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On the Updating of Structural Reliability Based on Inspection and Monitoring Data

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Abstract

Assessment of existing structures is getting more and more important due to the increasing number of structures and infrastructures close to the end of their service life in conjunction with severe economic constraints. The aim of this thesis is to investigate on how Inspection and monitoring can be effectively used to this purpose.

To date, an obstacle to the spread of their use is represented by the existing fragmentation of guidelines and standards on structural monitoring and the lack of an international standard universally recognised. In spite of this, main characteristics and classifications of inspection and monitoring systems, together with maintenance topics, are carefully investigated. The idea to regularly inspect most structures and monitor the behaviour of critical parts or of the whole structure to get early warnings may lead to immediate interventions and cost minimisation assuring an acceptable reliability level.

A crucial step of the process is the use of newly obtained measurements together with prior information to evaluate the actual structural reliability. This can be done using probabilistic methods or updating partial safety factors on the base of probabilistic considerations.

The application of theoretical principles is illustrated by the case study of a stadium roof subjected to high snow load and of the change in use of an office building with an increase of loads. In the first example the results coming from the continuous monitoring of the snow depth on the roof are used to decide about the closure of the stadium whilst, in the second, the influence of destructive tests on specimens and proof load tests on the updating of the resistance of the considered steel beam is investigated.

Riassunto

Il crescente numero di strutture ed infrastrutture esistenti prossime alla fine della loro vita di servizio e gli attuali vincoli economici hanno reso sempre più importante la valutazione delle strutture esistenti. Lo scopo di questa tesi è di indagare su come ispezione e monitoraggio possono essere efficacemente utilizzati a questo fine.

L'attuale frammentazione normativa e la mancanza di un riferimento a livello internazionale, specialmente nell'ambito del monitoraggio strutturale, ha rappresentato fino ad adesso un ostacolo alla diffusione delle applicazioni. Nonostante questo è stato possibile definire le principali caratteristiche di ispezione e monitoraggio, riportandone le possibili classificazioni e contestualizzandoli all'interno del processo di manutenzione. L'idea di ispezionare regolarmente la maggior parte delle strutture e di monitorare il comportamento delle parti critiche o di tutta la struttura al fine di ottenere allarmi preventivi può condurre ad interventi immediati ed alla minimizzazione dei costi riuscendo ad assicurare un livello di affidabilità accettabile durante tutta la vita della struttura.

Un passo fondamentale del procedimento è rappresentato dall'utilizzo delle nuove informazioni, ottenute tramite misurazione, assieme alle informazioni note a priori per valutare la reale affidabilità strutturale. Questo può essere fatto ricorrendo all'utilizzo di metodi probabilistici o aggiornando i fattori parziali di sicurezza sulla base di osservazioni probabilistiche.

L'applicazione dei principi teorici è illustrata dal caso studio della copertura di uno stadio sensibile al carico neve e dal cambio d'uso di un edificio adibito ad uffici con un conseguente aumento dei carichi. Nel primo esempio i risultati ottenuti dal monitoraggio continuo della profondità della neve sulla copertura sono utilizzati per decidere sulla chiusura dello stadio mentre, nel secondo, è studiata l'influenza che prove distruttive su provini e prove di carico sulla struttura hanno sull'aggiornamento della resistenza della trave in acciaio considerata.

Table of Contents

Figures

Tables

Chapter 1
Introduction

1.1 General framework

The number of structures and infrastructures close to the end of their service life time has increased considerably in the last years due to the numerous constructions completed in the fifties and sixties. Buildings, bridges, tunnels, dams, etc. were constructed to fulfil the standards of that time which are often inadequate if compared with the modern ones. The performance demand is changed and more severe load must be currently considered. In addition also poor maintenance can be frequently observed, not just in old structures and infrastructures but in most recent ones as well. Since a large portion of these structures are still in service and they probably cannot ensure an adequate safety, the society, the owners and authorities are facing the maintaining the ageing structures and infrastructures. The replacement of these structure requires a major economic effort that cannot be supported by the society thus inspection, monitoring and maintenance strategies represent a certain choice.

Testing and inspection are not new concepts, they have been conducted for thousands of years in an effort to prolong structures' service life and ensure public safety. The relatively new idea is to regularly inspect most structures and infrastructures and monitor the behaviour of critical parts or of the whole structure to get early warnings that may lead to an immediate intervention or at a regular maintenance intervention depending on the severity of the problem. This aspect is lacking in the civil sector, especially if comparisons with different industrial sectors are conducted. To date one of the primary factors that have led to unsatisfactory condition of our structures and infrastructures is precisely the unsatisfactory inspection and monitoring, with problems becoming apparent only once structures are in such dire need of attention that the cost of repair often approaches that of replacements. However the latest developments and decreasing cost of sensors and information technologies have made monitoring systems more attractive in civil engineering applications. In particular, the evolution of sensing technologies and data acquisition system has been impressive and a great research effort has also been dedicated to data analysis and interpretation algorithms.

Inspection and monitoring data may be effectively used in the updating of the structural failure probability and in the updating of the probability distributions of basic variables. The updating of prior information by newly obtained measurements permits to reduce the uncertainties existing during the design and to refer to the actual as-built conditions.

Probabilistic methods may be used to combine prior information about a variable with test results and measurements. This way to proceed allows to maximise knowledge on the structure and to minimise interventions and costs assuring an acceptable reliability, particularly important in case of doubts concerning actual reliability or serviceability, repair, strengthening, and change in use.

The present thesis is focused on the updating of structural reliability based on data acquirable by testing. The following chapters deal with theoretical aspects and practical applications.

Chapter 2 gives an overview on the guidelines and standards currently available highlighting the lacking of a landmark code. The topics already widely discussed and the ones that need further studies are examined. A proposal on the possible contents of such a reference code is given.

Chapter 3 describes inspection and monitoring in the context of the assessment process of existing structures reporting main characteristics and classifications. The attention is focused on the components of a modern monitoring system: treatment of data (acquisition, communication, storage etc.), sensors' classification, measurable physical quantities, static tests, and dynamic tests. At the end of the chapter two interesting applications of monitoring extracted from literature are reported.

Chapter 4 identifies different kind of maintenance: corrective, preventive, and operational, and examines the current practice in the civil sector, comparing it with different industrial sectors (automotive, aerospace, and marine). Useful recommendations to buildings classification and to realize effective maintenance plans are provided.

Chapter 5 introduces general concepts about structural reliability, necessary to effectively use inspection and monitoring data. Target reliability levels, corresponding to a certain probability of failure, and calculation methods are reported. The probabilistic derivation of the partial factors is examined as well as level II and level III probabilistic methods.

Chapter 6 shows a practical application where a stadium roof subjected to high snow load is considered. A probabilistic analysis is conducted to evaluate the results coming from the monitoring of the snow depth on the roof and to decide about the closure of the stadium when the snow depth reach a limit value.

Chapter 7 reports a second practical application. The change in use of a building from offices not open to the public to offices open to the public, and the consequently increase of loads, is investigated. The influence of destructive tests on specimens and proof load tests on the updating of the resistance of the considered steel beam is the main focus.

Chapter 8 contains the conclusions.

Annex A briefly describes a spreadsheet realised in the context of the present thesis to easily calculate the partial factors starting from their probabilistic definition. Some screenshots are included.

1.2 Brief history of building codes

It is fascinating to go back in history in order to understand how man gradually conquered enough 'certainties' to accept rationally the risk of his uncertainties. The history of structural engineering dates back to at least 2700 BC when the step pyramid of Djoser was built in Egypt. Pyramids were the most common major structures built by ancient civilizations because it is a structural form which is inherently stable and can be almost infinitely scaled as opposed to most other structural forms, which cannot be linearly increased in size in proportion to increased loads. It is necessary to go to circa 1755 BC to find the earliest known written building code, included in the Code of Hammurabi. The code related to the construction of houses, and the mason's responsibility was strongly binding. For instance, article 229 reports: if a mason build a house for someone and does not construct it properly, and the house which he built fall in and kill its owner, then the builder shall be put to death. It is interesting to note that the insistence on safety was then based on the transfer onto the builder of a risk that related to his own security.

Figure 1.1: Code of Hammurabi (c. 1755 BC)

If the knowledge of geometry and static mechanics advanced rapidly in ancient times (Archimedes, Euclid, etc.), the mastery of the uncertain in the construction of cathedrals in the Middle Ages proceeded by trial and error and led to well-known failures. Leonardo da Vinci was one of the first to look for a relationship between load effect and resistance in the case of beams. A little later, in the 17^{th} century the foundations of modern structural engineering were laid by Galileo Galilei, Robert Hooke and Isaac Newton with the publication of three great scientific works. Further advances came in the 18^{th} century when Leonhard Euler developed the Euler-Bernoulli beam equation with Daniele Bernoulli.

In this ages the first building regulations were issued: the Rebuilding of London Act, after the Great Fire of London in 1666, regulated the rebuilding of the city, required housing to have some fire resistance and authorised the City of London Corporation to reopen and widen roads; the Law of the Indies were passed in the 1680s by the Spanish Crown to regulate the urban planning for colonies throughout Spain’s worldwide imperial possessions.

The first systematic national building standard was established with the London Building Act of 1844. Among the provisions, builders were required to give the district surveyor two days' notice before building, regulations regarding the thickness of walls, height of rooms, the materials used in repairs, the dividing of existing buildings and the placing and design of chimneys, fireplaces and drains were to be enforced and streets had to be built to minimum requirements.

It is necessary to wait the early decades of the 20th century in order that rules or codes are developed in the different countries to define minimum criteria of safety design. For the first years the design was based on a deterministic safety concept and the code was essentially a set of rules based on prevailing good practice. As time progressed, the developments in industrial practice, the numerous new ideas, the development of computers to solve the equations, promised a new dawn of structural engineering.

Starting from the 1930s, probabilistic concepts have been introduced in the codes and semi-probabilistic methods have begun to spread, coupled with Limit State Design (LSD), also known as load and resistance factor design. A structure designed by LSD is proportioned to sustain all actions likely to occur during its design life, and to remain fit for use, with an appropriate level of reliability for each limit state. Building codes based on LSD implicitly define the appropriate levels of reliability by their prescriptions. A remarkable year in Europe is 1992, when CEN published ENV Eurocodes, based on semi-probabilistic design method and that currently represent a very important guidance.

Nowadays many design codes have reached a high level of reliability in many sectors and represent important landmarks. On the other hand the last years showed a growing number of new and interesting topics that need more attention and more codification efforts. One of these is the Structural Health Monitoring, treated in this thesis in the context of the updating of structural reliability.

Chapter 2
Guidelines and standards

2.1 Review

Standardization of Structural Health Monitoring (SHM) in the civil sector is an important topic which needs to be developed to contrast the actual fragmentation and to increase applications and benefits derivable from it. In the following the standards currently available are firstly reported matched with a short description and secondly they are analysed and compared starting from the topic treated and continuing with the themes that need to be developed and possible reference points.

To date the available interesting international standards are (this list is not considered to be comprehensive):
EN 31010:2008 Risk management – Risk assessment techniques
EN 15331:2009 Criteria for design, management and control of maintenance services for buildings
ISO 13822:2009 Bases for design of structures – Assessment of existing structures
ISO 13824:2009 Bases for design of structures – General principles on risk assessment of systems involving structures
ISO 14044:2006 Environmental management – Life cycle assessment – Principles and framework
ISO 14963:2003 Mechanical vibration and shock – Guidelines for dynamic test and investigations on bridges and viaducts
ISO 16587:2004 Mechanical vibration and shock – Performance parameters for condition monitoring of structures
ISO 18649:2004 Mechanical vibrations – Evaluation of measurement results from dynamic tests and investigations on bridges.
ISO 31000:2009 Risk management – Principles and guidelines
In terms of context some of them are more general and introduce the concepts of monitoring, maintenance and its planning, besides general concepts of risk management and risk assessment. The standards under the name "Mechanical vibration and shock" instead refer to the use of dynamic measurements to perform periodic SHM functions on bridges (ISO 14963 and ISO 18649) and provide general guidelines for the condition monitoring of structures (ISO 16587). Important is the introduction of life cycle assessment (LCA) studies and life cycle inventory (LCI) studies.

The theme of SHM is more widely discussed in several guidelines published by research organizations such as ISIS Canada or produced in the framework of international research projects such as the European SAMCO and IRIS.

ISIC Canada with its ISIS Manual n. 2 – Guidelines for Structural Health Monitoring consisting of eight chapter and three annexes. After a first introductory chapter on basic concepts, the chapter 2 deals with the composition of Structural Health Monitoring and the treatment of data. Chapters 3 and 4 describe field testing, respectively static and dynamic testing. Chapter 5 deals with the periodic monitoring. Several monitoring examples of bridges are provided in chapter 6. Chapters 7 and 8 report definitions and bibliography. Annexes A,B and C give further information about sensors used in different measurements, data acquisition system and algorithms for vibration-based damage detection.

SAMCO Final Report 2006 contains two notable papers: F08a – Guideline for the Assessment of Existing Structures and F08b – Guideline for Structural Health Monitoring. The first one is clearly more addressed to the general topic of assessment of existing structures and it is less interesting in this context. Briefly, the general scope of the guideline is defined first, the second chapter describes in detail the principles of structural assessment. A scheme of the assessment methodology is introduced and the procedures for data acquisition, structural analysis and safety verification are described. In the third chapter the proposed assessment levels (Level 0 – Level 5) and associated procedures are explained. The guideline F08b deals directly with the Structural Health Monitoring starting with an accurate classification of the actions, their determination and the importance of the definition of calibrated load models. The diagnostic of structures is treated in chapter 4, the main chapter of this document. It is composed by 3 paragraphs: the first – Structural Condition Analysis - deals mostly with the identification of the structure, particularly through not destructive testing (NDT) techniques and field tests (static and dynamic); the second – Monitoring of structure – is focused on the monitoring task and defines sensors and their characteristics in addition to the necessary measurement equipment. Much importance is given to the treatment of data starting from their acquisition and continuing with their selection, management and analysis. The third paragraph introduce numerical analysis, necessary for structural evaluation and carried out with the finite element approach. Chapter 5 describes the damage identification: causes for damage, procedures of identification and damage assessments. The five Annexes contain a useful sensor classification based on applications (measured value), experiences and examples of traffic load identification on bridges, monitoring of heritage buildings and identification of local damages and their effects on structures.

The IRIS project started in 2008 with the aim to focus on diverse industrial sectors' main safety problems as well as to transform its requirements into integrated and knowledge-based safety technologies, standards and services. Large-scale demonstrations, some at a nearly unprecedented scale for an EU project, have been the IRIS's main effort up to the present. The demonstrations have been done to cover and visualize as many as possible aspects of potential risks in the industry, while keeping the users safety in mind, in order to better mitigate future risks.

An interesting document is the CEN Workshop Agreement CWA 16633:2013 – Ageing behaviour of Structural Components with regard to Integrated Lifetime Assessment and subsequent Asset Management of Constructed Facilities. This CWA was prepared by CEN Workshop 63 "Condition Determination for Integrated Lifetime Assessment of constructed facilities and Components" and it was developed through close collaboration with experts from the IRIS project "Integrated European Industrial Risk Reduction System". The Focus of the CWA is on the area of bridge infrastructure, as in this field the most mature status within the IRIS Project has been reached. This paper deals with performance of bridge components and provides recommendations and examples concerning the prognosis of the remaining service life.
IRIS published a book (Industrial Safety and Life Cycle Engineering, 2013) as well, rich in examples and applications (particularly interesting is the chapter 7 on SHM) but not comparable to a code or a standard.

The Russian Federation issued a compulsory standard: GOST R 53778-2010. Focusing only on the most interesting aspects: firstly it defines the frequency of inspections or monitoring, for example the first conditioning inspection of a building or structure must be performed no later than two years after commissioning and that subsequent inspections must be carried out at least once every ten years in normal environments and at least once every five years in severe environments. Secondly it provides a classification of buildings or structures in: normal operating condition; serviceable condition; limited serviceable condition and failure state condition. Depending on the category periodic inspections or continuous monitoring (can be optional or mandatory) are required.
It also contains many useful recommendations on the inspection of soil and foundations, above-foundation structures (concrete, masonry, steel, timber and others building elements), utility systems (hot water, heating, cold water, sewage, waste disposal, gas supply and drainage) and power and communication systems. Of interest are the annexes as well, since they contain classification, possible causes of defects and damage in different structural elements, and a framework that can be systematically used in case of inspection.

VDI 6200, issued by The Association of German Engineers (VDI) in 2010, specifies that building constructions have to be classified in three classes according to the possible consequences in the event of global or partial failure. Depending on the consequences class regular inspection intervals that vary from 1 to 5 years (visual check), from 2 to 5 years (inspection by an engineer) or from 6 to 15 years (verification by an expert) are recommended. In the following there are indications about the fulfilment of the Structural Safety Building Logbook which should provide in compact form an overview of the building, instructions for planning and execution (as far as regular inspections and maintenance) and checklists and documentations usable during regular inspections.
This guideline is referred to every type of buildings with the exception of traffic structures, treated in Germany in DIN 1076 and in DS 803 for Deutsche Bahn's buildings. DIN 1076 on bridges demands simple inspections every 3 years (proof test, bearing, drainage, cracks, deformation, corrosion etc.) and main tests every 6 years (foundation, examination of

inaccessible parts of buildings, concrete cover, material analysis etc.). Additional tests are required after extreme or accidental events (flood, fire or traffic accident).

Another helpful guideline is the Austrian RVS 13.03.01 (2012). It provides general information about monitoring activities and is concerned with the monitoring system's configuration and measurable quantities. Some examples and additional reference standards are provided as well. A notable additional standard is the RVS 13.03.11 (2011). Austrian guidelines are now also introducing monitoring activities for the improvement of the assessment quality.

As far as the bridges are concerned an extensive volume of data have been collected in the United States by the Federal Highway Administration (FHWA) in its Long-Term Bridge Performance (LTBP) program. This program started in 2008 and is intended to be a minimum 20-years research effort aiming to provide a more detailed and timely picture of bridges health, improve knowledge of bridge performance, and ultimately promote the safety, mobility, longevity and reliability of the highway transportation assets. Long-Term Bridge Performance researchers conduct detailed periodic inspections, monitoring, and evaluation of the population of bridges representing the national bridge inventory by taking advantage of not destructive evaluation (NDE) techniques and visual inspections. This program has led to several publications, including reports, presentation and newsletters but not to a development of a real guideline, code or standard.

Many codes, standards and guidelines are available regarding instrumentations and tests on materials. References are ISO, EN, American Society for Testing and Materials (ASTM), British Standards (BS), Deutsches Institut für Normung (DIN) and Ente nazionale di unificazione (UNI).

2.2 Discussion

An obstacle to the diffusion and to the application of SHM is represented by the fragmentation and the lack of an international standard universally recognised. Current documents offer many references but can be complex to find which one is better addressed to a certain issue. Besides this, difficulties can be found in implementing in practice the recommendations. On one side many demonstrations and applications have been performed, on the other side the codification is a step behind not having yet implemented some of these results. Based on the available documentation a further effort to the standardization could break through the existing barriers in the SHM's applications and could lead to new developments starting from many new applications.

The analysis of the contents of different documents reported herein, can lead to useful observations, especially which topics have been already widely developed and which aspects need to be deepened. It can be seen that the main topic is obviously the performing of the SHM, meaning the basis of this complex activity: sensor classification, based on their working principles and application; performing tests, meaning static and dynamic tests; data treatment and damage identification, through not destructive methods. With respect to the tests it is possible to find information about the necessary equipment, the purpose of the tests and the different tests which can be performed, for example proof tests (static), stress history tests (dynamic), dynamic load tests (dynamic) or modal tests (dynamic). It is amply recognised the importance of the handling of all data: acquisition, communication, processing, storage and retrieval. Damages are classified and the different techniques usable for their detection are reported. Classification of actions and their determination is an interesting topic treated especially in the SAMCO guideline.

These are the main topics addressed in the standards currently available, in addition to general information, distinction between permanent and periodic monitoring, references to maintenance strategies and lifecycle cost optimization. These last two topics are discussed in the CWA 16633 which provides suggestions and examples concerning service life expectancy and prognosis on remaining service life.

A particular and important aspect that still needs to be investigated is related to the role of the SHM in the processes of risk assessment and its impact on design standards. We are able to monitor a structure starting from knowing the tests which can be performed and the sensors that have to be used, we are able to manage data in all the phases of the process as well to make deterministic assessments of safety basing on threshold values a priori assumed. What we are not yet able to do without uncertainty is, for instance, to evaluate the differences, in reliability terms, between monitored structures and not monitored ones. Basically there is no a codified systematic way that allows to assume appropriate safety factors which consider the reduced uncertainty due to the presence of a permanent monitoring system.

Deepening the knowledge in this sector and using the data deriving from structural monitoring to update the safety factor applied to characteristic values in the limit state design method currently represent import fields of study and research. Developments in this direction, focused to a future codification, are desirable to allow better monitoring planning, better management of financial resources with reduced life cycle costs, improved performance, extended life of structures, and improved safety.

A further interesting argument which needs to be studied and experienced is that related to updated structural and consequently FE models. Using the structural monitoring can be possible to update the representative model of a structure in order to obtain a better representation of its real behaviour. Once the representative model suits all the information coming from monitoring it will be also more accurate to perform real-time valuations and to predict the structural behaviour during possible future hazard scenarios. The construction and the use of these models is still an open field.

For this purpose it can be interesting and useful to look at different industrial sectors in which the standardization of products, less uncertainties of materials and smaller tolerances have fostered the development of the SHM since the 70s and therefore have today more experience and more advanced standards. An example is the aerospace industry in which the aircraft health management and the maintenance programs are fundamental aspects. Important and not obligatory guidelines are the Aerospace Recommended Practice (ARP) which deal with a wide range of topics. Interesting in this context is, for instance, the ARP 6461 – Guideline for implementation of Structural Health Monitoring on Fixed Wing Aircraft which describes how SHM aligns with current Maintenance Review Board process used by the industry to develop maintenance program. It also describes the principal benefits accrued by use of SHM system data for future design improvements or upgrades. [12]

Of particular interest is also the oil and gas sector with its standards and guidelines regarding inspection and monitoring of offshore structures and applications. References, in addition to some ISO's standards, are: the American Petroleum Institute Recommended Practice, the API RP 2A or API RP 8B, for instance; the NORSOK standards, developed by the Norwegian Petroleum Institute, in particular the N-005; the researches carried out by the DNV, Det Norske Veritas, which produced standards like the DNV-OSS-101; guides by the American Bureau of Shipping (ABS).

In addition petrochemical, automotive and marine are stimulating industrial fields that can provide important references.

2.3 Possible content of a code

Guidelines, research publications and experimental data are currently available on inspection and on monitoring and they represent the outcome of the remarkable work done. Not all the topics of interest are widely debated: some needs a better organization and some needs further studies and researches. It would be desirable to have available, in a near future, a comprehensive code containing theoretical aspects and that represents a landmark in practical applications.

Some recommendations on the possible content are briefly described in the following.

Bases and general principles

> Definitions on inspection and monitoring activities focusing on the possible advantages, benefits, practical applications in design and in assessment of existing structures, and on how they can successfully be implemented in the practice.
>
> General classification of inspection and monitoring specifying the main characteristics of each of them, meaning constitutive components, fields of application, strong points, weak points etc.

Analysis of actions

> Actions on structure determinate strains, displacements and deformations: exact knowledge about acting loads is the basis of the realistic evaluation of the structural load bearing capacity. Classification of actions basing on their effects and actions which mainly affect the different structures or structural elements.
>
> Measurement of actions and use of data to determinate and to calibrate load models.
>
> Probabilistic models and reference values to be used in different practical applications.

Sensor technology

Classification of the most important sensors currently available on the market reporting the operating principles, measurable physical quantities, practical applications, practical issues, and examples.

Static and dynamic testing of structures

Overview on static and dynamic testing (behaviour tests, proof load tests, modal tests, damage detection, etc.) including fields of application and benefits.

Types of equipment used for testing, practical recommendation and design and practical issues.

Treatment of data

Treatment of data is an important part especially when continuous monitoring is considered due to the big amount of data available. Appropriate attention should be put in all the process starting from data acquisition system and continuing with collection, communication, processing and storage of data. Fundamental are also diagnostics and retrieval of data.

Structural Health Monitoring system design

Once actions, sensors technology, static and dynamic testing and treatment of data are defined it is possible to deal with the design of the SHM system. The design issues start with the selection, installation, and placement of sensors and it continues with the treatment of data, with the design of data transmission and acquisition systems. Recommendations and design methodologies must be provided

Buildings classification and management

Buildings classification according to consequences classes. Classifications currently proposed by the codes and reported in the following may be accepted.

Recommendations on the elaboration of inspection and maintenance plans based on cost optimization criteria with the aim to minimize the life cycle cost of the building keeping the performances above a target level. Inspection and monitoring intervals should be specified.

Assessment of existing structures

This part is focused on the assessment process and on how concretely use inspection and monitoring data. Bases on the probabilistic derivation of partial safety factors and reliability analysis must be included as well as recommendations about updating of partial safety factors and probabilistic models

Important topics are also target reliability and acceptance criteria.

Further contents

Further contents such as damage identification (definition and classification of damage, algorithms for identification and assessment), numerical analysis with calibration of structural models and reliability of smart monitored structures could be investigated and included in the code.

Chapter 3
Inspection and monitoring

3.1 General framework

Inspection and monitoring are important parts of the assessment process of existing structures; they are strictly connected and often the boundaries between them are not well defined and in certain contexts some terms can be ambiguously used. Inspection is an investigation intended to update the knowledge about the present condition of the structure. Monitoring is the activity that permits to identify the behaviour and the characteristics of a structure or of a part of it through measurements carry out with technical equipment. Concisely, inspection is more general and can indicate any activities, monitoring is more specific and refers to quantitative measurements.

Testing and inspection are not new concepts, they have been conducted for thousands of years (Fig. 3.1) in an effort to prolong structures' service life and ensure public safety. The relatively new idea is to regularly inspect most structures and infrastructures and monitor the behaviour of critical parts of the whole structure to get early warnings that may lead to an immediate intervention or at a regular maintenance intervention depending on the severity of the problem. This aspect is lacking in the civil sector, especially if comparisons with different industrial sectors are conducted. To date one of the primary factors that have led to unsatisfactory condition of our structures and infrastructures is precisely the unsatisfactory inspection and monitoring, with problems becoming apparent only once structures are in such dire need of attention that the cost of repair often approaches that of replacements. However the latest developments and decreasing cost of sensor and information technologies have made SHM systems more attractive in civil engineering applications.

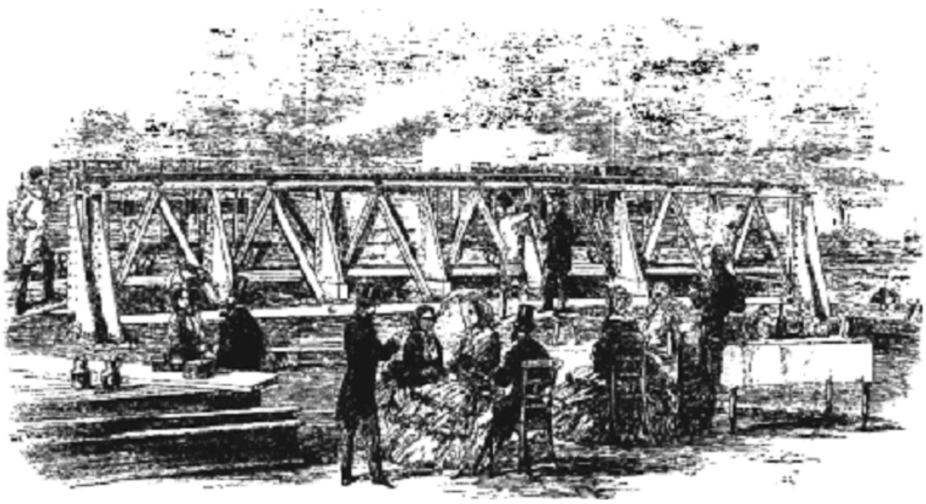


Figure 3.1: Testing of a steel bridge in England in the 19th century (ISIS Canada, 2001)

In the following they are first introduced two flowcharts that contextualize the present topic within the assessment process of existing structures without the will to go into details. Secondly main characteristics and classifications of inspection and monitoring systems are reported.

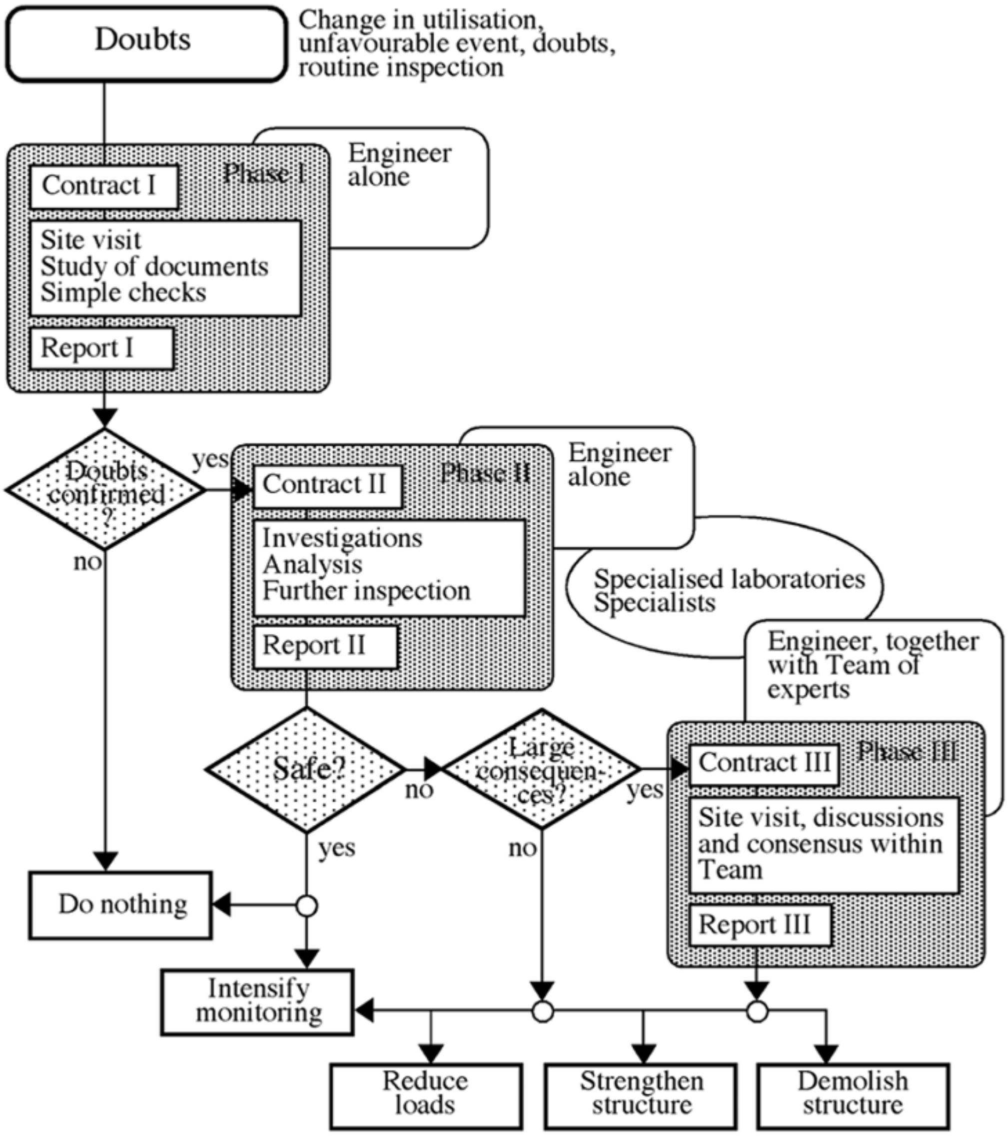


Figure 3.2: Illustration of the three phases approach (developed by J. Schneider in JCSS - Assessment of Existing Structures, 2001)

Figure 3.2 visualizes schematically the breaking down of the assessment of an existing structure in three phases, each of ones should be complete in itself. Phase *I* is the preliminary evaluation, phase *II* is the detailed investigation and phase *III* is the expert assessment. This subdivision is dictated by the experience. As it is possible to see, inspection and monitoring are fundamental parts of each phase and maintenance works can be the consequence of the report. An alternative schematization is the one proposed in figure 3.3 where the process showed can be repeated if necessary and there is no subdivision in phases.

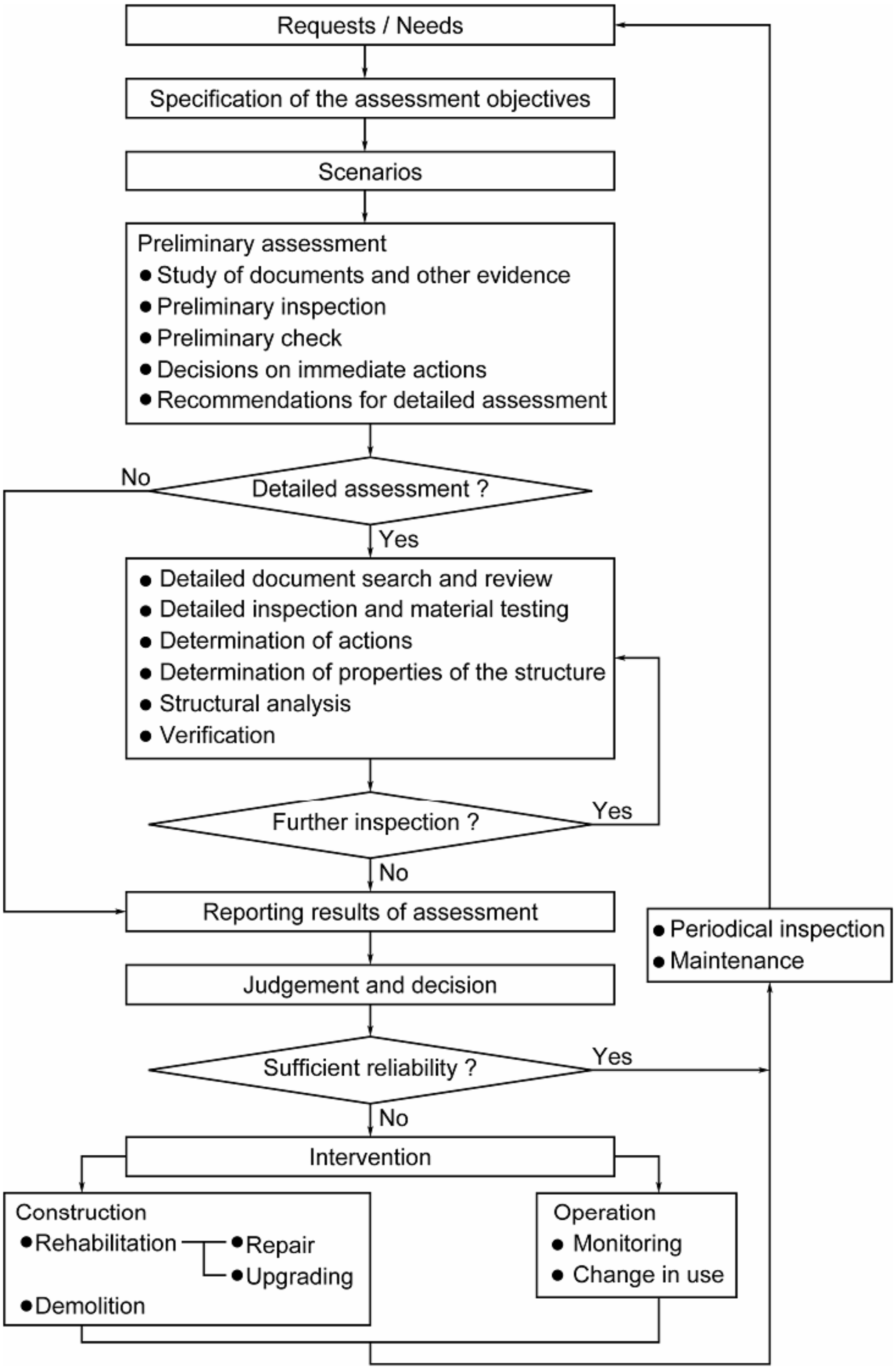


Figure 3.3: General flow of the assessment of existing structures according to ISO 13822 (2009)

Related to inspections typically two types of interrelated decisions have to be made:

- What inspections shall be performed?
  For example which are the parameters to be inspected, how many samples ad when shall be taken, what are the techniques to be used.
- What to do with the inspection results?
  For example types of measures to be taken (repair, strengthening, etc.), development of an inspection plan.

Two types of inspections can be in general distinguished:

Qualitative inspection: this type of information is related to the observation of parameters such as surface characteristics, visible deformations, cracks, spalling, corrosions etc.
The description of the damage of the structure will be in qualitative terms like: no damage, minor damage, moderate damage, severe damage etc. The ranges of each category shall be thereby specified. However it is possible and sometimes necessary to process the observation in a more formal way.

Quantitative inspection: this type of information results in a set of values of parameters that characterize the condition of the structural elements.
Examples of such condition parameters are: crack depth and length, corrosion area and depth, displacements, residual stresses, damping, eccentricities etc.

For both inspection types the related uncertainties such as the probability to detect some damage and/or the accuracy of the results shall be specified and taken into account. The results are usually compared to specified requirements or standards.

Monitoring is a complicated activity, a multidisciplinary task where subjects from numerous fields are involved. Currently we refer to the whole monitoring activity as Structural Health Monitoring or using the acronym SHM. SHM systems are applicable to all types of civil engineering structures, including bridges, building tunnels, pipes, highways and railways. While the specific details of SHM systems can vary substantially, a modern system will typically consist of six common components, namely:

1. Acquisition of data (a sensory system);
2. Communication of information;
3. Intelligent processing and analysing of data;
4. Storage of processed data;
5. Diagnostic (i.e. damage detection and modelling algorithms);
6. Retrieval of information as required.

A typical flow pattern between these six components is shown in Figure 3.4. However other flow patterns are also possible, and the flow of information between systems components can certainly take more than one path.

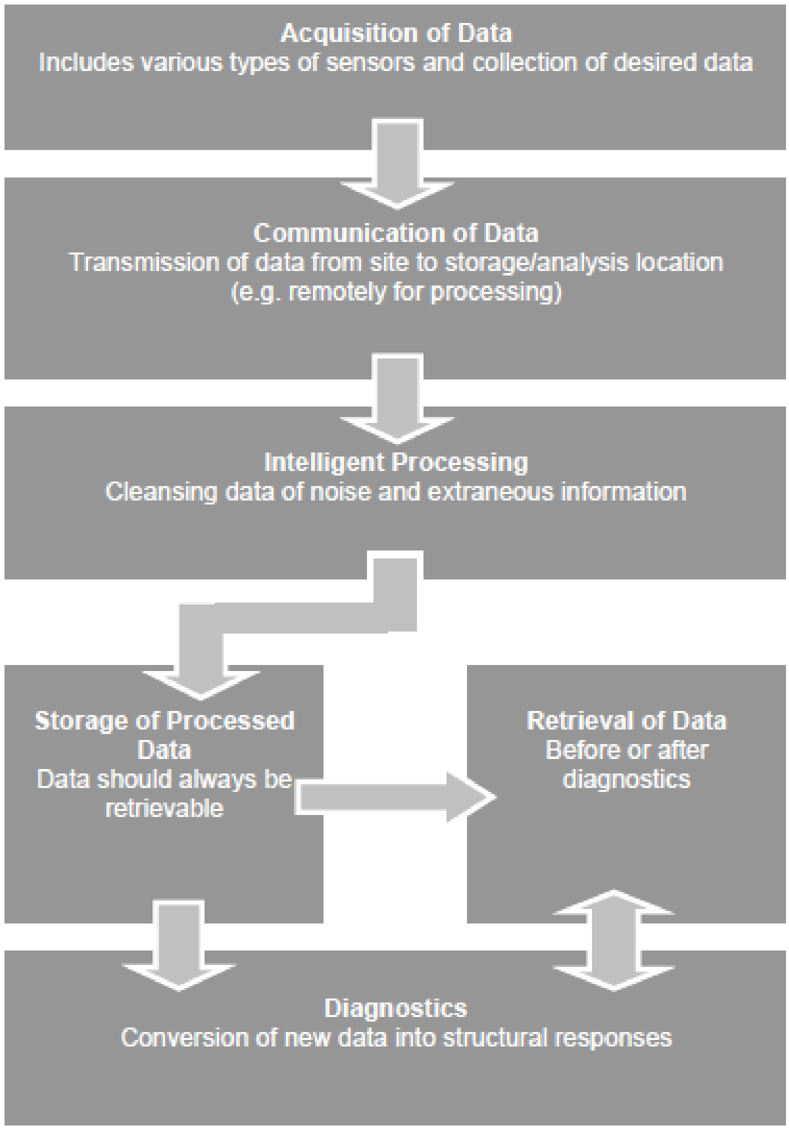


Figure 3.4: Components of a typical SHM system (ISIS Educational Module 5, 2004)

Each of the six components is discussed in the following more in detail :

1. The acquisition of data involves the collection of raw data such as strains, deformations, accelerations, temperatures, moisture levels, acoustic emissions, and loads. Essential to the effectiveness of an SHM is the selection of appropriate and robust sensors. In addition, the selection criteria should include accuracy, reliability, sensor installation limitations, durability and cost. Care should be also taken during the design of the SHM system to ensure that sensors can be easy installed within a structure without substantially changing the behaviour of the structure. Further important aspects of acquisition of data are the transfer to data acquisition system and data sampling and collection. The data acquisition system refers to the onsite system where signal demodulation, conditioning and storage of measured data are conducted prior to being transferred to an offsite location for

analysis. For most sensors an input signal is required, and then interpretation of the sensor output signal must be conducted to convert the analog sensor response into engineering terms. The most common and inexpensive method to transfer data is via physical link called a lead cable or wire. For very large structures or where long lead cables are otherwise impractical wireless communication technologies (currently more expensive, slower and not completely secure) can be used to transfer sensor signal. As sensor signals arrive at the data acquisition system, the data must be sorted for onsite storage. A well thought out data acquisition algorithm, which captures an adequate amount of data is a very important component of a successful SHM system. Sensors and data type that are typically monitored are reported in paragraph 4.2.

2. The communication of data of an SHM refers to the mechanism of transfer of data from the location where they are collected (the data acquisition system) to the location where they will be processed and analysed (normally some remote locations).

3. The intelligent processing and management of data consists in the removing of extraneous information and noise from the data obtained by the various sensors. The removal of this unwanted information is aimed to make data interpretation easier, faster, and more accurate. Various data management strategies have been developed to eliminate unnecessary data without sacrificing the integrity of the overall system.

4. The storage of processed data is the storage of intelligently processed data for later use in structural health diagnostics.

5. Diagnostics involves further interpretation of the collected, cleansed, and intelligently processed data to produce useful information about the response and health of the structure. This activity requires expert structural knowledge about the behaviour of structures as well as understanding of how that behaviour may be affected by damage, deterioration or other changes in condition.

6. When selecting data to store for retrieval, both the significance of the data and the confidence in its analysis should be considered.

In addition to the various components of SHM systems, it is possible to distinguish between different kind of monitoring on the basis of the time strategy, condition strategy, and load effect strategy. The time dependent strategies describes the duration and the frequency of the measurements and are here characterized as periodic and permanent monitoring. The selected strategy depends on the phenomena to be observed.

Short-term monitoring: it is performed by temporary installing an appropriate sensory system on the structure and gathering data for a short time.

Long-term monitoring: the monitoring system is permanently installed and maintained in operation on the structure. Data are acquired continuously or periodically and real time evaluations are possible.

Long-term monitoring allows sophisticated and accurate analysis but due to the high costs and complexity in design and treatment currently the applications are limited and involve mainly structures that are either extremely important or if there are doubts about their structural integrity. This is the case of ancient structures and monuments. Permanent monitoring should only be considered if the changes in loading are slow, such as gradual temperature changes, or if the loads are not predictable, e.g. natural hazards such as floods, hurricanes or earthquakes. Long-term monitoring can be subdivided on the base of the time interval of data collection in continuous or periodic. Frequent periodic is when data is collected at regular time intervals. Triggered periodic is when data collection is initiated or triggered by a specific event, e.g. when a measured parameter exceeds a threshold. The sampling interval for each data collection depends on the dynamic nature of the studied phenomena. The opposite is true for temporary monitoring, used to examine the structure at a specific point in time, often to evaluate a change. This kind of monitoring typically requires less complex sensor systems and it is easier to be managed.

The main features and the differences between permanent and periodic monitoring are summarized in the Table 3.1, prepared by A. Del Grosso (SMAR 2013):

	Permanent Monitoring	**Periodic Monitoring**
Sensor types	Extended	Restricted
Data management	Complex	Simple
Accidental events	Recorded	Not recorded
Damage identification	On-line	Off-line
Warnings & Alarms	Real-time	Deferred
Fatigue life evaluation	Direct	Indirect
Installation costs	High	Low
Operational costs	High	Low

Table 3.1: Permanent versus periodic monitoring by A. Del Grosso (SMAR 2013)

The condition strategy means what type of phenomenon to be observed. It is possible to distinguish between:

Local monitoring: it is the observation of local phenomenon, such as strain, crack opening, etc.

Global monitoring: it is defined as the observation of global phenomena of structures. An effective method to obtain the global behaviour of the structure is to monitor modal parameters, such as frequencies, mode shapes, and damping.

Local monitoring can also be defined as non-destructive localised evaluation and is useful for applications in the laboratory when certain parameters must be controlled. Global monitoring may include damage or deficiency detection. It is based on modal analysis and on the idea that when a structure is exposed to damage the corresponding modal properties are changed. Damage detection is normally defined in four steps or levels: determination that damage is present in the structure, location of the damage, quantification of the severity of the damage, and prediction of the remaining service life of the structure.

The load effect strategy is mainly a question of how the measurements should be collected over time. Based on the type of field testing undertaken it is possible to distinguish in:

Static monitoring: the loads are brought onto or placed on the structure very slowly so as not to introduce dynamic effects in the structure. They are most commonly used to determine the load carrying capacity of a structure and to provide data about a structure's behaviour and ability to sustain live loads.

Dynamic monitoring: tests used to determinate the dynamic properties of the structure and the interaction between the dynamic loads and the structures behaviour.

Measurements of phenomena such as deflection, inclination, settlements, crack widths, corrosion and phenomena caused by environmental properties, for example temperature, humidity, wind are most of the time quasi-static since they vary slowly over the time. When monitoring these parameters is often enough to measure the peak values over a longer time depending on the speed of actions that crate the phenomenon. This is a static monitoring. Dynamic monitoring is typically performed with a much higher sampling rate compared to static monitoring in order to obtain the structural behaviour. Further information about static and dynamic field testing are reported in paragraph 3.3.

3.2 Sensors and physical quantities

Essential for the monitoring of structures are sensors which are robust and operate stably and reliable. Sensors can be subdivided in such which concentrate on the monitoring of local properties like material and in those which observe structures from a global point of view. Some are embedded within the structure others are only placed on the surface of the structure.

The data types that are typically monitored by SHM systems are:

Load : it is interesting to determine if the loads on the structure are as expected, or if it is subjected to greater loads and to learn how the various loads are distributed within and supported by the structure. Loads can be measured directly using load cells installed within the structure, or it can be inferred through strains or other parameters measure on selected structural components.

Deformation : excessive deformation, or deformation in unexpected places, might signal deterioration or changes in structural condition and can be used to assess the need for rehabilitation or upgrade. Deformations and deflections can be measured with a variety of types of displacement transducers and tiltmeters.

Strain : Strain is a measure of the intensity of deformation of a structural component. The magnitude of the measured strains, and the variation of magnitudes recorded over the life of the structure, can be examined to evaluate the safety and integrity of the structure. Strains in structural components can be directly measured at the desired location using standard electrical resistance strain gauges, vibrating wire strain gauges or more recently developed fibre optic sensors.

Temperature : repeated cycles of heating and cooling can cause damage to structure through repeated cycles of deformation and thermally induced loads. Temperature may also affect the reading of certain sensors or sensing equipment used in SHM systems. Temperatures can be measured using thermocouples, integrated temperature circuits, thermistors, or certain types of fibre-optic sensors.

Acceleration : SHM can be used to determine how a structure is responding to acceleration, ground acceleration for instance, and the resulting loads via determination of the modal response parameters. This type of monitoring is now widespread especially in seismic regions. Even in non-seismic situations the modal response parameters of a structure can be monitored. Due to changes in support condition or material properties (damage or deterioration) , there can be a shift in these modal parameters. Accelerations are typically measured using a class of sensors called accelerometers.

Wind speeds and pressures : for tall buildings and long-span bridges wind can be a governing design criterion and should be recorded at various locations an SHM system. Wind speed can be measured using anemometers.

Acoustic emission : sound waves or acoustic emission waves can be used to determine the location and characteristics of damage in structure. Acoustic emission monitoring is based on the principle that the arrival times of sound waves at different sensors will be different depending on the distance between the sensors and the origin of the sound.

Many different types of sensors might be used in any specific application to measure various types of data. The most important sensors currently used within structural health monitoring are: strain gauges, fiber-bragg gratings, piezofilm sensors for strain measuring, displacement sensors for deflections, GPS based displacement sensors, hydrostatic levelling systems (HLS), displacement sensors for relative vibration measuring, vibrating wire strain gauges, vibration velocity sensors, vibration acceleration sensors, laser detector for vibration measurements, inclinometers for angular displacement measurements, fibre optic sensors, temperature, humidity and corrosion sensors.

The basic criteria for selection of sensors are minimal change of the measurand (resolution, linearity, accuracy), measuring range, type of measurement (static, dynamic etc.), test duration (long-term stability), test environment, installation environment and financial resources. More detailed information about sensors, sensor technology and measuring range can be find in ISIS Educational Module 5 (2004) and in SAMCO Final Report (2006) – F08b. To deepen this topic suggested reads are *Encyclopedia of Structural Health Monitoring* edited by *C. Boller, F.-K. Chang, Y. Fujino* and published by *John Wiley and Sons* in 2009 and *Inspection and Monitoring Techniques for Bridges and Civil Structures* edited by *G. Fu* and published by *Woodhead Publishing* in 2005.

In the following is reported an interesting table, translated from RVS 13.03.01 (2012), where sensor type and physical quantities are correlated.

	Physical quantity	Sensor type: Displacement transducer	Inclino-meter	Hydrostatic leveling systems	Distance measuring equipment (optic)	Strain gauges	Tachy-meter	Fibre-optic sensors	Load cells	Pressure sensors	Accelero -meter	Vibrating velocity sensor
static	Deformation/ Displacement (vertical) [m]	Well suitable		Well suitable	Well suitable	local (Well suitable)	Well suitable	Well suitable				
static	Deformation/ Displacement (horizontal) [m]	Well suitable			Well suitable	local (Well suitable)	Well suitable	Well suitable				
static	Inclination/ Rotation [°]	Conditionally suitable	Well suitable	Conditionally suitable	Conditionally suitable		Well suitable				Depending on sensor (Conditionally suitable)	Depending on sensor (Conditionally suitable)
static	Settlement [m]	Conditionally suitable	Conditionally suitable	Well suitable	Conditionally suitable		Well suitable	Conditionally suitable	Conditionally suitable	Conditionally suitable		
static	Expansion [‰]	Conditionally suitable				Well suitable		Well suitable				
static	Load [N]					Well suitable		Well suitable	Well suitable	Conditionally suitable	Tensile forces (Conditionally suitable)	Tensile forces (Conditionally suitable)
static	Stress [N/m^2]					Conditionally suitable			Conditionally suitable	Well suitable		
dynamic	Acceleration [m/s^2]										Well suitable	Conditionally suitable
dynamic	Vibrating velocity [m/s]										Conditionally suitable	Well suitable
dynamic	Eigenfrequency [Hz]	Conditionally suitable			Conditionally suitable	Conditionally suitable		Conditionally suitable			Well suitable	Well suitable
dynamic	Damping [%]	Conditionally suitable			Conditionally suitable						Well suitable	Well suitable

Legend

Well suitable sensor
Conditionally suitable sensor
Not suitable sensor

Table 3.2: Measured physical quantities corresponding to sensor type (translated from RVS 13.03.01, 2012)

3.3 Field tests

Field testing as part of structural identification is used as an inspection approach as well as part of monitoring in the way of cyclic or intermittent observation. We can distinguish between static field testing and dynamic field testing.

3.3.1 Static tests

Loads are slowly placed and sustained on the structure in order to not cause any dynamic effect such as impact, vibrations or resonance and hence the interpretation of data is less complex. Static field tests can be subdivided into:

Behaviour tests: the tests are carried out to study the mechanics of structural behaviour or to verify certain methods of analyses that can be used for the design and evaluation of structures with confidence. The loads imposed are less than or equal to the maximum allowed service load on the structure. A behaviour test provides information regarding how the load is distributed among various components of a structure, but no information is provided about the load capacity of the individual structural components. Results from these tests can be used to calibrate analytical methods.

Diagnostic tests: a diagnostic test denotes a test that is carried out to diagnose the effects of component interaction: if the response of a particular component of a structure is hindered of helped by another structural component. Through a large number of tests, it has been confirmed that diagnostic testing can be used with advantage: to locate the sources of distress that might exist in a structure due to inadvertent component interaction. Diagnostic testing has the benefit of explaining why the structure is performing differently than assumed.

Proof load tests: a proof test is carried out to establish the load-carrying capacity of a structure. During this test, the structure is subjected to exceptionally high static loads that cause larger responses in the structure than the responses that are induced by statically applied maximum service loads. Because of the very high loads applied to the structure in proof testing, there is always the possibility that the structure may be permanently damaged by the test. A well-planned proof test is carried out with gradually increasing loads, ensuring that the loads are not allowed to be beyond the limit of linear elastic behaviour. Care should be taken to ensure that all calculations are correct, all safety precautions are taken. The structure is continuously monitored during the testing.

3.3.2 Dynamic tests

During dynamic examinations the determination of the dynamic properties of structures and the interaction between the dynamic loads and the structures behaviour is in the focus of attention. Dynamic testing of structures can be subdivided into the following distinct categories:

Stress history tests: tests used to determine the range of stresses experienced for instance by parts of a bridge which are prone to failure by fatigue loading. After preliminary numeric investigations, for the determination of ''hot spot'' at structures, a larger number of sensors are attached and the stresses under operating conditions are measured. From the results of these investigations optimal sensor configurations for a continuous fatigue monitoring are determined. Stress history tests are accomplished whenever the dynamic actions in combination with the examined structure are too complex to obtain sufficiently exact results by numeric simulations.

Dynamic load tests: these tests serve to determine the dynamic increment from traffic loads. Realistic information is needed in order to control design acceptance after completion. Likewise, with same traffic volume, structural changes can leads to changed dynamic stresses in parts of the structure. Also a change of use due to planned passages of vehicles with changed dynamic characteristics lead to measure the new dynamic loads in advance. If during design the dynamic load effects are regarded as an increase of the static stresses, dynamic load tests are to be accomplished by the measurement of strains at those structural parts, which are of importance for the design.

Modal tests: modal tests are used for determination of modal properties of structures. The knowledge of the modal characteristics is used for damage identification, for quality control of structures after completion, for planning and assessment of repair work, for the assessment of structural safety, after extreme loading as well as for the calibration of structural models. The procedures for the determination of the natural frequencies, the mode shapes and the modal damping are differentiated regarding to the excitation of the structures in:

- *ambient vibration test*
- *forced vibration test*

In the first case the tests are accomplished under operating conditions. The excitation energy comes from the dynamic operating load of the structures (wind, weather, traffic, ground vibration). Since the systems responses due to natural excitation are often small, highly sensitive sensors must be used to their ascertainment. The usual kinds of excitation with forced vibration test are impulse (impulse hammer, drop weight etc.) and Heaviside function as well as regulated excitations (harmonic, periodic and stochastic) by electro-dynamic and electro-hydraulic exciter systems. The selection of the type of exciter depends on the dynamic characteristics of the structures as well as on the existing site conditions. Impulse excitations are unsuitable for large buildings. During regulated excitation arbitrary long measurement times are possible, with which higher frequency resolution can be achieved. Disadvantageous is the fact that equipment and operation of such exciter systems are substantially more expensive and require the exclusion of the normal operating conditions (traffic). The advantage is the almost complete identification of the modal characteristics of the structures.

In addition *pull-back tests* are usually conducted on bridges or on certain other types of structures to determine their response to lateral (sideways) dynamic excitation. In the case of bridges, since normal traffic loads do not significantly excite a bridge in the lateral direction, it is usually difficult to determine their lateral vibration characteristics from the results of ambient vibration tests. This type of test is conducted by pulling the structure laterally by means of cables anchored in the ground (or to some other fixed object) and releasing the cables suddenly. The response of the structure is monitored with the help of accelerometers, and the process of analysing the data is much the same as for an ambient vibration test.

3.4 Case study 1: Ölfusá bridge, Iceland

The Ölfusá suspension bridge, built in 1945 and located in Selfoss, about 60 km from Reykjavik, is the oldest but the most heavily loaded suspension bridge on Iceland. The structural system of the suspension bridge is an earth anchored system and consists of a steel truss girder with a concrete deck on top, locked coil strand cables with suspenders over the 84 m long main span, built-up steel section pylons and anchor-blocks (Fig. 3.5). The two-lane roadway of the main span was reconstructed in 1992 which involved the installation of a considerably heavier concrete deck: the original 8 m wide concrete deck was replaced with a new 8,7 m wide concrete deck consisting of 6,2 m roadway, a 1,8 m pedestrian lane and railing.

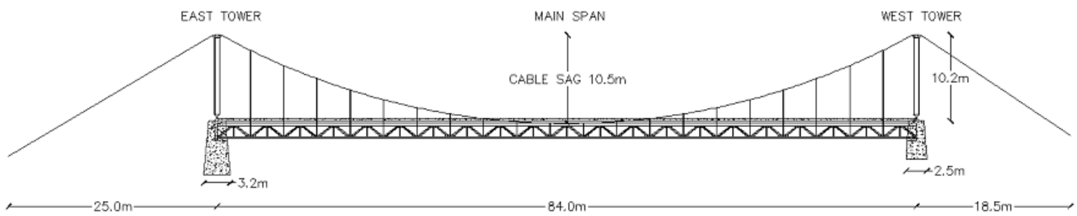


Figure 3.5: Main span of the Ölfusá suspension bridge (*Óskarsson, 2012*)

The uncertaintiy regarding the structural state of the Ölfusá Bridge is matter of some concern, especially regarding the actual condition and bearing capacity of the main cables. The preservation of the Ölfusá Bridge is of significant importance for the population in the south of Iceland, being a socially important link with regard to work commuting, tourism, and safety precautions.

The work conducted and herein presented is intended to provide an evaluation on the actual structural condition of the bridge and to implement a continuous monitoring system of the cable forces through the calibration of a finite elements model.

A first visual inspection was conducted. It revealed, in addition to increased self-weight and traffic loading, potential degradation due to corrosion and suggested that further investigations were necessary. For this purpose it was decided to realize a three dimensional model using commercial software (CSI SAP 2000 v15 and CSI Bridge v15) where cable, frame, solid and shell elements are utilized (Fig. 3.6):

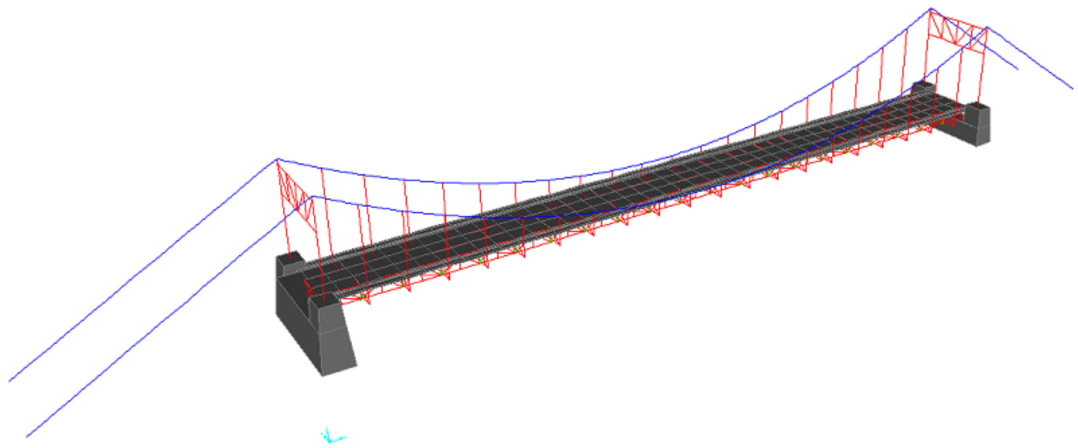


Figure 3.6: Finite elements model (*Óskarsson, 2012)*

The main focus during the modelling process was to represent the actual geometry as accurately as possible with careful placement of elements according to drawings, proper simulation and quantification of element mass and stiffness, and boundary conditions that represent real conditions. At first the original configuration of the bridge, comprising the old deck, is modelled to validate the accuracy of the modelling process and to start the calibration process by comparing with documented test results. According to load test documents from 1946 the suspension bridge cables were prestressed to have a deflection of +147 mm at the middle, above horizontal, under dead load condition. To achieve the correct amount of hogging effect of the bridge model, cable elements are subjected to strain loading:

Prestrain	Deflection δ	Main cable F	Suspenders F
[‰]	[mm]	[kN]	[kN]
2,25	147	2568	114

Table 3.3: Cable pre-strain and corresponding deflections and axial forces (*Óskarsson, 2012)*

The 2,25‰ value of strain is considered to adequately describe the actual behaviour of the bridge, resulting in values fairly close to the measured deflection of +147 mm and the design horizontal cable force of 2357 kN. Thus this initial cable strain is used in the comparison between the load test conducted in 1946 and the static analysis results from the computer model. These tests confirm an appropriate accuracy of the finite element model yielding differences of 22% and 13% under evenly distributed loading over a large area (30 m and 50 m respectively) at a global level. Basing on this first model the bridge deck is replaced with the new one and the current structural configuration is obtained. The static analysis highlights that the new heavier bridge deck increases the dead load of 49%, increases the tensile forces of 37% and of 40% in the main cable and suspenders respectively, and reduces the deformation of 139 mm (from +147 mm to +8 mm). The increased deflection induces higher compressive and tensile forces in the top and the bottom chord of the stiffening trusses. The modal analysis to determine the dynamic proprieties is conducted as well and the results are compared with the measured mode shapes. The bridge is closed to traffic and 20 minutes ambient vibration measurements are conducted using 7 accelerometers in 8 different setups to determine the actual dynamic properties of the bridge. The comparison returns suitable results with frequency differences up to 23%. Nevertheless better results can be achieved identifying the critical parameters of the FE-model (cable stiffness, boundary conditions, selfweight, stiffness of steel truss and concrete deck) and updating them. Results are summarized in Table 3.4:

Mode shapes		**FE-model**		**Measurements**	**Updated FE-model**	
No.	Type	f (Hz)	Diff.	f (Hz)	f (Hz)	Diff.
1	1^{st} vertical	0,88	23%	1,08	1,09	1%
2	1^{st} horizontal	1,42	14%	1,61	1,42	13%
3	2^{nd} vertical	1,48	16%	1,71	1,70	1%
4	1^{st} torsional	2,11	2%	2,07	2,17	4%

Table 3.4: Calibration of the FE-model with vibration tests (*Óskarsson, 2012)*

The following step is the cable force identification: load testing using a 60 t crane is performed: frequencies in the hangers and in the backstays of main cable at different positions are measured and the results are compared to the calibrated FE-model. The results highlight that the actual strength design criteria are satisfied with a rather low safety margin. This induces to implement a structural continuous monitoring system that can identify changes in the structure such as: vertical sag in main span; changes in cable force using vibration measurements; post-earthquake safety. Accelerometers on two backstays were installed in August 2014 (Fig. 3.7) and on-line continuous monitoring is on-going, together with post-processing and analysis of data. The initial results are promising.

Figure 3.7: Accelerometers positioning (VEGAGERDIN, 2014)

3.5 Case study 2: The New Svinesund Bridge, Sweden/Norway

The New Svinesund Bridge is a highway bridge across the Ide fjord joining Sweden and Norway. The total length of the bridge is 704m (Fig. 3.8) and consists of a substructure in ordinary reinforced concrete together with a steel box-girder superstructure. The main span of the bridge between abutments is approximately 247m and consists of a single ordinary reinforced concrete arch which carries two steel box-girder bridge decks, one on either side of the arch. The level of the top of the arch and the bridge deck are 91.7m and 61m, respectively. Over the part of the bridge where the arch rises above the level of the bridge decking, the two bridge decks are joined by traverse beams positioned at 25.5m centers. The traverse beams are in turn supported by hangers to the concrete arch.

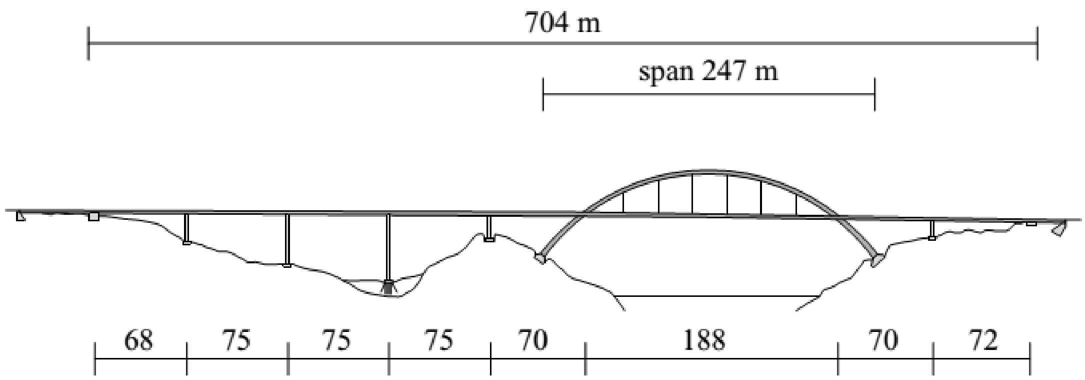


Figure 3.8: Sketch of the New Svinesund Bridge in its entirety, showing grid-line numbering and approximate dimensions (Wenzel, Health monitoring of bridges, 2009)

The bridge is a structurally complicated bridge and is the world's largest single-arched bridge and one of the most slender. As monitoring is an effective way to understand the real behaviour of the bridge, a monitoring project was initiated by the Swedish National Road Administration. The project, including measurements during the construction phase, the testing phase, and the first five years of operation, is coordinated by The Royal Institute of Technology (KTH). The primary objective of the monitoring programme is to check that the bridge is built as designed and to learn more about the as-built structure. This is achieved by comparing the measured structural behaviour of the bridge with that predicted by theory.

The data acquisition system consists of two separate data sub-control units located at the base of the arch on respectively the Norwegian and Swedish side. The sub-control system on the Swedish side contains the central rack-mounted industrial computer and is connected with ISDN telephone link for data transmittal to the computer facilities at NGI/KTH for further analysis and presentation of data. The logged data on the Norwegian side is transmitted to the central computer on the Swedish side via a radio Ethernet link. The selected logging procedure provides sampling of all sensors continuously at 50 Hz with the exception of the temperature sensors which have a sampling of once per 20 seconds or 1/20 Hz. At the end of each 10 minute sampling period, statistical data such as mean, maximum, minimum and standard deviation are calculated for each sensor and stored in a statistical data file having a file name that identifies the date and time period when the data was recorded. Raw data, taken during a 10 minutes period, is stored in a buffer if either of the programmed “trigger” values for the calculated standard deviations of acceleration or wind speed are exceeded. The instrumentation of the arch is reported in Table 3.5:

Type of sensors	Number	Location
Vibrating-wire strain gauges	16	Four at arch base and four just below the bridge deck, Norwegian and Swedish side
Resistance strain gauges	8	Two at arch base, two in a segment just below bridge deck, and four at the crown
Linear servo accelerometers on concrete arch	4	Installed pair-wise and are moved to new arch segments as construction of the arch progresses. When the arch is completed, two accelerometers will be moved to the arch midpoint and two to the arch's Swedish quarter point
Linear servo accelerometers on bridge deck	6	Three at midpoint and three at quarter point
Temperature gauges	28	At the same sections as the strain gauges
Outside air temperature gauge	1	At arch base on Swedish side
Three-directional ultrasonic anemometer	1	For measuring wind speed and direction at deck level close to the first support on the Swedish side
Load cells	2	Monitor the forces in the first hanger pairs on the Swedish side
LVDT	2	Monitor transverse movement of the bridge deck at the first bridge pier supports on both sides of the arch

Table 3.5: Sensor details (Wenzel, Health monitoring of bridges, 2009)

All the 24 strain gauges and 28 temperature gauges are embedded in the concrete section. In some sections both vibrating-wire and resistance strain gauges are installed side by side for instrument verification and quality control purposes.

On the whole, the sensors and data acquisition equipment appear to be operating satisfactorily and provide reasonable results. Installed strain gauges can now verify that the concrete arch is in compression and cracks developed at the top flange of the arch are now closed. Figure 3.9, for instance, shows the strains (10 minutes mean strain) at the top and bottom of arch segment S1 close to the arch base on the Swedish side. Casting of this segment was done in June 2003. It can be seen that the strains measured using the resistance strain gauge (RS) agree very well with the ones measured with the vibrating wire gauge (VW). The events on-site obviously play an important role in interpreting the results from the strain gauges. The casting of each subsequent segment, the tensioning and removal of the temporary back-stay cables and the lifting of steel deck sections can easily be followed in this diagram. Furthermore, Figure 3.9 verifies that the assembling of side span deck sections resulted in high concrete tension at the top of segment S1 and therefore one could observe cracking of the concrete roof. However, lifting of the 1450 tonnes main span deck on the 27th of July caused these crackes to close. The owner can now be sure that, when the asphalt layer is in place, the concrete arch will be fully in compression due to dead load.

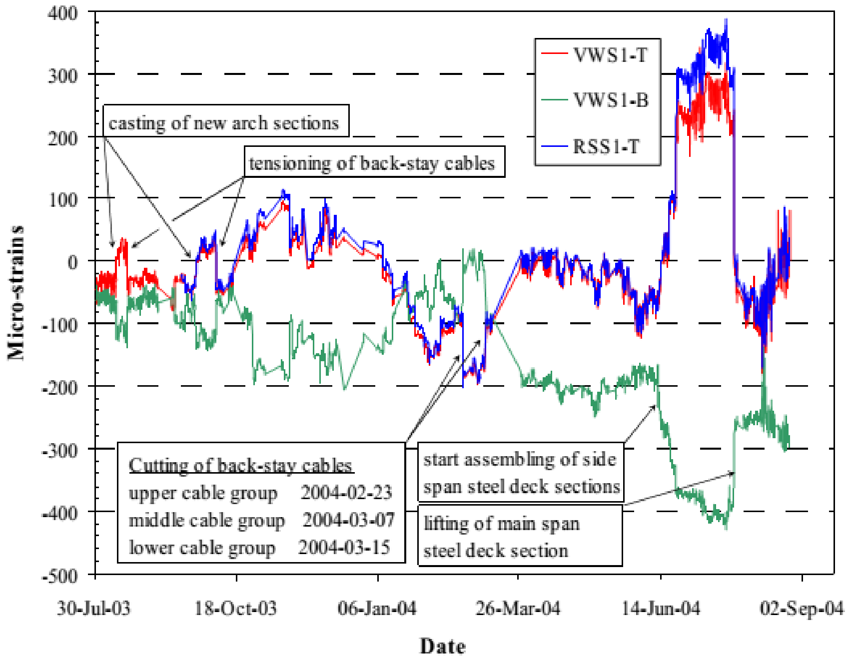


Figure 3.9: Results from the VWS1-T vibrating wire gauge ate the top of S1, VWS1-B vibrating wire gauge at the bottom of S1, RSS1-T resistance strain gauge at the top of S1

Using the readings obtained from the accelerometers, it has been possible to compare the theoretical and as-built first natural bending frequencies of the arch during different phases of its construction. The natural bending frequencies have been shown to compare closely to those of the original design in the stages prior to the completion of the bridge deck. However, measurements taken after the bridge deck was in place indicate that the bridge is stiffer in the vertical direction than that predicted in theory. This is shown by higher measured vertical natural frequencies, which was most noticeable for the first vertical mode. There was better agreement between the theoretical and the measured frequencies for the horizontal bending modes. It is possible that as the effects of creep and shrinkage in the concrete become larger with time, then the effective stiffness of the arch will decrease. However if this was the sole explanation, one would also expect the horizontal modes to show the same degree of difference between the theoretical and measured natural frequencies.

Further information can be found in several publications by *James* and *Karoumi*.

Chapter 4
Maintenance

4.1 General framework

The present chapter, in addition to the introduction of basic concepts and general information, deals with the maintenance practice in some different industrial fields where the standardization of the products, less uncertainties and smaller tolerances have permitted to develop advanced systems. References and comparisons with the civil field are illustrated as well as a proposal for recommended practice.

Maintenance is an essential part of keeping buildings and structures in an operable state, to avoid considerable damages and to protect human safety. The development of maintenance plans is of primary importance in the civil sector as well as in all the industrial sectors to reduce management costs and to optimize the performances, keeping acceptable the reliability level during the whole life of the structure. In this context inspection and monitoring become essential tools to reach the prefixed targets.

In general is possible to distinguish between different kind of maintenance:

Corrective maintenance in which the works start after a failure. This maintenance strategy should be adopted only when it is not feasible to adopt preventive measures and when the degraded state is acceptable, involving components that are not part of critical or safety systems.

Preventive maintenance in which the works start before the failure. There are different kind of preventive maintenance: *predetermined maintenance* (or scheduled maintenance) if there is a maintenance plan and the maintenance is periodically done; *condition based maintenance* (or predictive maintenance) if some component that have been identified as critical are to be checked periodically and subsequent interventions are determined by the condition of the item revealed by the inspection activity; *opportunity maintenance* when the maintenance is performed concurrently with other activities, leading to: financial saving, decreased maintenance time, reduced down-state time, less problems for the users.

Operational maintenance in which the maintenance is done during the use. This is the case of minor maintenance of equipment using procedures that not require detailed technical knowledge (ex. Inspecting, cleaning, servicing, preserving, lubricating and adjusting as required).

Interventions on existing structures, such as maintenance works, are subordinated to decision criteria. Many maintenance decisions require the evaluation of alternative solutions in terms of complex maintenance criteria such as cost, repairability, reliability and availability requirements. Possible decision criteria are briefly reviewed in the following:

Target reliability: is selected a target failure probability or the target safety level basing on different parameters like the importance of the structure, possible failure consequences, socio-economic criteria etc.

Economical consideration: are analysed expected benefits from the residual use of the structure and costs related to engineering and structural analyses, repair work, planned inspection and maintenance.

Time constrains: are to be considered several different aspects like desired and granted residual service life of the structure; mean service life of the structure; time for engineering, repair or strengthening operation; actions of building authorities.

Socio - economical and political preference.

Complex problems can be formulated as multi-criteria decision making problems in which the relative importance of maintenance criteria is often difficult to be assessed.
Differences and compatibility between codes and standards used at the design phase of the structure under consideration and actual valid standards or judgment play an important role.

4.2 Civil structures and infrastructures

The current practice for almost all residential and industrial buildings is to intervene when signs of deterioration or damage are observed (Table 4.1). In this thesis we refer to standard operating conditions, extraordinary or singular events are not considered as well as unique buildings and particular load conditions. The routine is to start with a preliminary assessment based on qualitative inspection, namely, visual observation with simple tools. The information collected can lead, if the structure is in a dangerous condition, to an immediate intervention or, if there is uncertainty, to a detailed assessment. Detailed assessment is based on quantitative inspection: examination of available documents (drawings, specification, structural calculation records, inspection and maintenance records, details of modifications, codes of practice which were used for constructing the structure, topography, subsoil conditions, groundwater level at the site etc.), material testing, measurement of actions, determination of property of the structure etc. The results of assessment shall be documented in a report which shows if the structural safety or serviceability is adequate or not. If inadequate, it can recommend construction interventions for repair, rehabilitation, upgrading, load restrictions, altering aspects of the use of the structure or implementing some form of in-service monitoring and control regime. If there are no uncertainties these interventions can be also prescribed after the qualitative inspection. It is clear that, in these situations, corrective maintenance is the only one applied.
The situation change when we refer to particular and important structures like dams, bridges, offshore installations, tunnels etc. Here is reported a brief overview.
Dams are historically the first class structure for the mandated application of inspection and monitoring and there is much to learn from this experience that can be applied to

other structures. Major dams are, for instance, equipped with transducers activated by central processor at regular intervals to measure static structural effects, such as relative or absolute displacements, strains with temperature correction, uplift pressures quantifying loads, and seepage rates. Transducers are also activated to record external influences to which the dam responds with structural effects, for example water level, structural temperature, and meteorological conditions. The variations of structural effects are evaluated for acceptability in the light of the environmental variations.
Regarding *bridges* permanent monitoring programmes have evolved in last years and today are implemented in major bridge projects. Being the important lifeline structures, modern long-span suspension bridges typically have elaborate inspection and maintenance programmes, so that significant damage and deterioration of the superstructure is likely to be picked up visually, whereas a monitoring system would require a high density of sensors to detect it. It is probably that only global changes such as changes foundation settlement, bearing failure or major defects, such as loss of main cable tension or rupture of deck element, are detectable by global monitoring procedures with a minimum of optimally located sensors. Less glamorous but possibly ultimately more beneficial developments of monitoring would be for optimal monitoring approaches for conventional short-span bridges where global response is more sensitive to defects, visual inspection is less frequent and monitoring systems can and do make a real contribution.
From the 1970s mandatory requirements for inspection have been developed for *offshore installations.* This conducted to the develop of different diagnostic systems (vibration-based diagnostics, operational modal analysis etc.) and to the elaboration of inspection and maintenance plans.
Tunnel monitoring is aimed to ensuring whether tunnel deformation is within limits in terms of stability and effects on or from adjacent structures. Monitoring of heritage and other structures during nearby tunnelling or mining is a major concern: these ground surface monitoring are temporary but feature all the technology of permanent monitoring systems. Interesting applications can be find in landslide monitoring as well.
Table 4.1 summarizes the standard inspection and maintenance practice related to different civil structures and infrastructures. It is important to note that both qualitative and quantitative inspection and both corrective and preventive maintenance are applicable to each one. The table emphasizes the most diffuse practice at present.

	Inspection		**Maintenance**	
	Qualitative	Quantitative	Corrective	Preventive
Residential and industrial buildings	X		X	
Dams		X		X
Long-span bridges	X	X		X
Short-span bridges	X		X	
Offshore installations		X		X
Tunnel excavations		X	X	

Table 4. 1: Inspection and maintenance practice for civil structures and infrastructures

4.3 Automotive industry

Scheduled inspection and maintenance are expected for all vehicles. Depending on the vehicle type, year of fabrication, driving conditions etc. manufacturers develop a program which indicate the maximum interval between two following inspections and maintenance. The interval is usually expressed in terms of time or distance travelled. Common car maintenance tasks are check/replace the engine oil and replace oil and fuel filters, inspect tires for pressure and wear, tire balancing and rotation, check or flush fluids (brake, transmission, power steering), check all lights, test electronics, inspect or replace spark plugs, air filter, timing belt etc.

In modern vehicles electronics controls most of the functions: embedded software takes care of the vehicle by constantly checking thousands of sensor signals. The latest applications use Automatic Vehicle Monitoring (AVM) systems that continuously measure, monitor and report the status of critical systems and components so maintenance issues can be identified and corrected before they become failures. These systems find application especially in public transportation networks to locate and track mobile vehicles.

4.4 Aerospace industry

Aircraft maintenance checks are periodic inspections that have to be done in all commercial/civil aircrafts after a certain amount of time or usage; military aircraft normally follow specific maintenance programmes which may or may not be similar to those of commercial/civil operators. Airlines and other commercial operators of large or turbine-powered aircraft follow a continuous inspection program approved by the designated organization.

The lowest-level maintenance event is the pre-flight check that precedes every flight and involves an inspection of the aircraft by the cockpit crew and, if necessary, by mechanics. This check for visible external damage or leaks lasts between 15 and 60 minutes, depending on the aircraft type (Table 4.2). The next maintenance event in the hierarchy is the ramp check, in which mechanics test individual functions of the aircraft, inspect the tires and brakes and replenish the oil and hydraulic fluids. A visual inspection of the aircraft is also carried out, both externally and in the cabin.

Detailed inspections, denoted as checks, are significantly more labor-intesive. Four different checks are defined: A and B checks are lighter checks, while C and D are considered heavier checks (Table 4.2).

A check is performed approximately every 125 flight hours or 200–400 cycles. It needs about 20–50 man-hours and is usually performed overnight at an airport gate or hangar. The actual occurrence of this check varies by aircraft type, the cycle count (take off and landing is considered an aircraft "cycle"), or the number of hours flown since the last check. The occurrence can be delayed by the airline if certain predetermined conditions are met.

B check is performed approximately every 4–6 months. It needs about 150 man-hours and is usually performed within 1–3 days at an airport hangar. A similar occurrence schedule applies to the B check as to the A check. B checks may be incorporated into successive A checks.

C check is performed approximately every 20–24 months or a specific amount of actual flight hours (FH) or as defined by the manufacturer. This maintenance check is much more extensive than a B check, requiring a large majority of the aircraft's components to be inspected. This check puts the aircraft out of service and until it is completed, the aircraft must not leave the maintenance site. It also requires more space than A and B checks—usually a hangar at a maintenance base. The time needed to complete such a check is generally 1–2 weeks and the effort involved can require up to 6000 man-hours. The schedule of occurrence has many factors and components as has been described, and thus varies by aircraft category and type.

D check is by far the most comprehensive and demanding check for an airplane. It is also known as a "heavy maintenance visit" (HMV). This check occurs approximately every 6 years. It is a check that, more or less, takes the entire airplane apart for inspection and overhaul. Also, if required, the paint may need to be completely removed for further inspection on the fuselage metal skin. Such a check can usually demand up to 50,000 man-hours and it can generally take up to 2 months to complete, depending on the aircraft and the number of technicians involved. It also requires the most space of all maintenance checks, and as such must be performed at a suitable maintenance base. It is also by far the most expensive maintenance check of all, with total costs for a single visit ending up well within the million-dollar range. Because of the nature and the cost of such a check, most airlines — especially those with a large fleet — have to plan D checks for their aircraft years in advance. On average, a commercial aircraft undergoes 2–3 D checks before it is retired.

Check	Time interval	Time consumption
Pre-flight	Every flight	15-60 minutes
A	125 flight hours/200-400 cycles	20-50 hours
B	4-6 months	150 hours/1-3 days
C	20-24 months	6000 hours/1-2 weeks
D	6 years	50000 hours/2 months

Table 4.2: Aircraft maintenance checks

4.5 Marine industry

Planned Maintenance System (PMS) has been developed to provide ships and applicable shore stations with a simple and standard means for planning, scheduling, controlling, and performing maintenance on all shipboard systems and equipment, according to class/classification society requirements. Its objective is to maintain equipment within specifications through preventive maintenance, identify and correcting potential problems before the equipment or system becomes inoperable. PMS provides:

- Comprehensive procedures for planned maintenance of systems and equipment.
- Minimum requirements for planned maintenance.

- Scheduling and control of the performance of tasks.
- Description of the methods, materials, tools, and personnel needed for maintenance.
- Detection of hidden failures or malfunctions.
- Test procedures to determine material readiness.
- Assessment procedures to determine material condition of equipment.

The planning and scheduling of the maintenance, as well as its documentation, must be made according to a system that is approved by classification societies. It's interesting to observe that there are items of equipment in the fleet which do not have PMS coverage. Reasons for this are numerous and include: insufficient funds, determination that planned maintenance is not required, equipment/systems that are planned for disposal and not economical for PMS development etc.

Many helpful software have been developed and are now available on the market to permit the shipping companies to carry out maintenance jobs in the easiest and most effective way possible. Programs today do not contain only maintenance, they offer almost entirely what is needed on board the ship or inside and outside the vessel.

Studies have shown that the use of planned maintenance systems, significantly decreased breakdowns and damage to ships.

4.6 Recommended practice

In this context is possible to define a desirable common methodology applicable to all civil buildings and infrastructures. The aforementioned maintenance practice highlighted that currently most residential buildings, industrial buildings, and short-span bridges, which represent most of the existing structures, do not have inspection and maintenance plans. In others industrial sectors, by numerous factors, the situation is considerable different. As mentioned before, scheduled inspection and maintenance are precisely defined or real time monitoring systems are expected.

In order to develop a better practice all new and existing constructions may be classified according to the consequences in the event of global or partial failure and basing on it regular inspection intervals may be recommended. Usually three classes are defined (VDI 6200, 2010) as shown in Table 4.3:

- CC 1 : Low consequences
- CC 2 : Medium consequences
- CC 3 : High consequences

Damage to life and health are the main assessment criteria to determine the classes. Typical structures are included in Table 4.3. A higher safety level is required for the higher consequence class structure and consequently more intensive monitoring and inspection procedures shall be applied.

Conse-quences class	Description	Building types and exposed construction elements	Building examples
CC1	Low consequences (material damage and financial loss, low environmental damage, risks to individual persons)	Robust and generally uncritical buildings with span widths less than 6 m Buildings for only temporary use by individual people	Detached residential houses, apartment houses Agricultural buildings,
CC2	Medium consequences (damage to life and health for many persons, serious environmental damage)	Construction of over 60 m in height Buildings and construction elements with span widths greater than 12 m and/or cantilevers greater than 6 m as well as large-area roofs Exposed construction elements in buildings insofar as they constitute a special risk potential	High rise buildings, television towers Office buildings, industrial and commercial buildings, power stations, production plants, train stations and airport buildings, indoor swimming pools, shopping malls, museums, hospitals, theatres, schools, discotheques, sports halls of all kinds Large canopy roofs, suspended balconies, suspended facades, domes
CC3	High Consequences (damage to life and health for a lot of persons, major environmental damage)	In particular: Assembly places for more than 5000 persons	Stadiums, bridges, congress halls, multi-purpose arenas

Table 4.3: Consequences classes for buildings according to VDI 6200 (2010)

In Table 4.3, column 3 " Buildings types and exposed construction elements" are specified illustrative criteria. A different classification, based on appropriate parameters, can be made.

Actually the bridges are not included in Table 4.3 since they are treated in Germany in DIN 1076 where simple inspections (proof test, bearing, drainage, cracks, deformation, corrosion etc.) every 3 years and main tests (foundation, examination of inaccessible parts of buildings, concrete cover, material analysis etc.) every 6 years are proposed. According with these inspection intervals and with the table reported in the following bridges are proposed to be included in CC3.

Depending on the consequences class, the regular inspection intervals given in Table 4.4 are recommended (VDI 6200, 2010):

Consequences class	Visual Check	Inspection (engineer)	Verification (expert)
CC1	3 to 5 years	as required	
CC2	2 to 3 years	4 to 5 years	12 to 15 years
CC3	1 to 2 years	2 to 3 years	6 to 9 years

Table 4.4: Intervals for periodic inspections (VDI 6200, 2010)

Visual check (or surveillance) by the owner/authorised representative includes the inspection of the building for obvious defects or damages and the documentation thereof. The inspection by an engineer (expert) is a visual inspection of bearing structure. It is usually carry out without the use of technical test equipment. The verification by a special expert is the thorough inspection of all the main load bearing elements and safety analysis based on structural calculations (may be necessary to take material samples to determinate the remaining strength or rigidities).

The proposed intervals represent guidance values, different indications can be find in others standards (e.g. GOST R 53778). It is important to note that the intervals to be selected in the actual case depend on a wide range of individual building characteristics, for instance the type of load-bearing structure, its robustness, age and state of preservation, its usage and the environmental conditions.

Basing on the regular inspections' result can be planned further investigations, maintenance interventions or extraordinary interventions.

Due to the large economic effort needed to keep the existing and future infrastructure systems in efficient and safe conditions, inspection, monitoring and maintenance should be planned according to cost optimization criteria, in order to minimize the life cycle cost of the building keeping the performances above a target level.

Chapter 5
Structural reliability

5.1 General framework

A structure is usually required to have a satisfactory performance in the expected lifetime: it does not collapse or becomes unsafe and that it fulfils certain functional requirements. Reliability of structural systems can be defined as the probability that the structure under consideration has a proper performance throughout its lifetime. Reliability methods are used to estimate the probability of failure or, fixed a desired maximum probability of failure, to conduct the verifications. The methods that can be used are the probabilistic methods and the partial factor method. It is important to note that the estimated reliability should be considered as a nominal measure of the reliability useful to make comparisons between comparable structures and take decisions and not as an absolute number. The aforementioned methods, based on probabilistic concepts of structural reliability and available experience, allow to account for uncertainties that affect the structural performance and that can never be entirely eliminated in a very effective way, especially if compared to the deterministic methods widely used in the past. According to the Eurocodes, consistent with most modern codes, the partial factor method and probabilistic methods only can be applied in the design of structures. The partial factor method, also called semi-probabilistic or level I method, is by far the most used in the practice due to its simplicity. Modern codes supply fixed calibrated factors to be applied to the calculated representative value of actions and resistances without further knowledge needed. The probabilistic methods (level II and level III methods) provide an effective tool for design, to evaluate the probability of failure (or the reliability index) and to conduct the risk evaluation, particularly important in the assessment of existing buildings and bridges. Nevertheless their application is more complex since it requires more experience and appropriate statistical data, therefore are normally used only in particular situations and as scientific bases of the partial factor method.

In order to be able to estimate the reliability using probabilistic concepts the main steps in a reliability analysis can be introduced (*Sørensen*):

1. Select a target reliability level (§5.2)
2. Identify the significant failure modes of the structure (typical failure modes to be considered are yielding, local and global buckling, fatigue and excessive deformation)
3. Formulate failure functions (limit state functions) corresponding to each component in the failure modes.

4. Identify the stochastic variables and the deterministic parameters in the failure functions. Further specify the distribution types and statistical parameters for the stochastic variables and the dependencies between them.
5. Estimate the reliability of each failure mode.
6. In a design process change the design if the reliabilities do not meet the target reliabilities. In a reliability analysis the reliability is compared with the target reliability.
7. Evaluate the reliability result by performing sensitivity analyses

The performance requirements are usually expressed in terms of maximum probability of failure p_f or, equivalently, in terms of minimum reliability index β. The numeric values of the reliability are often described on the basis of the reliability index β related, in the Level II procedures, to the probability of failure P_F by:

$$\beta = -\Phi^{-1}\,(p_F) \tag{5.1}$$

where Φ^{-1} is the inverse standardized normal distribution. The relationship between the failure probability and the reliability index is shown in Table 5.1:

P_F	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-5}	10^{-6}	10^{-7}
β	1.3	2.3	3.1	3.7	4.2	4.7	5.2

Table 5.1: Probability of failure and reliability index

In case the limit state function is described by a normal distribution, β can be calculated as:

$$\beta = \frac{\mu}{\sigma} \tag{5.2}$$

where μ and σ are the mean value and the standard deviation of the considered function.

The Eurocodes, in accordance to the majority of design codes, are based on the concept of the limit states, described by deterministic functions that depend on a set of basic variables and that separate desired states of the structure from undesired states. In mathematical terms:

$$g\,(F_d, X_d, a_d, \vartheta_d) = 0 \tag{5.3}$$

Equation (5.3) is called the limit state equation, F_d, X_d, a_d, θ_d are the design values of action, material properties, geometrical quantities and variables which account for model uncertainties. Another type of uncertainty which is not covered by these methods are gross errors or human errors. The limit state function can be also formulated by separating the resistance R and load effect E as follow:

$$g\,(F_d, X_d, a_d, \vartheta_d) = R - E \tag{5.4}$$

and the desired state is identified by the inequality:

$$g\ (F_d, X_d, a_d, \vartheta_d) > 0 \tag{5.5}$$

This verification shall be conducted for all relevant ultimate limit states (loss of equilibrium, attainment of the maximum resistance capacity of sections, excessive deformations, transformation into a mechanism, instability) and serviceability limit states (local damage, unacceptable deformations, excessive vibrations) identifying, for each of them, the relevant basic variables.

Generally, methods to measure the reliability of a structure can be divided in four groups (*Madsen* et al.):

- *Level I methods*: The uncertain parameters are modelled by one characteristic value, as for example in codes based on the partial safety factor concept.
- *Level II methods*: The uncertain parameters are modelled by the mean values and the standard deviations, and by the correlation coefficients between the stochastic variables. The stochastic variables are implicitly assumed to be normally distributed. The reliability index method is an example of a level II method.
- *Level III methods*: The uncertain quantities are modelled by their joint distribution functions. The probability of failure is estimated as a measure of the reliability.
- *Level IV methods*: In these methods the consequences (cost) of failure are also taken into account and the risk (consequence multiplied by the probability of failure) is used as a measure of the reliability. In this way different designs can be compared on an economic basis taking into account uncertainty, costs and benefits.

Level I methods can be calibrated using level II methods, level II methods can be calibrated using level III methods, etc.

5.2 Target reliability levels

An accurate determination of performance requirements is of extreme importance especially if people may be killed or injured as a result of collapse. In many cases, when considering the requirements for stability and collapse of a structure, the specification of the failure is not very complicated. In many other cases, in particular when dealing with various requirements of occupants' comfort, appearance and characteristics of the environment, the appropriate definitions of failure are dependent on several vagueness and inaccuracies. The transformation of these occupants' requirements into appropriate technical quantities and precise criteria is very hard and often leads to considerably different conditions. In the following the term failure is being used in a very general sense denoting simply any undesirable state of a structure which is unambiguously given by structural conditions.

In general there is a substantial difference between the notational probability of failure in the design procedure and the actual failure frequency (to a considerable extent is due to human error). For this reason, target levels for reliability are often based on calibration. Using calibrated reliability values, one should keep in mind that they are related to a specific set of structural and probabilistic models.

In EN 1990, ISO 2394, and ISO 13822 basic recommendations concerning a required reliability level for new structures are often formulated in terms of the reliability index β related to a certain design working life.

ISO 2394 taking the overall individual lethal accident rate of 10^{-4} per year as a reference (resulting from other activities), assumes an acceptable lethal accident rate of 10^{-6} per year, which corresponds to a reliability index $\beta_{t,1}$ = 4.7 (Table 5.1). The reliability index for a period of n years may be then calculated from the following approximate equation

$$\Phi(\beta_{t,n}) = [\Phi(\beta_{t,1})]^n \qquad (5.6)$$

where Φ denotes the distribution function of a standardised normal distribution. From eq. (5.6) the approximate value $\beta_{t,50}$ = 3,8 may be obtained from $\beta_{t,1}$ = 4.7. These values correspond to the target reliability indexes accepted in EN 1990 for the ultimate limit state. It should be emphasized that both values $\beta_{t,50}$ = 3.8 and $\beta_{t,1}$ = 4.7 correspond to the same reliability level, but to different reference periods considered for the assessment of the design values of some actions. Figure 5.1 shows the variation of β_n with β_1 for *n* = 5, 25, 50 and 100.

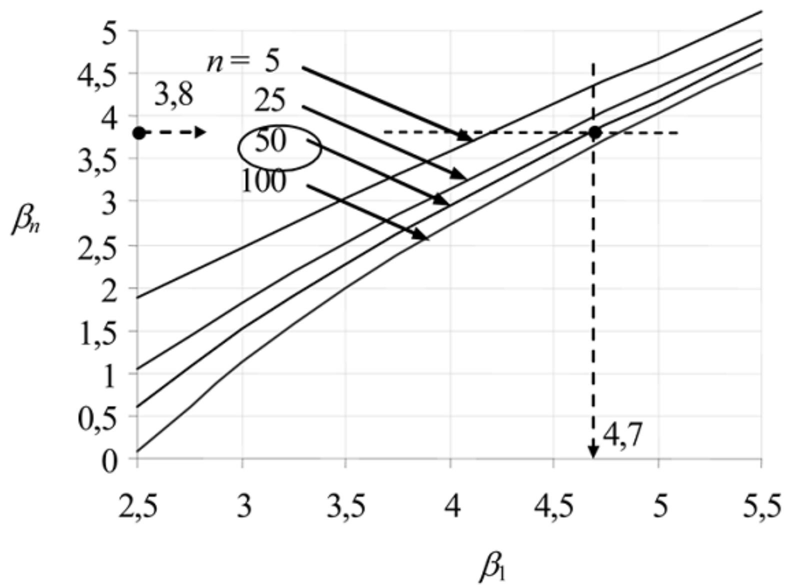


Figure 5.1: Variation of βn with β1 for n = 5, 25, 50 and 100 (Diamantidis, Holickỳ)

In Table 5.2 values for the target reliability index, calibrated on the life time, are given from ISO 2394. The indicated values depend on a balance between consequences of failure and the costs of safety measures. From an economic point of view the objective is to minimize the total working-life cost.

Relative costs of safety measures	Consequences of failure			
	small	some	moderate	great
High	0	1.5	2.3	3.1
Moderate	1.3	2.3	3.1	3.8
Low	2.3	3.1	3.8	4.3

Table 5.2: Target reliability index (life-time) in accordance with ISO 2394 (2015)

In Eurocode EN 1990 three consequences classes (CC) are established by considering the consequences of failure or malfunction of the structure: CC3 = high consequences; CC2 = medium consequences; CC1 = low consequences. Each consequences class is associated to a reliability class (RC1, RC2 and RC3) and minimum recommended values for the reliability index, associated with these classes, are provided (Table 5.3):

Consequence Class	Consequences for loss of human life, economic, social and environmental	Minimum values for β		Example of buildings and civil engineering works
		T = 1 year	T = 50 years	
RC3 – High	High	5.2	4.3	Bridges, public buildings
RC2 – Normal	Medium	4.7	3.8	Residential and offices
RC1 – Low	Low	4.2	3.3	Agricultural buildings

Table 5.3: Recommended minimum values for reliability index β (ultimate limit states) in accordance with EN 1990 (2002)

Note that a design using the partial factors given by the Eurocodes is considered generally to lead to a structure with a β value greater than 3.8 for a 50 year reference period. One way to achieve reliability differentiation is by applying a multiplication factor K_F to the partial factor for actions. The multiplication factor, equal to 0.9, 1.0, and 1.1 for reliability classes RC1, RC2, and RC3 respectively, is to be in fundamental combinations for persistent design situations. Reliability differentiation may also be applied through the partial factors on resistance γ_M. However, this is not normally used.

Similar recommendation is provided by JCSS (*Assessment of existing structures*) where target reliability indexes are related to both the consequences and to the relative costs of safety measures related to one year reference period and ultimate limit state.

ISO 13822 for the assessment of existing structures indicates four target reliability levels for different consequences of failure (ultimate limit states): small consequences: 2.3, some: 3.1, moderate 3.8 , high 4.3. The related reference period is “ a minimum standard period for safety (e.g. 50 years)”.

In general ISO 2394 and JCSS seem to provide a more appropriate reliability differentiation for existing structures than EN 1990 and ISO 13822 since costs of safety measures are taken into account. A clear link between the remaining working life and the target reliability level is not apparent from EN 1990 and JCSS and thus it may not be obvious what target reliability should be used for different working life periods. Recommendations on the target reliability levels are also provided in several national standards.

For existing structures is still unclear what reference value may be assumed. Certainly it is uneconomical to require that all existing structures comply with the target reliability levels for new structures. Normally a shorter design life is employed that results in a decrease of the representative values for the variable loads. Lower reliability levels can be used if adequately justified. For instance, *Steenbergen* et al. prove that two types of β values can be derived. First the level below which the structure is unfit for use. If this safety level is not reached the structure has to be closed and to be adapted. Secondly the safety level for repair of existing structures. Based on economic optimisation can proved that the life time reliability index for existing structures could be lowered by $\Delta\beta$ = 1.5 with respect to new structures (Table 5.4). Below this level, $\beta_u = \beta_n - \Delta\beta$, the existing structure is unfit for use. For repair a safety level comprised between β_n (new structures) and β_u (unfit for use) may be assumed. The abovementioned authors suggest $\beta_r = \beta_n - 0.5$ (Table 5.4). The cost optimisation is aimed at finding the optimum decision from the perspective of an owner of the structure. However, society commonly establishes limits at human safety. Based on the concept on individual risk and fixing the maximum acceptable probability to become the victim of structural failing, minimum reliability indexes for t_{ref} = 1, 15, 30 and 50 years can be derived (*Steenbergen* et al., Table 5.5).

Consequence Class	β_u unfit for use (T = 50 years)	β_r repair (T = 50 years)
CC3	4.3 – 1.5 = 2.8	4.3 – 0.5 = 3.8
CC2	3.8 – 1.5 = 2.3	3.8 – 0.5 = 3.3
CC1	3.3 – 1.5 = 1.8	3.3 – 0.5 = 2.8

Table 5.4: Reliability index for existing structures from economic optimisation (Steenbergen et al, 2010)

Consequence Class	Reference period			
	1 year	15 years	30 years	50 years
CC3	3.9	3.2	3.0	2.8
CC2	3.6	2.8	2.5	2.3
CC1	3.1	2.2	1.9	1.6

Table 5.5: Reliability index resulting from individual risk (Steenbergen et al, 2015)

Target values herein reported result from actual codes and specific studies carried out by the authors. Different reference values can be found in other publications or related to specific cases study.

5.3 Probabilistic methods

In the probabilistic methods the load effect E and the resistance R are generally random variables represented by mathematical functions and the probability p_F of the event $E > R$ is used to measure the reliability level of the element with regard to the considered limit state:

$$p_F = Prob(R \leq E) \tag{5.7}$$

The verification is satisfied if:

$$p_F \leq p_{F\ acceptable} \tag{5.8}$$

or, equivalently:

$$\beta \geq \beta_{target} \tag{5.9}$$

being $p_{F\ acceptable}$ or β_{target} provided by reference code (§5.2).
Eq. 5.7 can be rewritten in general terms referring to the limit state function as defined in §5.1:

$$p_F = P(g(\boldsymbol{X}) \leq 0) = \int_{g(X)\leq 0} \varphi(\boldsymbol{X}) d\boldsymbol{X} \tag{5.10}$$

where $\boldsymbol{X}$ is a vector of basic variables, $g(\boldsymbol{X})$ is the limit state function, $\varphi(\boldsymbol{X})$ is the joint probability density function of the vector of all the basic variables and $g(\boldsymbol{X}) \leq 0$ denotes the failure domain. However, such a function may be difficult to find or may be very complicated. The integral in equation (5.10) can also be written as multiple integral:

$$p_F = P(g(\boldsymbol{X}) \leq 0) = \int_{g(X)\leq 0} \varphi_{x1}(x_1)\varphi_{x2}(x_2) \dots \varphi_{xn}(x_n) dx_1 dx_2 \dots dx_n \tag{5.11}$$

being $x_1, x_2, \dots, x_n$ the realisations of the variables $X_1, X_2, \dots, X_n$.
In some special cases the integration indicated in equations (5.10) and (5.11) can be done analytically, in some other cases, when the number of basic variables is small (up to 5), various type of numerical integration may be effectively applied. In general (see ISO 2394), the failure probability p_F may be computed using:

- Exact analytical integration
- Numerical integration methods
- Approximate analytical methods (FORM, SORM, methods of moments)
- Simulation methods
- A combination of these methods

Exact analytical methods can be applied only in exceptional academic cases. Numerical integration can be applied much more frequently. The most popular computational procedures to determine the failure probability constitute approximate analytical methods. In complicated cases simulation methods or their combination with approximate analytical methods are commonly applied. Most of the commercially available software products include approximate analytical methods and various type of simulation methods.

In the following a calculation example that can be easily solved is reported. The example is extracted from *Handbook 2 – Reliability Backgrounds (Implementation of Eurocodes)*. Assume that both basic variables, the action effect *E* and the resistance *R* have a normal distribution. Then also the difference:

$$Z = R - E \tag{5.12}$$

called the reliability margin, has normal distribution with parameters:

$$\mu_Z = \mu_R - \mu_E \tag{5.13}$$
$$\sigma_Z^2 = \sigma_R^2 + \sigma_E^2 + 2\rho_{RE}\sigma_R^2\,\sigma_E^2 \tag{5.14}$$

where ρ_{RE} is the coefficient of correlation of *R* and *E*. It is often assumed that *R* and *E* are mutually independent and $\rho_{RE} = 0$. Equation (5.7) for the probability of failure p_F can be now modified to:

$$p_F = P(R < E) = \mathrm{P}(\mathrm{Z} < 0) = \Phi_Z(0) \tag{5.15}$$

and the whole problem is reduced to determine the distribution function $\Phi_Z(z)$ for $z = 0$, which leads to the probabilities of the safety margin *Z* being negative. The distribution function $\Phi_Z(0)$ is usually determined by transformation of the variable *Z* to standardised random variable *U*. The value u_0 corresponding to the value g = 0 is:

$$u_0 = (0 - \mu_Z)/\sigma_Z = -\mu_Z/\sigma_Z \tag{5.16}$$

and the probability of failure is given as

$$p_F = P(R < E) = \Phi_Z(0) = \Phi_U(u_0) \tag{5.17}$$

The probability density function $\varphi_Z(z)$ of the safety margin *Z* is shown in Figure 5.2, where the grey area under the curve $\varphi_Z(z)$ corresponds to the failure probability p_F.
Assuming that *Z* has a normal distribution, the reliability index β is calculated by eq. (5.2). Numerically, considering the resistance *R* and the load effect *E* mutually independent random variables ($\rho_{RE} = 0$) having a normal distribution and described respectively by μ_R = 100, σ_R = 10 (v = 0,10), μ_E = 80, σ_E = 8 (all expressed in dimensionless units). μ is the mean, σ is the standard deviation and v is the coefficient of variation. It follows from equations (5.13), (5.14) and (5.2):

$$\mu_Z = 100 - 80 = 20$$
$$\sigma_Z^2 = 10^2 + 8^2 = 12.81^2$$
$$\beta = 20/12.81 = 1.56$$

and the probability of failure follows from relation (5.17):

$$p_F = P(Z < 0) = \Phi_U(-1.56) = 0.059$$

If the variables *E* and *R* are not normal, the distribution of the safety margin *G* is not normal either and the above described procedure has to be modified. In a general case, numerical integration or transformation of both variables into variables with normal distribution can be used.

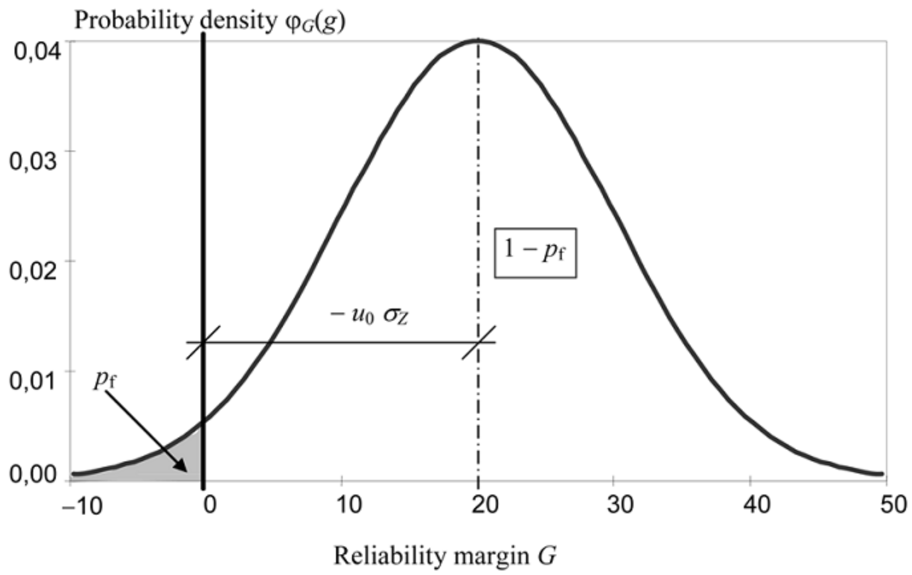


Figure 5.2: Distribution of the reliability margin G (Diamantidis, Holickỳ, 2012)

A brief overview about the computational methods that can be used to calculate the probability of failure is now reported. Only general principles and main characteristics are provided. Further information can be find in specific texts.

5.3.1 FORM and SORM

The FORM (First Order Reliability Method) is one of the basic and very efficient reliability methods: it is used by a number of software products and it is also mentioned in EN 1990. To obtain the reliability index the following steps have to be followed:

- Define the limit state function *g(**X**)* = 0 and characterize statistically the basic variables $X_1, X_2, \ldots X_n$.
- Transform the set of basic variables into a set of independent standard normal variables (with zero mean value and unit standard deviation). Hence the basic variable space (including the limit state function) is transformed into a standard normal space (Fig. 5.3)
- The failure surface is approximated by a tangent hyperplane at the design point, which is the point closest to the origin. It is found by iteration
- The failure probability p_F is given by $p_F = \Phi\,(-\beta)$, where β is the distance from the origin to the design point.

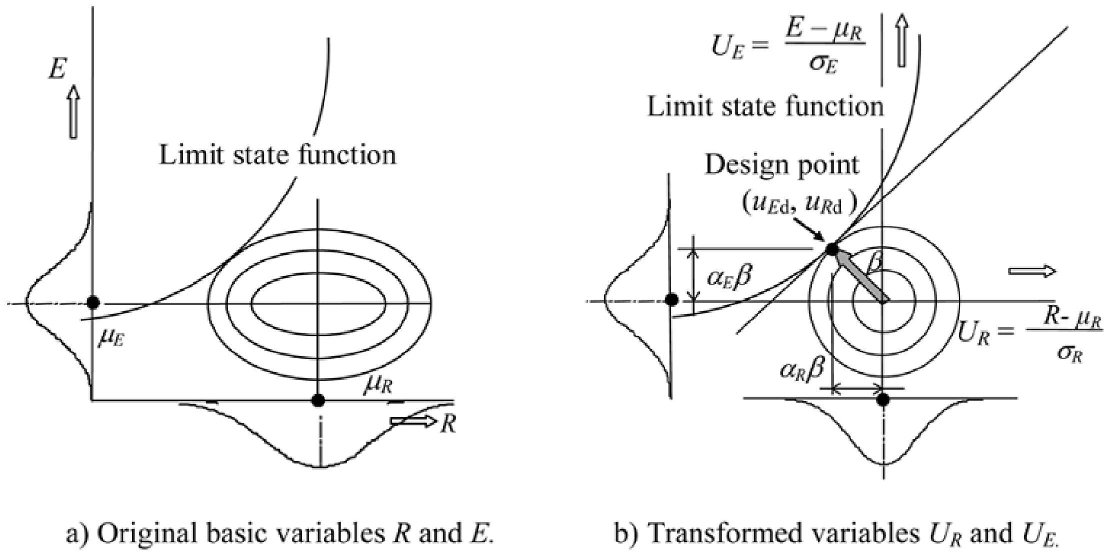


Figure 5.3: First Order Reliability Method (Diamantidis, Holickỳ, 2012)

The analytical method may be refined by approximating the failure surface by a quadratic surface in the design point. Such a method is called the Second Order Reliability Method (SORM). Experience shows that FORM/SORM estimates are adequate for a wide range of problems. However, these approximate methods have the disadvantage of not being quantified by error estimates, except for few special cases. Simulation (§5.3.2) may be used to verify FORM/SORM results. When using FORM/SORM, attention should be given to the ordering of dependent random variables and the choice of initial points for the search algorithm. Not least, the results for the design point should be assessed to ensure that they do not contradict physical reasoning.

5.3.2 Simulation methods

All the simulation methods are based on the generation of random variables of given distribution using available software products. Simulation methods can be divided into:

- Zero-one indicator based methods, which are non-analytical, and operate in the original space of variables ***X***
- Conditional expectation methods which are semi-analytical methods

The first group includes the Direct Monte Carlo simulation (when the original probability density is applied), the method of Importance Sampling (when the original probability density close to the design point is applied) and the Adaptive Sampling (updated importance sampling). The second group consists of Directional Simulation (suitable for unions of events) and the Axis Orthogonal Simulation (suitable for intersection of events). The Direct or Crude Monte Carlo method is a very simple simulation method in which the experiment is repeated many times and the probability of failure p_F is estimated from the fraction of trials leading to failure divide by the total number of trials. This method is not likely to be of use in practical problems because of the large number of trials required in order to estimate with a certain degree of confidence the failure probability. Note that the number of trials increases as the failure probability decreases. Simple rules may be found (e.g. if the expected failure probability is about 10^{-5} the number of trials should be about two orders greater, thus greater than about 10^{7}). The objective of more advanced simulation methods, currently used, is to reduce the number of trials needed. The method of the importance sampling introduces a sampling function, whose choice would depend on a priori information available, such as the co-ordinates of the design point and/or any estimates of the failure probability. In this way the success rate is improved compared to Direct Monte Carlo. Importance Sampling is often used following an initial FORM/SORM analysis. A variant of this method is Adaptive Sampling, in which the sampling density is updated as the simulation proceeds. Importance Sampling could be performed in basic variable or standard normal space, depending on the problem and the form of prior information.

A powerful method belonging to the second category is Directional Simulation. It achieves variance reduction using conditional expectation in the standard normal space, where a special result applies pertaining to the probability bounded by a hypersphere centred at the origin. Its efficiency lies in that each random trials generates precise information on where the boundary between safety and failure lies. However, the method does generally require some iterative calculations. It is particularly suited to problems where it is difficult to identify 'important regions'.

The two methods outlined above have also been used in combination, which indicates that when simulation is chosen as the basic approach for reliability assessment, there is scope to adapt the detailed methodology to suit the particular problem in hand.

5.4 Partial factor method

A more practical procedure is to use the partial factor method in which the calculation is conducted comparing the design values of actions and resistance, noted E_d and R_d respectively:

$$E_d < R_d \qquad (5.18)$$

These values are calculated multiplying the respective representative values for the partial safety factors γ such as illustrated in the following. The method is semi-probabilistic since statistics are applied in the evaluation of the input data, the formulation of assessment criteria and the determination of load and resistance factors. However, the designer conducts a deterministic verification and does not have relationships or procedures to evaluate the actual risk or reserve in carrying capacity.

In most cases the design value E_d of an action E can be approximated as:

$$E_d = \gamma_E E_{rep} \qquad (5.19)$$

where E_{rep} is the representative value of the action E (as taken into account in the relevant combination of actions) and γ_E is the partial factor. The characteristic value G_k is the representative value of permanent actions. The characteristic Q_k, the combination $\psi_0 Q_k$, the frequent $\psi_1 Q_k$ and the quasi permanent $\psi_2 Q_k$ values are the considered for the variable actions. $\gamma_E = \gamma_{Ed}\, \gamma_e$; γ_{Ed} is the partial factor for uncertainties in modelling the actions or their effect and γ_e partial factor for the variable load. The design value E_d of the load effect E can be expressed as:

$$E_d = E\left(\gamma_E E_{rep}; a_d\right) \tag{5.20}$$

In case of strong non-linearity these approximations may be unsafe and the general equations should be used:

$$E_d = \gamma_e E_{rep} \tag{5.21}$$

$$E_d = \gamma_{Ed} E\left(\gamma_e E_{rep}; a_d\right) \tag{5.22}$$

The design value X_d of a material or product property X is determined from the characteristic value X_k, the partial factor γ_M and, eventually, a conversion factor η:

$$X_d = \eta X_k / \gamma_M \tag{5.23}$$

where $\gamma_M = \gamma_{Rd}\, \gamma_m$; $\gamma_{Rd} = \gamma_{Rd1}\, \gamma_{Rd2}$, γ_{Rd1} is the partial factor accounting for model uncertainty, γ_{Rd2} partial factor accounting for geometrical uncertainties, γ_m reliability-based partial factor accounting for variability of the material and statistical uncertainty. According to EN 1990, a conversion factor η should be applied where it is necessary to convert the test results into values which can be assumed to represent the behaviour of the material or product in the structure or the ground.
The design value of R_d of the resistance R depends on the material properties X and the geometrical dimensions a:

$$R_d = R(\eta X_k / \gamma_M; a_d) \tag{5.24}$$

Similarly to the load effect, when a linear relationship between the resistance and the basic variables cannot be assumed, the general equations should be used:

$$X_d = \eta X_k / \gamma_m \tag{5.25}$$

$$R_d = R(\eta X_k / \gamma_m; a_d) / \gamma_{Rd} \tag{5.26}$$

The design values of geometrical quantities a are generally represented by nominal values:

$$a_d = a_{nom} \tag{5.27}$$

Note that the partial factors derived here are intended to be applied in conjunction with the load combination rules given in EN 1990 that is consistent with many different standards.

The calculation of the partial factors requires the definition of the design values and the characteristic values of the actions, material and product properties, geometrical data and model uncertainties. The Eurocodes recommend to use the First Order Reliability Method (FORM)(Level II), a simple and effective reliability method, to calculate the design values R_d and E_d and thus to the probabilistic calibration procedure for partial factors. The characteristic values can be calculated using the probability function once defined the corresponding fractile.
The design value of action effects E_d and resistances R_d should be defined such that the probability of having a more unfavourable value is as follows:

$$P(R \leq R_d) = \Phi(-\alpha_R \beta) \tag{5.28}$$
$$P(E > E_d) = \Phi(\alpha_E \beta) \tag{5.29}$$

where α_R and α_E are the sensitivity factors, β is the reliability index, Φ is the cumulative density function of the standard normal distribution.
If R and E are independent Gaussian random variables the joint probability distribution function can be represented by a concentric circle corresponding to different levels of the probability density in the space of normalized variables R/σ_R and E/σ_E. The design point is defined as the point of the limit state surface, approximated at a chosen given point by a tangent hyperplane, closest to the average point (μ_R, μ_E) as shown in Fig. 5.4:

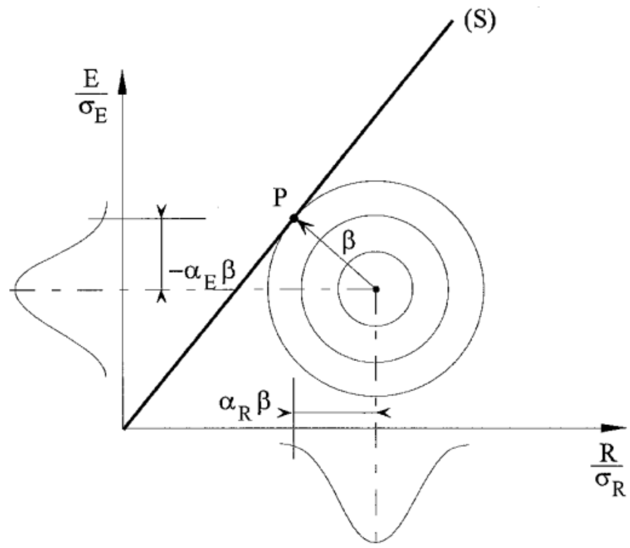


Figure 5.4: FORM design point (EN 1990, 2002)

This figure, based on the assumption that R and E are two independent Gaussian random variables, has a general meaning since the actual distribution of both the basic variables can be transformed at a given point into a normal distribution. This is one of the main steps of the FORM method. The equations (5.28) and (5.29) become:

$$R_d = \mu_R - \alpha_R \beta \sigma_R \tag{5.30}$$
$$E_d = \mu_E - \alpha_E \beta \sigma_E \tag{5.31}$$

It follows from Fig. 5.4 that the sensitivity factors α_R and α_E, direction cosines of the failure boundary, can be written as:

$$\alpha_R = \sigma_R / \sqrt{\sigma_R^2 + \sigma_E^2} \tag{5.32}$$
$$\alpha_E = -\sigma_E / \sqrt{\sigma_R^2 + \sigma_E^2} \tag{5.33}$$

In order to derive practical design rules for a wide-range of civil engineering structures, the values of α_R and α_E can be fixed to the following values:

$$\alpha_R = 0.8 \tag{5.34}$$
$$\alpha_E = -0.7 \tag{5.35}$$

The validity of such an approximation is delimited in EN 1990 by means of a condition for the ratio of the standard deviations in the form:

$$0.16 < \sigma_E / \sigma_R < 7.6 \tag{5.36}$$

When this condition is not satisfied $\alpha = \pm 1.0$ should be used for the variable with the larger standard deviation and $\alpha = \pm 0.4$ for the variable with the smaller standard deviation. This simplification is on the safe side as the sum of squared direction cosines should be equal to 1. When the load or resistance model contains several basic variables, the sensitivity factors of the non-dominant variables are given by equations (5.37) and (5.38):

$$\alpha_R = 0.8 \cdot 0.4 = 0.32 \tag{5.37}$$
$$\alpha_E = -0.7 \cdot 0.4 = -0.28 \tag{5.38}$$

At this point, considering the aforementioned definitions, analytical expressions for the partial factors γ_M and γ_F can be derived, taking into account their specific distributions type and distributional characteristics.

5.4.1 Material factor γ_m

The resistance R can be commonly described by a normal or a lognormal distribution. Considering that the most important variable is the material strength, suitable expressions for the partial factor γ_m are the following:

$$\gamma_m = \frac{X_k}{X_d} = \frac{(1 - 1.645 V_m)}{(1 - \alpha_R \beta V_m)} \qquad \text{Normal distribution} \tag{5.39}$$

$$\gamma_m = \frac{X_k}{X_d} = \frac{\exp(-1.645V_m)}{\exp(-\alpha_R \beta V_m)} \qquad \text{Lognormal distribution} \qquad (5.40)$$

where V_m is the coefficient of variation, X_d is the design value and X_k is the characteristic value, assumed to correspond to the 5% fractile of the theoretical distribution. The value 1.645 is obtained from the standard normal distribution table in correspondence of 5% fractile. Figures 5.5 and 5.6 show the variation of the partial factor γ_m with the coefficient of variation V_m for α_R = 0.8 and selected target reliabilities β = 3.3, 3.8, 4.3, 4.8 for normal distribution by equation (5.39) (Fig. 5.5) and lognormal distribution by equation (5.40) (Fig. 5.6):

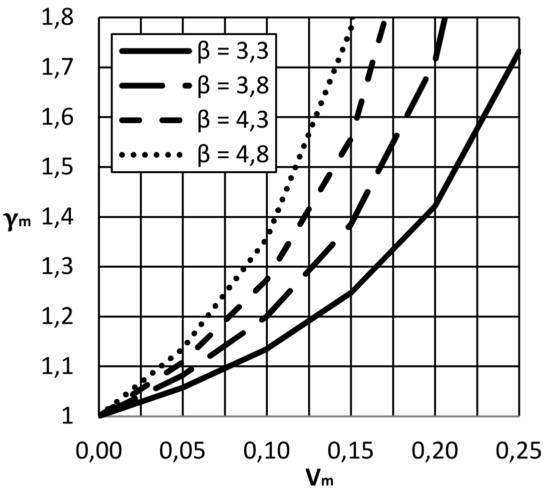


Figure 5.5: Variation of γ_m with V_m for α_R = 0,8 and for normal distribution

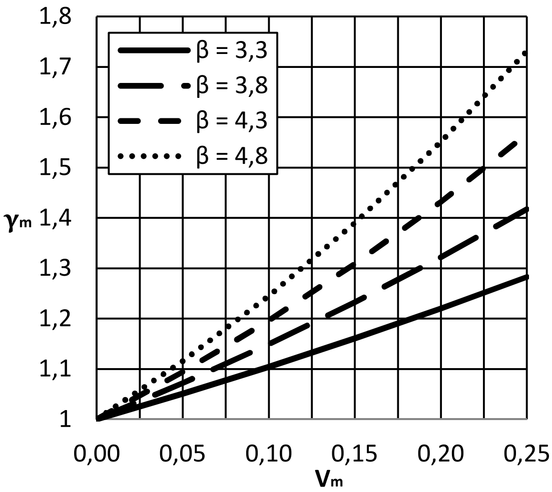


Figure 5.6: Variation of γ_m with V_m for α_R = 0,8 and for lognormal distribution

The partial factor γ_m increases with increasing β. The increase is considerably greater in the case of normal distribution than in the case of lognormal distribution. This effect is particularly obvious for coefficient of variation V_m greater than 0.10.

The partial factor γ_C = 1,5 (concrete) provided in Eurocodes has been derived assuming a normal distribution and considering γ_{Rd1} = 1,05, γ_{Rd2} = 1,05 , V_C = 0,15 and β = 3,8:

$$\gamma_C = \frac{(1 - 1.645 \cdot 0.15)}{(1 - 0.8 \cdot 3.8 \cdot 0.15)} \cdot 1.05 \cdot 1.05 = 1.53 \approx 1.5$$

The partial factor γ_S = 1,15 (steel reinforcement) has been derived considering γ_{Rd1} = 1,025, γ_{Rd1} = 1,05, V_S = 0,05 and β = 3,8 and a normal distribution.

$$\gamma_S = \frac{(1 - 1.645 \cdot 0.05)}{(1 - 0.8 \cdot 3.8 \cdot 0.05)} \cdot 1.025 \cdot 1.05 = 1.16 \approx 1.15$$

5.4.2 Permanent action factor γ_g

For the permanent actions usually a normal distribution can be assumed. Assuming that the characteristic value G_k of G is defined as the mean μ_G, the partial factor is given by:

$$\gamma_g = \frac{G_d}{G_k} = (1 - \alpha_E \beta V_G) \tag{5.41}$$

where V_G is the coefficient of variation, μ_G is the mean, G_d is the design value and G_k is the characteristic value. Variation of the partial factor γ_g with the coefficient of variation V_G is showed in Figure 5.7 for α_E = - 0.7 (unfavourable permanent action) and in Figure 5.8 for $\alpha_{E,fav}$ = 0.32 (favourable permanent action) and the selected reliabilities β.

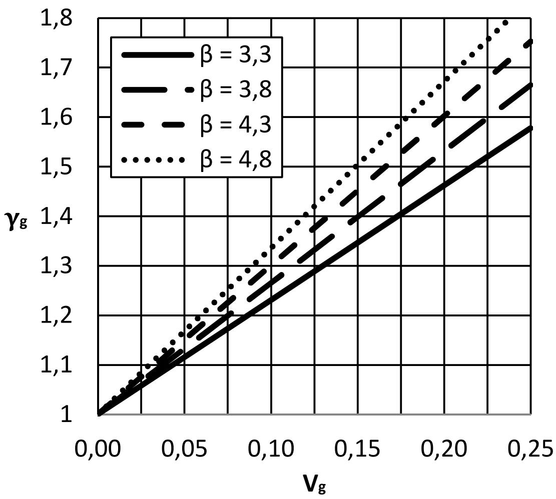


Figure 5.7: Variation of γ_g with V_g for α_E = - 0,7 (unfavourable action)

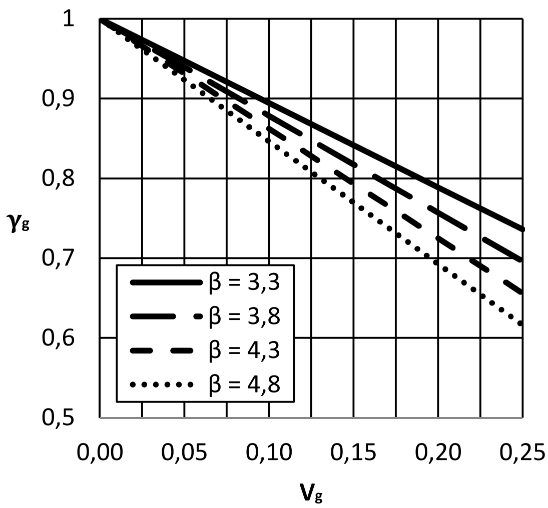


Figure 5.8: Variation of γ_g with V_g for α_E = 0,32 (favourable action)

The factor γ_g given in EN 1990 has been derived considering γ_{Ed} = 1,05, β = 3,8 and V_G = 0,05 and 0,10 for self-weight and other permanent actions, respectively.

$$\gamma_g = (1 + 0.7 \cdot 3.8 \cdot 0.10) = 1.33 \approx 1.35$$

5.4.3 Variable action factor γ_q

The variable load Q in general depends on a time-variant component and on a time-invariant component. Thus the design value Q_d of Q is determined on the base of the maxima variable load during the reference period t_{ref} and the partial factor γ_q is given as:

$$\gamma_q = \frac{Q_d}{Q_k} = \frac{F_{Q,tref}^{-1}[\Phi(-\alpha_E \beta, t_{ref})]}{Q_k} \tag{5.42}$$

where $F_{Q,tref}^{-1}$ denotes the inverse cumulative distribution function of maxima of the variable load during the reference period t_{ref}.

For variable loads it seems reasonable to assume a Gumbel distribution according to recorded data. The Gumbel distribution of maximum values in a basic reference period t_0 is given by (cumulative density function):

$$F_Q(Q)_{t_0} = e^{-e^{-a_{t0}(Q - b_{t0})}} \tag{5.43}$$

being a_{t0} the dispersion coefficient and b_{to} the mode of the distribution. The mean value and the standard deviation are:

$$\mu_{t0} = b_{t0} + \frac{0.577}{a_{t0}} \qquad \sigma_{t0} = \frac{\pi}{a_{t0}\sqrt{6}} \tag{5.44}$$

substituting:

$$F_Q(Q)_{t_0} = e^{-e^{\left(-0.577-\frac{\left(\frac{\pi}{\sqrt{6}}\right)(Q-\mu_{t0})}{\sigma_{t0}}\right)}} \tag{5.45}$$

If the reference period were t_{ref} years, under the hypothesis that the actions occurs independently in each year, the maximum values in t_{ref} years will also follow a Gumbel distribution:

$$F_Q(Q)_{tref} = \left(F_Q(Q)_{t_0}\right)^{t_{ref}/t_0} = \left(e^{-e^{-a_{t0}(Q-b_{t0})}}\right)^{t_{ref}/t_0} = e^{-\frac{t_{ref}}{t_0}e^{-a_{t0}(Q-b_{t0})}} = \\ e^{-e^{-a_{to}(Q-b_{t0})+\ln\frac{t_{ref}}{t_0}}} = e^{-e^{-a_{t0}\left(Q-\left(b_{t0}+\frac{\ln(t_{ref}/t_0)}{a_{t0}}\right)\right)}} = e^{-e^{-a_{tref}\left(Q-b_{tref}\right)}} \tag{5.46}$$

Consequently the Gumbel parameters b_{tref} and a_{tref} are correlated to the basic reference period parameters by:

$$a_{tref} = a_{t0} \tag{5.47}$$

$$b_{tref} = b_{t0} + \frac{\ln\left(t_{ref}/t_0\right)}{a_{t0}} \tag{5.48}$$

And, combining with (5.44), the corresponding mean and the standard deviation are:

$$\mu_{tref} = \mu_{t0} + \frac{\ln(t_{ref}/t_0)}{a_{to}} = \mu_{t0} + \frac{\sqrt{6}}{\pi}\sigma_{t0}\ln(t_{ref}/t_0) = \mu_{t0} + 0.78\,\sigma_{t0}\ln(t_{ref}/t_0) \tag{5.49}$$

$$\sigma_{tref} = \sigma_{t0} \tag{5.50}$$

The characteristic value of climatic actions is based upon the probability of 0.02 of its time-varying part being exceeded for a reference period of one year. This is equivalent to a mean return period of 50 years for the time-varying part. Thus, the characteristic load is defined as:

$$Q_k^{clim} = b_1 - \frac{1}{a_1}\ln(-\ln(0.98) \tag{5.51}$$

with a_1 and b_1 the parameters of the Gumbel distribution of the yearly maxima.

In case of imposed loads the characteristic value is defined in the Eurocode as the load that has a probability of exceedance of 5% for a reference period of 50 years, hence:

$$Q_k^{imp} = b_{50} - \frac{1}{a_{50}} \ln(-\ln(0.95) \tag{5.52}$$

with a_{50} and b_{50} the parameters of the Gumbel distribution associated to a reference period of 50 years.

According to eq. (5.29) and considering eq. (5.46) the design load for a reference period t_{ref} can be calculated as follow:

$$Q_{d,tref} = b_{t0} + \frac{\ln(t_{ref}/t_0)}{a_{t0}} - \frac{1}{a_{t0}} \ln(-\ln(\Phi(-\alpha_E \beta_{t0})) \tag{5.53}$$

where $\Phi(-\alpha_E \beta)$ is the probability of exceedance in the reference period t_{ref}. Can be easily proved that the same design value is obtained considering the probability of exceedance in the reference period t_0. The previous equations can be rewritten, according to equations (5.44), in terms of mean and standard deviation:

$$Q_k^{clim} = \mu_1 - 0.45\sigma_1 - 0.78\sigma_1 \ln(-\ln(0.98)) \tag{5.54}$$

$$Q_k^{imp} = \mu_{50} - 0.45\sigma_{50} - 0.78\sigma_{50} \ln(-\ln(0.95)) \tag{5.55}$$

$$Q_{d,tref}^{clim} = \mu_1 + 0{,}78\sigma_1 ln t_{ref} - 0.45\sigma_1 - 0.78\sigma_1 \ln(-\ln(\Phi(-\alpha_E \beta_1))) \tag{5.56}$$

$$Q_{d,tref}^{imp} = \mu_{50} + 0{,}78\sigma_{50} ln(t_{ref}/50) - 0.45\sigma_{50} - 0.78\sigma_{50} \ln(-\ln(\Phi(-\alpha_E \beta_{50}))) \tag{5.57}$$

which lead to the following partial factors, written considering a basic reference period t_0 = 1 year in case of climatic actions and, in case of imposed loads, considering a basic reference period of t_0 = 50 years:

$$\gamma_q^{clim} = \frac{1 + V_1(0.78 \cdot \ln t_{ref} - 0.45 - 0.78 \ln(-\ln(\Phi(-\alpha_E \beta_1))))}{1 + V_1(-0.45 - 0.78 \ln(-\ln(0.98)))} \tag{5.58}$$

$$\gamma_q^{imp} = \frac{1 + V_{50}(0.78 \cdot \ln(t_{ref}/50) - 0.45 - 0.78 \ln(-\ln(\Phi(-\alpha_E \beta_{50}))))}{1 + V_{50}(-0.45 - 0.78 \ln(-\ln(0.95)))} \tag{5.59}$$

where V_1 and V_{50} are the coefficient of variation and α_E = - 0.7 the sensitivity factor.

Figures 5.9, 5.10, 5.11 and 5.12 show the variation of the partial factor γ_q with the coefficient of variation $V_{Q,tref}$ and with the reference period for α_E = - 0,7 and selected target reliabilities β assuming Gumbel distribution of Q.

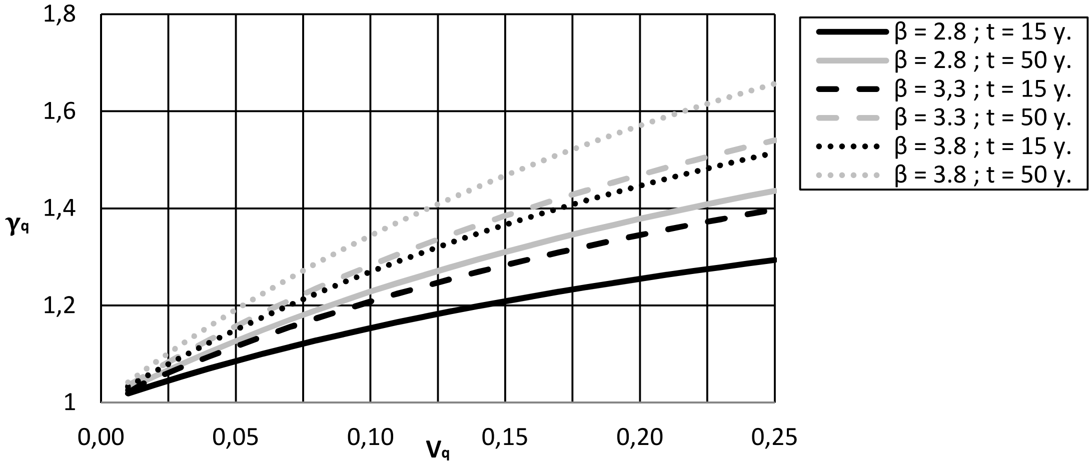


Figure 5.9: Variation of γ_q for climatic actions with V_q for α_E = -0.7 (t = 15 years; t = 50 years)

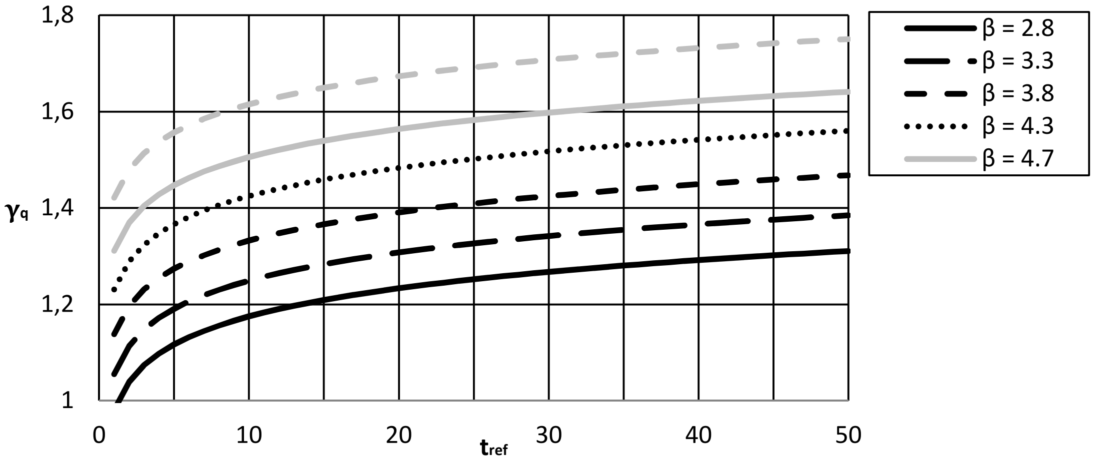


Figure 5.10: Variation of γ_q for climatic actions with t_{ref} for α_E = -0.7 (V_q =0.15)

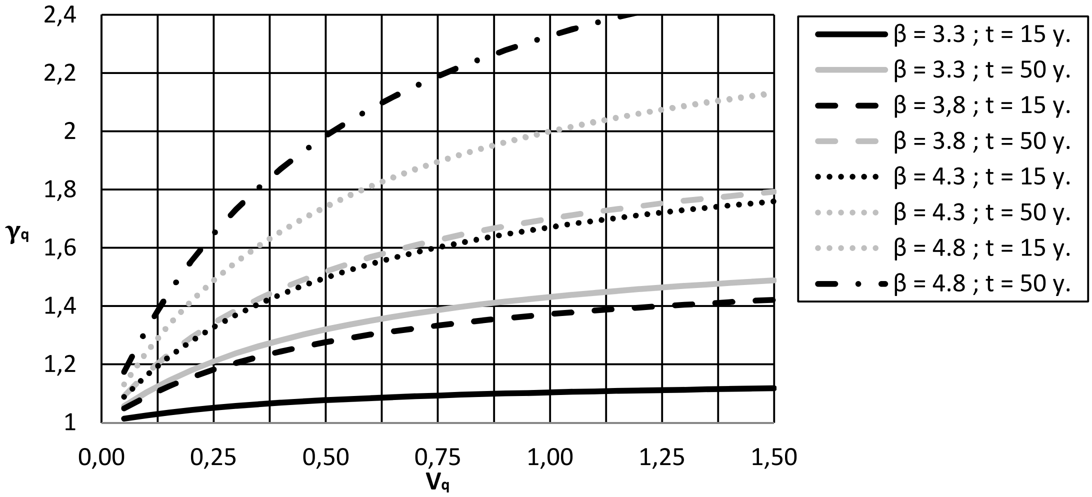


Figure 5.11: Variation of γ_q for imposed loads with V_q for α_E = -0.7 (t = 15 years; t = 50 years)

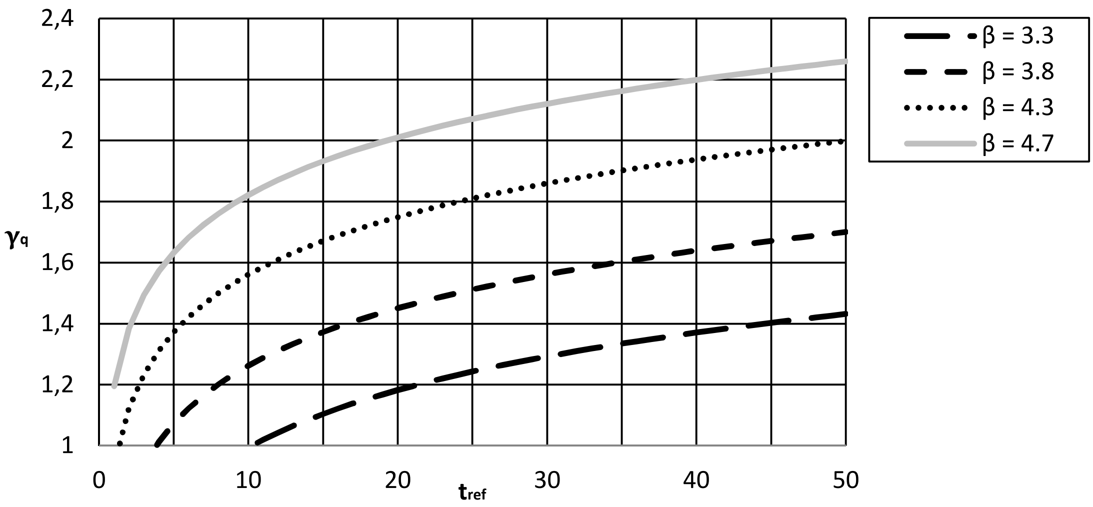


Figure 5.12: Variation of γ_q for imposed loads with t_{ref} for α_E = -0.7 (V_q =1.0)

The Eurocodes generally consider a reference time t_{ref} = *50 years* and define the characteristic value of a climatic action as the 98% fractile referred to 1 year. Considering, for instance, a wind action having $V_{Q,tref}$ = 0.16 and assuming β = 3.8, the calculated partial factor is:

$$\gamma_q = \frac{\left(1 + V_{Q,tref}\left(0.78 \cdot \ln 50 - 0.45 - 0.78\ln\left(-\ln\left(\Phi(0.7 \cdot 3.8)\right)\right)\right)\right)}{\left(1 - V_{Q,tref}\left(0.45 + 0.78\ln(-\ln(0.98))\right)\right)} =$$

$$= \frac{1 + 6.925 \cdot V_{Q,tref}}{1 + 2.594 \cdot V_{Q,tref}} = 1.49 \approx 1.5$$

Referring to the target reliability indexes proposed by the Eurocode (Table 5.3) is interesting to calculate the corresponding partial factors. The calculation is conducted for climatic actions (γ_q^{clim}) assuming as coefficient of variation the values 0.10, 0.15 and 0.20.

		RC3	**RC2**	**RC1**
T = 1 year	V_q = 0.10	1.31 (β =5.2)	1.23 (β =4.7)	1.16 (β =4.2)
	V_q = 0.15	1.42 (β =5.2)	1.31 (β =4.7)	1.21 (β =4.2)
	V_q = 0.20	1.51 (β =5.2)	1.38 (β =4.7)	1.26 (β =4.2)
T = 50 years	V_q = 0.10	1.41 (β =4.3)	1.34 (β =3.8)	1.28 (β =3.3)
	V_q = 0.15	1.56 (β =4.3)	1.47 (β =3.8)	1.38 (β =3.3)
	V_q = 0.20	1.68 (β =4.3)	1.57 (β =3.8)	1.47 (β =3.3)

Table 5.6: Variation of γ_q for climatic actions with V_q = 0.10, 0.15, 0.20 ; α_E = -0.7; t = 1 year, t = 50 years; β from EN 1990.

Starting from the results reported in Table 5.6 two observations can be made. Firstly the partial factor γ_q increases with the reference period, even though the value of the reliability index β decreases. This is in accordance to the fact that the design value of a variable action, represented by a Gumbel distribution, increases with the reference period. For instance, considering RC2 and V_q = 0.15, γ_q changes from 1.31 for a reference period of 1 year (β = 4.7) to 1.47 for a reference period of 50 years (β = 3.8):

$$\gamma_q^1 = \frac{Q_d^1}{Q_k} = \frac{1 + 0.15 \cdot (0.78 \cdot \ln 1 - \ 0.45 - 0.78 \ln(-\ln(\Phi(0.7 \cdot 4.7))))}{1 + 0.15 \cdot (-0.45 - 0.78 \ln(-\ln(0.98)))} = \frac{1.82}{1.39} = 1.31$$

$$\gamma_q^{50} = \frac{Q_d^{50}}{Q_k} = \frac{1 + 0.15 \cdot (0.78 \cdot \ln 50 - \ 0.45 - 0.78 \ln(-\ln(\Phi(0.7 \cdot 3.8))))}{1 + 0.15 \cdot (-0.45 - 0.78 \ln(-\ln(0.98)))} = \frac{2.04}{1.39} = 1.47$$

Secondly it can be observed that the partial factor decreases with decreasing coefficient of variation V_q. This is especially important in the context of this thesis: if inspections and monitoring can be used to reduce the uncertainties related to the evaluation of a certain action, a reduced partial factor can be assumed to ensure the same level of reliability. Thus, considering RC2 and T= 50 years, if as result of a monitoring activity is possible to pass from V_q = 0.15 to V_q = 0.10, the partial factor can be reduced from 1.47 to 1.34. Similar considerations are valid for the material factor γ_m and for the permanent action factor γ_g as well.

An alternative way to express the partial factor γ_q is to consider that it depends on the time variant component $Q_0(t)$ and on the time invariant component C_0. In most cases the maximum of the variable load related to t_{ref} can be obtained as a product of both components:

$$Q_{tref} = C_0 \cdot \max_{tref}[Q_0(t)] = C_0 \cdot Q_{0,tref} \tag{5.60}$$

Indicatives probabilistic models for time variant and time invariant components of some common variable loads are given in Table 5.7:

Variable	X	Distr.	μ_X/X_k	V_X
Time-inv. comp. – traffic load on road bridges including load model unc	C_{0Q}	LN	1	0.15
Annual max. traffic load on road bridges	Q	Gum	0.55–0.85[a]	0.03–0.15[a]
Time-inv. comp. – wind action	C_{0W}	LN	0.65	0.3
Annual max. basic wind velocity	v_b	LN[b]	$\sim 1/\{1 - V_{vb}[0.45 + 0.78\ln(-\ln 0.98)]\}$	[c]
Annual max. basic wind pressure	W	Gum	$\sim (1 + V_{vb}^2)/\{1 - V_{vb}[0.45 + 0.78\ln(-\ln 0.98)]\}^2$	[d]
Time-inv. comp. thermal action including model uncertainties	C_{0T}	LN Weib./	1	0.1
Annual max. bridge temperature (uniform component)	T	LN[b]	0.85–0.95[e]	0.03

[a] As a first approximation $\mu_X/X_k \approx 0.8$ and $V_X \approx 0.06$ can be accepted in common cases.
[b] Shifted lognormal distribution.
[c] Should be based on local meteorological data since it is dependent on terrain roughness, orography and altitude.
[d] $V_W \approx V_{vb}(4 - V_{vb}^2 + 6V_{vb}\omega_{vb})^{0.5}/(1 + V_{vb}^2)$ where ω is a sample skewness (in case of insufficient data, Gumbel distribution and ω_{vb} = 1.14 may be considered).
[e] For the Central Europe, $\omega_T \approx 0.3$–0.6 [35,37].

Table 5.7: Indicative probabilistic models of selected variable loads (Caspeele, Sỳkora, Allaix, Steenbergen)

Assuming a Gumbel distribution of the time variant component, the mean of $Q_{0,tref}$ is obtained as (equivalent to eq. (39)):

$$\mu_{Q0,tref} = \mu_{t0} + 0.78\,\sigma_{t0}\ln(t_{ref}/t_0) \tag{5.61}$$

where t_0 is the basic reference period for $Q_0(t)$ (1 year for climatic loads, 5 years for the sustained part of imposed loads in office building).
In many cases it can be considered, as an approximation, that Q_{tref} has Gumbel distribution with the following parameters:

$$\mu_{Q,tref} \approx \mu_{C0}\mu_{Q0,tref} \tag{5.62}$$

$$V_{Q,tref} \approx \sqrt{V_{C0}^2 + V_{Qo,tref}^2 + V_{C0}^2 V_{Qo,tref}^2} \tag{5.63}$$

where $V_{Q0,tref} = \sigma_{Q0}/\mu_{Q0,tref}$. Consequently the partial factor is assessed as:

$$\gamma_q \approx (\mu_{Q,tref}/Q_k)\cdot(1 - V_{Q,tref}(\,0.45 + 0.78\ln(-\ln(\Phi(-\alpha_E\beta))) \tag{5.64}$$

where Q_k is the characteristic value applied in the assessment and the ratio $\mu_{Q,tref}/Q_k$ can be obtained from Table 5.7.

Note that other distributions such as shifted lognormal or Weibull could be used to model the variable actions.

5.4.4 Model uncertainties factors γ_{Rd} and γ_{Ed}

In case model uncertainties are considered, the limit state function becomes:

$$g = \vartheta_R R - \vartheta_E E \tag{5.65}$$

where θ_R describes the uncertainties related to the resisting model and θ_E takes into account the uncertainties related to the load-effect model. In common cases the following model uncertainties factors can be assumed:

γ_{Rd1} = 1.10 for concrete strength
γ_{Rd1} = 1.025 for reinforcing steel
γ_{Rd2} = 1.10 for concrete section size
γ_{Rd2} = 1.05 for reinforcing steel position
$\gamma_{Ed,g}$ = 1.07 for unfavourable action (permanent action)
$\gamma_{Ed,g}$ = 1.00 for favourable action (permanent action)
$\gamma_{Ed,q}$ = 1.12 for unfavourable action (variable action)
Favourable variable actions are not considered in structural verification.

Alternatively, the partial factors γ_{Rd} and γ_{Ed} can be obtained assuming a normal or a lognormal distribution. If a normal distribution is assumed for both model uncertainties, the partial factors γ_{Rd} and γ_{Ed} can be written as:

$$\gamma_{Rd} = \frac{\mu_{\vartheta_R}}{\vartheta_{Rd}} = \frac{1}{1 - 0.4\alpha_R \beta V_{\vartheta_R}} \tag{5.66}$$

$$\gamma_{Ed} = \frac{\vartheta_{Ed}}{\mu_{\vartheta_E}} = 1 - 0.4\alpha_E \beta V_{\vartheta_E} \tag{5.67}$$

where μ_{ϑ_R} , μ_{ϑ_E} , V_{ϑ_R} , V_{ϑ_E} are, respectively, the mean values and coefficients of variation of the random variables θ_R and θ_E and the sensitivity factors correspond to non-dominant variables. However, *Taerwe* suggests to refer the partial factors to the characteristic values θ_{Rk} and θ_{Ek}:

$$\gamma_{Rd} = \frac{\vartheta_{Rk}}{\vartheta_{Rd}} = \frac{1 - 1.645 V_{\vartheta_R}}{1 - 0.4\alpha_R \beta V_{\vartheta_R}} \tag{5.68}$$

$$\gamma_{Ed} = \frac{\vartheta_{Ed}}{\vartheta_{Ek}} = \frac{1 - 0.4\alpha_E \beta V_{\vartheta_E}}{1 + 1.645 V_{\vartheta_E}} \tag{5.69}$$

Nevertheless, is generally preferred to adopt a lognormal distribution to model the uncertainties. The corresponding factors γ_{Rd} and γ_{Ed} are:

$$\gamma_{Rd} = \frac{\mu_{\vartheta_R}}{\vartheta_{Rd}} = \frac{1}{\exp\left(-0.4\alpha_R \beta V_{\vartheta_R}\right)} \tag{5.70}$$

$$\gamma_{Ed} = \frac{\vartheta_{Ed}}{\mu_{\vartheta_E}} = \exp\left(-0.4\alpha_E \beta V_{\vartheta_E}\right) \tag{5.71}$$

if referred to mean values and:

$$\gamma_{Rd} = \frac{\vartheta_{Rk}}{\vartheta_{Rd}} = \frac{\exp\left(-1.645 V_{\vartheta_R}\right)}{\exp\left(-0.4\alpha_R \beta V_{\vartheta_R}\right)} \tag{5.72}$$

$$\gamma_{Ed} = \frac{\vartheta_{Ed}}{\vartheta_{Ek}} = \frac{\exp\left(-0.4\alpha_E \beta V_{\vartheta_E}\right)}{\exp\left(1.645 V_{\vartheta_E}\right)} \tag{5.73}$$

if referred to characteristic values.
Reference distributions and values for mean and coefficient of variation to be assumed in different situations are provided by JCSS – Probabilistic Model Code.

Chapter 6

Monitoring of a stadium roof exposed to snow load

6.1 Introduction

Snow loads are important especially in northern and mountainous regions where heavy snowfalls and accumulation from many different storms during the winter season determine considerable loads. Buildings may be vulnerable to structural failure and possible collapse if basic preventive steps are not taken in advance of a snow event. Knowledge of the building roof framing system and proper preparation in advance of a snow event is essential in order to reduce the risk of the structure. Structural failure due to roof snow loads may be linked to several possible causes, including but not limited to the following:

- actual snow load significantly exceeds design snow load
- drifting and sliding snow conditions
- deficient workmanship
- insufficient operation and maintenance
- improper design
- inadequate drainage design
- insufficient design; in older buildings, insufficient design is related to inadequate load design criteria in the building code in effect when the building was designed

In recent years multiple major snow storms resulted in numerous building failures (Figure 6.1): collapses of a number of roofs in European countries such as Austria, Czech Republic, Germany and Poland during the winter 2005/2006 and in northeastern part of the United States in the winter of 2011 for example. The investigations following the collapses in Europe highlighted that the main observed causes of structural damage may subdivided into errors in design, during execution and use, and insufficient code provisions. An insufficient reliability level may be obtained by the partial factor design as indicated by probabilistic reliability analysis conducted by *Holický* and *Sýkora*. In several cases a model for snow loads recommended in standards underestimated actual loads and loads due to the combination of snow and ice on roofs are not considered. In many cases multiple causes such a combination of errors were observed.
Extreme snow loads may lead to four levels of damage: excessive deflection, failure of a member or few members, partial collapse, total collapse. Collapsed structures had mostly insufficient robustness (no tying, low resistance of key members or inappropriate structural detailing). Apparently, robustness is a key property affecting development of collapse from a local failure.

Figure 6.1: Collapse of a stadium

The current reference regarding snow loads in Europe is EN 1991-1-3 that accounts for roof slope, thermal characteristics of the structure, and exposure to wind to quantify the amount of snow that may be present on a roof over the course of a winter season. The scientific basis for harmonised definition of models for determining the actions of snow applied to the structural parts of construction works can be found in the outcomes of the research group coordinated by *Sanpaolesi*. The research work was divided into two consecutive phases. Each phase dealt with two specific items: phase I dealt with development of models for the determination of snow loads on the ground and development of models for exceptional snow loads; phase II dealt with definition of criteria to be adopted for serviceability loads and analytical study for the definition of shape coefficients. The final reports, and specifically the European snow maps, are quoted in the Eurocode 1. The basis for EN 1991-1-3 snow load computations is the ground snow load, s_k. This value is modified to become a flat roof snow load, s, by multiplying by a constant, μ_i, that accounts for roof snow loss that ground measurements do not see. In addition, the value is modified by coefficient that account for building exposure to wind, C_e, and the thermal characteristics of the building, C_t. Hence:

$$s = \mu_i \cdot C_e \cdot C_t \cdot s_k \tag{6.1}$$

s_k is intended as the upper value of a random variable, with the annual probability of exceedance set to 0.02, thus associated to a return period of 50 years. The Eurocode provides different equations to calculate it depending on the zone of the building and on the altitude of the site. Regional maps are available. The shape coefficient μ_i assumes different values depending on the type of the roof. For example for a monopitch roof μ_1 = 0.8. More values regarding different roof shapes, exceptional drifted cases and particular situations have been calibrated on a wide experimental campaign, both in situ and in wind tunnel, and are provided in EN 1991-1-3. C_e can be set equal to 0.8 (windswept

topography), 1.0 (normal topography) or 1.2 (sheltered topography). C_t is used to account for the reduction of snow load on roof with high thermal transmittance, in particular for some glass covered roofs, because of melting caused by heat loss. For all other cases C_t = 1.
The above reported equation is valid for the persistent and transient design situations with no exceptional snow falls or difts and can be applied for the site altitudes below 1500 m unless otherwise specified. The design value of the snow load is obtained multiplying the representative value (s) by γ_Q = 1.5, defined on the base of the equations reported and discussed in paragraph 5.4.3.
For the accidental design situations, where exceptional ground snow load or snow drift are the accidental action different equation are applied to calculate the representative value of the snow load (see EN 1991-1-3).
In other documents alternative definition of snow load can be found such as probabilistic model code prepared by JCSS. In this document definition and formulation about the calculation of snow loads at the ground level, conversion of snow loads from ground to roof level and regional coefficients for coastal and mountain regions are provided. Probabilistic definition of the snow load at the ground is generally represented by two probability distribution functions which define the total duration of loading and the maximum load intensity. These two cases are represented by gamma distribution functions and the parameters of the function can be determined by using local observations. Conversion of snow loads from ground to roof is affected from various parameters. The exposure level of building at the roof level is controlled by the slope and the shape of the roof. Thermal effects at the roof level may be also included in the calculation of snow loads. Definitively the snow load on roofs, S_r, is determined by the relation:

$$S_r = S_g \cdot r \cdot k^{h/h_r} \tag{6.2}$$

where
S_g is the snow load on the ground at the weather station. It depends on the snow depth d and on the average density of the snow $\gamma(d)$
r is a conversion factor of snow load on ground to snow load on roof
k is a coefficient: k = 1.25 for coastal regions, k = 1.5 for inland mountainous regions
h/h_r is the ratio between the altitude of the building site and the reference altitude (300 m)

The characteristics of the snow load S_g should be determined on the basis of observations from weather stations. The results of such observations are either water-equivalents of snow of depths of snow. In the first case the values can be used directly to determine the ground snow load. In the second case the data on snow depth must be converted to snow load by the relation:

$$S_g = d \cdot \gamma(d) = \lambda \cdot \gamma(\infty) \cdot \ln\left[1 + \frac{\gamma(0)}{\gamma(\infty)}(\exp\left(\frac{d}{\lambda}\right) - 1)\right] \tag{6.3}$$

where

d is the snow depth
$\gamma(d)$ is the average weight density of the snow
$\gamma(\infty)$ = 5 kN/m^3, $\gamma(0)$ = 1.7 kN/m^3, λ = 0.85 m
It is important to note that equation (6.3) provides values of the average weight density of the snow comprised between about 1.5 kN/m^3 and 2.0 kN/m^3 (referring to interval of practical interest of the snow depth)(Fig. 6.2) and it could underestimate the snow load in certain geographical areas more susceptible to snow accumulation and to higher weight density of the snow.

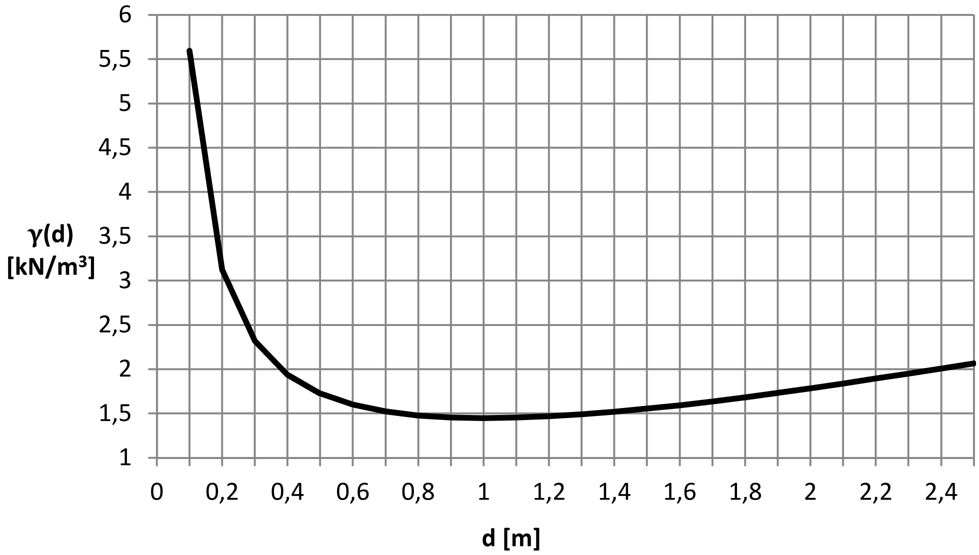


Figure 6.2: Variation of the weight density of the snow with the snow depth (eq. 6.3)

Higher snow loads may be obtained increasing $\gamma(\infty)$ and $\gamma(0)$ or reducing λ. Increasing $\gamma(\infty)$ the result changes very little if we consider interesting values of the snow depth. The increase of the snow load is more significant for high snow depths which are not of practical interest. The result is more influenced by $\gamma(0)$ and λ. Increasing $\gamma(0)$ the curve described by equation (6.3) is shifted upwards whilst reducing λ the curve changes shape and the increasing of the weight density of the snow is more important for high values of the snow depth. For instance, assuming $\gamma(\infty)$ = 6 kN/m^3, $\gamma(0)$ = 3 kN/m^3, and λ = 0.8 m, values of the average weight density of the snow comprised between about 2.5 kN/m^3 and 3.0 kN/m^3 can be found.
Thus further studies are necessary to investigate on the relation between the average weight of the snow γ and the snow depth d in the different climatic regions.

The conversion factor r is subdivided into a number of factors and terms according to the expression

$$r = \eta_a C_e C_t + C_r \tag{6.4}$$

where

the exposure coefficient, C_e and the shape factor η_a are reduction coefficients taking into account of the exposure to wind of a building and the slope of the roof.
C_t is a deterministic thermal coefficient, it is usually equal to 1.0. A value of 0.8 shall be used for roofs with high thermal transmittance, in particular glass covered roofs.
C_r is a redistribution (due to wind) coefficient. For monopitch roofs may be neglected. For other types of roofs additional indications are provided in JCCS probabilistic model code.

National codes such as the Italian D.M. 14.01.2008 (NTC 08) and the German DIN 1055-5 use similar equations to calculate the snow load.
However in some cases codes cannot reflect or formulate all the details of snow actions. Especially aging infrastructures, constructed before modern code regulation, should be assessed to check whether they meet to the requirements of updated regulations or not. It should be reminded that majority of today's infrastructures have been designed using climatic design values derived from historical climate data. Increasing snow actions due to climate changes will require modification to how infrastructures are engineered, maintained and operated.

6.2 Description of structure and monitoring system

Consider a hypothetical stadium erected at the beginning of the 1990s and located in Trento, Northern Italy, at an altitude of 190 m. The roof of the stadium consists of a cantilever steel beam (IPE 500) created by an extension of a simple supported beam. The lengths of the first span and of the overhang are 4 m and 8 m respectively for a total length of the structure of 12 m. The spacing between two following beams is 5 m. The inclination of the steel beam is negligible (α ≈ 0°). A schematic representation of the roof beam is reported in Figure 6.3:

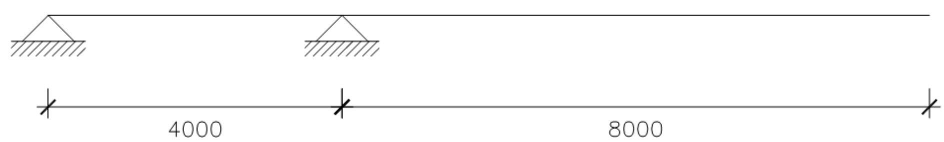


Figure 6.3: Scheme of the roof beam

The stadium can accommodate up to 4000 people and it is widely used in order to host sport events, concerts and shows. Given that the structure is located in the Alpine region and it is subjected to high snow loads, after the recent roof collapses and the related studies, it was decided to investigate its actual structural reliability. Analysing the available documents it can be immediately noted that the design snow load is rather low if compared with the one currently imposed by the Italian code.
D.M. 12.02.1982 recommended the following snow loads for zone I (Northern Italy):

$$q_s = 0.9\ kN/m^2 \qquad\qquad h_s < 300\ m \qquad (6.5)$$

$$q_s = 0.9 + 1.5(h_s - 300)\ kN/\ m^2 \qquad\qquad h_s \geq 300\ m \qquad (6.6)$$

$\alpha \leq 20°$: no reductions
$20° < \alpha < 60°$: 2,5% reduction (linear) for each degree of inclination of the roof

Considering the aforementioned characteristics a snow load of 0.9 kN/m^2 was assumed. The current code, D.M. 14.01.2008 (NTC 08), for the same site, imposes:

$$q_s = \mu_i \cdot q_{sk} \cdot C_e \cdot C_t = 0.8 \cdot 1.5 \cdot 1 \cdot 1 = 1.2\ kN/m^2 \qquad (6.7)$$

with C_e = 1 (exposure coefficient), C_t = 1 (thermal coefficient), μ_i = 0.8 (shape coefficient for a monopitch roof), q_{sk} = 1.5 kN/m^2 (characteristic ground snow load for the Alpine region and for altitude h < 200 m).
It is important to note that the direct comparison of these two values does not give the correct idea on how changed the importance of the snow load in the design process because the old code was based on the permissible stress method (the verifications were conducted referring to CNR-UNI 10011) while the current code uses a semi-probabilistic approach, the partial factor method, where the actions are combined using partial and combination factors. Nevertheless the calculated values indicate that snow loads currently assumed are higher than in the past and for this reason many existing structures subjected mainly to snow loads do not achieve the same reliability level imposed to the new structures in modern codes. In order to keep the reliability level acceptable it was decided to implement on the roof of the stadium a permanent monitoring system that can provide real time evaluation of the snow depth. The monitoring system is composed of new laser snow depth gauges mounted on mast poles at a height of 2 m above the roof. These sensors were preferred to the ultrasonic snow depth gauges because of several advantages (*Lanzinger and Theel*): lower measurement uncertainty by almost one order of magnitude; no influence of temperature and wind; no outages even during heavy snowfall; very little maintenance needed. The sensor contains a laser diode emitting eye-safe visible light at 650 nm. Reflected light is received and compared with the signal from a reference diode. A microprocessor calculates the phase shift and the distance to the target. Figure 6.4 shows a schematic representation of the measurement principle. Every 0.16 s a measurement with a single frequency is performed. Five modulation frequencies are used and measurements are averaged to abtain a more accurate measurement on critical targets, like snow. The laser snow depth sesnsor used allows probing distances up to 15 m with a resolution of 1 mm. The measurament accurancy for the snow depth measurement is specified at better than 5 mm. The longest possible averaging interval of 6 seconds is used for snow depth measurement.

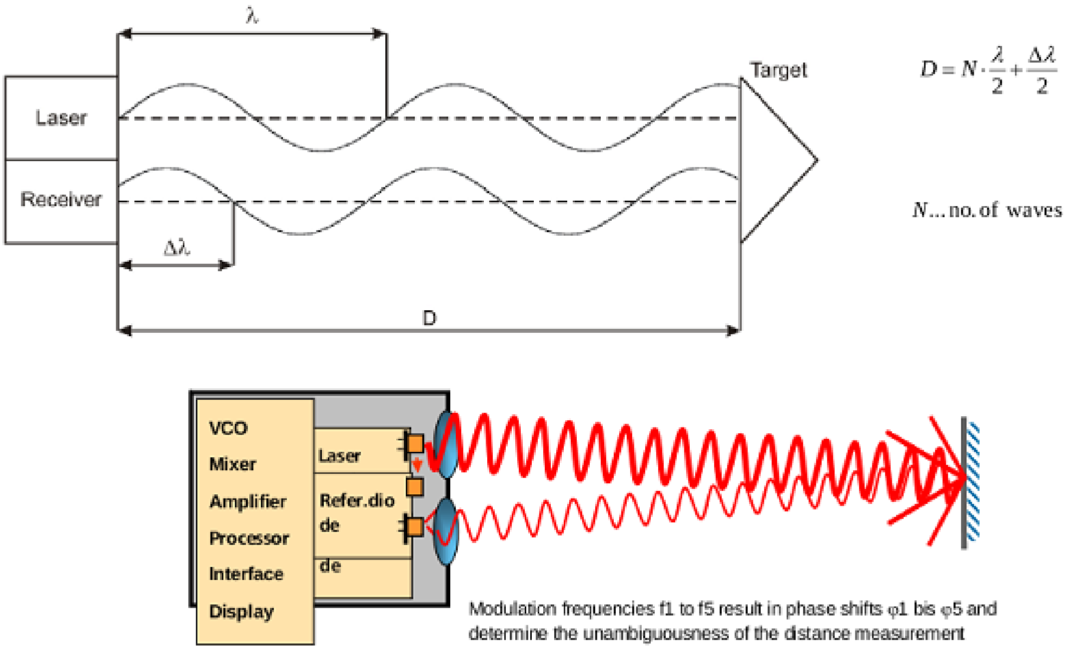


Figure 6.4: Illustration of the measurement principle used by the laser snow depth sensor (Haij, 2011)

The purpose of the implementation of this permanent monitoring system is to close the stadium and forbid the presence of people in it when the snow depth reach the limit level d_L. This decision is taken in order to avoid injuries and fatalities in case of a possible structural failure. The determination of the limit snow depth is based on a minimum acceptable reliability level as better explained in paragraph 6.3.

6.3 Reliability analysis and target reliability

The probabilistic reliability analysis is based on the limit state function Z(.) (eq. 6.8) for the section of the beam subjected to the maximum bending moment. The considered section is located in correspondence of the support between the first span and the overhang. The limit state function is expressed in terms of resistance due to the fact that instability is avoided by an effective bracing system and deformation is not considered since the ultimate limit state is examined to decide if the stadium can be let open or if it must be closed to the public. Thus, the limit state function is:

$$Z(\boldsymbol{X}) = \vartheta_R R - \vartheta_E\big(G + S(d)\big) = \vartheta_R W_{pl} f_y - \vartheta_E \frac{a^2}{2}\left(\gamma_g \cdot A + g_{roof} \cdot i + S_r \cdot i\right) \quad (6.8)$$

where

R is the resistance

G is the permanent action

S(d) is the snow load, function of the snow depth *d*

S_r snow load on the roofs, determined using equations (6.2), (6.3), and (6.4)

θ_R represents model uncertainties in structural resistance

θ_E represents model uncertainties in load effect

Notation and probabilistic models of all the basic variables are reported in Table 6.1

Basic variable	Symbol	Dimension	Distribution	Mean	CoV
Span of the overhang	a	m	DET	8	-
Sectional area (IPE 500)	A	m^2	DET	0.01155	-
Spacing of roof beams	i	m	DET	5	
Steel density	γ_G	kN/m^3	N	77	0.01
Weight of the roof cover	g_{roof}	kN/m^2	N	0.5	0.05
Section modulus (IPE 500)	W_{pl}	m^3	DET	0.002194	-
Yield strength (S275)	f_y	N/mm^2	LN	308.6	0.07
Shape coefficient	η_a	-	N	1.0	0.05
Snow depth	d	m	LN	Monitored	0.10
Resistance uncertainty	θ_R	-	LN	1.15	0.10
Load effect uncertainty	θ_E	-	LN	1.0	0.10

Table 6.1: Models of basic variables

Resistance of the steel beam, according to f_y, is described by a lognormal distribution with a coefficient of variation of 0.07. The mean of the resistance is obtained as 1.122 times the characteristic value. This factor derives from the definition of the characteristic value of a lognormal distribution: $X_k = \mu \cdot \exp(-1.645V_m) \rightarrow \mu = X_k \cdot \exp(1.645V_m) = 1.122 \cdot X_k$. The resistance is assumed to be time-independent and given that deterioration is not observed the reference value of the yield strength for steel S275 is adopted and in-situ measurements are not conducted.

The permanent action is described by the sum of two normal distributions: the first represents the weight of the steel beam and has a mean of 77 kN/m^3 and a coefficient of variation of 0.01; the second represents the weight of the roof cover and has a mean of 0.5 kN/m^2 and a coefficient of variation of 0.05.

The snow load, expressed in terms of snow depth d, is considered to be described by lognormal distribution with a coefficient of variation of 0.10. The mean value of the snow depth is constantly monitored and it is known in real time. Considering the single measurement by a laser sensor , the snow load could be assumed to be deterministic due to the high accuracy of the measurement itself. This is not done and a lognormal distribution is assumed because the snow depth measurement is a point measurement and do not account for the spatial variability of the snow depth along the stadium roof, which is dominant in this case. The shape coefficient for horizontal roofs is assumed to be normally distributed. The mean 1.0 and the coefficient of variation 0.05 describe the transition between the snow load in the location of sensors and the roof. These values differ from what usually reported in the literature where the shape coefficient has a different meaning and it describes the transition from ground snow load to roof loading.

The model uncertainties are described by the lognormal distribution. Assuming steel section subjected to bending about the strong axis when no stability phenomena are taken into account, the mean 1.15 and the coefficient of variation 0.10 of the model uncertainties

for resistance are accepted (*Nadolski* and *Sýkora*). The load effect uncertainty has a mean of 1.0 and a coefficient of variation of 0.10.
The distributions, the mean values, and the coefficients of variation assumed are in accordance with JCSS – Probabilistic model code, except where otherwise specified.

The reliability analysis is conducted assuming different values for the mean value of the snow depth *d* and calculating the corresponding value of the reliability index β. FORM and *VaP* 1.6 are used. The results are summarized in Figure 6.5:

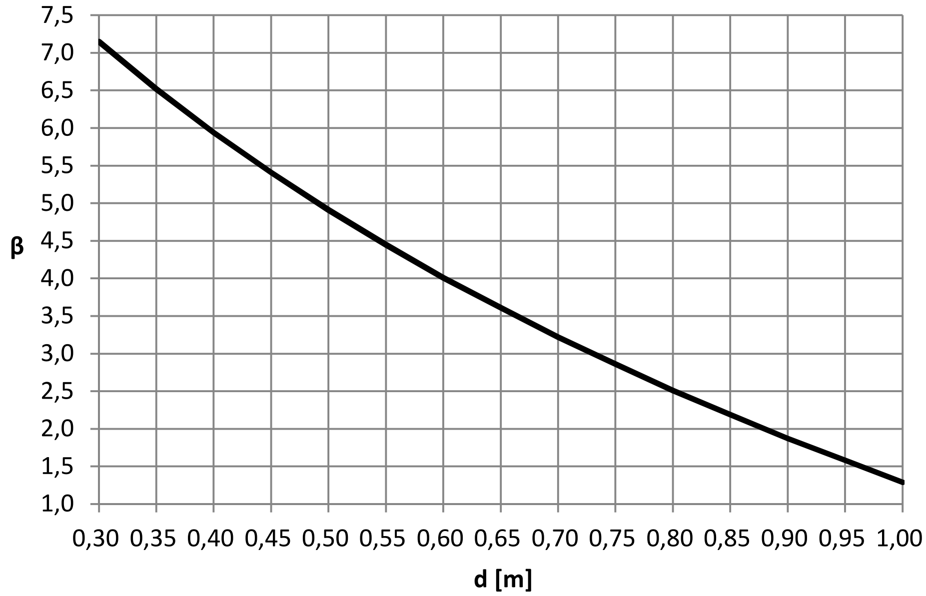


Figure 6.5: Variation of the reliability index β with the snow depth d

The figure shows how the reliability index decrease with the increasing of the snow depth. It follows that when the snow depth reach a limit value d_L the reliability become unacceptable and the stadium must be closed. Thus it is important to investigate on the limit value of the reliability index and the corresponding snow depth. The target values extracted from EN 1990, ISO 2394, ISO 13822, and JCSS and reported in §5.2 cannot be used in this case because they are about a reference period of 1 year, 50 years or the remaining working life. In this situation the reliability index β must be referred to a very short time interval and the use of equation (5.6) would conduct to meaningless values. It was decided to conduct an optimisation analysis and to permit the use of the stadium on the basis of the balance between the benefits and the consequences of failure. The use of the stadium is authorised when the benefit *B* associated with its use exceeds the probability of failure associated to a certain snow depth $p_F(d)$ multiplied for the consequences of failure C_f:

$$B \cdot [1 - p_F(d)] \approx B \geq C_f \cdot p_F(d) \qquad (6.9)$$

B and C_f need to be expressed in the same units for a considered snow depth. Usually monetary units are used. Realistically assuming that the benefit is less than the failure costs, $B < C_f$, then the target failure probability based on the economic optimisation, $p_{T,eco}$, is obtained from eq. (6.9):

$$p_F(d) \leq p_{T,eco} \approx B/C_f \tag{6.10}$$

The reliability index corresponding to the target probability is (for $B < C_f$):

$$\beta_{eco} = -\Phi^{-1}(p_{F,eco}) \approx -\Phi^{-1}(B/C_f) \tag{6.11}$$

where Φ^{-1} is the inverse cumulative distribution function of the standardised normal distribution. The reliability index β_{eco} is deemed independent of n and thus can be related to a short period of a single snowfall or to a longer period such as the accumulation from many different storms during the winter season.
Figure 6.6 indicates the variation of the target reliability index β_{eco} with the ratio B/C_f. The target level is approximately linearly proportional to the order of magnitude of the ratio.

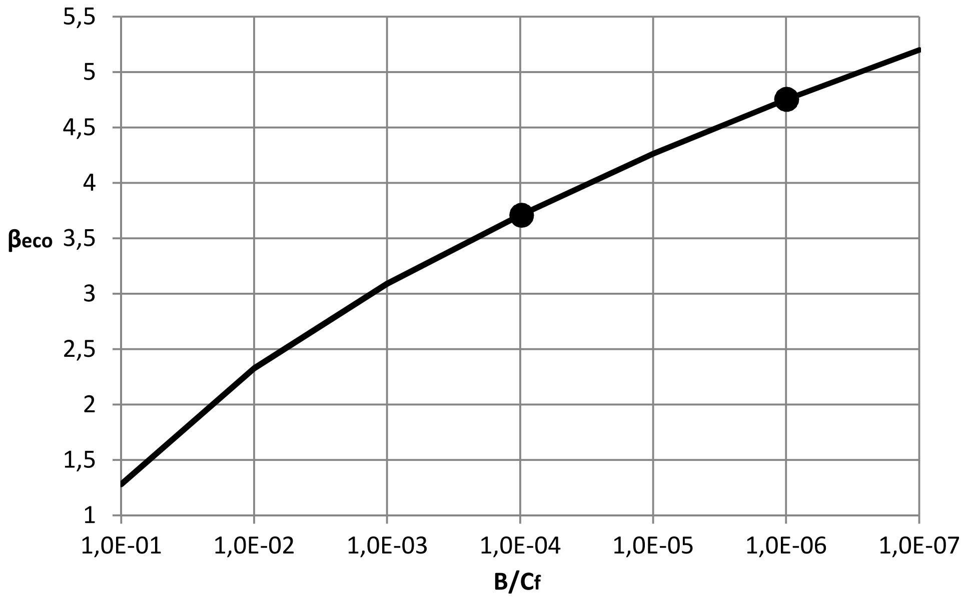


Figure 6.6: Variation of the target reliability index βeco with the ratio B/Cf

A possible approach is to consider the benefit as the average income deriving from the tickets sold to the public to enter into the stadium during a certain event and calculated multiplying the ticket cost and the number of tickets sold. Possible values for the ticket cost range from 1 € to 100 €. The estimation of the failure cost is very important but likely the most difficult step in the cost optimisation. In this case failure consequences need to explicitly account for societal consequences related to fatalities due to the roof failure. These consequences should be transformed into monetary units by multiplying the

expected number of fatalities and the Societal Value of a Statistical Life (SVSL) according to the Life Quality Index approach. The order of magnitude of the SVSL is 1000000 €.
It follows that plausible values for the ratio B/C_f are comprised between approximately 10^{-4} and 10^{-6}. The corresponding β_{eco} values are thus comprised between approximately 3.7 and 4.8 (Fig. 6.6). Entering in Fig. 6.4 with these values can be obtained the snow depth limits that determine the stadium closure. For β = 3.7 and β = 4.8 the snow depths are 64 cm and 51 cm respectively (Fig. 6.7), corresponding to a snow load on roof of 1.42 kN/m^2 and 1.08 kN/m^2 (equations (6.2), (6.3), and (6.4)).

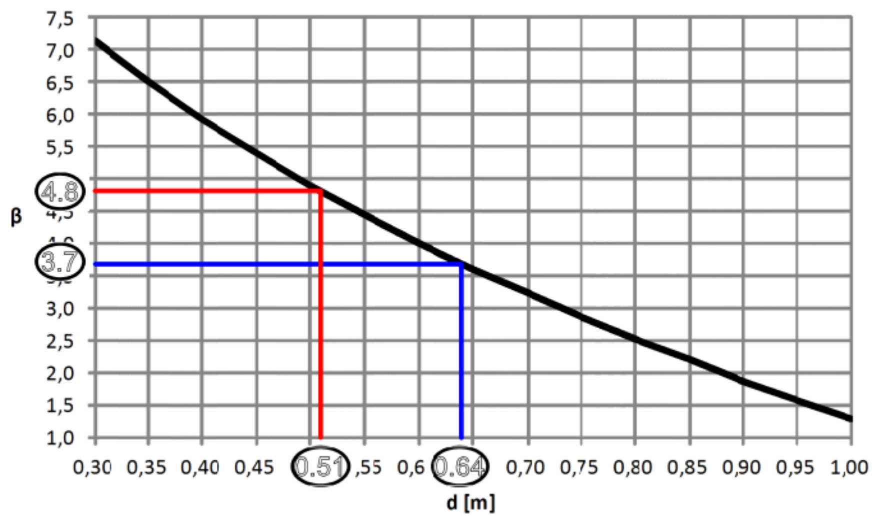


Figure 6.7: Determination of the snow depth limits

6.4 Concluding remarks

The reliability analysis highlighted that the roof of the stadium is able to sustain snow loads comparable with the snow load currently imposed by the Italian code for new structures, equal to 1.2 kN/m^2 for a monopitch roof located in the Alpine region at an altitude of 190 m. The calculated snow loads, equal to 1.08 kN/m^2 and 1.42 kN/m^2, have a probability comprised between about 0.05 and 0.003 to being exceeded for a reference period of one year, equivalent to a return period comprised between about 20 and 300 years. The calculation can be conducted assuming a Gumbel distribution (§5.4.3). Thus, regarding the ultimate limit state the structure can be considered safe and its use does not have to be restricted in normal situations and upgrading interventions are not necessary. This is an important and not obvious result due to the fact that the stadium roof was designed using a lower snow load and a different design method (permissible stress method). The second important result is related to the implementation of the permanent monitoring system of the snow depth on the roof of the stadium. This system provides a real time evaluation of the reliability level of the structure and supplies the necessary information that allow to decide if the stadium must be closed in case of an extraordinary heavy snowfall or when the accumulation from many different storms becomes excessive. This does not avoid possible damages or partial or total collapses but avoids multiple fatalities and injuries thus considerably reducing the total consequences of failure. In addition can be noted that the β values obtained through the optimisation analysis are lower than the value recommended by EN 1990 for reliability class 3 (RC3) for a period of 1 year, equal to 5.2.

This simple application shows the potentiality of the monitoring systems and how their implementation, together with the use of probabilistic methods, can improve the reliability of the structures. For this reason an extensive application to important structures and infrastructures such as stadiums, bridges, congress halls and multi-purpose arenas is desirable.

Chapter 7
Updating information in the change in use of an office building

7.1 Introduction

Inspection and monitoring data may be effectively used in the updating of the structural failure probability and in the updating of the probability distributions of basic variables. The updating of prior information by newly obtained measurements permits to reduce the uncertainties existing during the design and to refer to the actual as-built conditions. Information for the intents and purposes of assessment may be updated in a number of ways and may include:

- material properties determined by non-destructive or destructive testing
- geometric characteristics and permanent actions determined by component dimensions measured during inspection
- environmental effects identified during inspection
- damage and deterioration detected during inspection
- actual load carrying capacity estimated by proof loading

Probabilistic methods may be used to combine prior information about a variable with test results and measurements. This way to proceed allows to maximise knowledge on the structure and to minimise interventions and costs assuring an acceptable reliability, particularly important in case of doubts concerning actual reliability or serviceability, repair, strengthening, and change in use.

Direct updating of the structural failure probability can formally be carried out using the following basic equality from probability theory:

$$P(F|I) = \frac{P(F \cap I)}{P(I)} \tag{7.1}$$

where P denotes probability, F local or global failure, I inspection information, ∩ intersection of two events, and I conditional upon.

The updating procedure of the multivariate or individual probability distributions is given formally by:

$$f_X(x|I) = C \cdot P(I|x) \cdot f_X(x) \tag{7.2}$$

where X denotes a basic variable or statistical parameter, I an inspection result, $f_X(x)$ the probability density of X before updating, C a normalising constant, $f_x(x|I)$ the probability density of X after updating with information I, and $P(I|x)$ the likelihood to find information I for given value x of X.

Figure 7.1 shows corresponding prior and posterior probability density functions together with likelihood functions. In the first case the prior information is strong and the likelihood is weak (small sample size). In the second case the prior information is weak and the likelihood is strong. Finally in the last case the prior information and the likelihood are of comparable strength. As mentioned the likelihood is a measure for the probability of the observation given the true state of the structure. Note that in general the design value of the updated distribution might also be lower than the design value of the initial distribution.

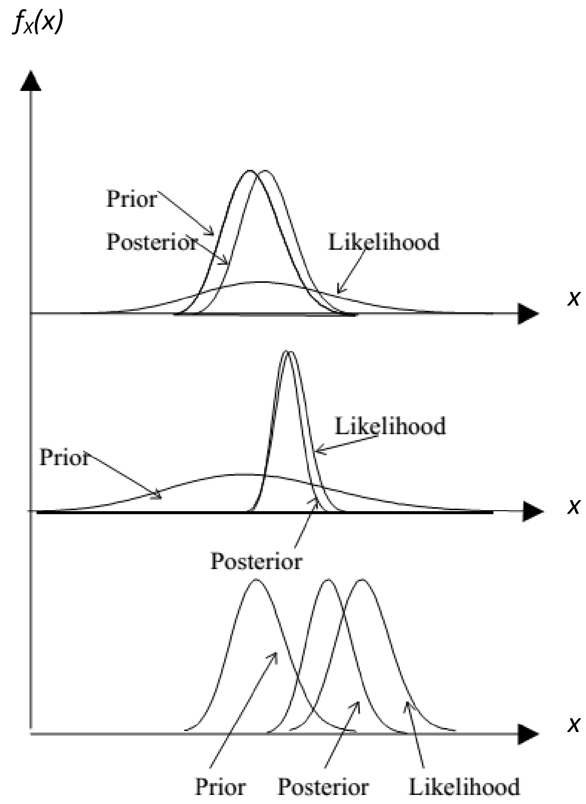


Figure 7.1: Illustration of updating of probabilistic models (JCSS - Assessment of Existing Structures, 2001)

A different way to express equation 7.2 is by application of Bayes theorem where the prior distribution functions are updated and transformed into posterior distribution functions. Assume that a random variable X has the cumulative distribution function $F_X(x)$ and density function $f_X(x)$. Furthermore assume that one or more distribution parameters, e.g. the mean value and standard deviation of X are uncertain themselves with probability density function $f_Q(q)$. then the probability density function for Q may be updated on the basis of observations of X, i.e. x_i. The general scheme for the updating is:

$$f_Q(q|x_i) = \frac{L(q|x_i)f_Q(q)}{\int_{-\infty}^{\infty} L(q|x_i)f_Q(q)dq} \tag{7.3}$$

where $L(q|x_i)$ is the likelihood of the observations or the test results contained in x_i, $f_Q(q)$ is the prior distribution function and $f_Q(q|x_i)$ is the posterior distribution function for the uncertain parameters Q. For discrete distributions the integral is replaced by summation. The observations x_i may not only be used to update the distribution of the uncertainty parameters Q but also to update the probability distribution of X. The updated probability density function of X is often called the predictive distribution of the Bayes distribution and may be assessed through:

$$f_X(x|I) = \int_{-\infty}^{\infty} f_x(x|q)\, f_Q(q|x_i)dq \tag{7.4}$$

In *Raiffa* and *Schlaifer* and *Aitchison* and *Dunsmore* a number of closed form solutions to posterior and predictive distributions can be found for special types of probability distribution functions. However, in practical situations there will always be cases where no analytical solution is available.

The updating procedure can be used to derive updated characteristic and representative values of basic variables to be used in the partial factor method or to compare directly action effects with limit values (cracks, displacements). Once the updated distributions for the basic variables $f_X(x|I)$ have been found, the updated failure probability $P(F|I)$ may be determined by performing a probabilistic analysis using common method of structural reliability for new structures. Symbolically it can be written

$$P(F|I) = \int_{g(X)<0} f_X(x|I)dx \tag{7.5}$$

where $f_X(x|I)$ denotes the updated probability density function and $g(x) < 0$ the failure domain ($g(x)$ being the limit state function).

7.2 Updating of material strength

Consider herein a common situation, the updating of resistance properties for structural elements and materials using tests. The results may be used in connection with the partial factor format or with a probabilistic design method to update a predefined prior distribution.

If the partial factor method is used either the classical method or the Bayesian method may be applied. Both methods are used in practice and in many cases the numerical values will not differ considerably. A recommendable procedure is to evaluate the tests using both methods, compare the results and select the most unfavourable. In the classical approach the design resistance R_d is to be calculated by the formula (ISO 2394):

$$R_d = \frac{R_{k,est}}{\gamma_m} \cdot \frac{\bar{\eta}}{\gamma_{Rd}} \tag{7.6}$$

where $R_{k,est}$ is the lower characteristic value R_k of the resistance determined statistically from tests, γ_m is the partial factor for material, $\bar{\eta}$ is the mean value of the conversion coefficient or modification factor, γ_{Rd} is the model uncertainty coefficient.

The lower characteristic value $R_{k,est}$ is estimated from the test results, taking into account a confidence level of at least 0.75. In the absence of other information, the characteristic value is assumed to be the 0.05 fractile of a normal distribution. The characteristic value is estimated by:

$$R_{k,est} = m_R - k_S s_R \tag{7.7}$$

where m_R is the sample mean value, s_R is the sample standard deviation, k_S is the coefficient depending on the sample size. The value k_S depend on the number of tests, n, and on the chosen confidence level. Table 7.1 gives k_S-values for the 0.01, 0.05 and 0.10 fractiles and confidence level of 0.75. The standard deviation s_R is to be established from the test results. In some cases the standard deviation may be considered to be known a priori. In that case:

$$R_{k,est} = m_R - k_\sigma \sigma_R \tag{7.8}$$

where m_R is the sample mean value, σ_R is the sample standard deviation, k_σ is the coefficient depending on the sample size. The value of k_σ should be taken from Table 7.2.

Probability	**Number of tests, *n***								
P	3	4	6	8	10	20	30	100	∞
0,10	2,50	2,13	1,86	1,74	1,67	1,53	1,47	1,38	1,28
0,05	3,15	2,68	2,34	2,19	2,10	1,93	1,87	1,76	1,64
0,01	4,40	3,73	3,24	3,04	2,93	2,70	2,61	2,46	2,33

Table 7.1: Values of k_s, s_R unknown (Confidence level = 0.75), (ISO 2394, 2015)

Probability	**Number of tests, *n***								
P	3	4	6	8	10	20	30	100	∞
0,10	1,67	1,62	1,56	1,52	1,50	1,43	1,40	1,35	1,28
0,05	2,03	1,98	1,92	1,88	1,86	1,79	1,77	1,71	1,64
0,01	2,72	2,66	2,60	2,56	2,54	2,48	2,45	2,39	2,33

Table 7.2: Values of k_σ, σ_R unknown (Confidence level = 0.75), (ISO 2394, 2015)

In the above procedure the normal distribution is used. This assumption can be regarded as a relatively conservative one. In reality, one may also consider Lognormal or Weibull distributions to find more economic design values.
In the Bayesian method the design value may be estimated directly from test data:

$$R_d = \eta_d \left\{ m_R - t_{vd} s_R \sqrt{\left(1 + \frac{1}{n}\right)} \right\} \tag{7.9}$$

where m_R is the sample mean value, s_R is the sample standard deviation, t_{vd} is the coefficient of the Student distribution (Table 7.3), n is the number of tests, η_d is the design value of the conversion factor.
Values for t_{vd} follow from Table 7.3, where $v = n-1$, $\beta_R = \alpha_d\beta$, where β is the target reliability index and α_d the design value for the FORM influence coefficient. Without further indication, one should use α_d = 0.8 if the uncertainty of R is dominating and α_d = 0.3 otherwise. Equation 7.9 can be used directly within the design value method. For use within the partial factor method, two ways are possible:

a) The characteristic value R_k is used, using the same equation, but with β_R = 1.64 (the partial factor follows from $\gamma_m = R_k/R_d$).
b) The γ_m value normally used for the type of material and failure mode is used; in this way, the characteristic value R_k is defined as $R_k = \gamma_m R_d$; note that in this case R_k may have a probability of exceeding the limit value different from 0.95.

Equation 7.9 is based on a normal distribution for R and a non-informative prior distribution for both the standard deviation and the mean. If the standard deviation is known in advance, one may replace the same standard deviation by the distribution standard deviation and take $v = \infty$.
The Bayesian method as presented in this subcase is very sensitive to the observed standard deviation σ_R, if this quantity is not known in advance. It might be advisable to eliminate excessively small and large values of the posterior standard deviation in order to avoid unsafe or uneconomic results.

		Degrees of freedom, v								
β_R	$\Phi(-\beta_R)$	1	2	3	5	7	10	20	30	∞
1,28	0,10	3,08	1,89	1,64	1,48	1,42	1,37	1,33	1,31	1,28
1,64	0,05	6,31	2,92	2,35	2,02	1,89	1,81	1,72	1,70	1,64
2,33	0,01	31,8	6,97	4,54	3,37	3,00	2,76	2,53	2,46	2,33
2,58	0,005	63,7	9,93	5,84	4,03	3,50	3,17	2,84	2,75	2,58
3,08	0,001	318	22,33	10,21	5,89	4,78	4,14	3,55	3,38	3,09
NOTE — If σ_R is known, $v = \infty$ should be used.										

Table 7.3: Values of t_v (ISO 2394, 2015)

As an example, consider a sample of n = 3 test pieces, having a sample mean m equal to 100 kN and a sample standard deviation s_R equal to 15 kN. The 5% characteristic value is given by (v =2):

$R_k = m_R - 2.92 s_R \sqrt{1 + \frac{1}{3}} = 100 - 3.37 \cdot 15 = 49.5\ kN$

Note that the classical method would lead to: $R_k = m_R - 3.15 s_R = 52.8\ kN$. The result is almost the same.

In a full probabilistic treatment the first step is the establishment of a so-called prior distribution function. Given this prior distribution and given the statistical test data, a posterior distribution can be derived. Consider the case that *R* has a normal distribution the prior distribution is given by:

$$f'(\mu, \sigma) = k\sigma^{-(v' + \delta(n') + 1)} exp\left\{-\frac{1}{2\sigma^2}\{v'(s)^2 + n'(\mu - m')^2\}\right\} \tag{7.10}$$

where *δ(n') = 0* for *n' = 0* and *δ(n') = 1* otherwise.

The parameters *s'* and *v'* characterize the prior information about the standard deviation. The expectation and the coefficient of variation of the standard deviation σ can be expressed as:

$$E(\sigma) = s' \tag{7.11}$$

$$V(\sigma) = \frac{1}{\sqrt{2v'}} \tag{7.12}$$

The prior information about the mean is characterized by *m'*, *n'* and *s'*. The expectation and the coefficient of variation of the mean *μ* can be expressed as:

$$E(\mu) = m' \tag{7.13}$$

$$V(\mu) = \frac{s'}{m'\sqrt{n'}} \tag{7.14}$$

It is also possible to interpret the prior information as the result of hypothetical prior test series, one for the mean and one for the standard deviation. In that case, *m'* and *s'* represent the best estimates for the mean and the standard deviation, *v'* is the hypothetical number of degrees of freedom for *s'* and *n'* is the hypothetical number of observations for *m'*.

Also note that for a test, we normally have *v = n – 1*, but that the prior parameters *n'* and *v'* may be chosen independently from each other. According to ISO 2394, if very little information is available, *n'* and *v'* should be chosen equal to zero. In many cases it seems reasonable to assume that there is very little or no prior information on the mean (so *n' = 0*), but that it is possible to obtain a fairly good estimate of *σ'*.

Using equation 7.2 one may combine the prior information characterized by equation 7.10 and a test result of *n* observations with sample mean *m* and sample standard deviation *s*. The result is a posterior distribution for the unknown mean and standard deviation of *R*, which is again given by equation 7.10, but with parameters given by the following updating rules:

$$n'' = n' + n \tag{7.15}$$
$$v'' = v' + v + \delta(n') \tag{7.16}$$
$$m''n'' = m'n' + mn \tag{7.17}$$
$$v''(s'')^2 + n''(m'')^2 = v'(s')^2 + n'(m')^2 + vs^2 + nm^2 \tag{7.18}$$

Using equation 7.2 the predictive value of *R* can be found from:

$$R = m'' - t_{v''}s''\sqrt{\left(1 + \frac{1}{n''}\right)} \tag{7.19}$$

Values of $t_{v''}$ for given probabilities of exceeding the limits are given in Table 7.3.
If *R* has a lognormal distribution, *Y = ln(R)* has a normal distribution. The previous relations can be used introducing *Y* and the results can be obtained as $R = e^Y$.

In order to update material strength, load tests may be executed as well. Proof loading or load testing is a special kind of quantitative inspection in which the structural element is subjected to an increasing load intensity to assess its actual load carrying capacity. A successful proof load test demonstrates immediately that the resistance of the structural element is greater than the proof load. This reduces uncertainty associated with the resistance and increases its reliability even though it does not reveal its actual capacity and does not provide a meaningful measure of the safety of the structural element as well. The probability of failure in the redesign stage can be evaluated by using the conditional probability expression:

$$p_f = P[R - E < 0 | R' - E' > 0] = \frac{P[R - E \leq 0 \cap R' - E' > 0]}{P[R' - E' > 0]} \tag{7.20}$$

where *R'* and *E'* denote the resistance and the total load effect at the moment of the load test. Equation 7.20 has a greater impact if the load successfully borne was high.
If the failure functions in 7.20 are assumed to adopt a simple fundamental two-dimensional form:

$$Z = R - E \tag{7.21}$$

and *E'* is deterministic, the resistance distribution $f_R(x)$ can be truncated on the lower side as:

$$f_{R'}(x) = \frac{1}{1 - F_R(E')} f_R(x) \qquad for\ x > E' \tag{7.22}$$

where $f_R(x)$ is the original strength distribution.
Assuming that the strength is normally distributed with a mean μ_R and a standard deviation σ_R the following can be defined:

$$\lambda = \frac{E' - \mu_R}{\sigma_R} \tag{7.23}$$

The mean and the standard deviation of the calibrated strength distribution $f_R'(x)$ are obtained as follows:

$$\mu_R{}' = \mu_R + \frac{\varphi(\lambda)}{1 - \Phi(\lambda)} \sigma_R \tag{7.24}$$

$$\sigma_R{}' = \left[1 + \frac{\lambda\varphi(\lambda)}{1-\Phi(\lambda)} - \left(\frac{\varphi(\lambda)}{1-\Phi(\lambda)}\right)^2\right]^{1/2} \sigma_R \tag{7.25}$$

where $\varphi(.)$ Is the probability density function for the standardised normal variable, $\Phi(.)$ is the standard normal integral.
If load *E'* is not deterministic but random (normal or Gumbel distributions may be assumed), function $f_R'(x)$ can be evaluated numerically with the probability density function $f(E')$ from:

$$f_{R'}(x) = \int_0^\infty \frac{f_R(x)}{1 - F_R(E')} f(E') dE' \tag{7.26}$$

The truncated standard normal distribution, with μ = 0 and σ = 1, is illustrated for E' = -0.5, 0, and 0.5 in Figure 7.2.

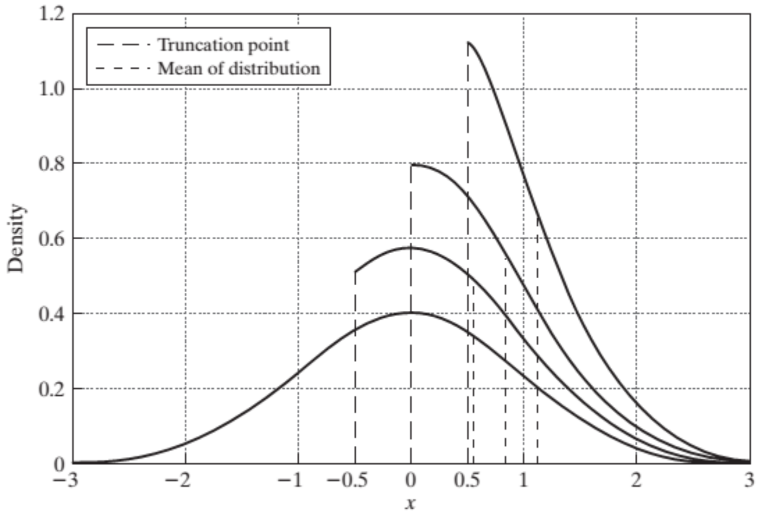


Figure 7.2: Truncated normal distributions (Greene, 2011)

In case the resistance has a lognormal distribution the previous equations can be used substituting $ln(E')$ to E', μ_{lnR} to μ_R, and σ_{lnR} to σ_R:

$$\sigma_{\ln R} = \sqrt{\ln(v_R^2 + 1)} \tag{7.27}$$

$$\mu_{\ln R} = \ln(m_R) - \frac{1}{2}\sigma_{\ln R}^2 \tag{7.28}$$

It should be also recognised that there is a risk that the structural element will be damaged or not survive a proof load test and so proof load testing may not always be cost-effective. A reliability analysis may be conducted to determine the target proof load including information from all resistance and loading variables that influence the assessment process. In Figure 7.3 the probability of failure during the test and after the test are shown. A steel bar subjected to tension is considered.

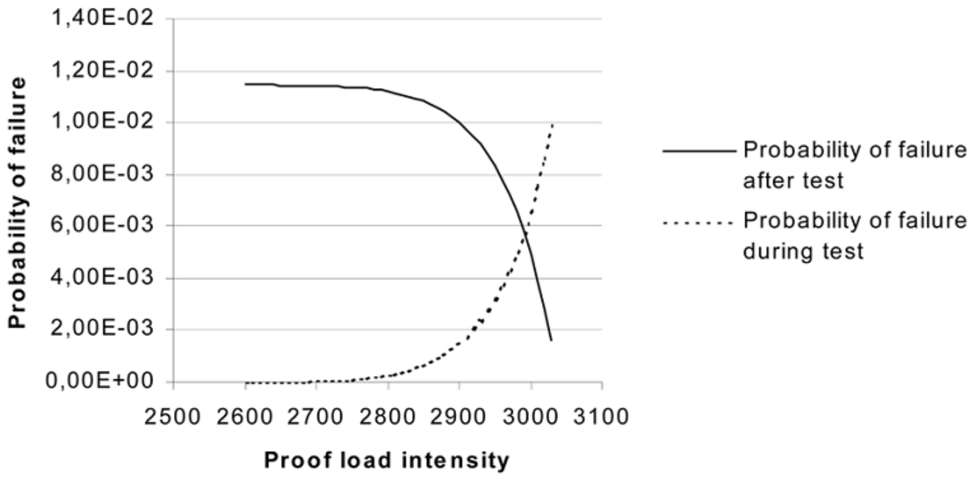


Figure 7.3: Probability of failure as function of the proof load intensity (Diamantidis , Holickỳ et al., 2012)

It is seen that there is a close relationship between the benefit of the proof test i.e. a decrease in the failure probability after the test and the risk of failure during the test. A decision analysis where the costs of failure during the test, cost of failure after the test and costs of the test itself are included, can assist in deciding whether a proof load test should be performed.

Note that the same approach is used if the structure has survived an extreme load during its past lifetime. The load may be considered as the load test and the previous relations may be applied.

7.3 Description of the structure and reliability analysis

The structure examined is a steel beam that sustains a floor in an office building. Due to the change in use of the building from offices not open to the public to offices open to the public, and the consequently increase of loads, the semi-probabilistic verifications are not satisfied thus a reliability analysis is conducted to verify the actual reliability. The steel beam (IPE 400) is simply supported, it has a span length *l* of 7 m and it was designed using Eurocodes. The distance between two following beams is *d* = 5 m.

The limit state function Z(.) is:

$$Z(\boldsymbol{X}) = \vartheta_R R - \vartheta_E(G+Q) = \vartheta_R W_{pl} f_y - \vartheta_E \frac{l^2}{8}\left(\gamma_g \cdot A + g \cdot d + q \cdot d\right) \qquad (7.29)$$

Notation and probabilistic models of all the basic variables are reported in Table 7.4:

Basic variable	Symbol	Dimension	Distribution	Mean	CoV
Span of the beam	l	m	DET	7	-
Sectional area (IPE 400)	A	m^2	DET	0.008446	-
Distance of beams	d	m	DET	5	
Steel density	γ_G	kN/m^3	N	77	0.01
Weight of slab and floor layers	g	kN/m^2	N	5.5	0.05
Section modulus (IPE 400)	W_{pl}	m^3	DET	0.001307	-
Yield strength (S275)	f_y	N/mm^2	LN	308.6	0.07
Initial imposed load	q_i	kN/m^2	GAMMA	0.62	1.10
Final imposed load	q_f	kN/m^2	GAMMA	0.94	1.10
Resistance uncertainty	θ_R	-	LN	1.15	0.10
Load effect uncertainty	θ_E	-	LN	1.0	0.10

Table 7.4: Models of basic variables

Resistance of the steel beam, according to f_y, is described by a lognormal distribution with a coefficient of variation of 0.07. The mean of the resistance is obtained as 1.122 times the characteristic value. This factor derives from the definition of the characteristic value of a lognormal distribution: $X_k = \mu \cdot \exp(-1.645 V_m) \rightarrow \mu = X_k \cdot \exp(1.645 V_m) = 1.122 \cdot X_k$. The resistance is assumed to be time-independent and given that deterioration is not observed the reference value of the yield strength for steel S275 is adopted and in-situ measurements are not conducted.

The permanent action is described by the sum of two normal distributions: the first represents the weight of the steel beam and has a mean of 77 kN/m^3 and a coefficient of variation of 0.01; the second represents the weight of slab and floor layers and has a mean of 5.5 kN/m^2 and a coefficient of variation of 0.05.

The imposed load q is described by a Gamma distribution with a coefficient of variation of 1.10. The mean value increases from 0.62 to 0.94 kN/m^2 from initial to final situation. These values are calculated assuming a Gamma distribution with a representative value, corresponding to 95% fractile, of 2 kN/m^2 for the initial situation and of 3 KN/m^2 for the final situation, according to Eurocodes. Note that referring to JCSS – Probabilistic model code higher values of the coefficient of variation may be obtained. In the present case the calculated coefficient of variation of 1.42 is considered too high, even though can be observed that it conducts to results that are on the safe side. Assuming 1.42, β values are lower of about 0.20-0.25 than the values calculated in the following for initial and final situation.

The model uncertainties are described by the lognormal distribution. Assuming steel section subjected to bending about the strong axis when no stability phenomena are taken into account, the mean 1.15 and the coefficient of variation 0.10 of the model uncertainties for resistance are accepted (*Nadolski* and *Sýkora*). The load effect uncertainty has a mean of 1.0 and a coefficient of variation of 0.10.

The reliability analysis is conducted using the software *VaP 1.6* and FORM. The reliability indexes for initial and final situation are:

Initial situation: $\beta_i = 3.85$
Final situation: $\beta_f = 3.17$

The reliability index for the initial situation is very close to the minimum β value recommended by the Eurocode for new structures considering CC2 and reference period of 50 years (Table 5.3). The final β, according to Table 5.4 for existing structures, indicates that the structure is fit for use even though the target value for repair is not reached. However, the target value for repair, equal to 3.3, is here considered as the minimum reliability index to be achieved in the final situation to judge the structure safe and not to repair or strengthen it.

7.4 Probabilistic updating

Due to the existing uncertainties related to the resistance of the steel beam and to the actual safety of the structure, it has been decided to plan and execute a number of tests. Material properties are determined testing miniaturised specimens drilled from structural members without reducing their resistance and evaluating to conduct proof load test on the structural element. Non-destructive testing as Brinell hardness tests for metallic materials are not conducted in the present.
In normal daily practice only a limited number of tests on specimens can be carried out for economical reasons. In the present case for example, the number of tests is five. If only a limited number of tests on material samples are available the evaluation of test results according to standard statistical methods may lead to unrealistic low characteristic or design values. This drawback can be avoided, if the evaluation of test samples with a limited number of tests is carried out according to statistical models which permit the introduction of prior knowledge. Based on prior knowledge about the distribution of the investigated variable, a posterior distribution is derived in combination with the obtained test results. It is known from previous experience and according to Table 7.4 that for the yield strength of steel S275 a lognormal distribution can be assumed. The following information is available regarding its strength:

$m' = 308.6\ MPa$, $V(\mu) = 0.50$, $s' = 21.6\ MPa$, $V(\sigma) = 0.32$

For the unknown characteristics n' and v', according to ISO 2394, can be assumed:

$$n' \approx 0,\ v' = \frac{1}{2 \cdot 0.32^2} \approx 5$$

The following strength characteristics have been obtained from the five miniaturised specimens:

n = 5, v = n – 1 = 4, m = 324.6 MPa, s = 28.4 MPa

Using equations 7.15, 7.16, 7.17, and 7.18 and considering that yield strength has a lognormal distribution, the updated characteristics are:

$$n'' = 0 + 5 = 5$$
$$v'' = 5 + 4 = 9$$
$$m'' = 324.6\ MPa$$
$$s'' = \sqrt{[5 \cdot 21.6^2 + 4 \cdot 28.4^2]/9} = 24.9\ MPa$$

Thus, using the previous information, the standard deviation of the new measurements can be decreased from s = 28.4 MPa to s = 24.9 MPa. However, it should be noted that the combination of the previous information with the current measurements might not always lead to favourable results.

The applied simplified technique is not the only procedure to combine data affected by different uncertainties. More advanced procedures based on the Bayesian approach or on the likelihood representation of uncertainties may be find in the literature and may be applied.

In addition further data regarding cross-section area and actions may be collected. In case of metallic materials cross-section area may be consistently influenced by corrosion. When severe corrosion is observed an equivalent cross-section can be introduced following an extensive measurement of the actual dimensions. Measurements of the effects of actions on the structure can lead to interesting results as well and avoid unnecessary interventions and costs. This is especially true in case of structures mainly subjected to climatic actions or bridges subjected to particular traffic actions. In the present case severe corrosion is not observed and actions cannot be precisely measured thus these investigations are not conducted.

Based on the updating of material properties a new reliability analysis is now conducted. The results, expressed in terms of reliability index, are reported in the following:

Initial situation: β_i' = 4.00

Final situation: β_f' = 3.31

The results highlight that new information, derived from tests, lead to an increased reliability and to a reduced probability of failure. In particular, the reliability index in the final situation reach the minimum acceptable value here considered. Thus the structure can be considered safe and fit for use and structural interventions are not necessary.

As mentioned in the previous paragraph proof load tests can be applied to a structure to verify its load carrying capacity as well. Before testing the steel beam, gradually raising the load intensity, a theoretical study to determine the suitable proof load intensity is conducted. The event of primary importance is the probability of test induced failure. A second concern is the reliability index, β, which can be deduced from the field test after reaching a given level of proof loading. The difference between the reliability index estimated after the test and the initial reliability index computed with standard calculation procedures is regarded as the measure of the benefit of testing.
The reliability analysis of the proof load test is performed replacing the imposed load q in equation 7.27 with the proof load p_{PL}. Three probabilistic models for the proof load may be assumed: Gumbel distribution, Normal distribution and deterministic. In the present the proof load is assumed to be deterministic even though assuming a Normal distribution with a small coefficient of variation, equal to 0.05 for instance, could be more appropriate. Assuming a Gumbel distribution the theoretical probability of failure during load testing is greater and it seems to be too conservative in this case. Different β values are obtained for different p_{PL} . Results are plotted in Figure 7.4:

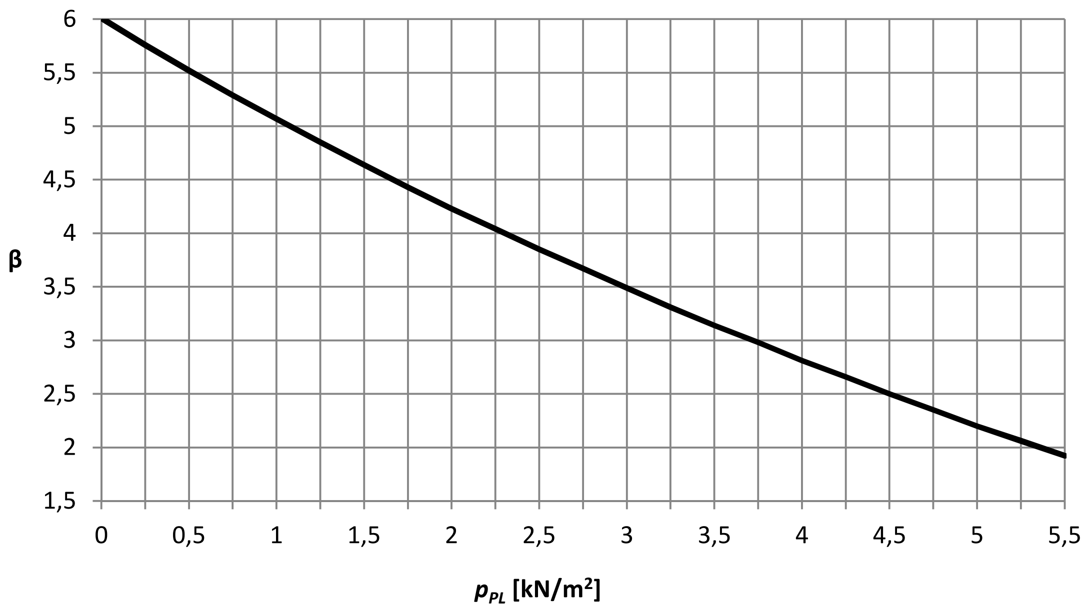


Figure 7.4: Variation of the reliability index

Figure 7.4 shows, according to paragraph 7.2, the decreasing of the reliability index and correspondingly the increasing of the probability of failure during the test with the increasing of the intensity of the proof load used during the test.
After the proof load test the resistance distribution, described by a lognormal distribution, is truncated on the left side. For the different values of p_{PL} is calculated the corresponding truncation values and the updated β values, denoted as β''. The updated reliability indexes may be calculated approximating the truncated lognormal distribution of f_y (yield strength)

with a not truncated lognormal distribution that has the same mean and standard deviation.

p_{PL} *[kN/m^2]*	***$f_{y,min}$*** *[MPa]*	***μ_R''*** *[MPa]*	***σ_R''*** *[MPa]*	**β''**
0.0	131.9	308.6	21.6	3.17
0.5	143.6	308.6	21.6	3.17
1.0	155.4	308.6	21.6	3.17
1.5	167.1	308.6	21.6	3.17
2.0	178.8	308.6	21.6	3.17
2.5	190.5	308.6	21.6	3.17
3.0	202.2	308.6	21.6	3.17
3.5	213.9	308.6	21.6	3.17
4.0	225.6	308.6	21.6	3.17
4.5	237.4	308.6	21.6	3.17
5.0	249.1	308.6	21.5	3.17
5.5	260.8	309.0	21.0	3.18

Table 7.5: Truncation values and updated reliability indexes β''

The results highlight that, for the considered p_{PL}, the proof load test does not improve significantly the reliability of the steel beam (Table 7.5). This is connected to the fact that the truncation value $f_{y,min}$ is really small and thus the truncation only involves a small part of the left tail. Assuming higher values for p_{PL}, higher β'' values could be obtained but the reliability index during the testing would be unacceptably low.

7.5 Concluding remarks

This chapter introduces the basic theory of updating in the assessment of existing structures, focusing on the most common situation: updating of material strength.

Some observations can be derived from the example examined. First it is shown how is possible to update the reliability of a structure introducing new information derived from tests. The combination of prior and newly obtained information permits to reduce the uncertainties about the current state of the structural element. This is useful in different applications: cases which doubts on deterioration and material performance are present, cases which are so particular that the data commonly applied for calculation do not reflect the actual circumstances, cases when the existing design formulae seem to lead to very conservative results, and derivation of new design formulae. In the present case the destructive tests on specimens permits to obtain an higher reliability index and to reduce the probability of failure. This is a particularly favourable situation. In general the updating can lead to opposite results as well: lower reliability index and increased probability of failure.

Secondly the application of a proof loading is examined. The results above obtained show that the proof load test gives modest results in the updating of structural reliability. The reliability index of the test decreases from about 6.0 to about 2.0 increasing the proof load intensity from 0 kN/m^2 to 5.5 kN/m^2 whilst the reliability index calculated using the approximated truncated resistance distribution does not change significantly due to the fact that the truncation involves only a small part of the left tail. It is possible to conclude that the proof load test in the present case and in similar situations i.e. ordinary residential and office buildings, cannot be effectively used to update the reliability index and the probability of failure.

Chapter 8
Conclusions

This thesis faced the inspection and monitoring topics with respect to the updating of structural reliability. The following concluding remarks can be drawn:

The realisation of a comprehensive reference code representing a landmark in inspection and monitoring planning, design and in practical applications is desirable. A breakthrough point could be to focus the attention on practical aspects such as the updating of structural reliability and the decision making process.

Development of effective maintenance plans extended to all structures and infrastructures will simplify the inspection tasks and minimize the life cycle cost keeping the performance above a minimum target level.

Based on inspection and monitoring data both partial factors and probabilistic distribution for basic variables can be updated to account for reduced uncertainties and actual as-built conditions. These advanced verifications may avoid expensive and unnecessary interventions.

Practical examples show the possible applications and the potentiality of tests and monitoring systems. Interesting and fascinating applications are represented by continuous monitoring and real time evaluations. These applications permit to reduce uncertainties, maximize reliability, and minimize costs.

Annex A
Spreadsheet for partial safety factors

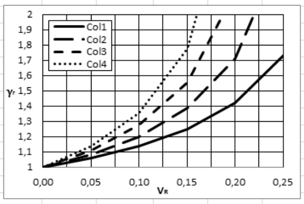
In the context of this thesis has been realised a spreadsheet that allows to easily calculate the partial safety factors using equations 5.39, 5.40, 5.41, 5.48, 5.49. Input data are the FORM sensitivity factors α_R or α_E, reliability index β, coefficient of variation *V*, and reference period t_{ref} for variable actions. The spreadsheet also plots diagrams where the partial safety factor and the coefficient of variation or the reference period are reported.
In the following some screenshots are reported.

Partial factor for resistance R

NORMAL distribution: $\gamma_r = \dfrac{(1-1.645V_R)}{(1-\alpha_R\beta V_R)}$

FORM sensitivity α_R = 0,8
Reliability index β = 3,8 ⟶ Partial factor γ_r = 1,20
Coefficient of Variation V = 0,1

	β			
CoV	3,3	3,8	4,3	4,8
0	1	1	1	1
0,05	1,057316	1,082252	1,108394	1,135829
0,1	1,13519	1,200431	1,273628	1,356331
0,15	1,247103	1,384651	1,556302	1,776533
0,2	1,42161	1,711735	2,150641	2,892241
0,25	1,731618	2,453125	4,205357	14,71875



LOGNORMAL distribution $\gamma_r = \dfrac{\exp(-1.645V_R)}{\exp(-\alpha_R\beta V_R)}$

FORM sensitivity α_R = 0,8
Reliability index β = 3,8 ⟶ Partial factor γ_r = 1,15
Coefficient of variation V_R = 0,1

	β			
CoV	3,3	3,8	4,3	4,8
0	1	1	1	1
0,05	1,051008	1,07224	1,093901	1,115999
0,1	1,104618	1,149699	1,196619	1,245454
0,15	1,160963	1,232753	1,308982	1,389925
0,2	1,220182	1,321807	1,431897	1,551155
0,25	1,282421	1,417295	1,566353	1,731088

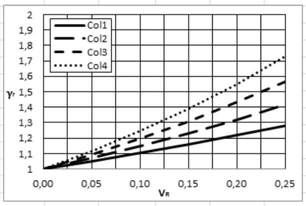


Figure A.1: Partial factor for resistance *R*

Partial factor for permanent loads G

NORMAL distribution: $\gamma_g = \dfrac{\mu_G(1-\alpha_E\beta V_G)}{\mu_G}$

FORM sensitivity α_E = -0,7
Reliability index β = 3,8 ⟶ Partial factor γ_g = 1,27
Coefficient of variation V_G = 0,1

	β			
CoV	3,3	3,8	4,3	4,8
0	1	1	1	1
0,05	1,1155	1,133	1,1505	1,168
0,1	1,231	1,266	1,301	1,336
0,15	1,3465	1,399	1,4515	1,504
0,2	1,462	1,532	1,602	1,672
0,25	1,5775	1,665	1,7525	1,84

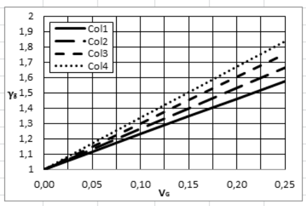


Figure A.2: Partial factor for permanent loads *G*

Partial factor for variable actions Q

GUMBEL distribution:
(CLIMATIC ACTIONS)

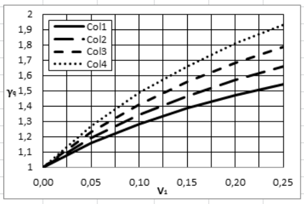
$$\gamma_q^{clim} = \frac{1 + V_1 (0{,}78 ln t_{ref} - 0.45 - 0.78 \ln(-\ln(\Phi(-\alpha_E \beta_1))))}{1 + V_1 (-0.45 - 0.78 \ln(-\ln(0.98)))}$$

FORM sensitivity α_E =	-0,7
Reliability index β_1 =	3,3
Coefficient of variation V_1 =	0,1
Reference period t_{ref} =	50

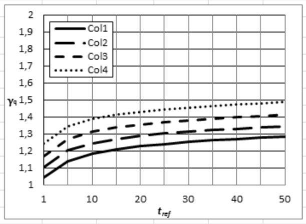
$\longrightarrow$ Partial factor γ_q = 1,28

The coefficient of variation and the reliability index refer to the period of 1 year

	β			
CoV	3,3	3,8	4,3	4,8
0	1	1	1	1
0,05	1,157652	1,191711	1,22958	1,271352
0,1	1,282837	1,343941	1,411881	1,486822
0,15	1,384649	1,467747	1,560143	1,66206
0,2	1,469074	1,570411	1,683087	1,807373
0,25	1,540215	1,656922	1,786686	1,929822



	β			
t_{ref}	3,3	3,8	4,3	4,8
1	1,04054	1,101643	1,169583	1,244524
5	1,140223	1,201326	1,269266	1,344207
10	1,183154	1,244258	1,312198	1,387138
15	1,208267	1,269371	1,337311	1,412251
20	1,226085	1,287189	1,355129	1,43007
25	1,239906	1,30101	1,36895	1,44389
30	1,251199	1,312302	1,380242	1,455183
35	1,260746	1,321849	1,38979	1,46473
40	1,269017	1,33012	1,39806	1,473001
45	1,276312	1,337415	1,405355	1,480296
50	1,282837	1,343941	1,411881	1,486822



GUMBEL distribution:
(IMPOSED LOADS)

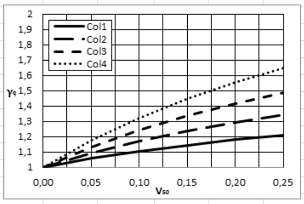
$$\gamma_q^{imp} = \frac{1 + V_{50} (0.78 \cdot \ln(t_{ref}/50) - 0.45 - 0.78 \ln(-\ln(\Phi(-\alpha_E \beta_{50}))))}{1 + V_{50} (-0.45 - 0.78 \ln(-\ln(0.95)))}$$

FORM sensitivity α_E =	-0,7
Reliability index β_{50} =	3,8
Coefficient of variation V_{50} =	0,16
Reference period t_{ref} =	50

$\longrightarrow$ Partial factor γ_q = 1,25

The coefficient of variation and the reliability index refer to the period of 50 year

	β			
CoV	3,3	3,8	4,3	4,8
0	1	1	1	1
0,05	1,056584	1,091774	1,130902	1,174062
0,1	1,104266	1,169111	1,241212	1,320743
0,15	1,144994	1,23517	1,335435	1,446032
0,2	1,180187	1,292249	1,41685	1,55429
0,25	1,2109	1,342064	1,487903	1,64877



	β			
t_{ref}	3,3	3,8	4,3	4,8
1	0,776502	0,871306	0,976719	1,092992
5	0,931165	1,025969	1,131382	1,247656
10	0,997775	1,092579	1,197991	1,314265
15	1,036739	1,131543	1,236956	1,353229
20	1,064384	1,159189	1,264601	1,380875
25	1,085828	1,180632	1,286045	1,402319
30	1,103349	1,198153	1,303565	1,419839
35	1,118162	1,212967	1,318379	1,434653
40	1,130994	1,225799	1,331211	1,447485
45	1,142313	1,237117	1,34253	1,458803
50	1,152438	1,247242	1,352654	1,468928

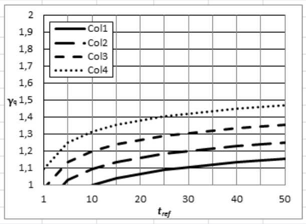


Figure A.3: Partial factor for variable actions *Q*

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