is appropriately high. Reliability analysis of structures in general and for the purpose of code calibration has developed over the last half century and can be considered now as a well-established tool in structural engineering.

It is very important to appreciate that structural reliability assessments performed for the purpose of code calibration are indeed performed under the same best practice simplifications and idealisations as are prevailing in the code based structural design verifications. From this perspective, it is to be recognised that the reliabilities are conditional on a number of assumptions that may or may not be fulfilled.

In principle, due to all the simplifications and idealisations introduced into a design, some aspects of structural performance are obviously not fully achieved – and this is the core reason for the requirement of sufficient structural robustness, in addition to the requirements on reliability. Many codes and standards require that structures should be robust in the sense that the consequences of structural failure should not be disproportional to the effect causing the failure.

Most, if not all, design codes include prescriptive requirements which in some way or another implicitly add to the robust performance of structures. This concerns e.g. requirements on structural ties and ductility of joints and mechanisms of failures. Modern design codes also provide more explicit requirements for certain types of structures subject to extreme load conditions. However, the more general code requirement for sufficient structural robustness remains largely unspecified and thus leaves the structural engineer with the problem and the responsibility of dealing with this appropriately.

Modern code based design and assessment of structures is based on a set of documents typically comprising:

- a code on general principles and the code safety format
- code(s) on specification of loads
- code(s) on specification of resistances and design equations (material specific)
- standards for the production of materials
- standards for execution, protection, maintenance and repair
- standards for quality control

In some cases, special codes are provided for the consideration of extraordinary loads. Together, the codes and standards stipulate the required reliability towards the fulfilment of safety, serviceability, durability and consequently the life cycle performance of structures. In the context of code based design and assessment of structures, the requirements may be viewed as comprising of two types:

- fulfilment of design equations or standard safety format dealing with the 'design envelope'
- 'deemed to satisfy' provisions for robustness

This is illustrated in Figure 6.2. The provisions to ensure robustness are typically in the form of requirements for tying of structural components, performance of joints, design for member removal situations and design for pressure loads. These provisions specified by the design codes address and aim to ensure robust structural performance.

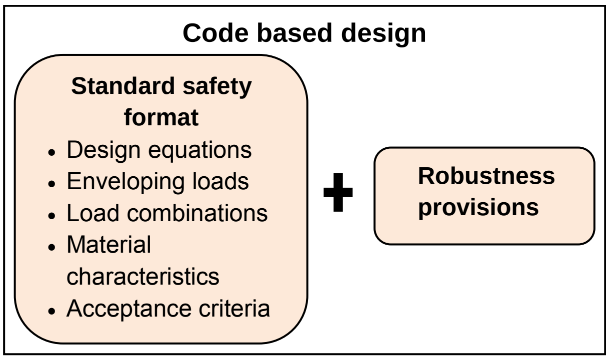

Figure 6.2 Illustration of the main components of requirements in design and assessment codes (Narasimhan and Faber 2011).

It is to be appreciated that codes do not provide a method for quantifying robustness but rather aim to ensure the sufficiency (of robustness), e.g. in terms of the potential consequences that are to be limited to prescribed levels (for buildings). See Section 2.3.

As mentioned in Chapter 1 and elsewhere, the definition of robustness in the Eurocode EN 1991-1-7:2005 relates to the ability of a structure to be not damaged to a level that is disproportionate to the original cause. According to this definition, a structure which is designed to have strong key elements, but which may progressively collapse when a key element is removed, is a robust structure. However, risk and reliability-based indices (see below), robustness is defined in relation to the follow up consequences because they deal with the situation of progressive collapse.

6.3 Risk- and Reliability-based Quantification

Recognising that present code-based analysis procedures cannot give a complete picture of structural robustness, a variety of probability-based quantification procedures have been proposed (e.g., Ellingwood 2005). Common aspects of these approaches include attempts to quantify the complete range of potential loading on structures, as well as associated probabilities of occurrence for those load scenarios, and quantification of uncertainty in structural properties or structural response. The approaches described below relate to both redundancy and robustness, with individual approaches placing more emphasis on one aspect or another.

Three early attempts to quantify robustness, carried out prior to the World Trade Centre attacks, were as follows.

- (Frangopol and Curley 1987) and (Fu and Frangopol 1990) considered probabilistic indices to measure structural redundancy, based on the relationship between damage probability and system failure probability.
- (Lind 1995; 1996) proposed a generic measure of system damage tolerance, based on the increase in failure probability resulting from the occurrence of damage.
- In a study somewhat related to reliability based methods, (Ben-Haim 1999) proposed a robustness quantification approach using information-gap theory. This approach did not require the complete probabilistic description of loading that is needed for reliability-based assessments, and it can be applied to general systems. Challenges remained, however, for using this method to balance robustness improvements with their associated costs.

Recently, taking Faber et al., (2004) as the basis, an index and metric of robustness was proposed by Baker et al. *(*2008) for engineered systems, with the term “system” used to refer to both a physical structure as well as its associated inspection, maintenance and repair procedures. The approach divides consequences into direct consequences associated with local component damage (that might be considered to correspond to and proportional to the initiating damage) and indirect, or “follow-up”, consequences associated with subsequent system failure (that might then be considered disproportional to the initiating damage), see also (Faber and Maes, 2005).

An index was formulated by comparing the risk associated with direct and indirect consequences, with the idea that systems having high risks associated with indirect consequences are more likely to suffer disproportionate ‘consequences’ and, thus, be less robust. In addition to quantifying the effect of the physical system’s design, this approach could potentially account for the effect of inspection, maintenance and repair strategies as well as preparedness for accidental events. Also presented by these authors was a discussion of how decision analysis theory can be used to make decisions regarding acceptable robustness.

6.4 Theoretical Basis for Quantifying Robustness

An illustration related to several key definitions of robustness is presented in Figure 5.2. In there, due to an exposure of any kind (shown as Step 1), local damage shown in Step 2 may occur. This local damage is defined as the direct consequence of the exposure. Given this local damage, the structure may survive without further damage or a substantial part of it may collapse. Robustness requirements are especially related to the 2nd and 3rd Steps of the Figure 5.2, i.e. to the aim of preventing local direct damage from developing into further damage, including total collapse.

Approaches to quantify robustness via a robustness index can be classified into three levels, with each of the following being of decreasing complexity:

- A risk-based robustness index based on a complete risk analysis where the consequences are divided into direct and indirect risks
- A probabilistic robustness index based on probabilities of failure of the structural system for an undamaged structure and a damaged structure
- A deterministic robustness index based on structural measures, e.g. pushover load bearing capacity of an undamaged structure and a damaged structure

Of the above, the basic and most general approach to quantify robustness is the first, i.e. via a risk analysis, where both probabilities and consequences are taken into account.

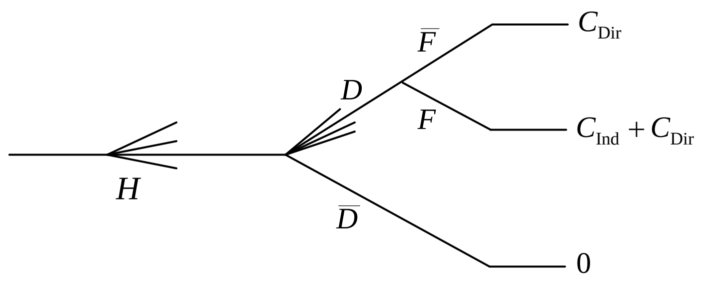

Figure 6.3: An event tree for robustness quantification, (Baker et al. 2008).

The same concepts as in Figure 5.2 are presented in a more general way in the form of an event tree in Figure 6.3. The assessment starts out with the consideration and modelling of exposures (H) that can damage components of the structural system. The term “exposures” is used to refer to extreme values of ‘normal’ design actions, accidental actions and deterioration processes, but it could also include human errors in the design, execution and use of the structure. Here, the term “damage” is used to refer to a reduction in performance or the failure of one or more individual components of the structural system. After an exposure event has occurred, all the components of the structural system either remain in an undamaged state ($\overline{D}$) as before or change into a damaged state (*D*). Each local damaged state can then either lead to the partial or complete failure of the structure (*F*) or its survival without any further damage ($\overline{F}$).

Consequences are associated with each of the possible damage and failure scenarios, and classified as either direct (C_{dir}) or indirect (C_{ind}). Direct consequences are considered as the direct result of the exposure and, depending on the intensity of the exposure, may correspond to damage to one or more individual components. Indirect consequences in principle comprise all consequences in excess of the direct consequences (Faber and Maes, 2003) and are incurred due to, for example, a loss of system functionality, or failure, and can be attributed to lack of robustness (Baker *et al.* 2008) and (JCSS 2008).

The framework for risk analysis is based on the following equation where risk contributions from local damage (direct consequences) and comprehensive damage (follow-up / indirect consequences), are added, see (Faber et al. 2007), (Baker et al. 2008) and (JCSS 2008):

$$R = \sum_i \sum_j C_{\mathrm{dir},ij} P(D_j|H_i) P(H_i) + \sum_k \sum_i \sum_j C_{\mathrm{ind},ijk} P(S_k|D_j \cap H_i) P(D_j|H_i) P(H_i) \qquad (6.1)$$

where

$C_{dir,ij}$	expected consequence (cost) of damage (local failure) D_j due to exposure H_i
$C_{ind,ij}$	expected consequence (cost) of comprehensive damages (follow-up / indirect) S_k given local damage D_j due to exposure EX_i
$P(H_i)$	probability of exposure H_i
$P(D_j\|H_i)$	probability of damage D_j given exposure H_i
$P(S_k\|...)$	probability of comprehensive damages S_k given local damage D_j due to exposure H_i

The optimal design (decision) is the one minimizing the sum of costs of mitigating measures and the total risk *R* (JCSS, 2008). It is to be noted here that, as mentioned in Chapter 5, an important step in the risk analysis is to define the system and the system boundaries.

6.5 Robustness Indices

While the risk from failure can be determined as given in Equation 6.1, a non-dimensional index could help to provide a quantified measure of robustness that can help to compare different solutions and make related decisions. Some examples of different types of indices developed by various researchers are given below.

6.5.1 Risk-based robustness index

The robustness index proposed in (Baker et al. 2008) divides consequences into:

- direct consequences associated with local component damage (that might be considered proportional to the initiating damage), and
- indirect consequences associated with subsequent system failure (that might be considered disproportional to the initiating damage).

The index of robustness (I_{rob}) is defined as

$$I_{rob} = \frac{R_{Dir}}{R_{Dir} + R_{Ind}} \tag{6.2}$$

where R_{Dir} and R_{Ind} are the direct and indirect risks associated with the first and the second term in Equation (6.1). The index takes values between zero and one, with larger values indicating larger robustness.

Considering that the optimal decision is the one minimising the total risk, and which can be attained by reducing the first or the second term of Equation 6.1, this robustness index is seen to be not always fully consistent with a full risk analysis. However, it can be considered as a helpful indicator based on risk analysis principles. It is to be noted that since the direct risks typically are related to code based limit states they can generally be estimated with higher accuracy than the indirect risks.

The index accounts not only for the characteristics of the structural performance but also for the performance of the system after damage and all relevant consequences. Furthermore, all measures (decision alternatives), which can be implemented either to improve structural performance with respect to robustness or to decrease the vulnerability (increasing component reliability), are explicitly accounted for by the index. It should be noted that the robustness index is "conditional" in the same principal manner as is structural reliability (see also Schubert and Faber, 2008). The index is conditional on the level of reliability of the individual components/failure modes of the system as well as the ratio between direct and indirect consequences. The reliability of structural components and failure modes is conditional on the probabilistic modelling and the best practice technology and design practices.

6.5.2 Reliability-based robustness index

Frangopol & Curley [1987] and Fu & Frangopol [1990] proposed some probabilistic measures related to structural redundancy – which also indicates a level of robustness. It is a "redundancy index", (*RI*), defined by:

$$RI = \frac{P_{f\,(\mathrm{damaged})} - P_{f\,(\mathrm{intact})}}{P_{f\,(\mathrm{intact})}} \tag{6.3}$$

where $P_{f\,(\mathrm{damaged})}$ is the probability of failure for a damaged structural system and $P_{f\,(\mathrm{intact})}$ is the probability of failure of an intact structural system. This redundancy index provides a measure on the redundancy of a structural system. The index takes values between zero and infinity, with smaller values indicating larger robustness.

They also considered the following related redundancy factor:

$$\beta_R = \frac{\beta_{\text{intact}}}{\beta_{\text{intact}} - \beta_{\text{damaged}}} \tag{6.4}$$

where β_{intact} is the reliability index of the intact structural system and β_{damaged} is the reliability index of the damaged structural system. The index takes values between unity and infinity, with larger values indicating larger robustness.

6.5.3 Deterministic robustness index

A simple and practical measure of structural redundancy used in the offshore industry is based on the so-called RIF – value (Residual Influence Factor), (Faber et al. 2006) and (ISO19902: 2007).

A Reserve Strength Ratio (*RSR*) is defined as:

$$RSR = \frac{R_c}{S_c} \tag{6.5}$$

where R_c denotes characteristic value of the base shear capacity of an offshore platform (typically a steel jacket) and S_c is the design load corresponding to ultimate limit state.

In order to measure the effect of full damage (or loss of functionality) of structural member "*i* " on the structural capacity, the so-called 'RIF' value (sometimes referred to as the Damaged Strength Ratio) is defined by:

$$RIF_i = \frac{RSR_{\text{fail},i}}{RSR_{\text{intact}}} \tag{6.6}$$

where RSR_{intact} is the RSR value of the intact structure and $RSR_{\text{fail},i}$ is the RSR value of the structure where member "*i* " has either failed or has been removed. The *RIF* takes values between zero and one, with larger values indicating larger redundancy.

A combination of the *RSR* and *RIF* can be considered to provide an indication of the robustness of the structure, with RSR reflecting more a situation similar to 'Key Element Design' for buildings (see Chapter 2).

6.5.4 Other quantification methods

Several researchers have considered vulnerability of specific classes of structures to specific damage scenarios (Agarwal et al. 2003; Ellingwood and Leyendecker 1978; Feng and Moses 1986). A relatively well-studied case is the progressive collapse of frame structures (*ref recent special issue*). This work is important for characterising failure probabilities for specific scenarios, but it is often difficult to generalize these findings to other types of systems or other damage.

Other simple measures of robustness have been proposed based on e.g. the determinant of the stiffness matrix of structure with and without removal of elements.

6.5.5 Examples

The risk-based framework for assessing the robustness index for systems involving structures has been applied on a number of cases and examples are available in the literature. In Narasimhan and Faber (2008), the robustness of high rise buildings are assessed for various structural configurations and damage scenarios. In Schubert and Faber (2007), the robustness of infrastructure subject to rare events is addressed and in Von Radowitz et al. (2008) the robustness of an externally reinforced concrete bridge subject to deterioration is analysed in some detail. In Sørensen et al. (2011), examples of application of the robustness framework and robustness measures described above to timber structures are given. These examples include a Norwegian sports hall and a Croatian truss structure.

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7. Designing for Robustness[9]

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Information useful in the design of structures for robustness is provided here. This presentation is heavily biased towards the design of buildings, with only a limited amount of information provided with respect to other types of structures. The information provided here covers both conventional robustness design and risk-based design. However, the former is covered well by other documents [e.g. IStructE 2010a, IStructE 2010b, Knoll and Vogel 2009, NIST 2006]. Therefore, only basic information that can act as a starting point for prescriptive design of Class 2 buildings of Eurocode EN 1991-1-7:2006, while following its philosophy, is given here.

7.1 Design Framework

Methods for assessing the potential of a structure to withstand damage without developing further damage or a general structural collapse have been thoroughly presented in Chapters 5 and 7 (see also Ellingwood and Dusenburry, 2005; Canisius, 2007). The difference of such progression of damage from a sudden general collapse is associated with:

a) the initiation through a relative localized damage and then
b) its evolution time to global collapse

Based on the methodological aspects presented before, the problem of *global failure* (assuming it is the only concern) can be expressed with a probabilistic formulation using the probability P_F, of a progressive collapse due to an abnormal event, E as follows:

$$P_F = P(F|LE) \times P(L|E) \times P(E) < P_A \qquad (7.1)$$

where

P_F	Probability of global failure associated structural collapse.
P(E)	Probability of occurrence of hazard E (accidental action)
P(L|E)	Probability of local damage, L, given that E occurs
P(F|LE)	Probability of collapse given that E and L both occur
P_A	Acceptable probability of global failure

Note: *The global failure F need not be the complete collapse of a building, but can be a partial collapse greater than a considered limit such as the "100m² or 15% floor area" criterion of the Eurocode EN 1991-1-7:2006 (see Chapter 2).*

If the risk is considered then the following inequality can be formulated:

[9] Revised September 2011

$$R = P_F \times C(F) < R_A \tag{7.2}$$

where:

R: risk

C (F): expected consequences of global failure (total collapse) or partial failure

R_A: acceptable risk

The parameters in expressions (7.1) and (7.2) can be obtained based on observed data, information in literature, analysis and expert opinion as illustrated in the previous chapters.

Design criteria shall be defined such that inequalities (7.1) or (7.2) are satisfied. It is consequently important to establish acceptance criteria based on the framework presented in Chapter 3. In particular, the question of whether specific design considerations against disproportionate collapse are necessary for a given structure must be clarified first. The answer depends on the classification of the structure according to its Consequences Class (see Table 2.1 of Chapter 2). A classification of structures has been prepared in various standards and guidelines such as the UK Building Regulations which were adopted in Eurocode EN 1991-1-7:2006, the ASCE[2005], and the US DoD[2009] and these were discussed in Chapter 2 of this document. The three consequence classes, CC1 to CC3, specified in the Eurocodes are used in the examples presented in this Chapter.

The design objectives comprise

- the identification of hazard scenarios,
- the formulation of performance objectives, and
- the specification of acceptability criteria.

Definitions and several aspects used in this section 1.1 are taken from Starossek and Haberland, (2010, 2012). Accordingly, **hazard scenarios** are the abnormal conditions that are assumed to occur during construction and the lifetime of a structure. In a hazard-specific approach, they are specific abnormal events. In a non-hazard-specific approach, they are either notional actions or notional damage, without regard to the cause. The terms hazard-specific and non-hazard-specific design categorise the manner hazard scenarios are specified.

Hazard-specific design is based on specific hazards to the structure that could possibly occur (specific abnormal events); these abnormal events and the ensuing effects must be derived and quantified based on available data, statistical analysis and engineering judgement. Non-hazard-specific design excludes the strategy of reduction of exposure described in Chapter 3. It is intended to provide collapse resistance where potential threats cannot be specified and was previously discussed as the ‘Scenario Approach’ in Chapters 3 and 4.

The performance objectives are used to specify the acceptable or tolerable response of a structure to the hazard scenarios (Starossek and Haberland, 2010, 2012). They should be defined at a global level, that is as an acceptable extent of collapse and acceptable other consequences of a hazard with a specified intensity (event size) and associated return period. Other damage includes damage to the non-collapsed remaining structure, damage to the surroundings, and indirect consequences resulting, for instance, from an impairment of the surrounding infrastructure. An example of a performance matrix, expressed in terms of acceptable degrees of damage, which can form the basis for performance-based design with global performance objectives is shown in Table 7.1. Local performance objectives can alternatively serve as simplified and substitute criteria for achieving global objectives. Specifying performance objectives is not intrinsically an engineering problem. It can be supported by professionals but must reflect the desires of the owner, the concern of parties affected by a given project and its possible collapse, and public opinion (Starossek and Haberland, 2012). During such as exercise, for example, the expected consequences in terms of fatalities can be the major parameter that sets the failure limits to a particular structure. An example set of fatality criteria, produced as F-N curves by Trbojevic (2005), is given in Figure 7.1 While F-N curves deal with life safety issues, there is no comparable method to consider non-life safety related consequences.

Event size	**CC1**	**CC2**	**CC3**
Very large	Severe	high	Moderate
Large	High	moderate	Mild
Medium	moderate	mild	Mild
Small	Mild	mild	Mild

Table 7.1 A performance matrix with acceptable degrees of damage for consequences classes CC1 to CC3.

7.2 Design Methods

7.2.1 General

Once the appropriate performance objectives have been set, in the next step, appropriate design methods that aim to prevent intolerable performance, such as global collapse, should be selected. The available design methods for this purpose can be classified as follows (also see Table 7.2)

a) Event control: Affects the probability of occurrence of hazard E. (Note: This method is applicable only when designing for identified hazards.)

b) Specific load resistance (SLR): Influences the probability of local damage, L, given that E occurs, i.e. reduces the vulnerability of the structure and its members. Local damage is more generally known as direct damage.

c) Alternative load paths (ALP): Influences the probability of further (i.e. “indirect” or “follow up” failure, such as collapse, given local failure. Structural provisions such as ties can help to provide ALP.

d) Measures that reduce the consequences of failure, especially of follow-up failure.

The measures a) and d) are indirect methods while b) and c) are direct methods for preventing disproportionate collapse. Direct and indirect design are defined and discussed in Starossek and Haberland, (2010, 2012). According to this work direct design aims at explicitly ensuring collapse resistance in the design process by demonstrating that the structure meets the specified performance objectives when specified hazard scenarios occur and affect the structure. Direct design thus strongly relies on structural analysis. Indirect design, on the other hand, aims at reducing the effects of a hazard implicitly by incorporating agreed design features that help to achieve the performance objectives.

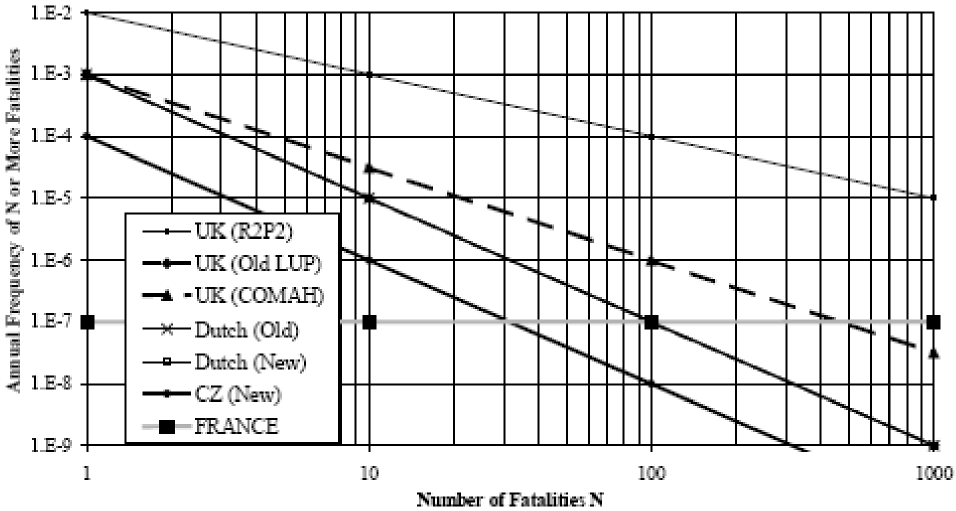


Figure 7.1: F-N curves relating expected fatalities (N) from an accidental event and the annual frequency of occurrence (F) of events with not less than N fatalities Trbojevic (2005).

7.2.2 Event control

Event control refers to avoiding or protecting against an incident that might lead to disproportionate failure. This approach does not increase the inherent resistance of a structure to disproportionate failure (Starossek and Haberland, 2010). Once the building is in use, the effectiveness of this method depends on how those operating and using it comply with the designer's specifications and recommendations regarding even control. However, there are preventive measures that can reduce the probability of a hazard materialising at a high intensity, such as:

- Planning of the location of the building
- Provision of stand-off perimeter
- Provision for surveillance systems such as alarm and security
- Prohibiting the storage of explosives
- Placing fenders around the columns to prevent vehicle impact
- Placing barriers around the ground area
- Gas detectors and automatic cut-off devices for gas.
- Control or limiting of fire ignition sources
- Limiting fire loads
- Fire suppression systems
- Installation of smoke detectors and alarms
- Use of Structural Health and Monitoring Systems
- Quality control during construction, maintenance and repair activities.

The preventive safety measures can lead to a reduction of the probability of a hazard P(E) occurring at a high intensity and to an increase of the associated return period. Therefore, with such measures, it is easier to satisfy the performance criteria as for example shown in Table 7.1 and by Canisius [2008] with respect to internal gas explosion risks in Large Panel System buildings.

7.2.3 Specific Load Resistance (SLR)

The methods presented next i.e. in § 1.2.3, 1.2.4 and §1.2.5 are described and discussed In this method, sufficient strength to resist failure from accidents or misuse is provided to structural members in certain regions of a building to allow them to resist accidental loads. For this, it is necessary to classify members according to their importance to the survival of the structure and identify the so-called key elements. Their failure is expected to cause further damage that violates the performance objectives because, in their absence, the structure as a whole is unable to develop sufficiently strong alternative load paths. Examples of potential key elements could be columns and load bearing walls of a building, a pier of a continuous bridge, or a cable in a cable-supported structure (Starossek and Haberland, 2012). If robustness shall be verified by using key elements, these can be for example designed by increasing the material safety factor by 20% (Soerensen and Christensen, 2006). In the Eurocode EN 1991-1-7:2006, a uniformly distributed load of 34 kN/m^2 is specified as the design load (for unidentified actions).

It is necessary to bear in mind that ensuring higher safety against initial damage requires more than the use of higher design loads or recourse to protective measures. An initial damage can also be caused by occurrences such as corrosion or fire – events that are more effectively counteracted by 'Event Control' measures such corrosion protection, regular inspection, fire protection and fire fighting systems than by increasing design loads (Starossek and Haberland, 2012).

The SLR method is suitable and cost-effective for structures with a limited number of identifiable key elements. However, it needs to be applied to structures with a larger number of key elements when other methods are not easy or practical to implement: e.g. where the structural system lacks alternative load paths and event and consequence controls are difficult, if not impossible, to implement.

Method	Reduces	Issues to address
a) Event control (EC)	Probability of occurrence and/or the intensity of an accidental event	- Monitoring, quality control, correction and prevention
b) Specific load resistance (SLR)	Probability of local, i.e. direct, damage due to an accidental event	- Strength and stiffness - Benefits of strain hardening - Ductility versus brittle failure - Post-buckling resistance - Mechanical devices
c) Alternative load path method (ALP), including provision of ties	Probability of further, indirect, damage in the case of local damage	- Multiple load path or redundancy - Progressive failure versus the zipper stopper - Second line of defence - Capacity design and the fuse element - Sacrificial and protective devices - Testing - Strength and stiffness - Continuity and ductility
d) Reduction of consequences	Consequences of follow up, i.e. indirect, damage such as progressive collapse	- Segmentation - Warnings, active intervention and rescue - Redundancy of facilities

Table 7.2: Classification of design methods

7.2.4 Alternative Load Paths

Alternative load path (ALR) is a direct method to enhance the robustness of a structure. and is critically reviewed in Starossek and Haberland (2010, 2012). In this approach, alternatives for a load to be transferred from a point of application to a point of resistance are provided. This enables redistribution of forces originally carried by failed components to, provided the new paths are sufficiently strong, prevent a failure from spreading. Alternative load paths can also form through load-transfer mechanisms, for example via:

- inversion of flexural load transfer (from hogging to sagging above a failing column)
- transition from flexural to tensile load transfer (catenary action)
- transition from plane to spatial load transfer (in one-way slabs turning into two-way slabs).

This direct method requires the designer to prove that a structure is capable of fulfilling its performance objectives by bridging over one or more failed (or notionally removed) structural elements, with a potential additional damage level lower than a specified limit. The method can be applied in both hazard-specific and non-hazard-specific situations because the notional damage to be considered in the application of the alternative-paths method is non-threat-specific. When using the alternative-path method in a hazard-specific manner, the initial damage that could result from an accidental event is first determined from a preliminary analysis (Starossek and Haberland, 2010). It appears that the aim of most of the codified measures is to provide, in one way or another, alternative load paths. In the standards and guidelines reviewed in Chapter 2, ties are recommended via prescriptive design rules such that catenary action can be generated and ductility ensured in a building structure.

Basically the ALP strategy considers the situation that one or more structural elements (beams, columns, walls) have been damaged, by whatever event, to such an extent that their normal load bearing capacity has vanished completely. For the remaining part of the structure it then requires, for some relatively short period of time (the time to repair, T) the structure to withstand the "normal" loads with some prescribed reliability (JCSS, 2011):

$$P(R < S \text{ in } T \mid \text{one/more element(s) removed}) < P_T \qquad (7.3)$$

The requirement in (7.3) is applicable, for example, in the case of a fire accident. The target reliability P_T in (7.3) depends on:

- the normal safety target for the building (see 7.1)
- the period under consideration
- the probability that the element under consideration is removed (by other causes then already considered in design).

The probability that some element is removed by some cause depends on the sophistication of the design procedure and on the type of structure. In the case of non-hazard-specific design, the ALP strategy starts with the assumption of reasonable scenarios of initial damage. The structure is then designed such that the spread of this local initial damage remains limited to an acceptable extent (Starossek and Haberland, 2012).

The generation of alternative load paths, even in locations or buildings where they are not explicitly considered, is helped by the provision of minimum levels of strength, continuity, and ductility to a structure. For example, the following are good practices that can enhance robustness of a building:

- Good plan lay-out
- Integrated tie system
- Returns on walls
- Redundancy
- Ductile Detailing
- Fire resistance of structural members

However, sometimes it is good to incorporate segmentation into large structures, without tying all its parts together, so that any failure can be stopped from progressing beyond a segment (see Table 7.1 and Section 7.2.5).

7.2.5 Consequence reducing measures

The implementation of consequence reducing measures aims at reducing the direct and indirect consequences of failure and thus the total risk. Such measures can be for example:

- Structural and architectural
- Electro-Mechanical (equipment)
- Organisational, including emergency planning
- Self-rescue and rescue by others
- Backup facilities

Important structural and architectural measures are, for example, the possible segmentation, or compartmentation, of the structure and the provision of effective escape/evacuation routes. The former is sometimes also referred to as the provision of 'structural fuses', similar to electrical fuses that protect circuits and appliances. Segmentation, in fact, is a potential way of enhancing the robustness of a structure as discussed first by Starossek (2007). In this approach, a spreading of failure following an initial damage is prevented or limited by isolating the failing part of a structure from the remaining structure by so-called segment borders. The locations of the segment borders are chosen by the design engineer within the scope of the design objectives. Two examples where structural segmentation, accomplished by discontinuity, possibly prevented widespread disproportionate collapses are the Pentagon Building in Washington, D.C. and the Charles de Gaulle Airport Terminal in Paris (Starossek, 2007). The use of appropriate escape routes (staircases and possibly lifts) has been widely discussed in relation to many accidents, including after the WTC towers collapse.

Another example is the Piper's Row Car Park failure which was confined to a local region by unintentional structural fuses that was present due to poor continuity of reinforcements.

Electro-mechanical equipment measures include, for example, automatic sprinkler systems, warning systems for evacuation, control centres including video monitoring systems etc. Organisational measures are, for example, a clear emergency management system and safety consciousness of all staff and occupants of a building. Self-rescue, and rescue by other means, can be made more effective by having regular trial exercises such as 'fire drills'.

The consequences of the loss of an important building can be mitigated by having alternative facilities that can be used for the same purpose within a short time. In the case of bank, this may involve the duplication and storage of records elsewhere so that a different branch could take over the function of the one that has become dysfunctional.

The consequence reducing measures shall be implemented in a cost effective way fulfilling the overall risk acceptability criteria. They can be selected based on their reduction of risk ΔR and the costs ΔC necessary to achieve it. Uncertainties in these parameters should be taken into account (Stewart, 2008).

7.2.6 Differentiation of implementation of methods

The implementation of the design methods and of the associated measures depends on the type of building under consideration i.e. on its classification (see Chapter 2). For buildings of low importance with minor consequences in case of collapse, specific consideration of robustness is not necessary. On the other hand, for buildings of the CC3 consequences class a sophisticated level of analysis, including advanced structural analysis and risk analysis, should be implemented.

The differentiation of procedure used in relation to robustness is common in codes such as the Eurocodes. The design considerations recommended in the Eurocodes for the three different consequence classes CC1, CC2 and CC3 defined in Chapter 2, are shown in Table 7.3.

Class 1	No special considerations
Class 2, Lower Group Frames	Horizontal ties in floors
Class 2, Lower group Wall structures	Full cellular shapes Floor to wall anchoring.
Class 2, Upper Group	Horizontal ties and effective vertical ties OR limited damage on notional removal OR special design of key elements
Class 3	Risk analysis and/or advanced structural analysis recommended

Table 7.3: Differentiation of robustness measures in Eurocodes

For CC3 buildings a risk analysis is required. In order to perform risk analysis acceptability criteria are required. Useful information on risk assessment and risk appraisal are briefly discussed next.

7.3 Risk Assessment and Risk Appraisal

7.3.1 Acceptable Reliability

Current structural design codes do not provide acceptable values for reliability or the risk related to, for example, the global failure (collapse) of a structure. Only target reliability values for components are provided (see for example JCSS, 2011, Diamantidis and Bazzurro, 2007), and they cannot be directly used when global failure i.e. partial or full structural failure, is to be considered.

Target and acceptable values of probability of global failure are available for in some case as summarised below:

- Acceptable failure probability for global failure of pipelines in case of accidental loads (ALS: accidental limit state) $p_A = 10^{-6}$ per year and for global failure of offshore structures $p_A = 10^{-5}$ per year (Moan, 2007)

- Acceptable failure probability for global failure of buildings in earthquake regions in the U.S.A. $p_A = 2x10^{-5}$ per year (Hamburger et al, 2003)

- Inherent risk of CC3 buildings in Spain (Tanner, 2008) $p_A = 10^{-6}$ fatalities per year per m^2 net floor area.

7.3.2 Risk Matrix approach

In many structural engineering projects the computation of inequalities (7.1) or (7.2) seems a difficult task due to scatter in available data, model uncertainties etc. Therefore, for buildings, a simplified risk matrix approach can be implemented as in other types of structures such as tunnels, pipelines, platforms, chemical plants.

Many practical studies of the societal risk of a project are in the form of a numerical F-N-curve, which is usually a straight line in a log-log plot (see Figure 7.1). An F-N-curve shows the relationship between the annual frequency F of accidents with N or more fatalities. It expresses both the probability and the consequence associated with given activities. Usually these risk curves are shown in a log-log plot with the annual frequency given on the vertical axis and the number of fatalities depicted on the horizontal axis. Upper and lower bound curves are recommended based on gained experience with similar projects/activities and the ALARP (As Low As Reasonably Practical) acceptability criterion is obtained as the domain between the aforementioned limits. The upper limit represents the highest risk that can be tolerated in any circumstances while below the lower limit represents the risk which is of no practical interest. Such acceptability curves have been developed for various industrial fields, as mentioned above, including the chemical and the transportation industry.

The ALARP recommendations can be represented also via a so-called risk-matrix. For this purpose, *qualitative* hazard probability levels have been defined together with hazard severity levels in terms of their consequences. These hazard probability levels and the hazard severity levels can be combined to generate a risk classification matrix, the principle of which in Tables 7.4 and 7.5 as proposed for CC3 buildings by Harding and Carpenter [2009]. The information provided in the tables represents an example how events can be classified according to their likelihood (probability of occurrence) and the severity of their outcome (consequences). Acceptability criteria can be then set by combining both parameters, for example rare events with a severity degree up to serious may be acceptable.

Likelihood of the event: Chosen by the designer	**Frequency**
Frequent	More than 10 per year
Likely	Between 1 and 10 per year
Occasional	Between 1 each year and 1 every decade
Unlikely	Between 1 every decade and 1 every century
Rare	Between 1 every century and 1 every 1000 years
Improbable	Between 1 every 1000 years and 1 every 10000 years

Table 7.4: Example likelihood categories

Severity of the event; Chosen by designer	**Assumed consequences of the chosen event**
Disastrous	20% to 100% collapse
Extreme	15% collapse of floor to 20% collapse of building
Serious	Up to 15% collapse of floor
Magnificent	Loss of structure member local to hazardous event but no collapse of floor
Minor	Local structure damage but no loss of structural members
Negligible	Superficial damage only

Table 7.5: Example severity categories

7.4 Design examples

Some design examples related to the aforementioned design methods are presented below. In the first example the direct design methods, the Specific Load Resistance (SLR) and the Alternative Load Path (ALP), are illustrated in the case of the Murrah Building in Oklahoma. The practical implementation of the latter, indirectly via indirect prescriptive rules, i.e. tie design, is then illustrated. Following this, an example risk assessment is provided.

7.4.1 Direct Design: SLR and ALP methods

The Murrah Building in Oklahoma City, USA, was designed in the early nineteen seventies and constructed during 1974-1976. It had 9-storeys of reinforced concrete frames and shear walls. The building before and after damage from a bomb attack can be seen in Figure 7.2.

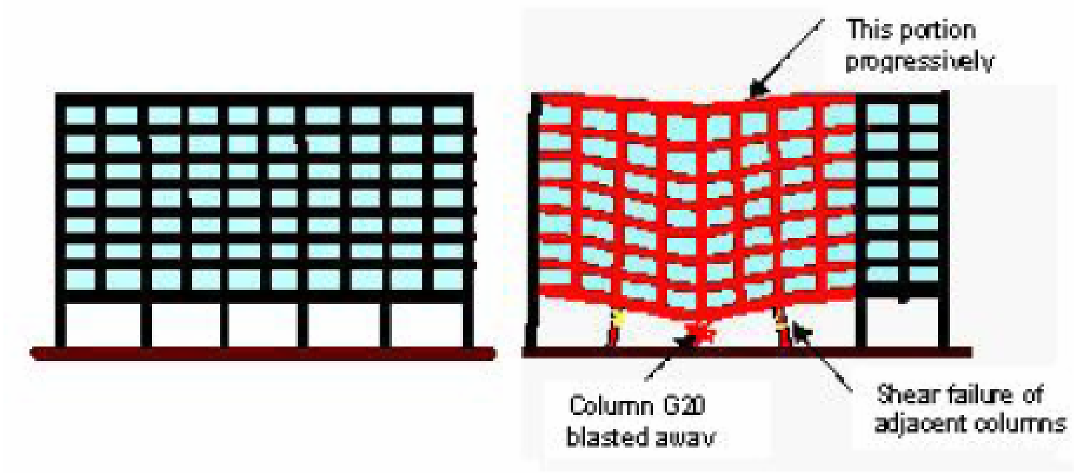
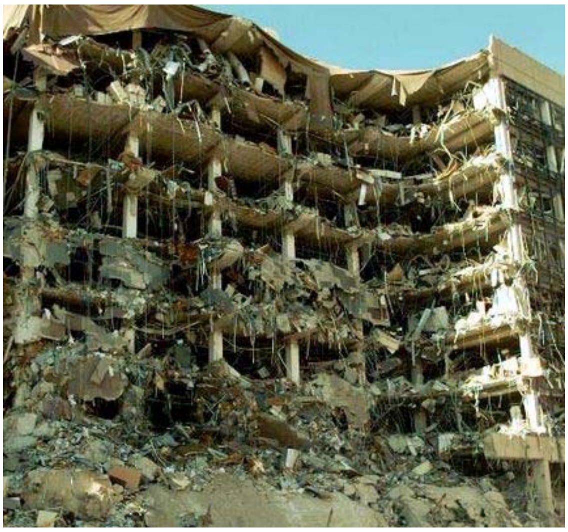


Figure 7.2: Illustration of the building before and after the collapse.

A bomb destroyed the Murrah Building in April 1995. The bomb, which was detonated 3.05 m away from the building in a truck at the base level, destroyed three columns. Loss of support from these columns led to failure of a transfer girder. Failure of the transfer girder caused the collapse of columns supported by the girder and floor areas supported by those columns. The result was a general collapse where about one-third of the building was destroyed. A total of 168 people were killed and over 608 were injured. Figures 7.3 and 7.4 show the destroyed building.



Figures 7.3 and 7.4: Views of the Murrah Building after the bomb attack.

The Murrah building collapsed progressively, initiated by the destruction of a relatively small part (the three columns) of the structure. However, the cause of collapse was a large bomb (4000 lb kg TNT equivalent) which was capable of causing damage over an area of several city blocks. Investigations have also shown that with some modest changes in the design, the damage from the bomb could have been reduced significantly. The changes, based on a study by Corley et al. [1998], could have been:

a) Improvement of the local resistance of the columns as illustrated on Figures 7.5 and 7.6 (taken from Starossek, 2006).

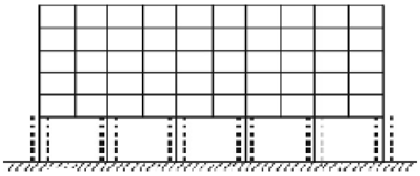


Figure 7.5: Protecting the columns in the ground floor

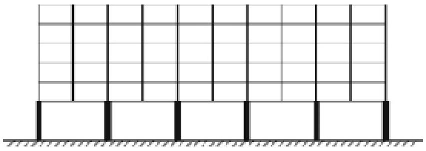


Figure 7.6: Strengthening of the columns in the ground floor

b) Improvement of the effectiveness of alternative load paths as shown on Figures 7.7 and 7.8 (taken from Starossek, 2006).

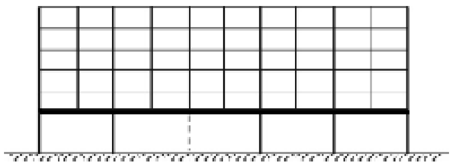


Figure 7.7: Strengthening the transfer girder

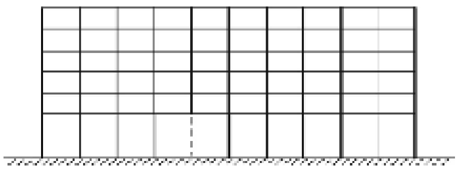


Figure 7.8: Strengthening the ground floor with more columns (but, this would have conflicted with architectural requirements)

The proposed improvements demonstrate that both direct design methods could have been useful in order to avoid the collapse of the building.

7.4.2 Indirect design of ALP via ties

Indirect design of alternative load paths is implemented in the codes in terms of prescriptive rules for the provision of continuity, ductility and redundancy through horizontal and vertical ties. An example tie design according to the Eurocodes of a CC2 framed office building, on having been obtained from Vrouwenvelder [2005], is presented below.

CC2 Buildings 2, Lower Group, Framed structures - Tie design

The design according to the Eurocodes is applied in a typical CC2 building of the lower group. Horizontal ties should be provided around the perimeter of each floor (and roof) and internally in two right angle directions to tie the columns to the structure (Figure 7.9). Each tie, including its end connections, should be capable of sustaining the following force in [kN]:

internal ties: $T_i = 0{,}8\ (g_k + \Psi q_k)\ s\ L$ (but > 75kN) (7.4)

perimeter ties: $T_p = 0{,}4\ (g_k + \Psi q_k)\ s\ L$ (but > 75kN) (7.5)

In here g_k and q_k are the characteristic values in [kN/m^2] of the self weight and imposed load respectively; Ψ is the combination factor, s [m] is the spacing of ties and L [m] is the span in the direction of the tie, both in m.

Edge columns should be anchored with vertical ties capable of sustaining a tensile load equal to 1% of the vertical design load carried by the column at that level.

Consider a 5-storeyed building with story height h = 3,6 m. Let the span be L = 7,2 m and the tie spacing s = 6 m. The loads are $q_k = g_k = 4$ kN/m^2 and Ψ=1,0. In that case the required internal tie force may be calculated as:

$$T_i = 0{,}8\ \{4+4\}\ (6 \times 7{,}2) = 276 \text{ kN} > 75 \text{ kN}$$

For the considered steel quality (FeB 500) this force corresponds to a steel area A = 550 mm^2 or 2 ø18 mm.

The perimeter tie is simply half the value, for the same spacing. Note that in continuous beams this amount of reinforcement usually is already present as upper reinforcement anyway.

For the vertical tying force we find:

$$T_v = (4 + 4)\ (6 \times 7{,}2) = 350 \text{ kN/column}$$

This corresponds to A = 700 mm^2 or 3 ø18 mm.

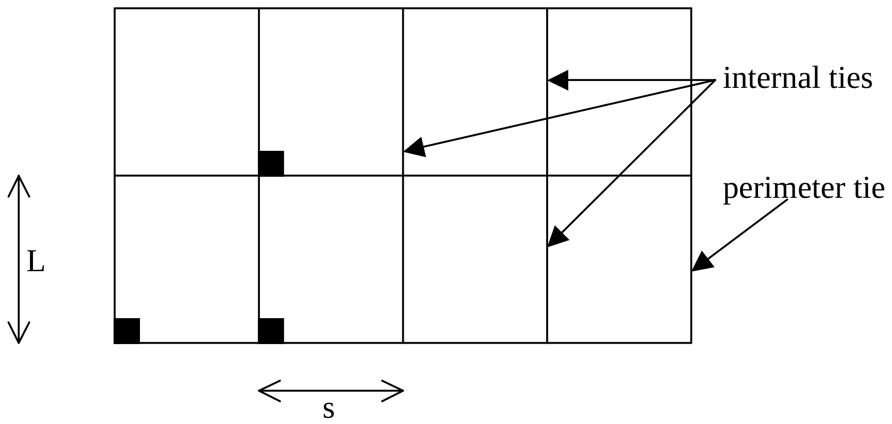


Fig. 7.9: Example of effective horizontal tying of a framed office building.

Class 2 - Upper Group, Load-bearing wall construction.

Rules for horizontal ties similar to those for framed buildings except that the design tensile load in the ties shall be as follows:

For internal ties $T_i = \dfrac{F_t(g_k + \psi \cdot q_k)}{7{,}5}\dfrac{z}{5}$ kN per m width but > F_t (7.6)

For perimeter ties $T_p = F_t$ (7.7)

Where F_t = (20 + 4 n) with a maximum of 60, in which n represents the number of storeys; g, q and Ψ have the same meaning as before, and z is the lesser of 5 h, with h the clear height of the storey in (m), and the greatest distance in the direction of the tie in [m], between centres of columns or other vertical load bearing members, whichever is smallest.

In vertical direction of the building the following expression is presented:

For vertical tie $T_v = \dfrac{34A}{8000}\left(\dfrac{h}{t}\right)^2$ N , (7.8)

but at least 100 kN/m (of length of wall) times the length of the wall. In this equation (7.8) A is the load bearing area of the wall in mm^2, h is the storey height and t is the wall thickness.

For the same starting points as in the previous case we get F_b = min (60, 40) = 40 and z = L = 12 m whichever is the smallest and from there for the internal and perimeter tie forces:

$$T_i = 40\ \frac{4+4}{7{,}5}\frac{7.2}{5} = 61 \text{ kN per m width}$$

$$T_p = 40 \text{ kN per m width.}$$

The vertical tying force is given by:

$$T_v = \frac{34 \cdot 0{,}2}{8}\left(\frac{3{,}6}{0{,}2}\right)^2 = 300 \text{ kN per m width.}$$

For many countries this may lead to more reinforcement then usual for this type of structural elements.

7.4.3 Risk analysis

As mentioned previously, in Eurocodes, a complete risk analysis is recommended for CC3 buildings. This is applicable in general also to any building or structure associated with high risks, such as:

- buildings in railway stations
- high-rise buildings
- hotels in regions with terrorist threat
- embassy buildings
- museums
- broadcasting centres, etc.

A scheme of the procedures to be followed in conducting a risk analysis, as given in Eurocodes, is shown in Figure 7.10.

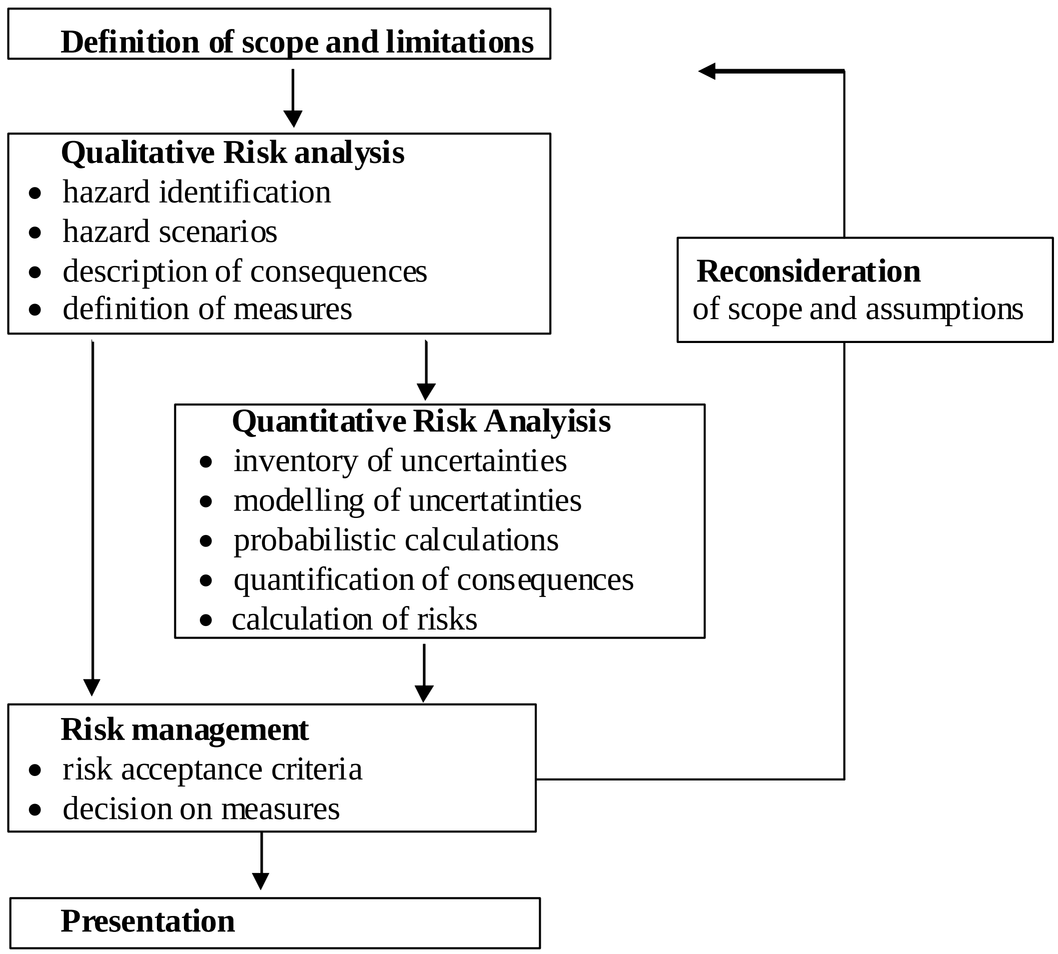


Figure. 7.9: Risk analysis scheme

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8. Robustness during Construction

A. (Hash) Maitra

8.1 Introduction

A major objective of the design process is to prevent structural failures. Unfortunately, too often this is taken to mean failures of the permanent structure, with its temporary form not being given much consideration, sometimes with catastrophic results. This is not acceptable, because a structure which is being built is as much a person's workplace as the permanent structure and, as such, avoiding failure of the "temporary" structure deserves the same considerations as does the avoidance of failure of the permanent structure. Such a failure could also give rise to significant economic and reputation losses to those responsible for the event.

Over the years, there have been many examples of structures that collapsed during the course of their construction. Already, In the second chapter of this document, the case an Army Officers' Mess in England, was presented. In each know case of failure, had some thought been put into considering the effects of getting things slightly wrong at the construction phase, the disasters could have been avoided. Further examples are given below.

Collapse of a hangar-roof steelwork

In 2001, a hangar at an international airport was being extended. The hangar was in the form of a portal frame with trussed steel rafters which spanned about 60 m. The plan for the erection of the roof was to build it on the ground and then lift it on to its supporting columns by means of a dual lift. The steelwork in the roof structure weighed approximately 200 tonnes. While it was being lifted, the roof structure collapsed in a sideways buckling mode.

No one died as a result of this collapse. Nevertheless, the effects on the contract were significant. The steel used in the original erection was not reusable and the whole contract was delayed for a significant period. Therefore, although the collapse could not be described as catastrophic, or disproportionate, in terms of human injury or impact on the environment, it was a fairly serious engineering failure that had significant other consequences disproportionate to the cause.

An investigation by the enforcing authorities showed that the temporary bracing supplied for the purposes of the lift was inadequate.

This example is fairly typical of the collapses that are witnessed on construction sites. A reason for such collapses seems to be the designers' ignorance of the fact that, in order to achieve the final stable configuration, most structures must pass through a temporary unstable phase, which, if not managed properly, could lead to collapse.

In this chapter, some background to collapse of structures during construction, with special consideration given to disproportionate collapse, is given below by reference to case-studies of collapses that took place in the UK between 1980 and 2004. This is followed by some potential rules that could help ensure that disproportionate failure in the construction phase is avoided.

8.2 Background Information and Special Definitions

8.2.1 Failure: a definition

Intuitively, it is understood that when a structure collapses, even partially, then it has failed. This is the obvious definition of failure. However, it is also necessary to appreciate that not all failures are as serious as others. For example, a functional failure such the deformation of the temporary support system of a permanent structure during construction could have different effects depending on whether the latter is deformation sensitive or not. In the case of permanent structures, similar examples can be the failure of a fail-safe signal system of a railway which, although undesirable, need not be as catastrophic as a human error that causes a collision with many associated deaths, whereas there could be catastrophic follow-up consequences when a train derailment is caused by a deformation of the rail track alignment. Nevertheless, a failure to function as envisaged by the client and the designer cannot be ignored, which allows 'failure' to be defined as follows.

> *A structure may be considered to have failed when it does not fulfil its purpose, either functionally or structurally.*

While this is a general definition applicable to all that has preceded this section, it is given special consideration here.

8.2.2 Types of failure

There are three types of failure, as follows:

a) *Overload failure*, which, is usually brought about by the overloading of a component. In the absence of alternative load paths, this local failure causes overloading of adjacent components and, eventually, progressive collapse. Sometimes, it renders a structure (or a large part of it) and the materials used to build it useless. The collapse of a section of bridge into the valley below will render the bridge unfit for use and, inevitably, lead to the scrapping of the components that have collapsed. The only remedy for this type of failure is to rebuild that section of the bridge, if not the whole bridge, i.e., the consequences are catastrophic.

b) *Serviceability failure*, which, when it happens, prevents a structure being used to its full potential. For example, when a component in a building exhibits deflections under load that are enough to prevent things like doors opening properly, it is said to have suffered a serviceability failure. The problem can be remedied, so it is not catastrophic.

c) *Functional failures*, which, when it happens, prevents a structure from fulfilling completely the reason for it being built. For example, a bridge built with insufficient clearance will not allow all types of vehicle that use a road to pass under it. Clearly, this means that the product, the bridge, does not completely fulfil all of the attributes required of a bridge. The problem can be remedied, so it is not catastrophic. However, the costs associated with returning the bridge to full functionality could be significant.

In the construction phase, the functional and serviceability failures are usually not considered because, by definition, they are related to end use. (An example of non-overload failure during construction is where the two parts of a launched bridge or a bored tunnel, built starting from the two ends, do not meet in the middle.) Therefore, the only type of failure considered here is overload failure where the effect of a component failure could have an effect disproportionate to the cause, i.e., the structure in its temporary state is not sufficiently robust.

8.2.3 Importance of robustness during construction

There are a number of reasons for robustness during construction being important. These can be categorised generically as:

a) The ethical reason;

b) The legal reason; and

c) The economic reason;

There are very good ethical reasons for requiring a structure being built not to collapse. Engineers should attach the highest importance to protecting people, which should include the safety of construction workers. Similarly, there are very good economic reasons for wanting to avoid accidental collapse of structures while they are being constructed. No professional engineer can be indifferent to the economic impact that an accidental collapse would have on a project. However, a deeper investigation into these issues is beyond the scope of this chapter.

This leaves us with most important of the reasons listed above: for example, the requirement in the UK’s Health and Safety law to ensure that structures or any part of a structure must remain stable throughout the construction phase. This is an absolute requirement on the person building the structure. In addition, for example, the UK’s Health and Safety law also requires designers of the permanent works to undertake actions to achieve such good performance. The designers are required by law (put into the context of accidental collapse) to:

a) Eliminate the possibility of accidental collapse;

b) When, and only when, (a) is not reasonably practicable,

c) Reduce the possibility of accidental collapse; and then

d) Provide the builder with adequate information about accidental collapse to allow him to manage its avoidance.

These conditions are applicable to a structure during its whole life, beginning with the temporary construction stage.

Clearly, condition (a) cannot be guaranteed absolutely. This means that designers should concentrate on (b): reducing the possibility of a collapse, and (c): providing sufficient information to allow the avoidance to be managed.

Currently, there is very little guidance on how engineers should go about providing the necessary stability during the construction phase. To make the construction case the governing design is not always a feasible approach. However, some design resources needs to be expended when a designer recognises that a structure in its temporary (construction) condition is vulnerable.

8.3 Vulnerable 'Temporary' Structures

The first stage in designing to prevent accidental disproportionate collapse (ADC) is recognising when a structure or a component part of that structure is vulnerable (to failure). The Oxford English dictionary gives a meaning of "vulnerable" as: *susceptible of injury*, which is helpful but does not adequately define the problem. Therefore, a definition of this for use with temporary structures is developed below by starting with some brief examples to illustrate the basic problems.

Example 1: Accidental collapse of a suspended scaffold during construction

A suspended scaffold was being erected over a production hall in an industrial premise. The scaffold spanned some 20m, with proprietary scaffold beams at close centres supporting a timber platform. The ends of the proprietary beams connected into a lattice-type frame of scaffold components. These end frames were attached to the walls of the production hall. While laying the boarding on the supporting structure, the whole suspended scaffold collapsed. (Fortunately, no one was seriously injured.)

An investigation into the collapse showed that the weight of timber boards temporarily stacked on the structure being constructed caused local loading in excess of the design load. This was exacerbated by a lack of diagonal bracing in the end frames supporting the proprietary beams. Consequently, the end frames, at one end, lozenged, allowing the proprietary beams to rotate, initiating the collapse. The overloading of a number of beams caused the whole structure to collapse, progressively.

Example 2: Collapse of a tunnel while under construction

In 1992, the tunnels which would eventually house the underground section of the Heathrow Express railway collapsed while they were being built. A major catastrophe was averted purely by chance and no one died as a result of this collapse. Nevertheless, the effects on the contract were, as in the previous example, significant.

The tunnels were being constructed by the New Austrian Tunnelling Method (NATM), which had been used successfully a number of times elsewhere. The investigation into the collapse showed that poor workmanship at the soffit of the tunnels meant that the compression applied by the egg-shaped arch at this point caused the soffit to heave, destroying the structural integrity of the arch system. The report published by the investigation team, among other things, criticised the designers for producing a design that lacked robustness, i.e., foreseeable, minor errors in workmanship should not have caused the catastrophic collapse of the tunnel. In the construction phase the tunnel was a vulnerable structure.

In each of the examples above, it is clear that the structure was vulnerable because its components relied on being part of a complete system to provide integrity. While both collapses were due, eventually, to a lack of "strength", the routes by which they achieved this "weak" condition were completely different.

In the first example, the whole structure failed because the construction method caused a number of isolated components to carry more lateral load than the inadequately braced end supports could sustain. The critical construction case loading condition was not foreseen by the designers. Therefore, the design did not reflect the reality of how the structure was being built and it was inadequate for this, the governing, load case. As a direct result of this oversight, the loaded members failed, causing the whole structure to collapse into the production hall.

In the second example, poor, but not woefully inadequate, workmanship at the soffit of the tunnel ring prevented it from developing the necessary strength to support the forces applied by the arch structure during construction. This compromised the integrity of the arch and a section of the tunnel collapsed.

In both cases, the designers had not recognised that a method of construction could create components that were so weak that the structure during its temporary (construction) phase would be inherently unstable.

Based on the above, the definition could be proposed for a vulnerable 'temporary' structure:

> *A vulnerable 'temporary' structure is an incomplete structure that in its permanent configuration relies on the stabilising effects of the completed system to perform its structural function.*

In other words, in isolation, these components are structurally inadequate and a designer should recognise these as temporary service conditions worthy of analysis. Based on this definition, a vulnerable temporary structure is not robust because the failure of a single member would lead to further failure.

8.4 Designing to Prevent Disproportionate Collapse of Structures in their Temporary State

The first step in preventing disproportionate collapse is to have in mind the way in which the structure being designed can be built. A clear idea of this could allow situations where the construction case could govern the design to be identified and accounted for early. It could even initiate a change in the structural system to a more robust one.

Having identified how a structural system might be built, it is necessary to take into consideration how vulnerabilities in it could be increased during construction, for example, via :

a) Excessive loads applied during construction;

b) Incorrect sequencing of construction;

c) Temporary weaknesses in a system;

d) Temporary instability.

Each of the initiating actions listed above could lead, eventually, to "overload" of a component which could, in turn, lead to failure of that component. Where there is no sufficiently strong alternative load path, shedding of load to neighbouring components could lead to their overload, and so on, setting up a progressive collapse. Alternatively, there could be a loss of overall stability of the structural system or a major part of it.

8.4.1 Excessive loads applied during construction

On any construction site, delivered components are stored prior to their installation. For example, when constructing a composite floor, i.e. concrete on steel decks, prior to laying them a contractor needs to store the decking sheets somewhere close to where they will be fixed later. Therefore, it is foreseeable that the contractor will store the decking on the structural skeleton. These loads can be significant, depending on whether the contractor splits the packs in which the decking are delivered. If the structural skeleton is not properly checked for the ability to resist these loads, the structure may be vulnerable.

In order to prevent collapses caused by this type of (over)loading, the designer must inform the contractor about the maximum loads that can be applied to the incomplete skeleton. A competent contractor should be able to use this information to distribute the loads accordingly, as long as the constraints are reasonable. In this case, it is inadequate to restrict the construction case load to one as low as 0.05 kN/m^2 which would require a contractor to spread the steel roof sheets or decking over a wide area; it is not feasible and will not happen.

The above principle can be illustrated by reference to a simple (trivial) example. Consider a (permanent) composite beam spanning 24 m supporting a 200 mm thick concrete slab @ 4.8 kN/m^2 and a live load of 5 kN/m ^{2} on permanent decking @ 0.1 kN/m^2. The steel beams are at 6 m c/c.

In the permanent condition, the concrete floor will restrain the top flange, which allows the beam to be designed on the slenderness ratio L_E being zero (continuous restraint is provided by the slab) and allows the beam to support a moment M_{MAX}

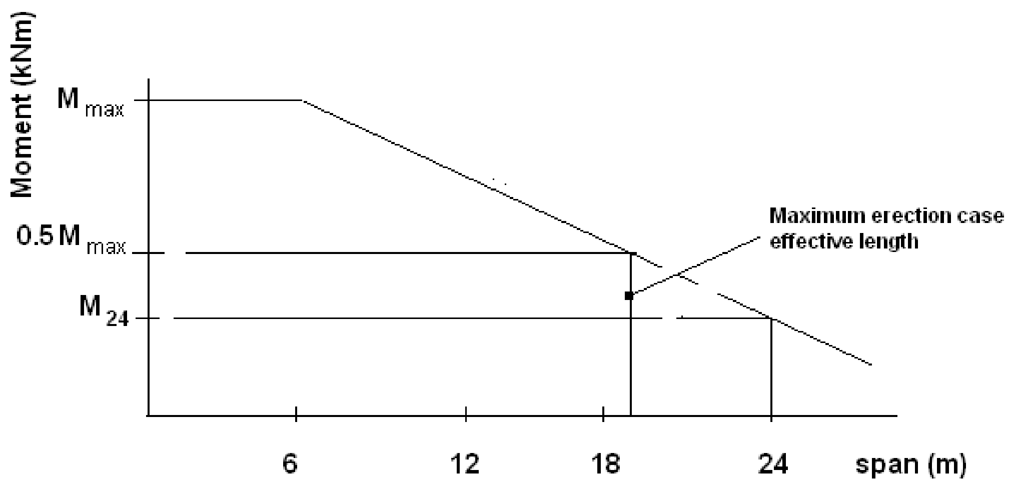

Figure 8.1: Moment capacity-span relation for a beam

During the erection phase, in the absence of any restraint to the top flange, the slenderness ratio of the beam must be based on the full 24 m span, which means that the maximum moment that can be sustained (M_{24} in Figure 8.1) is approximately $0.4M_{MAX}$. Assuming a simply supported beam, the moments are as follows:

Load case	**Dead load Moments** (for characteristic loads)	**Live load moments**	**Total moment**
In-service	$(4.8 \times 6 \times 24^2) / 8 = 2074$ kN-m	$(5 \times 6 \times 24^2) / 8 = 2160$ kN-m	4234 kN-m
Erection	$(4.8 \times 6 \times 24^2) / 8 = 2074$ kN-m	$(4.2 \times 24) / 4 = 25$ kN-m The LL moment assuming 4 men and some equipment equates to 0.75 kN/m^2 as a patch load over 6 m^2 but applied as a point load at the beam centre.	2099 kN-m

Table 8.1: Moments acting on the beam

A comparison of the moments in Table 8.1 shows that the moment during construction is approximately 0.5 x the in-service design case moment. A quick check on figure 8.1 shows that the beam, in its erection case, may have exceeded its capacity and buckling is a real possibility. Therefore, the construction case could be the governing design case and should be checked. If this beam buckles, it is possible that there could be further collapse of the structure. Therefore, the ideal situation would be to prevent buckling completely and provide sufficient top flange restrain, by the use of bridging members, to ensure that failure, if any, will be in bending, i.e., the failure will be local and there would not be further failures resulting from it. From figure 8.1, the effective length of the top flange should not exceed 18 m (precaution against bucking with partial load factor of about 1.1).

8.4.2 Incorrect sequencing of construction

When the sequence of erection is fundamentally important, this information must be transmitted to a contractor. Similarly, when components are critical for safety, this information must be transmitted to the contractor.

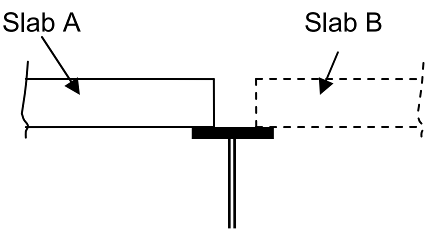

Figure 8.2: Unequal loading of a beam by precast panels

Consider figure 8.2, which shows the placing of pre-cast floor slabs on a universal beam. Where the placing of slabs on side A of the beam (called slab A in figure 8.2) gets too far ahead of slabs on side B (called slab B in figure 8.2), the beam could be subjected to significant torsional forces. If the beam-end connections are not able to support these forces, the beam could collapse, leading to the collapse of all of the precast slabs. Therefore, this type of failure must be prevented, for example, by a good sequence of construction.

Another way to avoid this type of failure is to design the beams for this load case. However, if it is not possible to provide an end connection to support this load, then contractor must be informed so that he can avoid an incorrect sequencing of construction.

8.4.3 Inadequate information to allow development of effective temporary works

A portal frame, by its inherent nature, generates a lateral thrust at its base. For long-span low pitch frames these thrusts can be significant. Unless a contractor knows the magnitude of this force, the temporary works to support this load may be inadequate. There is a danger that the frames can kick out, causing collapse.

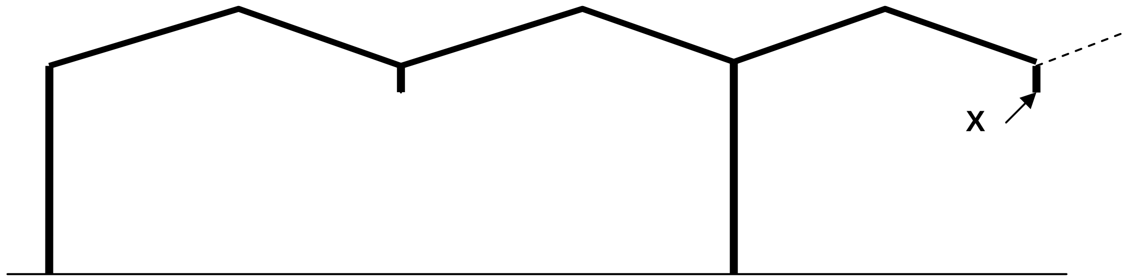

Figure 8.3: Multi-bay portal frame construction

Similarly, for multi-bay portal frames, the lateral thrust on the valley beams, X in Figure 8.3, must be supported during erection: until the next rafter is in place and connected to its supporting column. If the existence of this force is ignored or its values is not known, then it is impossible to be certain that the temporary works will be adequate. If the temporary works fail, there is a likelihood of progressive collapse of the structure. Consequently, it is important to pass on this type of information to a contractor and to highlight the importance of temporary works being able to support this load. Alternatively, the valley beam and the internal columns can be designed to resist the lateral loads applied by the (unbalanced) rafter load. Collapses of this type have occurred on construction sites.

It is also worth noting that this type of collapse can happen if the demolition sequence removes the support to beam X; a situation that is also in breach of , for example, UK health and safety law.

9. Effects of Quality Control and Deterioration

T.D. Gerard Canisius

9.1 Introduction

Engineers are generally aware of the importance of having good quality design, materials, construction, maintenance and repair for the proper performance and durability of a structure. However, whether this knowledge is always practised is questionable because nearly 90% of structural failures have been caused by poor quality or human error (Allan, 1992). Owing to the importance of the quality in preventing structural failure, many publications such as that by Ellingwood [1987], Blockley [1992], Thorburn & MacArthur [1993], Canisius [2000] and Ellingwood and Kanda [2006] on related topics have appeared at various times

In recognition of their importance to good structural performance, codes of practice, such as the Eurocodes, have given prominence to quality and durability. For example, in its Annex B, EN 1990:1992 has provided a method of adjustment of partial safety factors on materials to reflect the level of Quality Control (QC) and supervision provided: There, a lower factor partial factor is recommended when the quality and supervision are better than 'normal' and a higher factor is recommended when these aspects are lower than normal. (Note: However, a partial safety factor cannot be expected to mitigate the effects of gross errors. See below.)

In this document, Quality Control (QC) means the processes of ensuring that any activity or artefact involved in any stage of the design, construction, maintenance and use of a structure does not adversely affect the performance intended by the designer. The designer too may be subject to QC because poor quality can play a part in reducing robustness by providing structural resistance and/or stiffness inferior to that desired or specified.

Supervision and checking are some activities that help to control the quality, although the former needs its own QC because bad supervision is detrimental to a structure. Deterioration, when not arrested and reversed or repaired, can be considered as a form of poor quality. Deterioration and poor quality have a dual role in robustness, because:

- they reduce robustness by weakening structures, and
- robustness can counter their ill effects.

If a disproportionate failure did not result from poor quality construction or maintenance, or improper use, then it could be related to poor quality design, provided the cause of failure was not beyond the state of the art.

9.2 Examples of Poor Quality that Affected Robustness

As mentioned in the previous section, any robustness reducing effect can be considered as due to poor quality. Some examples of these are shown below.

(a) Ronan Point (Poor concept))

The Ronan Point failure (Figure 1.1) was caused by a design that considered the friction between wall panels and floors as sufficient to resist lateral loads on the former. In the case of an internal explosion that lifted the ceiling floor of a storey, the inter-component friction became almost negligible, allowing a wall to slide off its 'support'. This collapse is due to poor quality design, although it may be argued that it was beyond the state of the art of the time. Although progressive failures had occurred before this partial collapse, it was only following this disaster that the world's first disproportionate collapse regulations came into being in the UK.

(b) Officers' Mess, Aldershot, UK (Poor quality construction and instability under 'temporary' conditions)

On the 21st July 1963 one of four identical buildings being constructed in the UK collapsed (BRS, 1963). These buildings consisted of three storeys and a penthouse, with a total height of 40 ft (Figure 9.1). Each building had a concrete frame built with precast beams and columns, using in-situ concrete joints, and clad with precast concrete panels. At the time of the collapse, the building frame had been erected to its full height and many of the cladding panels had been attached. After the collapse it was decided to demolish the other three buildings but one of them collapsed before this was done!

The initiation of the collapse of the structure was attributed to the local failure of a beam to beam or beam to column joint, due to a poor quality connection between them. Then, the general instability of the building which was under construction, due to the absence of any wall panels or bracings in the middle storey at the time, had contributed to the disproportionate collapse.

(c) Various Large Panel System (LPS) Buildings (Poor quality construction)

The Ronan Point failure occurred because some components were held together only by friction. The resulting changes to regulations resulted in the provision of mechanical connections between all precast components. Limited intrusive inspections carried out on several similar buildings during subsequent retrofit operations have revealed that not all joints between components in these buildings had been well constructed. There was an extreme case with no vertical dowel placed within the loops of reinforcement that extended out of wall adjoining panels. In another case, to make construction easier, an ill-fitting floor reinforcement loop had been bent away from the wall dowel to which it should have been connected. Such defective joints could have contributed towards the potential progressive collapse of these buildings under internal overpressure such as that from a gas explosion.

Figure 9.1 Aldershot building prior to the collapse (Photo: UK Crown Copyright.)

(d) Reinforced Concrete Buildings during the 1999 Kocaeli Turkish Earthquake (Non-application of standards and poor quality material/construction)

The collapse of many reinforced concrete buildings during the 1999 earthquake in Turkey has been attributed to designs which were not carried out according to the applicable standards and to the poor quality of materials and construction (Figure 9.2).

Figure 9.2: Failure due to poor quality design and/or construction – the absence of stirrups in the column. (Photo by courtesy of EEFIT, 1999)

(e) Hyatt Regency Hotel Walkway (Incorrect implementation of designer's intentions)

This progressive collapse of a walkway was caused by poor implementation of a design which was originally "good" in terms of safety. Although the contractor had changed the difficult-to-construct original detail, the new defective joint had not been checked by the Designer. The poor joint failed from 'overloading' (Figure 9.3), and that led to a progressive collapse.

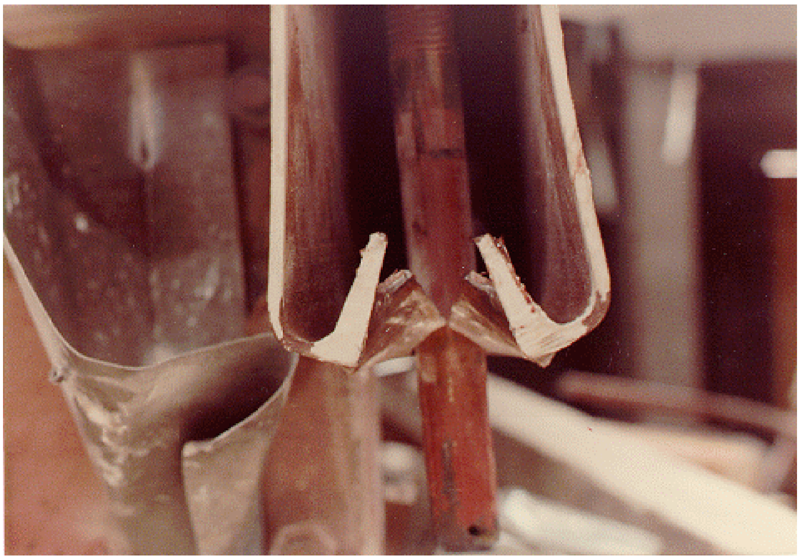

Figure 9.3: The failed connection at Hyatt Regency Hotel Walkway. (Photo © Dr. Lee Lowery, Jr, Texas A&M University)

(f) Collapse of a wedding hall floor, Jerusalem, Israel (Bad structural modifications)

This is an example of poor quality structural changes. The building had been extended to provide an additional storey on the existing flat roof. The new 'floor', which was not strong enough for the imposed new live load, had survived by also resting on the 'non-structural' partitions of the storey below. However, once the partitions had been removed later to create more unobtrusive space, the floor had sagged and then collapsed during a wedding celebration on 24 May, 2001. The floor collapsed through another floor, making it a progressive collapse via overloading of a region. Further details available in:

- *Versailles wedding hall disaster*, Wikipedia.
- *http://news.bbc.co.uk/onthisday/hi/dates/stories/may/24/newsid_4530000/4530071.stm*

(g) Pipers Row Car Park, UK (Poor design and maintenance)

The horizontal progressive collapse of a floor of this structure is attributed to both poor concrete patch repair and poor structural design [Wood, 2003]. However, the unintended non-continuation of reinforcement between different areas of the roof slab prevented a more significant progressive collapse.

Figure 9.4. Pipers Row Car Park, Wolverhampton, England, progressively collapsed under its own weight. (Photo by courtesy of the HSE, UK).

9.3 Quality Control and Deterioration in Codes of Practice

In many codes of practice, such as the Eurocodes, the primary design requirement is the sufficiency of reliability of each component/element/connection of a building. This is usually achieved via the limit state design equations that use characteristic values and partial safety factors calibrated to the required reliability level. This reliability level typically corresponds to an annual probability of failure of the order 10^{-6}. However, additional requirements/measures have to be used when ensuring that a structure has sufficient reliability also as a system which is unlikely to suffer disproportionate collapse.

In addition to the above, provisions in design and construction can help to reduce or eliminate the effects of design errors, execution errors, unexpected deterioration of components, etc. Robustness requirements in codes of practice should cover such aspects and also quality control systems and application of best practices in design, execution and operation & maintenance as illustrated in Figure 9.5. However, in most structural design codes, quality control and durability are not addressed in sufficient detail as may be desirable.

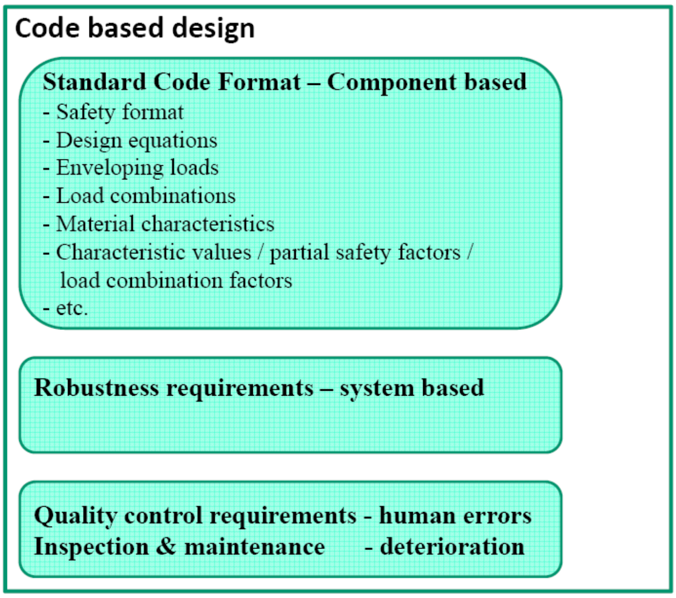

Figure 9.5: Position of Quality Control and Durability in Code-based design

The current suite of Eurocodes have given significant importance to quality control, supervision, deterioration/durability via the main Eurocode, EN 1990: Basis of Design. Its principles and rules are applicable to constructions of any material. Additional considerations are available in the Eurocodes for different materials.

9.4 Quality Control and Robustness

There are two main issues to be addressed when controlling the quality of structures [Canisius 2000]. These are

- Gross errors, or ‘human errors’, and
- Quality errors in materials and fabrication.

In addition, it is necessary to be aware that errors can occur during research and codification, potentially leading to deficient design guidance. This issue, which is beyond the remit of the ordinary designer, is not considered here.

9.4.1 Gross errors

A gross error, by definition, will affect a component or a connection of the structure to a level that is beyond the ranges assumed or considered in a design. Such an error can significantly alter the structural performance in a detrimental manner. Gross errors and robustness are very closely connected because the latter could help to mitigate the effects of the former while the former itself could cause the absence of the latter.

Gross errors may occur at any stage of the life of a structure, such as design, detailing, construction, use, inspection and maintenance. For example, during use, damage or detrimental changes can happen to a component. Overloading by unanticipated changes of use too can be considered as a gross error.

Designing against gross errors could be either very costly or even impossible. Therefore, the best method of guarding against them is to provide good supervision and quality control. Such measures can help to ensure that a design is carried out properly and the resulting structure is constructed, maintained and used in accordance with the designer's brief

Although structural reliability methods have been developed to consider gross errors and their effects, they are still in their infancy and cannot be considered as a tool for practising engineers.

9.4.2 Quality errors

Poor quality need not affect the performance of a structure as drastically as could a gross error. However, when quality is poor, especially during either the design or some aspects of construction, the result could be a gross error. That is, although identified separately, gross errors too are a result poor quality related to certain types of activities. Therefore, for the purpose of this discussion, quality errors are those that do not result in gross errors but affect one or more parameters in a relative small way.

Poor quality material or components can be present in a structure due to various reasons such as:

- damage in transportation or use
- poor production techniques
- poor supervision and checking of material supplies
- poor production and storage
- imprudent saving of resources at any stage of the design-resourcing-manufacture-supply-erection-maintenance chain,
- poor maintenance and repair of the completed structure, and
- neglect of a structure, that gives rise to deterioration [Canisius 2000].

The issues highlighted above are situations beyond those usually accounted for by the use of partial safety factors.

In most structural design codes, structural safety aspects of quality control and supervision are not explicitly addressed using mathematical methods. Instead, a certain level of quality control and supervision is expected or specified by the designer whose design assumptions are usually implemented (as far as possible) by the presence of the Engineer's team on site and the use of control tests, the results which have to fall between specified limits.

The Eurocode EN 1990 has made a first attempt at incorporating quality effects in its informative Annex B. However, it does not consider the gross errors that result from poor quality control.

9.5 Assuring Quality for Assuring Robustness

As discussed above, various causes can create poor quality, i.e. unexpected variations in important properties and parameters or gross changes. Poor quality can materialise to affect a structure at anytime during its life, from conception to demolition. Some of the causes of poor quality that can arise at various stages of life of a structure are given below.

a) During design - Errors or lack of knowledge related to the concept, calculations and detailing. Lack of knowledge of important design parameters can result also from insufficient investigations/data gathering (for example of soil parameters). Errors can also happen by oversight (for example, water pressure arising from heavy rainfall).

b) In procurement – Insufficiency of controls put in place (e.g. supervision)

c) During construction – Neglect of duty, use of poor material, damaged material or components, poor fabrication, errors in fabrication, errors in setting out, insufficient curing measures, insufficient protection from accidental damage.

d) After construction – Overloading/misuse, improper maintenance, bad changes to the structure, wear or damage to protective measures against, for example, fire and moisture.

e) Work conditions that affect worker motivation and care (see Canisius and Maitra, 2004).

Note: A designer's performance can be checked by:

- the designer himself (self-checking, or Category I),
- colleagues or superiors (Category II), and
- an independent party. (Category III).

However, the quality control process will not work properly if the designers and the checkers themselves lack adequate knowledge of the subject and, importantly, also an awareness of what they lack.

9.5.1 Quantification of Quality Effects

The quantification of quality aspect is a difficult exercise and may not provide much advantage in some situations. While issues related to poor materials and fabrication tolerance is easier to deal with by considering past data, it is not so for gross errors because it is not possible to predict them. The gross errors can be very complex and are affected by different reasons such as a lack of training, lack of motivation, over-work, and a simple disregard for quality of staff. Supervision and checking are good ways of preventing gross errors and, thus, of assuring robustness of a structure while robustness itself will help to mitigate consequences when a gross error does occur.

9.5.2 Methods of considering and avoiding quality problems

A non-exhaustive list of methods for preventing quality related problems in structures is given in Table 9.1 below.

Activity during the life a structure	Type of Quality Issue that can affect robustness and safety	Methods of preventing occurrences and reducing or eliminating their effects
Conceptual design	Poor concept, giving a structural type sensitive to errors and poor quality	Employment of knowledgeable Engineers, preferably with experience
Engineering design and detailing	Erroneous calculations	Capable staff. Check calculations. Use verified software.
	Not consider or erroneously consider an important safety aspect	Capable staff who understand structural behaviour. Staff to be conversant with Codes to be employed. Staff to be aware when to seek help. Independent checking. Use validated software.
	Wrong assumptions on structural or material behaviour	
	Incorrect detailing	Capable staff. Checking.
	Erroneous notes in drawings	Capable staff. Checking.
Procurement	Poor control measures	Contracts to have good quality control measures and acceptance testing.
	Priority of costs over quality	Adequate funds.
	Inconsistent quality	Choose suppliers with good QC measures.
Construction	Poor material quality on site	Test for quality. Reject poor quality. Good suppliers.
	Damaged components	Inspect prior to and after construction. Reject/Repair.
	Incorrect setting out	Well trained staff. Checking prior to start of construction. Early discovery would reduce complexities of correction.
Activity during the life a structure	**Type of Quality Issue that can affect robustness and safety**	**Methods of preventing occurrences and reducing or eliminating their effects**
	Incorrect dimensions of components	Check.
	Poor curing (where necessary)	Good supervision and procedures. Well trained staff.

	Badly applied protection measures	Good supervision and procedures. Well trained staff.
	Poor connection of components	Good supervision and procedures. Well trained staff.
	Poor (too low or too high) prestressing	Good supervision and procedures. Well trained staff.
	Contractor changing details without permission of designer. Designer not checking a contractor's changes.	Good supervision and procedures. Safety conscious staff.
	Inadequate site investigations	Good specifications. Good supervision and procedures. Well trained staff.
	Other poor construction practices.	Good specifications. Good supervision and procedures. Well trained staff.
In use	Overloading components.	Awareness of occupiers and users. Load limit signs.
	Bad structural changes, usually without an engineer's involvement	Awareness of occupiers and users. Awareness of contractors. Availability of as constructed drawings.
	Damage to components or their protective measures	Good periodic inspection and maintenance.
	Not maintaining regularly	Owner/Manger commitment to maintain.
	Improper repair	Good specifications. Good supervision and procedures. Well trained staff.
	Changes that affect foundations	Awareness of occupiers and users. Awareness of contractors. Availability of as constructed drawings.
During demolition	Bad demolition sequence, caused by defective understanding of structural behaviour.	Awareness of contractors. Good contractors. Availability of as-constructed drawings.

Table 9.1: Various quality issues that may arise during the life of a structure and methods of preventing them or reducing/eliminating their effects.

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10. Conclusions and Recommendations

T.D. Gerard Canisius

This document provides information to the practising engineer concerned with the design of buildings using a risk-based approach. Such an approach is required for 'Class 3' Buildings of the suit of first-generation Eurocodes which are in use now. While not claiming to be complete in every sense, the document provides sufficient information to guide an engineer who is contemplating such a design.

In order to help designers appreciate the reasons for the existence of disproportionate collapse related regulations and code requirements, information on the history and the current state of disproportionate collapse regulations, particularly in Europe, is presented. This historical exposition starts from that of the Ronan Point collapse in the UK in 1968.

Considering that safety of occupants and fears of the public are the drivers for safer design, issues related to management of risks and public perception of acceptable damage/consequences, especially in relation to disproportionate collapse, are discussed in Chapter 3. It is recommended here that the presented novel concepts be applied in the present European context and examples be produced for the benefit of practising engineers. These and further research could also support any new practical recommendations that can be implemented in Codes and Regulations.

Any disproportionate collapse starts with the realisation of a hazard. It can be either a natural hazard or one initiated by humans (i.e. a 'man-made' hazard). Design of buildings, with or without the explicit consideration of risk, depends significantly on the availability of information on, for example, the rate of occurrence and intensity of such hazards. Therefore, information on these are given in Chapter 4, with further details provided in Appendix B. However, more efforts are required to gather and collate information and generate further knowledge related to some actions (loads) and methods suitable to model them.

The realisation of hazards has consequences to buildings, their users and occupants, and the general public. The types of consequences that should be considered in an analysis depend on the boundary of the 'system' considered in a risk assessment. The consequences can be in the spheres of life safety, economic and business costs, environmental effects, reputational costs, etc. Therefore, a description of various consequences and an explanation of how they depend on the definition, or the boundary, of the considered system are provided in Chapter 5.

'Decision making' is what engineers frequently do during design or assessment of a structure. These decisions can relate to, for example, the form of the structure and strength and stiffness of its components. In relation to robustness, the protective measures and consequence reduction measures too form a part of this process. The 'risk' concepts described in Chapter 6 provide a rational and comprehensive way of making robustness related decisions. The concepts presented there are useful for conducting fully-fledged analyses and for developing and calibrating, simpler and more practical, risk-matrix or F-N curve type criteria presented in Chapter 7.

The concept of a robustness index and example developments are presented in Chapter 6. These include the historical semi-probabilistic or reliability based indices and a novel risk-based index. It is noted that the concept of 'robustness' in codes and that of reliability and risk-based robustness indices are not identical. This is because the indices deal with progressive collapse, i.e. situations related to relatively large 'follow-up' consequences', whereas the codes' definition encompasses also the potential initiating failure. For example, the definition of robustness in the Eurocode EN 1991-1-7:2006 relates to the ability of a structure to be not damaged to a level that is disproportionate to the original cause. According to this definition, a structure which is designed to have strong "key elements", but which may progressively collapse once a key element is removed, i.e. a structure with a low robustness index, is robust. This difference in definition should be kept in mind by engineers. Although these do not create practical difficulties and both are equally valid and useful, it is recommended that the two concepts be unified before the next generation of Eurocodes are developed.

The practical methods of robustness design are presented in Chapter 7. Although this document's concern is Class 3 buildings, also the design of Class 2 buildings using conventional tying and key elements is presented as useful background material. Various methods of risk-based design, such as the use of a risk matrix or an F-N curve and with explicit risk considerations, are presented and discussed. While results of previous work discussed in this document exist, useful exercises for the future can be to:

- study consequences of past structural failures to develop information beyond the traditional F-N curve, to consider non-safety related risks,
- develop further example calculations and bench marking studies to aid the practising engineers, and
- reach consensus on risk analysis methods for use with important (Class 3) buildings, including on ways to consider uncertainties.

Following the completion of the first seven chapters that deal with the design of completed buildings, two further chapters are used to present aspects of construction that may be overlooked by an engineer who is under pressure to deliver a design on time and to budget. First of these two aspects is robustness during construction of a building. It is described in Chapter 8, with case studies. The importance of considering the method of construction, so that a structure can be kept robust also in its temporary state, is discussed there. The presented calculations are based on standard code-based limit state methods. The application of risk-based methods to this phase of construction is being given importance at present.

The opinion that any robustness related failure can be attributed to poor quality at any stage of the life of a structure, starting from concept to use to maintenance and demolition, is presented in Chapter 9. The reasoning is demonstrated with real-life examples. A call for good quality assurance during every stage of the life of the structure is made.

The information presented in this document is such that a practising engineer can embark on risk-based structural design of Class 3 buildings of Eurocodes. However, owing to the complexity of concepts, an engineer who is not familiar or comfortable with the proposed methods must seek advice from a competent person.

The information collected, developed and presented here can form the basis for the development of new Eurocode requirements for the design of Class 3 structures. Towards this, it is recommended that the Joint Committee of Structural Safety and the European Union consider embarking on benchmarking studies that can feed into the development of the next generation of Eurocodes and further help practising engineers.

Appendices

A: Terms and Definitions

B: Hazard Modelling

C: Robustness in Other Disciplines

Appendix A: Terms and Definitions

A list of basic terms and definitions related to structural robustness is given below.

Abnormal Load: A load that is not considered to act, or does not act, on a structure during its normal service life.

Accidental Action: An action or a high intensity of an action not considered to act during the normal service life of a structure. Formally, in Eurocodes, these are earthquakes and those mentioned in EN 1991-1-7:2006.

Alternative Load Path (ALP): The methods a structure will use to transfer loads to the foundations, using a different load path, following the removal of a load-bearing component.

As Low As Reasonably Practicable (ALARP): Risk levels that fall between **Tolerable** and **Intolerable** levels. When a risk is within the ALARP region, the designer is expected implement risk reduction measures as long as their costs are justifiable in relation to the benefits achieved.

Catenary Action: The resistance of lateral loads by the generation of tensile forces in a longitudinally restrained beam or slabs.

Consequence: An outcome of an event.

Direct damage/consequences: The damage or consequences that result when a load acts on a component.

Disproportional collapse: The collapse of a structure, or of a significant part of it, initiated by a relatively small triggering event.

Follow up damage/consequences: See Indirect damage/consequences.

General collapse: The failure of a whole structure by a single triggering event. Only "Direct damage" is present in a general collapse.

Hazard: A set of circumstances with the potential to cause events with undesirable consequences.

Identified hazard: A hazard that has been identified as can occur and is relevant to the performance (failure) of a structure.

Indirect damage/consequences: The damage or consequences that could occur in a region beyond the locality of an action. They follow failure of one or more load bearing components as a direct result of the action.

Intolerable risk: A level of risk that is not acceptable and must be reduced.

Key element: A load bearing component, the survival of which is necessary for the stability of a structure. When the failure of a component could lead to more consequent structural damage than is tolerable, then the Eurocodes require the former to be designed as a Key Element that can survive a specified accidental load.

Limited local collapse: Failure of a structural component without affecting the adjacent components (e.g. destruction of a few columns in a multi-bay structure, which does not lead to total collapse).

Progressive collapse: The spread of an initial local failure from component to component, eventually resulting in either the collapse of an entire structure or a disproportionately large part of it.

Probability: The likelihood or degree of certainty of occurrence of a particular event during a specified period of time.

Redundancy The presence of more components than necessary for structural stability. A statically indeterminate structure has redundancy and, thus, alternative load paths.

Reliability: The ability of either a structure or a structural component to fulfil its specified requirements during a given period (e.g. during the design life).

Risk: A measure of the danger that an undesired event represents for humans, the environment or the economy. In general, the risk is the combination (product) of the probability of an event and its expected consequences.

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

Tying: Provision of means of transferring forces between various components of a structure so that it can act as a whole and generate alternative load paths.

Tolerable risk: The level of risk that is acceptable.

Unidentified hazard: A hazard not considered in a design because either its importance to the structure or its potential presence was not known at the time of design.

Appendix B: Modelling of Hazards

A. (Ton) Vrouwenvelder

Some examples of hazard modelling are given in this appendix. The hazards dealt with are as follows, with the hazard number referring to the sub-section within this appendix.

B.1 Earthquakes
B.2 Fire
B.3 Vehicle impact
B.4 Ship collision
B.5 Explosions

B.1 Earthquake Modelling

Frequency and magnitude

For all seismic active areas in the environment of the construction site statistics on the earth quake magnitude M for faults should be known. Usually the number of earthquakes per year exceeding a given magnitude may be expressed as:

$$\ln N(m) = A - B(m - m_o) \qquad \text{for } (m > m_o)$$

Here $N(m)$ is the average number of earthquakes per year with a magnitude M larger than m, which occurs in the given seismic active region Earthquakes of which $M < m_o$ are not taken into account. They are considered not to be relevant for the structures (m_0 for example may be equal to 3.0). The number of earth quakes per year with $M > m_o$ is equal to

$$N_o = \exp(A)$$

For a given earthquake it can be stated that the probability exceeding m is given by:

$$P\{M > m\} = \exp\left[-B(m - m_o)\right]$$

In many cases there are several areas with different earthquake characteristics.

Attenuation

The relation between an earthquake with magnitude M occurring at a distance R and the peak acceleration at the building site (see Figure 3.1) can be written as:

$$\hat{a}_g = f(M, R) * \varepsilon$$

with M the magnitude of the earthquake, R the distance to hypocenter, ε a model uncertainty factor with a mean equal to 1.0 and $\hat{a}_g$ the peak ground acceleration at the construction site. In some formulations also the depth H of the epicentre is considered. For the deterministic part of above formula, for example the relation stated by Donovan (1973) can be used:

$$\hat{a}_g = \frac{C \quad e^{0.5M}}{(R+25)^{1,32}} * \varepsilon \quad ; \quad \text{C = 10.8, } \hat{a}_g \text{ in m/s}^2 \text{ and } R \text{ in km}$$

The distance R is a random variable too as the hypocentre of the earth quake may happen anywhere along a fault or within some area. In a seismic hazard analysis the contributions of all seismic zones shall be considered and summed up.

The earthquake motion description

The soil movement during an earth quake is a function of time that can be modelled as a zero mean Gaussian stochastic process. In order to describe the frequency content, Kanai and Tajimi have derived the following expression for the (one sided) spectral density function for the stationary strong motion part of the earth quake:

$$S_{aa}(\omega) = \frac{G_o [1+4\zeta^2{}_g(\omega/\omega_g)^2]}{[1-(\omega/\omega_g)^2]^2 + 4\zeta^2{}_g(\omega/\omega_g)^2}$$

In this model G_o is a scaling factor, the parameters ζ and ω_0 are chosen on basis of the dynamic properties the local soil. The variance corresponding to this spectrum can be determined to be:

$$\sigma^2{}_A = \int_0^\infty S_{aa}(\omega)d\omega = \frac{\pi\, G_0\ \omega_g}{4\zeta_g}(1+4\zeta^2{}_g)$$

Usually ζ is in the order of 0.60 and ω_o may vary from 5 to 50 rad/s, depending on local geological conditions.

The strong motion duration represents the time interval over which the motion intensity is almost constant and near its maximum. It is preceded by a relatively short rise time and followed by a relatively long decay period. Figure B.1 gives a schematic impression. In the decay time the frequency content of the signal may shift to the lower range. Different strong motion duration definitions can be used for different seismic records.

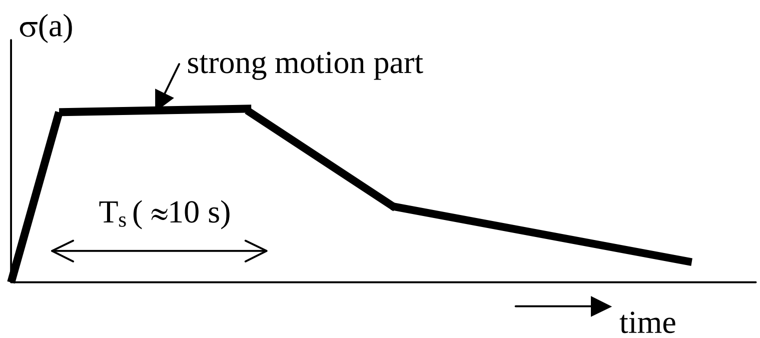

Figure B.1 Shape of earthquake intensity

The above time intervals depend on the intensity of the ground shaking. For rock conditions and magnitudes of order of *M* = 5 - 8, one may assume a rise time of 1-3 s and a strong motion time of 5-15 s; the decay time is usually longer then the strong motion time. According to the theory of random processes, peaks in (a narrow band). Gaussian random processes have a Rayleigh distribution:

$$P\left\{\hat{a} > a\right\} = exp\left\{-\frac{a^2}{2\sigma_a^2}\right\}$$

The number of peaks to be expected in the strong motion part is equal to $N = T_s / T_o$ where T_s is the strong motion duration and $T_o = 2\pi / \omega_o$.If the peaks are stochastically independent, the expected maximum peak may follow from:

$$\mathrm{E}\{\hat{a}\} = \sigma_{\mathrm{a}} \sqrt{2 \ln(2N+1)}$$

Using this equation we may in reverse calculate the standard deviation for an earthquake with a given peak ground acceleration.

The above vibrations are horizontally in the direction of the earth quake wave propagation. In some cases it may be necessary to include also the two other directions.

B.2 Fire Modelling

Fire frequency

For most applications, it is sufficient to consider fire ignition as a Poisson process with constant occurrence rate:

$$P\{\text{ignition in } (t,t+dt) \text{ in a compartment}\} = \nu_{fire}dt$$

The occurrence rate $\nu_{fire} = \iint \lambda(x,y)\, dxdy$, where $\lambda(x,y)$ corresponds to the probability of fire ignition per year per m^2 for a given occupancy type; integration is over the floor area A_f of the fire compartment. A very straightforward assumption is to take $\lambda(x,y)$ as a constant:$\nu_{fire} = A_f\lambda$

After ignition there are various ways in which a fire can develop. The fire might extinguish itself after a certain period of time because no other combustible material is present. The fire may be detected very early and be extinguished by hand. An automatic sprinkler system may operate or the fire brigade may arrive in time to prevent flash over. Only in a minority of cases does a fire develop fully into a complete room or compartment fire; sometimes the fire may break through a barrier and start a fire in another compartment. From the structural point of view only these fully developed or post flashover fire may lead to failure. For fire compartments having a very large volume, e.g. industrial buildings and sports halls, a local fire of high intensity also may lead to structural damage.

The occurrence rate of flashover is given by:

$$\nu_{\text{flash over}} = P\{\text{flash over} \mid \text{ignition}\}\, \nu_{fire}$$

The probability of a flashover once a fire has taken place, can obviously be influenced by the presence of sprinklers and fire brigades. It should be noted that the effectiveness of a fire brigade depends on their equipment and size of staff but primarily on the time required to commence operations. Hence, the presence of the fire brigade near or within the building or plant and the presence of fire detection and alarm systems determine their efficiency. Basically, the effectiveness of fire brigades and sprinklers in keeping the fire small is strongly restricted to the pre-flashover stage. Once the post flashover stage has been reached these fire fighting systems are of limited value. Of course, fire brigades may still be of a help to limit the fire to one compartment.

Fire load intensity

The available combustible material can be considered as a random field, which in general might be nonhomogeneous as well as non-stationary. The intensity of the field q at some point in space and time is defined as:

$$q(x,y,t) = \lim_{\Delta A \to 0} \frac{\Sigma \mu_i \Delta m_i H_i}{\Delta A}$$

x, y	=	point in the horizontal plane
μ_i	=	derating factor between 0 and 1, describing the degree of combustion
Δm_i	=	combustible mass present at ΔA for material i
H_i	=	specific combustible energy for material i

ΔA = considered area (= $\Delta x \Delta y$)

The ventilation conditions are governed by the ventilation parameter $A\sqrt{h}$, where A is the total amount of door and windows area and h is the (weighed) average height of the ventilation openings:

$$h = \frac{\sum (A_i h_i}{\sum A_i}$$

A_i = area [m^2] of each opening i in the fire compartment with $\sum A_i = A$
h_i = height [m] of each opening i in the fire compartment

The ventilation parameter is random, as generally the area of opened windows, doors, etc. is not known. During fire, people may open or close doors, while in addition door and windows may collapse. This would indicate a modelling as a temperature dependent random process. However, modelling ventilation as a random variable with a large coefficient of variation seems to be adequate for the time being.

Temperature-time relationship

For known characteristics of both the combustible material and the compartment, the post flash over period of the temperature time curve can be calculated from energy and mass balance equations (see [9, 10, 17]). The fire load density determines the total duration of the fire and so the final temperature while the shape of the time-temperature-relation is determined by the opening factor f:

$$f = \frac{A\sqrt{h}}{A_t}$$

A_t = total internal surface area of the fire compartment, i.e. the area of the walls, floor and ceiling, including the openings [m^2]
A = total area of the vertical openings in the fire compartment, i.e. the windows, ventilation openings and other vertical openings [m^2]
h = mean value of the height of these openings, weighted with respect to the opening area [m].

In many engineering applications, use is made of standard temperature-time-relationship according to ISO 824:

$$\theta = \theta_0 + \theta_A \log_{10} \{\alpha t + 1\} \text{ for } 0 < t < t_{eq}$$

θ = temperature in compartment
θ_0 = temperature at the start of the fire (basic random variable)
θ_A = parameter (basic random variable)
α = parameter (basic random variable)
t = time
t_{eq} = equivalent time of fire duration (basic random variable)

B.3 Vehicle Impact Modelling

Frequency of collapse

Consider a structural element in the vicinity of a road or track. Impact will occur if some vehicle, travelling over the track, leaves its intended course at some critical place with sufficient speed (see Figure B.2). Which speed is sufficient depends on the distance from the structural element to the road, the angle of the collision course, the initial velocity and the topographical properties of the terrain between road and structure. In some cases there may be obstacles or even differences in height.

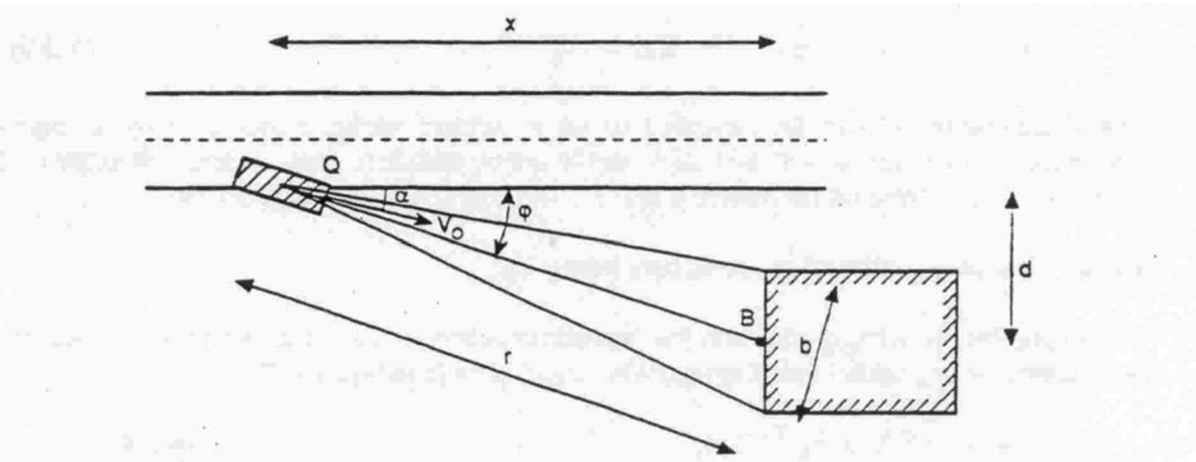

Figure B.2: A vehicle leaves the intended course at point Q with velocity v_0 and angle φ. A structural element at distance r is hit with velocity v_r.

The event that the intended course if left is modelled as an event in a Poison process. In most countries statistics are available for various road types, mainly for highways. To give some indication: according to [4.4] the probability of leaving a highway is about 10^{-7} per vehicle per km. Higher and lower values will occur in practice, depending on the local circumstances.

The main parameters describing the kinematics of a vehicle at the point of departure are the velocity v_0 and the angle φ. There are no indications that v_0 and φ are dependent for straight sections of the road. The direction angle φ varies from 0 to 30 or 40^0.

The velocity of a vehicle on a road depends on the type of road, the mass of the car, the weather conditions, the local situation and the traffic intensity at the time. Statistics are available in all countries. The distribution of the velocity conditional upon the event of leaving the track, however, is not known. As long as no specific information is available, one might assume the conditional and unconditional velocity distributions to be the same.

Confining ourselves to the case of a level track, the vehicle will as a rule slow down after the point of leaving the track, due to roughness of the terrain, obstacles or the driver's action. It is assumed, that the car maintains its direction. Further, assuming a constant deceleration or friction, the speed and the distance can be calculated as a function of t:

$$v(t) = v_0 - at$$

$$r(t) = v_0 - \tfrac{1}{2} a t^2$$

Both formulas hold as long as $v(t) > 0$. Eliminating t leads to v as a function of r:

$$v(r) = \sqrt{(v_0^2 - 2\,ar)} \qquad \text{for } 2\,ar < v_0^2$$

The deceleration 'a' can be assumed to be a random variable modelled by a lognormal distribution with mean 4 m/s^2 and 30% coefficient of variation. This means that in 90% of the cases the deceleration is between 2.0 and 7.0 m/s^2, which seems reasonable.

Combining the 'leaving model' and the 'speed reduction model', it is possible to calculate the (approximate) probability that a structural element is hit (see figure x):

$$P_c(T) = n\,T\,\lambda\,\Delta x\,P(v^2 > 2ar)$$

n = number of vehicles per time unit
T = period of time under consideration
λ = probability of a vehicle leaving the road per unit length of track
Δx = part of the road from where collisions may be expected
v = velocity of the vehicle when leaving the track
a = deceleration
r = the distance from "leaving point" to "impact point"

For r we may substitute:

$$r = d/\sin\alpha$$

d = distance from the structural element to the road
α = angle between collision course and track direction

In (4.3.4) nT is the total number of vehicles passing the structure during some period of time T; $\lambda\,\Delta x$ is the probability that a passing vehicle leaves the road at the interval dx. Note that the distance Δx also depends on α, where α is a random variable. So, in fact, equation (4.3.4) should be considered as being conditional upon α, and an additional integration over α is needed. We will, however, simplify the procedure and calculate Δx on the basis of the mean value of α:

$$\Delta x = b / \sin \Delta(\alpha)$$

The value of b depends on the structural dimensions. However, for small objects such as columns a minimum value of b follows from the width of the vehicle. Possible data is given in Table x.

Table x: Data for probabilistic collision force calculation

variable	Designation	type	mean	stand dev
N	number of lorries/day	deterministic	5000	-
T	reference time	deterministic	100 years	-
λ	accident rate	deterministic	10^{-10} m^{-1}	-
B	width of a vehicle	deterministic	2.50 m	-
α	angle of collision course	rayleigh	10 □	10□
V	vehicle velocity	lognormal	80 km/hr	10 km/hr

a	Deceleration	lognormal	4 m^2/s	1.3 m/s^2
M	vehicle mass	normal	20 ton	12 ton
K	vehicle stiffness	deterministic	300 kN/m	-

Mechanical impact model

Even then, impact is still an interaction phenomenon between the object and the structure. To find the forces at the interface one should consider object and structure as one integrated system. Approximations, of course, are possible, for instance by assuming that the structure is rigid and immovable and the colliding object can be modelled as a quasi elastic single degree of freedom system (see Figure 4.1). In that case the maximum resulting interaction force equals:

$$F = v_r \sqrt{(km)}$$

v_r = the object velocity at impact
k = equivalent stiffness of the object
m = mass of colliding object

This result can be found by equating the initial kinetic energy ($mv_r^2/2$) and the potential energy at maximum compression ($F^2/2k$).

Theoretical Design values for impact forces

We shall calculate the collision force probabilities and derive from there a design value for a bridge column near a highway track. We may write:

$$P(F>F_d) = n\ T\ \lambda\ \Delta x\ P\{\ 1.4\ \varphi\ (\ m\ k\ (v^2 - 2ar)) > F_d\ \}$$

It follows roughly that a life time probability of 0.001, corresponding to the Eurocode standards α=0.7 and β=3.8, leads to a force close to 6000 kN. If we accept the reduced exceedance probability of 10^{-4} per year as according to ISO [4.8], one finds about 4000 kN.

B.4 Ship Collision Modelling

Frequency of collision

The probability of a ship colliding with a particular object in the water (offshore platform, bridge deck, bridge piers, sluice) depends on the intended course of the ship relative to the object and the possibilities of navigation or mechanical errors. In order to find the total probability of an object being hit, the total number of ships should be taken into account. Finally, the probability of having some degree of structural damage also depends on the mass, the velocity at impact, the place and direction of the impact and the geometrical and mechanical properties of ship and structure.

When discussing ship collisions, it is essential to make a distinction between rivers and canals on the one side and open water areas like lakes and seas on the other. On rivers and canals the ship traffic patterns can be compared to road traffic. On open water, shipping routes have no strict definitions, although there is a tendency for ships to follow more or less similar routes when having the same destination.

Typical possible models for the ship distribution within a traffic lane is presented in figure x. In general it will be possible to model the position of a ship in a lane as a part with some probability density function. Details will of course depend on the local circumstances. It should be noted that sometimes the object under consideration might be the destination of the ship, as for instance a supply vessel for an offshore structure.

Navigation errors are especially important for collisions at sea. Initial navigation errors may result from inadequate charts, instrumentation errors and human errors. The probabilistic description of these errors depends on the type of ship and the equipment on board, the number of the crew and the navigation systems in the sea area under consideration. Given a ship on collision course, the actual occurrence of a collision depends on the visibility (day or night, weather conditions, failing of object illumination, and so on) and on possible radar and warning systems on the structure itself.

Mechanical failures may result from the machinery, rudder systems or fire, very often in connection with bad weather conditions. The course of the ship after the mechanical failure is governed by its initial position and velocity, the state of the (blocked) rudder angle, the current and wind forces, and the possibility of controlling the ship by anchors or tugs. These parameters together with the mass and dimensions of the ship should be considered as random. Given these data, it is possible to set up a calculation model from which the course of the ship can be estimated and the probability of a collision can be found.

For further discussion a co-ordinate system (x,y) is introduced as indicated in Figure B.3. The x co-ordinate follows the centre line of the traffic lane, while the y co-ordinate represents the (horizontal) distance of the object to the centre. The structure that potentially could be hit is located at the point with co-ordinates x=0 and y=d.

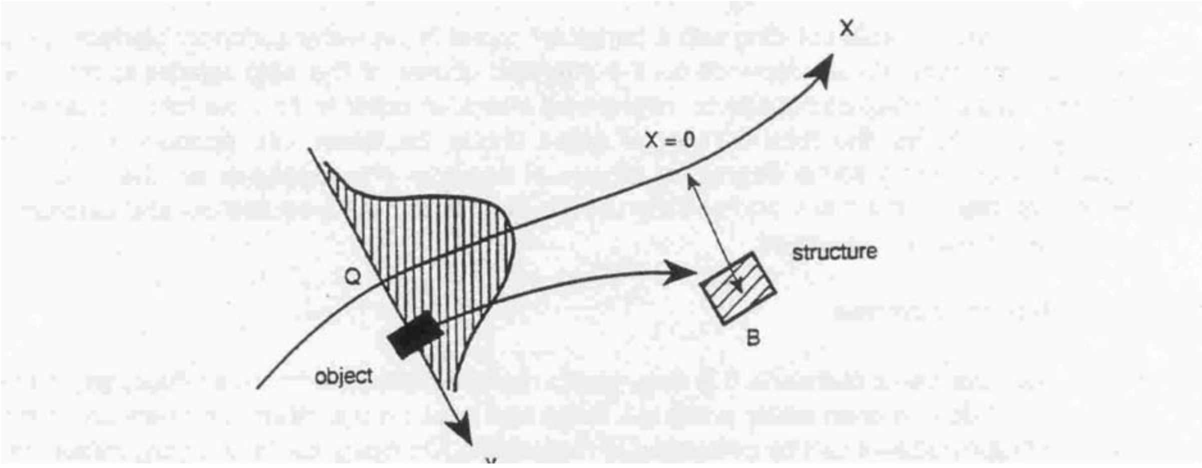

Figure B.3: Ingredients for a probabilistic collision model

The occurrence of a mechanical or navigation error, leading to a possible collision with a structural object, can be modelled as an (inhomogeneous) Poison process. Given this Poison failure process with intensity $\lambda(x)$, the probability that the structure is hit at least once in a period T can be expressed as:

$$P_c(T) = nT(1 - P_a) \int\int \lambda(x)\, P_c(x,y)\, f_s(y)\, dx\, dy \qquad (4.5.1)$$

where:

T	=	period of time under consideration
n	=	number of ships per time unit (traffic intensity)
$\lambda(x)$	=	probability of a failure per unit travelling distance
$P_c(x,y)$	=	conditional probability of collision, given initial position (x,y)
$f_s(y)$	=	distribution of initial ship position in y direction
P_a	=	the probability that a collision is avoided by human intervention.

For the evaluation in practical cases, it may be necessary to evaluate P_c for various individual object types and traffic lanes, and add the results in a proper way at the end of the analysis.

To give some indication for λ, in the Nieuwe Waterweg near Rotterdam in the Netherlands, 28 ships were observed to hit the river bank in a period of 8 years and over a distance of 10 km. Per year 80 000 ships pass this point, leading to λ = 28/(10*8*80000) = 10^{-6} per ship per km.

Mechanical Models

For practical applications, especially in offshore industry, some rules have been developed to calculate the part of the total energy that is transferred into the structure. Some of these rules are based on empirical models, others on a static approximation, starting from so called load indentation curves (F-u diagrams) for both the object and the structure (see figure 4.5.2). According to this model the interaction force during collapse is assumed to raise form zero up to the value where the sum of the energy absorption of both ship and structure equal the available kinetic energy at the beginning of the impact.

The forces in Table 4.7.1 of Eurocode 1, Part 2.7 have been calculated on the basis of (4.2.2). In order to get an idea of the accuracy of the mechanical model a comparison can be made with for instance the forces proposed in (see [4.4.3]) a number of bridge projects (figure 4.5.3) and the values estimates on the basis of real collisions (figure 4.5.4). In both cases the simple model gives reasonable results. Note that the plotted points are based on (4.2.1): that is the dynamic amplification is not included.

Failure frequency

If data about types of ships, traffic intensities, error probability rates and sailing velocities are known, a design force could be found from:

$$P(F > R) = nT(1 - p_a) \int\int \lambda(x)\, P[v(x,y)\sqrt{(km)} > R]\, f_s(y)\, dx\, dy$$

v(x,y)	=	impact velocity of ship, given error at point (x,y)
k	=	stiffness of the ship
m	=	mass of the ship
R	=	resistance

The values in Table 4.3 of Part 1.7 have not been derived on the basis of an explicit target reliability. In fact, the values have been chosen in accordance with ISO DIS 10252. For a particular design it should be estimated which size of ships on the average might be expected, and on the basis of those estimates, design values for the impact forces can be found.

B.5 Explosion Modelling

Nature of the action

Gas explosions account for by far the majority of accidental explosions in buildings. Gas is widely used and, excluding vehicular impact, the incidents of occurrence of gas explosions in buildings is an order of magnitude higher than other accidental loads causing medium or severe damage.

Many gas explosions within buildings occur from leakage into the building from external mains. Reference 5.1 concluded : “There should be no relaxation ... for buildings without a piped gas supply, since a risk would usually remain of gas leaking into the building from outside”. It would be impractical in most circumstances to ensure gas will not be a hazard to any particular building. Therefore it seems reasonable to take a gas explosion as the normative design accidental action, excluding impact.

In this context an explosion is defined as rapid chemical reaction of dust or gas in air. It results in high temperatures and high overpressure. Explosion pressures propagate as pressure waves.

The following are necessary for an explosion to occur (reference 5.6):

- fuel, in the proper concentration :
- an oxidant, in sufficient quantity to support the combustion :
- an ignition source strong enough to initiate combustion

The fuel involved in an explosion may be a combustible gas (or vapour), a mist of combustible liquid, a combustible dust, or some combination of these. The most common combination of two fuels is that of a combustible gas and a combustible dust, called a “hybrid mixture”.

Gaseous fuels have a lower flammability limit (LFL) and an upper flammability limit (UFL). Between these limits, ignition is possible and combustion will take place. Combustible dusts also have a lower flammability limit, often referred to as the minimum explosive concentration. For many dusts, this concentration is about 20g/m³. The oxidant in an explosion is normally the oxygen in air.

Moisture absorbed on the surface of dust particles will usually raise the ignition temperature of the dust because of the energy absorbed in vaporizing the moisture. However, the moisture in the air (humidity) surrounding a dust particle has no significant effect on an explosion once ignition has occurred.

The pressure generated by an internal explosion depends primarily on the type of gas or dust, the percentage of gas or dust in the air and the uniformity of gas or dust air mixture, the size and shape of the enclosure in which the explosion occurs, and the amount of venting of pressure release that may be available.

In completely closed rooms with infinitely strong walls gas explosions may lead to pressures up to 1500 kN/m^2, dust explosions up to 1000 kN/m^2, depending on type of gas or dust. In practice, pressures generated are much lower due to imperfect mixing and the venting which occurs due to failure of doors, windows and other openings.

Windows and other relatively lightweight elements of the enclosure provide such vents that are themselves expelled by the initial pressure rise. This has the effect of greatly reducing the maximum pressure generated. Windows respond in a brittle manner because the thinness of the glass makes very little deformation possible before there is complete disintegration. For this reason, coupled with their relatively light weights and low static strengths, they make good explosion vents. But venting is also afforded by failure of non-structural, relatively weak wall panels.

It must be borne in mind that the response in real structures is highly complex: the geometry of the space, obstacles to free expansion producing turbulence, etc. The value of theoretic analyses of structural responses to such [explosive] loadings is limited by the impossibility, at least at present, of determining with any degree of accuracy, even after the event, what they have been at all significant points in any particular case. And it is part of the nature of accidental loading that prediction before the event will always remain impossible. The response of complete structures are, moreover, highly complex.

Model for the unconfined explosion

An explosion can be defined as "a rapid combustion with a marked and measurable pressure increase" [VDI guideline]. A hemispherical cloud with a volume V_0 consisting of a homogeneous combustible gas/air mixture will, after ignition in the centre, expand to a hemisphere with volume V_1. The characteristic properties can be calculated as follows:

1) The peak overpressure

In case of detonation:

$$P_{peak} = P_0 \cdot 0.518 \cdot (\frac{r}{L_0})^{-1.7} \qquad \text{for} \qquad 0.29 \leq \frac{r}{L_0} \leq 1.088$$

$$P_{peak} = P_0 \cdot 0.2177 \cdot (\frac{r}{L_0})^{-1} + 0.1841 \cdot (\frac{r}{L_0})^{-2} + 0.1194 \cdot (\frac{r}{L_0})^{-3} \qquad \text{for} \qquad \frac{r}{L_0} \geq 1.088 \wedge$$

In the case of deflagration:

$$P_{peak} = P_0 \cdot \phi \cdot (\frac{r}{L_0})^{-1}$$

P_{peak}: peak overpressure of shock wave [Pa]
P_0: atmospheric pressure [Pa]
r: distance to the centre of the explosion [m]
L_0: characteristic explosion length [m], which is given by:

$$L_0 = \left(\frac{V_0 \cdot E_c}{p_0} \right)^{\frac{1}{3}}$$

E_c: combustion energy of mixture per unit volume

2) The positive phase duration t_P

$$t_P = c_0 \cdot \frac{t_t}{L_0}$$

c_0: local sound velocity [m/s]

3) The impulse:

$$i_S = \int_{t_P} (p_S(t) - p_0)dt$$

Using the schematic simplified pressure-time curve, the impulse, for both shock wave or
pressure wave, is then equal to:

$$i_S = \frac{1}{2} \cdot P_S \cdot t_P$$

The time course of the static overpressure can be approximated during the phase of overpressure by the following equation, known as the Friedlander-Approximation:

$$p(t) = p_e(1 - \frac{t}{t_p})\exp^{-\alpha t/tp}$$

Loads due to internal explosions

Numerous empirically methods predicting explosion overpressures based on explosion venting are published in the literature. The empirical relationships produced by Cubbage and Simmonds, Cubbage and Marshall, Rasbash, and Rasbash et al, are commonly used. They were determined for a limited range of variables such as volume, burning velocity, mass of fuel (air mixture), and vent areas. The empirical correlations Cubbage and Simmonds are based on the concept of a vent coefficient K.

$$K = \frac{A_s}{A_v}$$

where A_s means the area of side of enclosure, and A_v the area of the vent opening. The commonly used equations and its ranges of application are listed below:

(1) Cubbage and Simmonds

Probably the most widely used of the formulae presented. The Cubbage and Simmonds' equations contain terms expressing the effect of characteristics of both the gas-air mixture and the enclosure in which the explosion occurs. They may be used for any type of gas-air-mixtures since the influence of combustion characteristics of different gases on the pressure generated is allowed for by the burning velocity. S_0 means the burning velocity. This is the velocity with which the flame front moves relative to the unburned mixture immediately ahead of it.

$$P_1 = S_0 \cdot \frac{(4.3 \cdot K \cdot W + 28)}{V^{\frac{1}{3}}}$$

$$P_2 = 58 \cdot S_0 \cdot K$$

P_1: pressure of the vent removal phase [mbar]
P_2: pressure of the venting phase [mbar]
S_0: burning velocity [m/s] (natural gas 0.45 m/s)
K: vent coefficient, dimensionless
W: weight per unit area of the vent cladding [kg/m^2]
V: volume of room [m^3]

Range of application:

- Max and minimum dimensions of room have a ratio less then 3:1: $L_{max} : L_{min} \leq 3 : 1$
- The vent area coefficient; K, is less then 5: $K \leq 5$
- The weight per unit area of the vent cladding W must not exceed 24 kg/m^2

(2) Rasbash et al

The equation of Rasbash et al. can be expected to predict the maximum overpressure generated in a given situation, irrespective of whether this relates to P_1 or P_2.

$$P_m = 1.5 P_v + S_0 \{[\frac{(4.3K \cdot W + 28)}{V^{\frac{1}{3}}}] + 77.7K\}$$

P_m: maximum overpressure [mbar]
P_v: uniformly distributed static pressure at which venting components will response [mbar]
S_0: burning velocity
K: vent coefficient
W: weight per unit area of the vent cladding
V: volume of room

Range of application:

- Maximum and minimum dimensions of room have a ratio less than 3:1: $L_{max} : L_{min} \leq 3 : 1$
- The vent area coefficient; K, is between 1 and 5: $1 \leq K \leq 5$
- The weight per unit area of the vent cladding does not exceed 24 kg/m^2: $W \leq 24$ kg/m^2
- The response pressure of the vent cladding, overpressure required to open it, does not exceed 70 mbar: $P_v \leq 70$ mbar

(3) NFPA 68, Guide for Venting of Deflagrations, 2002 Edition for low strength buildings

The Guide for Venting of Deflagrations of the National Fire Protection Association proposes for low strength buildings the following equation to determine the maximum pressure developed in a vented enclosure during a vented deflagration of a gas- or vapour-air-mixture:

$P_{red} = (C^2 \times A_s^2) / A_v^2$

P_{red}: maximum pressure developed in a vented enclosure during a vented deflagration in bar
A_v: vent area in m^2
A_S: internal surface area of enclosure in m^2
C: venting equation constant in $(bar)^{1/2}$

The maximum pressure P_{red} can not be larger than the enclosure strength P_{es}. P_{red} should not be greater than 0.1 bar.

From the following table the values for the venting equation constant can be seen:

gas- or vapour-air-mixture	venting equation constant C in $(bar)^{1/2}$
anhydrous ammonia	0.013
Methane	0.037
gases with fundamental burning velocity < 1.3 that of propane	0.045
Hydrogen	not available

There are no dimensional constraints on the shape of the room besides that the shape is not extremely one dimensional. As a check the following equation should be used:

$l_3 < 8 \times (A / U)$

where l_3 : is the longest dimension of the enclosure, A the cross-sectional area in m^2 normal to the longest dimension and U the perimeter of cross section in m. The vent closure should weight not more than 12.2 kg/m^2.

(4) NFPA 68, Guide for Venting of Deflagrations, 2002 Edition for high strength buildings

The required vent area for rectangular enclosure is determined according to the following equation:

$A = [(0.127 * \log_{10} K_G - 0.0567) * p_{Bem.}^{-0.582} + 0.175 * p_{Bem.}^{-0.572} (p_{stat.} - 0.1)] * V^{0.667}$

A vent area $[m^2]$
p_{max} maximum explosion overpressure of the dust
K_G deflagration index of gas $[bar\ m\ s^{-1}]$
p_{Bem} design strength of the structure [bar]
p_{stat}: static activation overpressure with size of existing vent areas [bar]
V: volume of enclosure $[m^3]$

This equation is valid for the following conditions:

- $V \leq 1'000\ m^3$

- $L/D \leq 2$, where L greatest dimension of enclosure, $D = 2 * (A / \pi)^{0.5}$, A is cross-sectional area normal to longitudinal axis of the space
- $p_{stat} \leq 0.5$ bar, $p_{stat} < p_{Bem.}$
- $0.05 \leq p_{Bem.} \leq 2$ bar
- $K_G \leq 550$ bar m s^{-1}

For elongated rooms with $L/D \geq 2$ the following increase for the vent area has to be considered:

- $\Delta A_H = A * K_G (L/D - 2)^2 / 750$
- ΔA_H increase for vent area [m^2]

(5) Gas and fuel / air explosions in road and rail tunnels

According to Annex B of ENV 1991-2-7 for the case of detonation, the following pressure time function should be taken into account:

$$p(x,t) = p_0 \exp\{-(t - \frac{|x|}{c_1})/t_0\} \qquad \text{for} \frac{|x|}{c_1} \leq t \leq \frac{|x|}{c_2} - \frac{|x|}{c_1}$$

$$p(x,t) = p_0 \exp\{-(\frac{|x|}{c_2} - 2\frac{|x|}{c_1})/t_0\} \qquad \text{for} \frac{|x|}{c_2} - \frac{|x|}{c_1} \leq t \leq \frac{|x|}{c_2}$$

$$p(x,t) = 0$$

for all other conditions

p_0: peak pressure (=2000 kN/m2)
c_1: propagation velocity of the shock wave (~1800 m/s)
c_2: acoustic propagation velocity in hot gasses (~800 m/s)
t_0: time constant (=0.01s)
¦x¦: distance to the heart of the explosion
t: time [s]

In case of deflagration the following pressure time characteristic should be taken into account:

$$p(t) = 4 p_0 (\frac{t}{t_0})(1 - \frac{t}{t_0}) \qquad \text{for } 0 \leq t \leq t_0$$

(6) Dust explosions in rooms and silos

Dust explosions are treated in Annex B, Section B.3, based on the ISO 6184-a, published in the VDI Richtlinie 3673. These references are based on work of Bartknecht. The three basic conditions for a dust explosions are:

- combustible dust
- dispersive air
- ignition source

Under conditions of complete confinement most common inflammable dusts mixed with air, at atmospheric pressure, may produce a maximum explosion pressure in excess of 14 bar. (Bussenius, 1996). Dust explosions occurring at elevated initial temperatures tend to show lower maximum explosion pressures than those occurring at ambient temperatures. Bartknecht, 1993 (pg. 205ff) gives a linear relationship between the reciprocal temperature and the P_{max}:

$$\frac{P_{max}}{P_{initial}} \sim \frac{T_{max}}{T_{initial}}$$

The cubic law is an important tool in estimating the explosion severity of dusts in vessels. Dusts are classified according to their K_{st}-value (K_{st} is the VDI designation; the ISO designation for the same quantity is K_{max}).The cubic law is given by (see Bartknecht, 1993, pg. 175):

$$K_{st} = V^{\frac{1}{3}} (\frac{dP}{dt})_{max}$$

where V is the volume of the vessel [m^3] and $(\frac{dp}{dt})$ max is the maximum value of the rate of pressure increase during explosion

For the standardisation of the dust explosion classes Bartknecht developed an explosion vessel (Bartknecht, 1993, pg. 169). In his 1 m^3 explosion vessel. Bartknecht (1971) used a dust dispersion system by which the dust was forced at high velocity by high pressure air through a number of 4-6 mm diameter holes in a U-shaped tube of 19 mm in diameter. Bartknecht's 1 m^3 vessel and dust dispersion system has later been adopted as an ISO standard (International Organisation for Standardisation (1985)).
Definition of dust explosion classes according to Bartknecht, 1993, pg. 177 (1m^3 apparatus, 10 kJ ignition source)

Dust explosion class	K_{st} [bar m/s]	Characteristics
St 0	0	Non-explosible
St 1	$0 < K_{st} \leq 200$	Weakly to moderately explosible
St 2	$200 < K_{st} \leq 300$	Strongly explosible
St 3	$K_{st} > 300$	Very strongly explosible

Vent sizing methods for dust explosion

Numerous methods for vent sizing have been proposed. The process of choosing the most effective method of vent sizing can be complex, depending on several factors like K_{st}, P_{max}, vessel volume and length to diameter ratio. The vent ratio in general is defined as:

Vent ratio = Area of vent / Volume of the vessel

A method for scaling vent areas for rooms and silos is the Radandt Scaling Law. Bartknecht, 1987, Appendix 8.1 indicated the Equation, derived by Radandt:

$$A = \left[a + \frac{b}{P_{red}} \right] \cdot V^c$$

A: vent area [m^2]
P_{red}: maximum explosion pressure in the vented vessel [bar]

V: volume of vessel [m^3]
a, b, c: empirical constants depending on the dust explosion class (see Tables 1 and 2)
P_{stat}: static relief pressure and size of existing vent areas [bar]
P_{red}: reduced maximal explosion pressure [bar]

Table 1: Factors for the calculation of the vent area A in relation of the cubical volume V.
P_{stat} is assumed to equal 0.1bar, P_{max} = 9 bar and P_{red} must not exceed 2 bar.

Dust explosion class	$p_{red, max}$ (bar)	a	b	c
St 1	< 0.5	0.04	0.021	0.741
	≥ 0.5	0.04	0.021	0.766
St 2	< 0.5	0.048	0.039	0.686
	≥ 0.5	0.048	0.039	0.722

Table 2: Factors for the calculation of the vent area A in relation of the silo volume V.
P_{stat} is assumed to be equal 0.1bar, P_{max} = 9bar.

Dust explosion class	$p_{red, max}$ (bar)	a	b	c
St 1	≤ 2	0.011	0.069	0.776
St 2	≤ 2	0.012	0.114	0.720

Cubic and elongated vessels, silos and bunkers according VDI, 1995

The sizing of the vent area of cubic vessel is based on experimental investigations that were carried out under conditions that represent the actual situation. The equations should cover unfavourable conditions. For a inhomogeneous dust distribution the size of the vent is smaller than for homogeneous distribution. Therefore only the homogeneous situation is considered:

$$A = [\ 3.264 * 10^{-5} * p_{max} * K_{st} * p_{red.max}^{-0.569} + 0.27 * (p_{stat} - 0.1) * p_{red.max}^{-0.5}\ ] * V^{0.753}$$

A vent area [m^2]
p_{max} maximum explosion overpressure of the dust
K_{st} dust specific characteristic [bar m s^{-1}]
$p_{red.max}$ anticipated maximum reduced explosion over pressure in the vented vessel [bar]
p_{stat}: static activation overpressure with size of existing vent areas [bar]
V: volume of vessel, silo, bunker [m^3]

This equation is valid for the following conditions:

- o $m^3 \leq V \leq 10'000\ m^3$
- H/D ≤ 2, where H high and D diameter of elongated vessel
- o bar ≤ p_{stat} ≤ 1 bar
- o bar ≤ $p_{red.max}$ ≤ 2 bar
- 5 bar ≤ p_{max} ≤ 10 bar for 10 bar m s^{-1} ≤ K_{st} ≤ 300 bar m s^{-1} 5 bar ≤ p_{max} ≤ 12 bar for 300 bar m s^{-1} ≤ K_{st} ≤ 800 bar m s^{-1}

Rectangular enclosure according VDI, 1995

The required vent area for rectangular enclosure is determined according to the following equation:

$$A = [\ 3.264 * 10^{-5} * p_{max} * K_{st} * p_{Bem}^{-0.569} + 0.27 * (p_{stat} - 0.1) * p_{Bem.}^{-0.5}\ ] * V^{0.753}$$

A vent area [m^2]
p_{max} maximum explosion overpressure of the dust
K_{st} dust specific characteristic [bar m s^{-1}]
p_{Bem} design strength of the structure [bar]
p_{stat}: static activation overpressure with size of existing vent areas [bar]
V: volume of vessel, silo, bunker [m^3]

This equation is valid for the following conditions:

- $m^3 \leq V \leq 10'000\ m^3$
- $L_3/D_E \leq 2$, where L_3 greatest dimension of enclosure, $D_E = 2 * (L_1 * L_2/\ \pi\ )^{0.5}$, L_1 , L_2 other dimensions of enclosure
- bar $\leq p_{stat} \leq$ 1 bar
- 0.02 bar $\leq p_{Bem.} \leq$ 0.1 bar
- 5 bar $\leq p_{max} \leq$ 10 bar for 10 bar m $s^{-1} \leq K_{st} \leq$ 300 bar m s^{-1}
- 5 bar $\leq p_{max} \leq$ 12 bar for 300 bar m $s^{-1} \leq K_{st} \leq$ 800 bar m s^{-1}

For elongated rooms with $L_3/D_E \geq 2$ the following increase for the vent area has to be considered:

- $\Delta A_H = A * (- 4.305 * \log p_{Bem} + 0.758) * \log L_3/D_E$

ΔA_H increase for vent area [m^2]

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Appendix C: Robustness in Other Disciplines

Dimitris Diamantidis

Robustness is an issue not confined to these civil engineering structures but considered by others such as electrical engineers and mechanical engineers in their designs. It is considered also by economists, medical professionals, etc., but with their own definitions and procedures. These non-civil and non-structural engineering aspects are beyond the scope of this document.

Considering that this document deals mainly with the robustness of buildings, some information on other civil engineering structures is given below.

Offshore structures

Progressive collapse analysis has been considered in the design of offshore structures for nearly thirty years. The accidental actions (impact scenarios, fire and explosion, flooding, etc.) are usually determined through risk analyses and by accounting for the relevant factors of influence. The magnitude of the accidental event can be controlled by using passive or active measures. For passive measures there are recommendations given, for example fenders can be installed to reduce the damage due to impact.

In principle an offshore structure can be designed to resist the accidental action. It must be decided whether a local damage may be avoided or is tolerable. For this case it is crucial to provide alternative load paths to ensure that a small damage does not lead to disproportionate consequences through a progressive collapse. This design criterion leads to a robustness of the structure and ensures that loss of stability and capsizing can be avoided within an acceptable probability.

For verification of these accidental events the NORSOK (2004) standard can be used. It provides an Accidental Limit State (ALS) for the consideration of accidental loads. The ALS applies to all relevant failure modes. The structural integrity criterion in NORSOK is a two-step procedure. The first step is to analyze the resistance of the structure against accidental loads, i.e. the structure must be checked whether it can maintain its intended load carrying function. The second step is to check the structure for the damaged condition. Hereby is important that the damaged condition is analyzed for defined (reduced) load combinations (e.g. for steel structures load and resistance factor is set to 1.0). A summary of design of offshore structures against accidental actions is presented by Moan (2007). The consideration of accidental loads to obtain a robust offshore structure is essential. The accident rates for platforms demonstrate the need for more robustness of the structures. The ALS criteria of NORSOK is a first step to implement global failure modes and progressive failure in structural design.

Bridges

The requirement to avoid progressive collapse in case of local failure is an important design criterion for multi-span bridges. It can have strong impact on both conceptual design, including choice of structural system, and detailed design. The triggering events of collapse are manifold. This extraordinary event could either be a ship impact, strong ice formations collision on a pier or fire and explosion (CIB, 1992).

In view of the accidental nature of imaginable and unimaginable circumstances, in relation to structural robustness, it would be unrealistic to design against progressive collapse just by preventing local failure at any expense. In case of bridges it is more reasonable to allow local failure (e.g. loss of pier) and investigate the behaviour of the damaged structure. It must be demonstrated that a progressive collapse due to the local failure can be avoided. It can be seen that not only the redundancy (that ensures alternative load paths) is important for the robustness as shown in the design of the Confederation Bridge (Starossek, 2006). The increase of a system's degree of static indeterminacy may be used to avoid progressive collapse caused by accidental events, thus increase the robustness of the entire structure.

Current design codes do not strictly require the prevention of progressive collapse of bridges. Recent disasters and theoretical considerations on the basis of risk theory indicate that codes should be improved to more clearly address this problem.

A disproportional nature of consequences may arise if also affects the transport system, results in fatalities or other effects on what is underneath (a road or railway, shipping channel).

Tunnels

Robustness of tunnel structures is implemented mainly through fire resistant materials. Accidental loads include internal and external hazard scenarios. Accidental loads are usually derived based on a site specific study. Outcome of such studies are protective measures such as protective layers. The tunnel with the protective layer shall namely be able to resist, without puncturing of an exterior waterproofing membrane or spalling of interior concrete, the accidental loads specified for the project. Only one of the accidental loads is thereby assumed to act at any time on any session of the tunnel. However, a fire that results due to an impact can be a likely scenario and should be considered in the design.

Since the consequences of accidents in tunnels can be extreme, risk studies are performed in order to verify the acceptability of the risk and to select appropriate (cost-effective) safety measures (see for example Diamantidis, 2011).

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