APPRAISAL OF METHODS FOR SAFETY EVALUATION AND RISK MANAGEMENT

IM-SAFE.EU

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Preface

This report is part of the H2020 CSA IM-SAFE project results and is the outcome of WP3 (Data-informed safety evaluation and maintenance strategies) Task 3.1 (Safety evaluations and risk management methods) activities, listed as delivery D 3.1. It constructs a part of the technical background for the formulation of the proposal for the mandate to CEN for a further amendment to the existing EU standards on data-informed safety assessment taking into account inspections, monitoring and testing and for a new standard for preventive maintenance of transport infrastructure.

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WP3 contributes to the identification of the normative gaps with regard to data-informed safety assessment, risk assessment and maintenance decision-making based on review of the current state-of-the-art as represented by standards, guidelines, other regulations as well as current practice and research. The particular focus of Task 3.1 is on data-informed assessment taking into account inspections, monitoring and testing.

This report includes the review of the methodologies for performance assessment, including data-informed approaches, the review of the damage indicators (DIs) and performance indicators (PIs) used for the management of infrastructure networks and infrastructure objects and the review of the methodologies for risk assessment and risk management of infrastructure networks and infrastructure objects. The conclusions from the review are used to formulate suggestions with regard to developments in standardization of concepts and methods for assessment of existing structures, including a proposal for the framework for data-informed assessment of transport infrastructure.

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1 Problem statement

1.1 Introduction

Road and railway infrastructure networks form the backbone of European transportation systems, carrying more than 80% of passenger and 50% of goods transport in Europe. In particular, large infrastructure assets are crucial for the availability and safety of the Trans-European Transport Network (TEN-T) that has over 1234 km of large bridges (bridges with >100m span) and 775 km of tunnels. Malfunction of these infrastructure assets will cause huge negative impacts and long-term drawbacks on the economy and society (WG3.5, 2018). Bridges and tunnels which are critical elements of the transport infrastructure networks, have in many cases reached their design service life and keep ageing. Besides, most bridges currently carry significantly more vehicles / traffic loads than what they were originally designed for. Such a condition brings high safety risks. At the same time, resources and capacity for conservation and care are too limited and should be used in an optimised way to counteract the growing back-log of maintenance. Maintenance deficiency accelerates the structural deterioration and the safety risk of infrastructure. This urgent issue is both European and global: in the last two decades there have been nearly 30 major failures of road and railway bridges and tunnels in Europe with hundreds of people killed and injured. The collapse of the Morandi Bridge in Genoa, Italy (2018, 43 people killed) has led to a year-long state of emergency in the Liguria region, an extensive analysis of the structural failure, and widely varying disputes of liability. Such incident cannot be singled out: in the last two decades, around 20 bridges in different European countries (Italy, France, Portugal, Spain, Denmark, Finland, Norway, Ireland, UK, Greece, Romania, Czech Republic) have collapsed or severely damaged with nearly 120 casualties. Beside bridges, similar concerns affect tunnel and other types of infrastructure. Although the most notorious examples of major tunnel disasters in Europe are related to catastrophic fire events, at the end of 2019 severe damages have occurred in the highway E26 Berté tunnel near Genoa, Italy where heavy concrete tunnel lining fell down and caused a major traffic disruption.

Aiming to ensure the safety of the transport infrastructure during operation through the improvement of maintenance policies across Europe, the European Commission opened in 2019 the call for the Coordination and Support Action (CSA) “Monitoring and safety of transport infrastructure”. The main goal of this CSA is to support the preparation of a mandate for a CEN standard for the maintenance and control of the European transport infrastructure. In 2020, the CSA was granted to the IM-SAFE project consortium (H2020 CSA IM-SAFE, 2020).

The expected result of IM-SAFE is, among others, to propose further amendment to the existing EU standards on safety assessment taking into account inspections, monitoring and testing and to propose a new standard for preventive maintenance of transport infrastructure Activities in the scope of Work Package 3 (WP3) of the IM-SAFE project, which are reported in this document, intend to identify the normative gaps with regard to data-informed safety assessment and maintenance decision-making based on review of the current state-of-the-art as represented by standards, guidelines, other regulations as well as current practice and research.

1.2 Challenge of implementation and standardisation of methods for safety evaluation and risk management

In engineering it is common to demonstrate the practical applicability of theory and methods by case studies. In this regard, case studies inform about potential, relevance but also about obstacles and limitations of theory and methods for solving problems in the real world. The case studies about maintenance, assessment of the structural performance and monitoring of

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existing bridges and tunnels that are publicly available, however, are mostly written by researchers and follow the research interest of demonstrating a particular methodology. Beside the real case addressed, these case studies often contain constructed data or assumptions to “make the demonstration work”. Consequently, case studies seldomly serve as a blueprint that can be used in a real reassessment context. Due to the progress of the project, the highlighted aspects have the status of a hypothesis reflecting the expected outcome of the formal study currently performed by the project consortium. In the following the main challenges and obstacles in real maintenance, structural reassessment and monitoring situations are highlighted and briefly discussed.

The need for reassessment is in general based on doubts about the structural performance and this can have several reasons as change of use/loads, detected or suspected damage or deterioration process, etc. Structural performance might refer to the absence of different adverse states that compromise the intended purpose of the structure. Structural failure of a component or the entire structure are obvious examples of such adverse states, excessive deflection, deformation or vibration are others. In the context of aging structures, the so-called condition limit states may be considered to describe adverse states that have the potential to lead to critical states for the structural integrity (such critical states often relate to tolerance to material deterioration or partial damage of structural elements). Requirements on structural performance can be formulated by normative direct assessable limits, e.g. a limit on maximum observable crack width in concrete structures. The detection of an exceedance of limits might initiate mitigation actions as e.g. physical intervention, or the initiation of a more careful analysis of the structure. For adverse states with larger consequences as the structural failure limit state, requirements on structural performance are expressed as explicit reliability criteria for structural safety.

Since 2010 the Eurocodes have reached the final stage of national implementation by the Member States as they are now replacing all national standards, assuring more uniform safety levels for buildings and critical infrastructures within the EU. For new structures reliability criteria are formulated and the target reliability indices for three different consequence classes are given explicitly (EN 1990:2002 “Eurocode - Basis of structural design”). However, in practical structural design it is assumed that these reliability requirements are complied with when applying the partial safety factors and design equations given in the code, i.e. the reliability requirements are only considered implicitly. For the assessment of the reliability of existing structures, however, the partial safety factors and design equations given in the code are no longer valid and cannot serve as an implicit poof of compliance. In the absence of a partial factor format that is applicable for the assessment situation at hand, the engineer is left with some ambitious challenges to evaluate the compliance with the reliability criteria:

- To which event should the reliability criterion be related to? Commonly, limit states on which basis reliability can be assessed, represent single failure modes. And the realization of failure modes implies a large possible variety of consequences. Consequences, in turn effect the criterion for reliability. The representation and analysis of failure of the entire structure, i.e. a critical combination of failure modes, often requires considerable efforts and is not always feasible.
- The specification of the reliability criterion itself is also very difficult in a structural assessment situation. The reliability criteria for new structures as specified in the design standard might be only of limited use as it not only refers to different consequence classes but implicitly also to the models, uncertainties and design cost (risk mitigation costs). These aspects might be entirely different for an assessment situation of an existing structure. The models to represent the limit states are often much more detailed in an assessment situation, and uncertainties are surely also different (e.g. larger in the presence of an unclear and spatially variable deterioration process). The costs for increasing the reliability are also much different: whereas in the design and planning phase the increase of reliability requires only the investment

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in e.g. some percent more reinforcement, for an existing structure the reliability can often only be increased by expensive strengthening measures. This clearly affects the magnitude of a reasonable reliability criterion.

- The limit states should be formulated such that they contain the suspected damage or deterioration mechanisms. This is particularly challenging when damage or deterioration mechanisms vary in space and time, as this would possibly necessitate an explicit representation of the complex system interactions in the structure (i.e. prohibit the simplified representation by a few single failure modes).
- Based on the developed limit states, in case of explicit reliability verification, the prior reliability has to be computed, i.e. the reliability that results when all random variables (or processes) in the limit state represent the prior knowledge (the knowledge that did lead to doubts about the structure and initiated the reassessment). The knowledge is in general scarce and the uncertainties are large. Experience shows, that the quantification of large uncertainties is particularly difficult.
- The explicit reliability verification requires a very high expertise level that is at present not common to the engineering practice. Moreover, suitability of the high level of the refinement in the reliability verification is very much depending on the availability and relevance of the information that can be obtained about the loads and load effect on structure, and on the justification of the costs of the effort-demanding analysis. It is therefore essential to provide the engineer with sufficient flexibility in choosing his approach, both with respect to the verification format for compliance check and level of approximation of the load and resistance models used for the evaluation of structural performance.

1.3 IM-SAFE project scope

Project scope and application domain of this report is consistent with the NMBP-36-2020 Call, which focuses on bridges and, where relevant, on tunnels. It is noted that the general concepts to a large extent apply to other types of infrastructure and transport infrastructure networks at large. This report is relevant to the domain of structural performance (including safety, serviceability, durability and robustness of structures) with a focus on the appraisal of existing infrastructure, which is addressed at the level of the network, object, and structural component. Where relevant and feasible, differentiation is made between bridges and tunnels. With regard to the type of infrastructure, information, analysis, and conclusions presented in this report apply to modern European transport infrastructure (i.e. transport infrastructure constructed in 1960's or later). With regard to construction materials, concrete structures (i.e. civil engineering objects in plain, reinforced and prestressed concrete) and steel structures (i.e. civil engineering objects in steel) are covered, recognizing that to a large extent the fundamental considerations and general procedures may apply to civil engineering objects for other types of structural materials.

1.4 Objectives of the deliverable

This report forms the technical background for the formulation of the proposal for the mandate to CEN for a further amendment to the existing EU standards on data-informed safety assessment considering inspections, monitoring and testing. The report also serves as the background document for a proposal for a new standard for preventive maintenance of transport infrastructure. The activities in Task 3.1 of WP3 of IM-SAFE project, which are documented are as follows:

- the review of the methodologies for reliability assessment at risk-informed, reliability-based and the semi-probabilistic level, including data-informed approaches accounting for prior information, information from inspection, testing, and monitoring including principles of :
    - probabilistic characterization of the hazardous events,

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  - evaluation of the probability of occurrence of adverse events,
  - principles of predictive performance models accounting for deterioration and damage,
  - cost models for inspections, monitoring, maintenance, direct and indirect consequences of failures;
- the review of the damage indicators (DIs) and performance indicators (PIs) used for the management of infrastructure networks and infrastructure objects;
- the review of the methodologies for risk assessment and risk management of infrastructure networks and infrastructure objects the evaluation of the current approaches to the use of information from inspection, testing and monitoring in the assessment and management of bridges and tunnels including:
  - the proposal of the framework for data-informed assessment,
  - the appraisal of concepts and methods for assessment of existing structures with regard to future standardization.

1.5 Report contents

Chapter 1 of this report explains the context, the approach and the aim of the of the activities carried on in Task 3.1 of WP3 of the IM-SAFE project. Chapter 2 provides a classification of types of infrastructure based on typology, vulnerable zones and vulnerability (in terms of both functionality and structural loading for bridges). In Chapter 3 a classification of hazards, actions related to hazards is given and the probabilistic and data-informed representation of hazardous events is described.

Chapter 4 introduces the concepts of condition, performance and risk. In the scope of the condition concept, the classification of deterioration and damage is described, the damage indicators for assessment are given and the implementation of the condition concept in through-life management is introduced/described. In the context of the performance concept the principles of the limit-state, reliability-based approach to performance modelling are given. For the risk concept, general principles of risk representation are described, and a generic framework for risk management of infrastructure objects is introduced.

In Chapter 5, performance indicators are categorized for different levels of assessment, namely network, object, and component level. Additionally, key performance requirement indicators for bridge and tunnel management systems are listed.

Chapter 6 describes the principles of performance verification of existing structures and provides a description of performance verification methods. In Chapter 7, several methods for assessment and management of risk for infrastructure networks, objects and components are given. In Chapter 8, the principles of data-informed approached in assessment are given and the use of data in performance verification is elaborated.

Finally, Chapter 9 an appraisal with regards to standardization is given, including a proposal for a framework for data-informed assessment of transport infrastructure.
Summary and conclusions of the report are given in Chapter 10, which is followed by the list of references and the appendixes with additional detail information on typology of bridges and tunnels and example of vulnerability analysis for arch bridges.

In the Appendix 1 and 2 the typology of bridges and tunnels is described in more detail and in Appendix 3, an example of application of vulnerability analysis to a bridge case is included.
The report is complemented by the proposal for harmonisation of terminology for data-informed safety assessment. To set the basis for a common understanding of the glossary within Europe and to respond to the need of resolving conflicting definitions that may cause misleading interpretation the IM-SAFE terminology proposal is include in IM-SAFE online Knowledge Base, https://imsafe.wikixl.nl/.

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2 Classifications for infrastructure

2.1 Types of classification

In this Chapter, the focus associated with the classification for infrastructures is mainly on bridges and partly processed for tunnels. The classification is based on vulnerability and consequence class concepts and comprises objects built after the 1960s.

2.2 Typology

2.2.1 Typology of bridges

A typology of bridges is the result of a classification of the bridges according to their physical/mechanical or other properties. The resulting bridge classes are also called types. A standardized typology helps to manage a large number of bridges and systematically assign vulnerable zones to the types for further analysis. Classification according to the mechanical and static concepts allows a distinction to be made e.g. between the following types and subtypes, that can be combined to more complex systems as well.

In general, bridge typology can be classified as follows:

- **Function of Bridges**
  - Railway Bridges,
  - Road Bridges,
  - Highway Bridges,
  - Motorway Bridges,
  - Footbridges,
  - Aqueducts,
  - Canal Bridges,
  - Taxiway Bridges,
  - Green Bridges,
  - Game Bridges,
  - Double-decker Bridges,
  - Protective Bridges,
  - Residential Bridges,
  - Dam Bridges,
  - Skyway.

- **Beam Bridges**
  - Single Beam,
  - Multi Beam,
  - Multi Beam including a Gerber Beam.

- **Frame Bridges**
  - Single Frame,
  - Multi-jointed Frames:
    - Vertical columns,
    - Inclined columns/braces.

- **Arch Bridges**
  - Tied Arch Bridges,
  - Truss Arch Bridges,
  - Sickle Arch Bridges,
  - Deck Arch Bridges.

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- **Suspension Bridges**
  - Simple (classic) Suspension Bridges,
  - Standard Suspension Bridge:
    - with one/two columns,
    - with many columns.

- **Cable Stay Bridges**
  - Simple Pylon Cable Bridges,
  - Single Pylon Harp Cable Bridges,
  - Double Pylon Cable Bridges,
  - Many Pylon Cable Bridges.

- **Truss bridges**
  - Queenpost Truss Bridges,
  - Supporter Truss Bridges,
  - Warren Truss Bridges,
  - Howe Truss Bridges,
  - Cantilever Truss Bridges.

- **Moveable bridges**
  - Drawbridge,
  - Bascule Bridges,
  - Folding Bridges,
  - Curling Bridges,
  - Vertical-lift Bridges,
  - Table Bridges,
  - Retractable Bridges,
  - Rolling bascue Bridges,
  - Submersible or Floating Bridges,
  - Tilt Bridges,
  - Swing Bridges,
  - Transporter Bridges.

Figure 2.1 to Figure 2.3 show main typologies of bridges with in the left column from the simplified static systems and the right column shows side views for a better assessment of the body relationships. Basic specifications of the main bridge types with regard to static system and form are given in Appendix 1.

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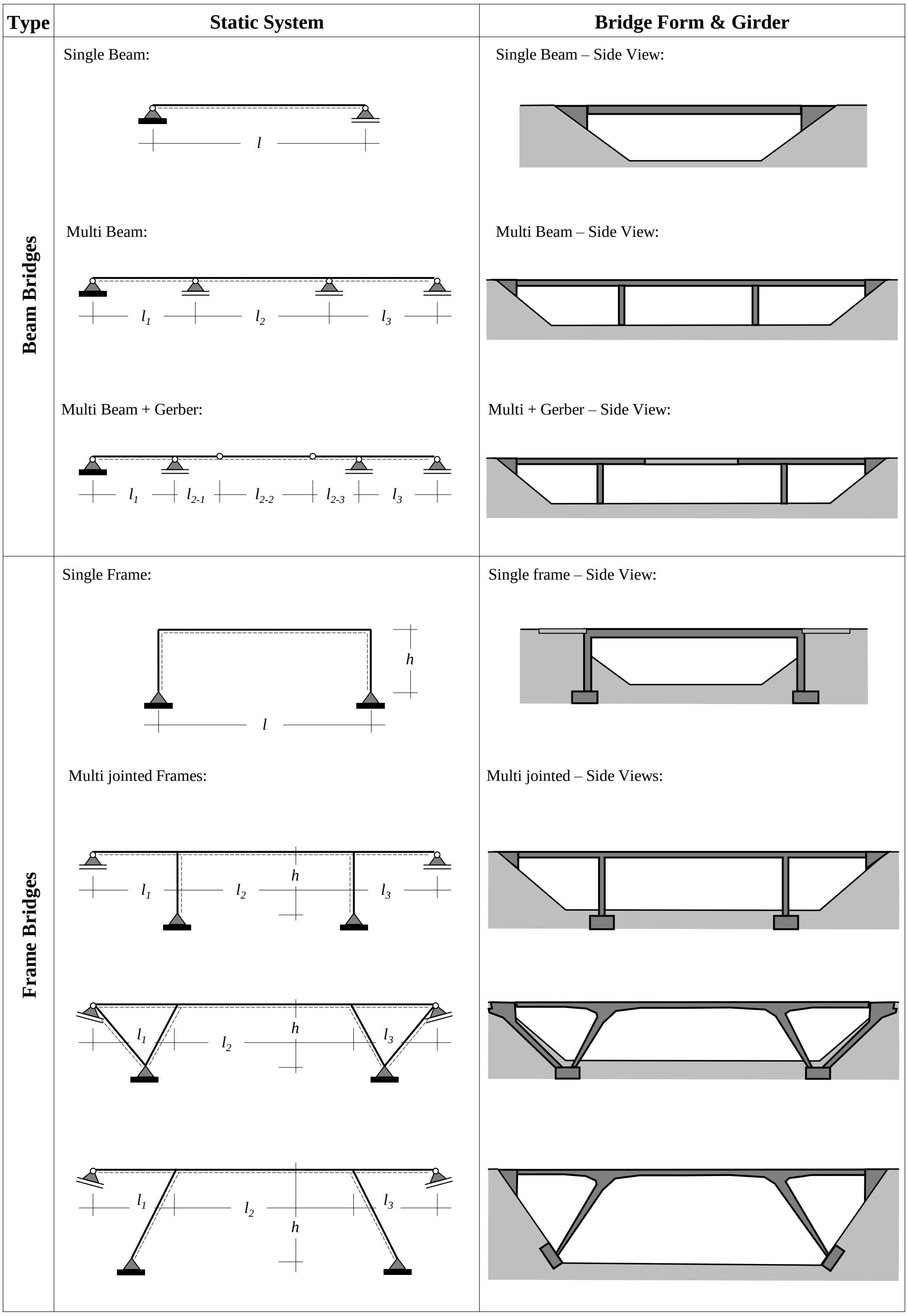

Figure 2.1 - Overview of bridge types based on typology with static system and girder form – Part 1: beam bridges and frame bridges.

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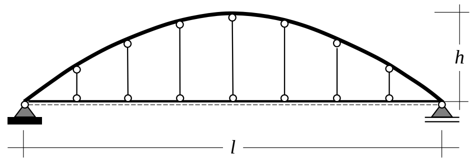
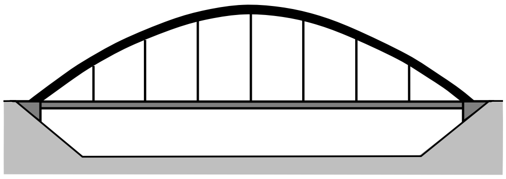
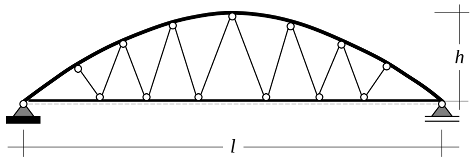
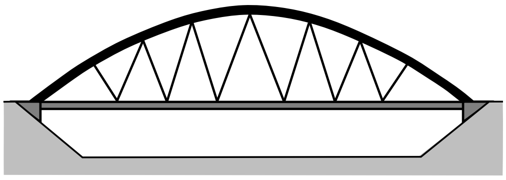
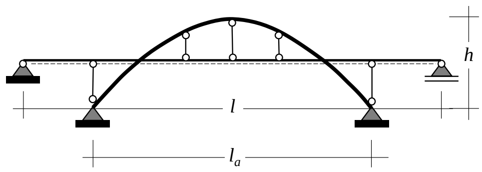
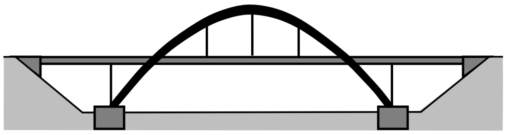
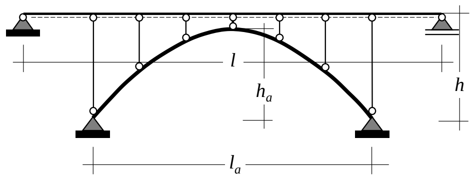
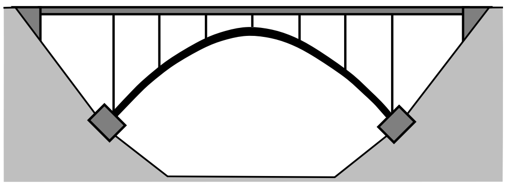
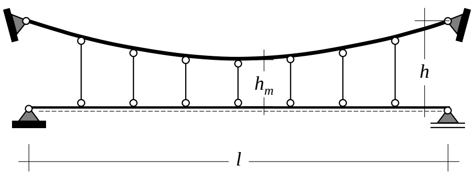
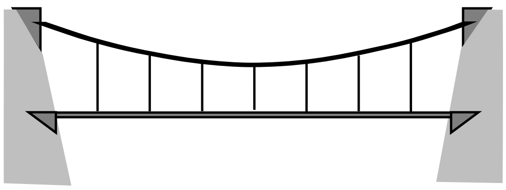
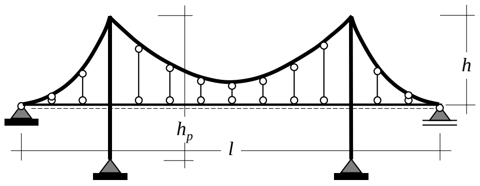
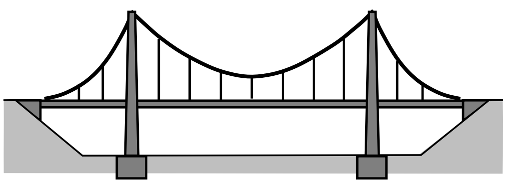
Type	Static System	Bridge Form & Girder
Arch Bridges	Tied Arch Bridges: 	Tied Arch – Side View: 
	Truss Arch Bridges: 	Truss Arch – Side View: 
	Sickle Arch Bridges: 	Sickle Arch – Side View: 
	Deck Arch Bridges: 	Deck Arch – Side View: 
Suspension Bridges	Simple Suspension Bridges: 	Simple Suspension – Side View: 
	Standard Suspension Bridges: 	Standard Suspension – Side View: 

Figure 2.2 - Overview of bridge types based on typology with static system and girder form – Part 2: arch bridges and suspension bridges.

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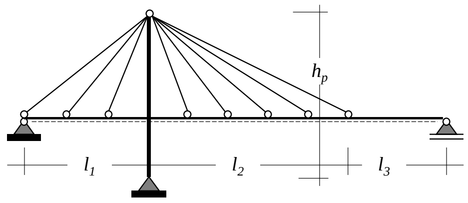
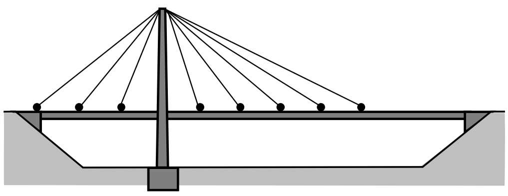
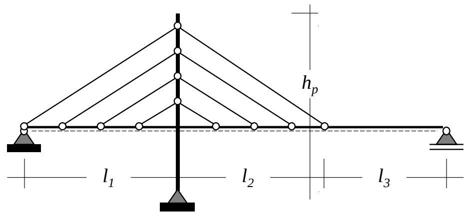
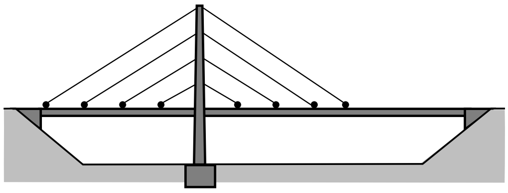
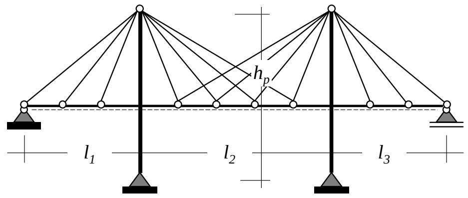
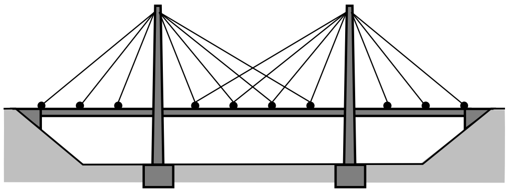
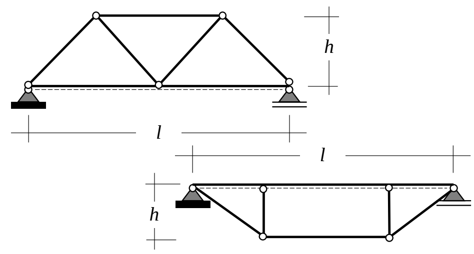
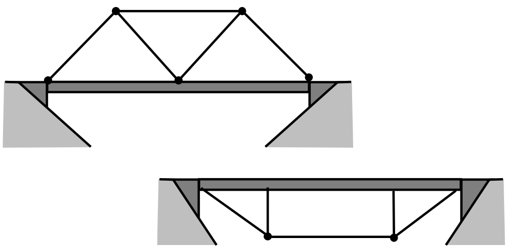
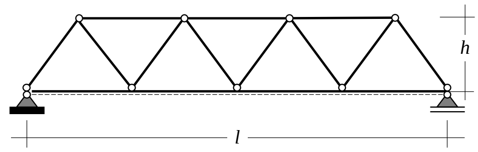
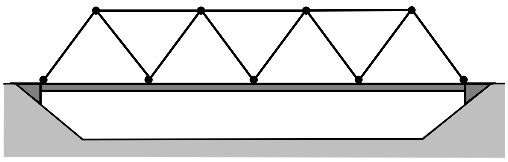
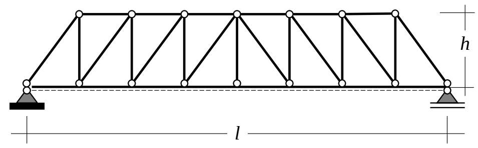
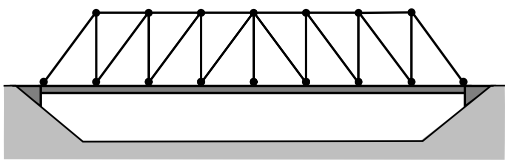
Type	Static System	Bridge Form & Girder
Cable Bridges	Single Pylon Cable Bridges: 	Pylon Cable – Side View: 
	Single Pylon Harp Cable Bridges: 	Pylon Harp – Side View: 
	Double Pylon Cable Bridges: 	Double Pylon – Side View: 
Truss Bridges	Queenpost/Supporter Truss Bridges: 	Queenpost/Supporter – Side View: 
	Warren Truss Bridges: 	Warren Truss – Side View: 
	Howe Truss Bridges: 	Howe Truss – Side View: 

Figure 2.3 - Overview of bridge types based on typology with static system and girder form – Part 3: cable bridges and truss bridges.

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2.2.2 Typology of tunnels

The typologies of tunnels presented in this document are the result of their classification according to their function, shape and construction method. Similar as described in Section 2.2.1, a standardized typology helps to manage many tunnels and systematically assign vulnerable zones to the types for further analysis.

A classification list about typology of tunnels is presented as follows:

- **Function of Tunnels**
  - Railway Tunnels
  - Highway Tunnels
  - Metro Tunnels
  - Irrigation Tunnels
  - Drainage Tunnels
  - Hydropower Tunnels
  - Submerged Tunnels

- **Shape of Tunnels**
  - Polycentric Shaped Tunnels (D Shape)
  - Circularly Shaped Tunnels
  - Horseshoe Shaped Tunnels
  - Egg-Shaped Tunnels
  - Elliptical Shaped Tunnels
  - Segmental Shaped Tunnels

- **Tunnel Construction Method**
  - New Austrian Tunnelling Method (NATM)
  - Cut and Cover Method
  - Bored Tunnel Method
  - Clay Kicking Method
  - Shaft Method
  - Pipe Jacking Method
  - Box Jacking Method
  - Underwater Tunnels

The static systems represent all forces, actions, and restrictions that keep the structure in equilibrium. Meantime, these systems are useful to understand how the structures are working on and how vulnerable zone could be identified.

Figure 2.4 - and Figure 2.5 - show simplified schemas of the typologies of tunnel described in this section, the left column presents the static systems and the right column the tunnel cross-section. Action of soil in these cross-sections is represented by springs, which represent the boundary conditions of the structure.

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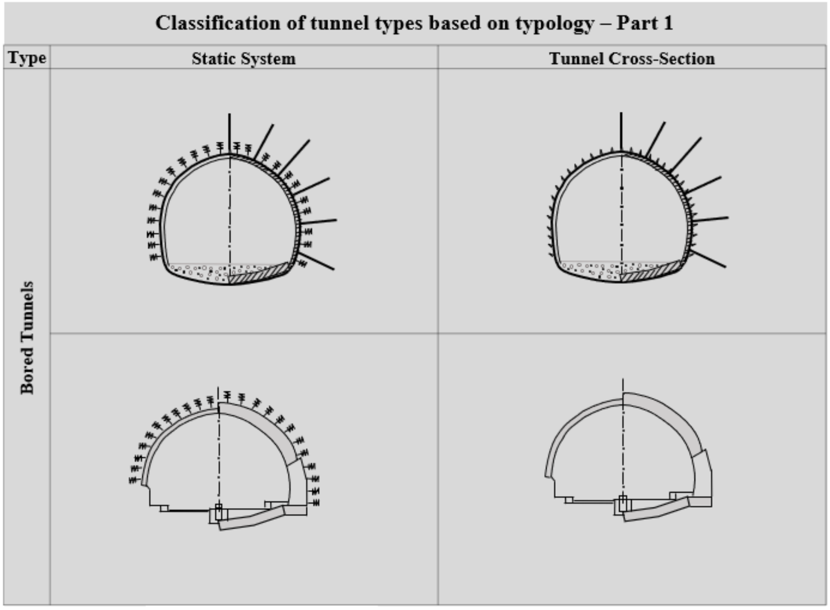

Figure 2.4 - Overview classification of tunnel types based on typology - Static system and Tunnel Cross-Section- Part 1.

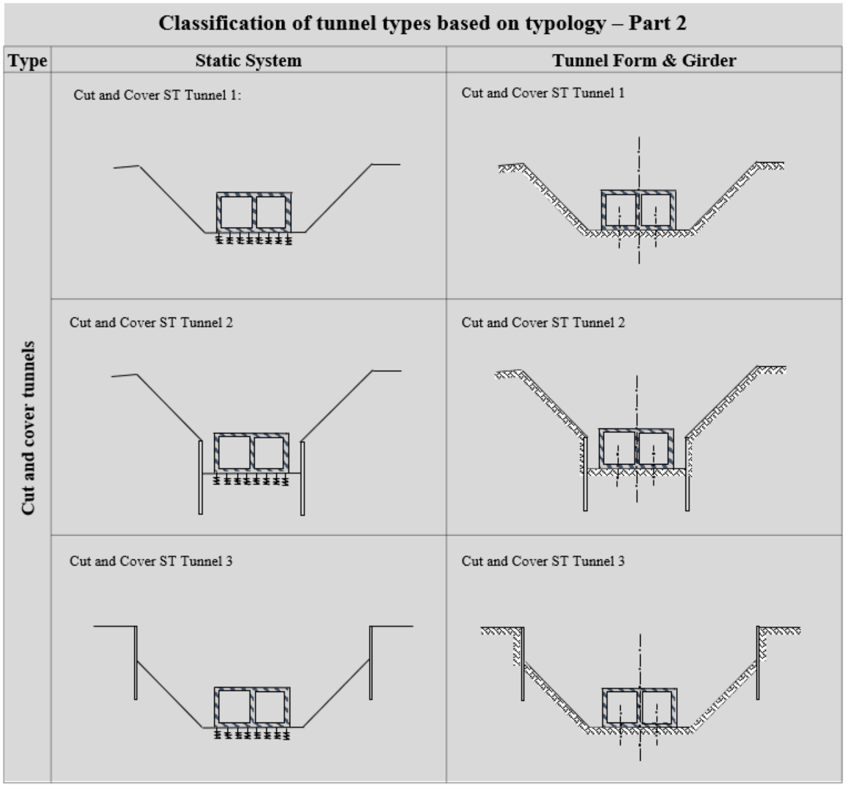

Figure 2.5 - Overview classification of tunnel types based on typology with static system and girder from - Part 2.

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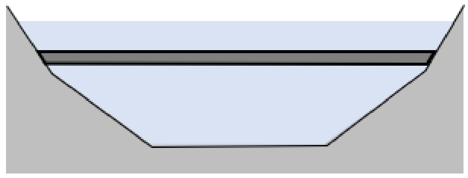
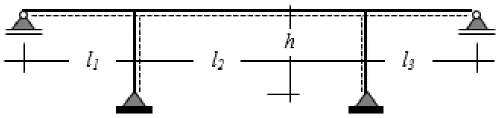
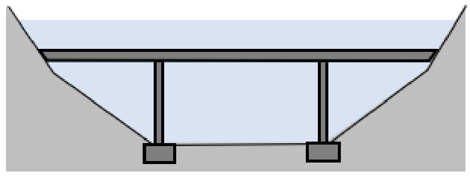
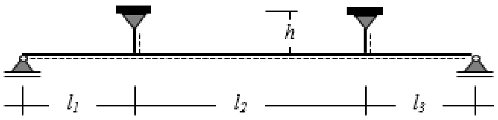
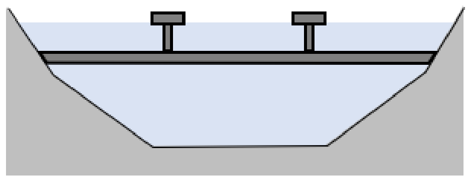
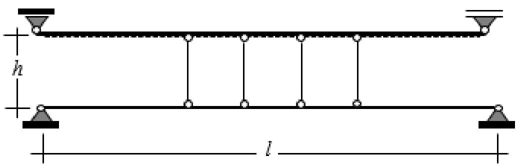
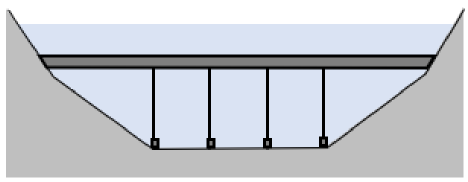
Type	Static System	Bridge Form & Girder
Submerged Floating Tunnels	Free SFT: 	Free SFT: 
	Column support SFT: 	Column support SFT: 
	Pontoon SFT: 	Pontoon SFT: 
	Tension Leg SFT: 	Tension Leg SFT: 

Figure 2.6 - Overview classification of tunnel types based on typology with static system and girder from - Part 3.

Appendix 2 describes the general characteristics of these typologies.

2.3 Vulnerability

2.3.1 Vulnerability concept

Vulnerability is generally understood as the degree to which a system, or part of it, may react adversely during the occurrence of a hazardous event. This concept of vulnerability implies a measure of risk due to the failure of a system (e.g. bridges or tunnels) caused by an event and its physical, social and economic aspects and impacts. The term vulnerability is already a well-known and often used term in engineering. However, a wide variety of definitions and understandings of vulnerability exist nowadays.

Different definitions can be found for the term of Vulnerability, as it was proposed by precedent projects such as: (a) SAFEWAY Project (see https://www.safeway-project.eu) definition, "Vulnerability refers to the propensity of exposed elements such as physical or capital assets, as well as human beings and their livelihoods, to experience harm and suffer damage and loss when impacted by single or compound hazard events"; and the (b) COST TU 1406 Project

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(see Action TU1406 - COST) definition, which differentiates between structural vulnerability and functional vulnerability. In course of the IM-SAFE project, definition of vulnerability is formulated follows: *Vulnerability is an intrinsic property of an entity (e.g. network, object, component or element, or the parts thereof) resulting in its susceptibility to a risk, which may be quantified by the ratio between the risks due to direct consequences of an event and the total value of the considered entity, determined bearing in mind all relevant hazards for a specified time frame.*

Nevertheless, it is important to clarify that in this Chapter we are considering vulnerability from the perspective of condition survey, and where those areas or elements subjected to higher stresses or deformation might be, In this context, the following should also be considered: Areas with higher probability of material deterioration/damage, which are also important.

2.3.2 Methods for the characterization of vulnerable zones

2.3.2.1 Robustness related detection method.

The *alternative load path strategy* in accordance with the classic robustness considerations is ideal for finding vulnerable zones in a structural component, structure or network. With this method, the resistance to a progressive collapse (i.e. “indirect” or “consequential” failure) is to be analysed explicitly for possible *vulnerable* zones. Therefore, the alternative load path strategy is understood as a possibility to reduce the probability

$$Pr(S|D \cap H)$$

with
S…System-specific damage
D…local damages
H…Hazard

Measures to achieve this include the provision of redundancy and integrity or the use of ductile elements, see for instance section 30.9.4.2 of (fib MC2020, 2022).

The assessment of the vulnerable zones should be performed by using a method like the *consequence reduction strategy*. The method is a part of robustness considerations and aims to limit unacceptable (disproportionate) consequences

$$Cind$$

and/or

$$Pr(S|D \cap H)$$

with
C_{ind} …indirect cost

resulting from the local damages D in the vulnerable zone. Measures associated with the consequence reduction strategy can include the structural segmentation or compartmentalization, and changing the context of the structure, see for instance section 30.9.4.3 of the (fib MC2020, 2022).

The *event control strategy* consists of preventing the occurrence of a previously identified set of hazards and limiting its occurrence rate to an acceptable value. This strategy does not increase the intrinsic resistance. Measures associated with event control might include: changes of the building site or access to it, restrictions of the use of the structure, installation of warning systems; implementation of passive protective measures, but also quality management plans to prevent human errors and maintenance exercises, see for instance section 30.9.4.4 of (fib MC2020, 2022).

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2.3.2.2 Sensitivity related detection method.

In a numerical model, sensitivity analysis (SA) is a method that allows the evaluation of uncertainties in one or more input variables to uncertainties in the output variables. This analysis therefore allows (a) the assessment of the predictive quality of the model for the phenomenon of interest, (b) the assessment of the effects of changes in the input variables on the phenomenon, and (c) the relationships/correlations between the input variables and the characteristics of the phenomenon.

In other words, the expected values of various parameters involved can be used to evaluate the robustness and, i.e. 'sensitivity' of the results from which changes and identify the values and associated vulnerable zones beyond which the results change significantly. SA identifies priority needs for improving knowledge and hence vulnerable zones (Kala, et al., 2019).

2.3.2.3 Force and loading based vulnerability analysis

The force and loading based vulnerability follows the basic principles of the COST TU1406 vulnerability definition where the focus is on the loads and force profiles associated with related hazards for a specified time frame.

2.3.2.4 Deformation based vulnerability analysis

The basic principle of deformation-based vulnerability is the same as for force and load vulnerability, but with a focus on deformations that causes damage consequences associated with related hazards for a specified time frame.

2.3.2.5 Performance based vulnerability

The performance-based vulnerability includes, among other things, details and design concepts that have been observed or discovered not to work well, and therefore develop into critical elements or zones in relation to a bridge type or cross section. The considerations on performance are not limited to highly stressed parts due to loads, forces or deformations, but can also arise from environmental influences, poor processing, etc.

2.3.3 Vulnerability of bridges

In this first version of D3.1, the assessment of vulnerable zones for bridges is illustrated through two different approaches: i) the Force and loading based approach; and ii) the Performance-based approach. These two examples generally work well for undamaged structures, however for in-service structures other more sophisticated methods that can detect and locate vulnerable zones considering the damaged condition are typically required.

2.3.3.1 Force and loading based vulnerability

In Figure 2.7 to Figure 2.9, the left-hand columns show the shear force V(x) and moment lines M(x), which can then be used for a first approach to localize the vulnerable zones/elements. As can be seen from the figures in the right-hand columns of Figure 2.7 to Figure 2.9, with this approach the vulnerable zones are mostly congruent with the areas of maximum moment stress or maximum shear force stress. It should be noted, however, that vulnerable zones/elements are derived solely from the state lines, as shown in Section 2.1. Nevertheless, it should be noted, however, that the critical areas - i.e. the points where the utilisation of the design load-bearing capacity is critical - are not necessarily at the points with the greatest load actions.

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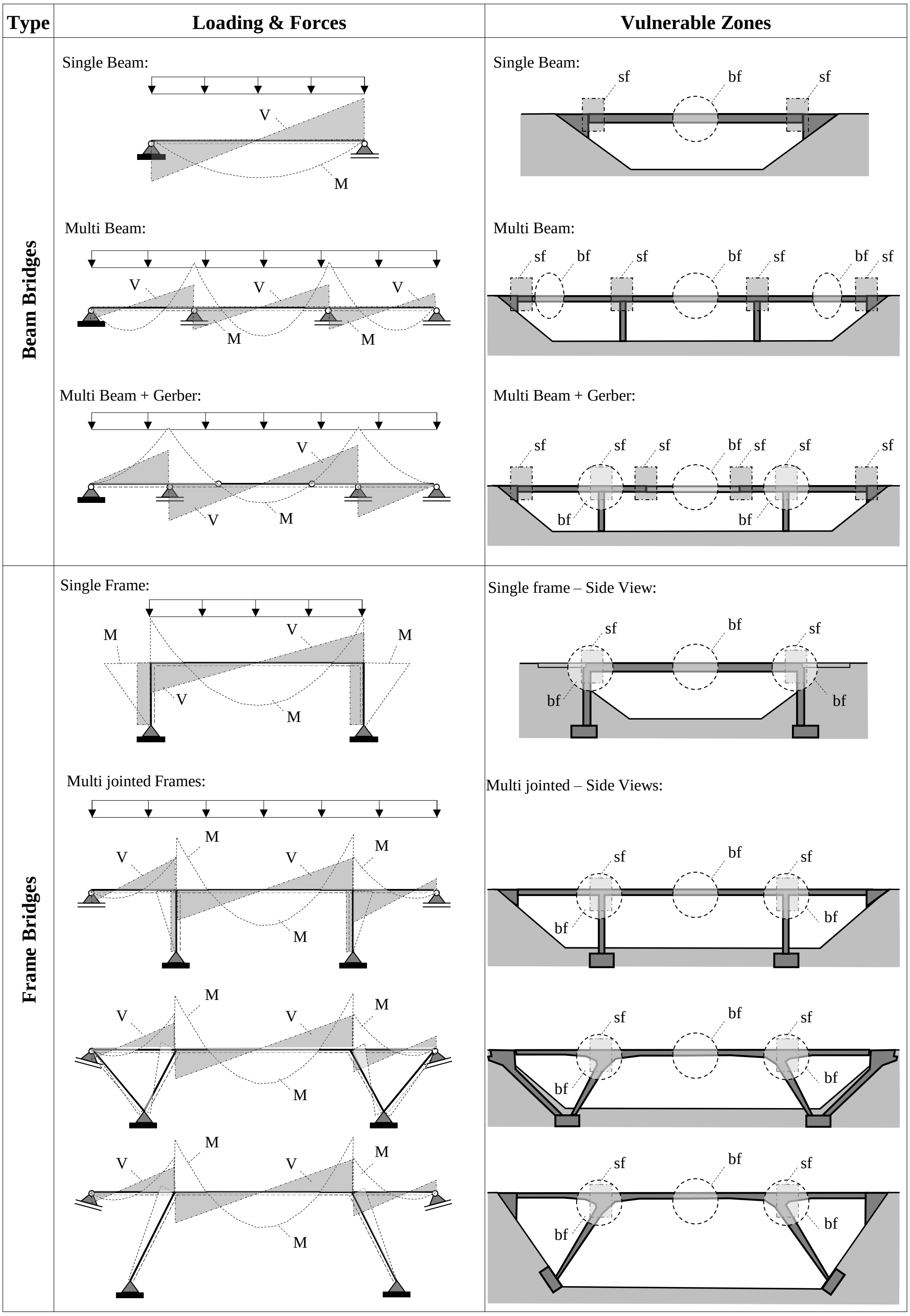

Figure 2.7 - Identification of vulnerable zones – Part 1: beam bridges and frame bridges.

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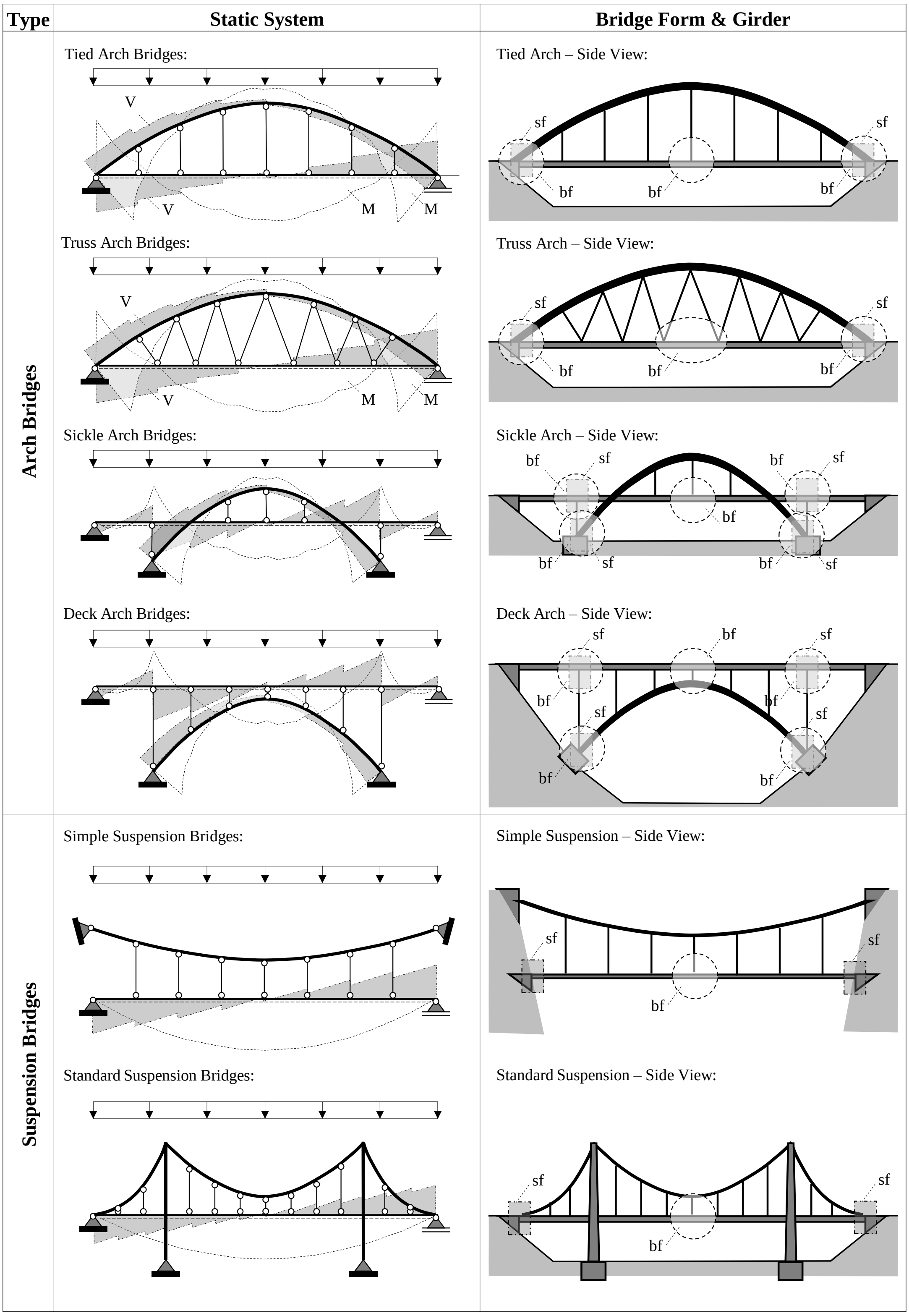

Figure 2.8 - Identification of vulnerable zones – Part 2: arch bridges and suspension bridges.

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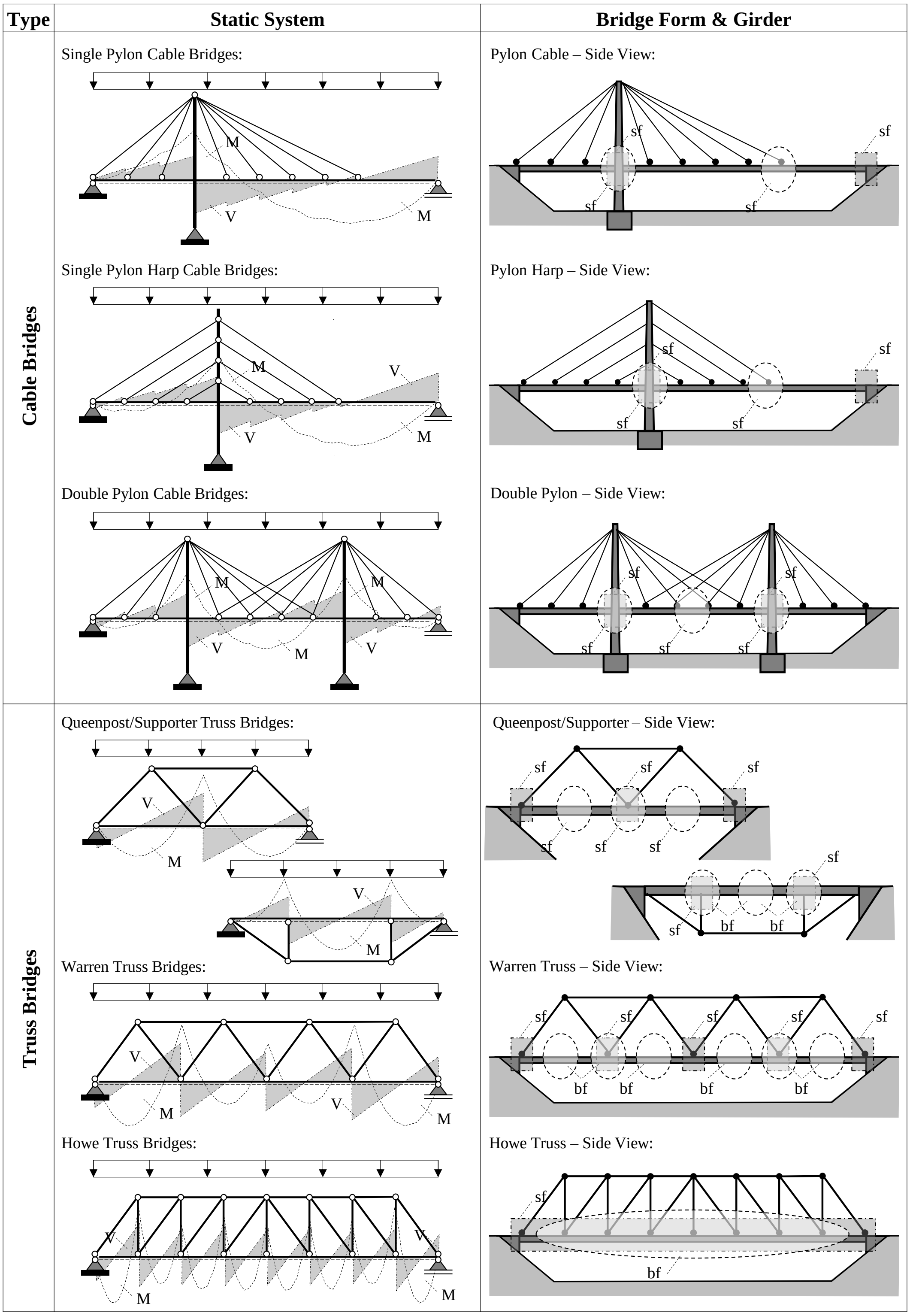

Figure 2.9 - Identification of vulnerable zones – Part 3: cable bridges and truss bridges.

2.3.3.2 Performance based vulnerability

In COST TU1406, vulnerable zones were defined according to design concepts and details which can lead to poor performance in terms of robustness, security and durability. This classification can be assigned to the robustness approach outlined at the beginning. The vulnerable zones shown in the following sections and example of performance based vulnerability analysis for arch bridges is given in Appendix 3 are based on the elaborations of

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COST TU1406 (see Action TU1406 - COST), WG3 and were largely extracted from the WG3 report (Hajdin, et al., 2018). It noted that in general application of more refined analyses should be consider for cases that are not covered in the simplified schemes described in Chapter 2 of this report and which is consistent with the general assessment flow that will be described in Chapter 9.

2.3.3.2.1 Conceptual weaknesses

History has shown that some design concepts do not perform well, i.e. there are critical elements related to a bridge type or a bridge cross-section type. Those elements should be carefully evaluated before intervention strategies are formulated. A typical example is a Gerber hinge in a girder or frame bridges. This might be addressed as a conceptual weakness and labelled as "CW" in the following sketches:

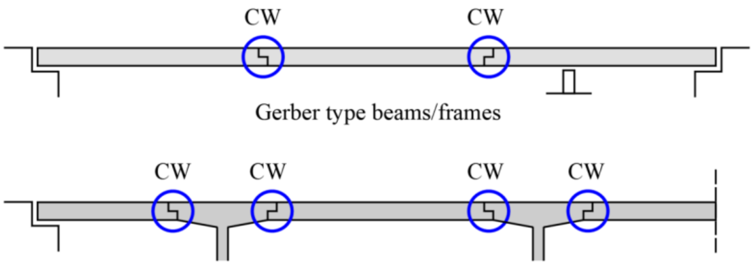

Figure 2.10 - Detail of conceptual weaknesses in a concrete bridge deck.

Conceptual weaknesses may also be associated to details of cross-sections. The precast multicellular cross section is one such example:

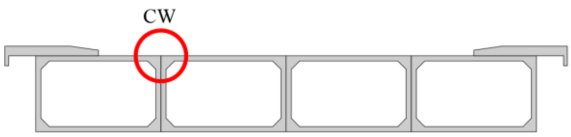

Figure 2.11 - Conceptual weaknesses in a precast multicellular cross section.

In addition, there are elements buried in the ground or embankment, which can hardly be inspected without a traffic disruption. If those elements were not properly designed and/or protected during construction and operation, costly interventions are needed. Those elements form a special subgroup of the substructure. Besides the foundation, this subgroup includes: buried tie, buried inclined leg, run-on slab (transition slab), abutment back wall, etc., labelled as "BE" in the next sketch. Some of them are related to structural safety, while others to serviceability. Deflection/settlement of the structure or embankment might indicate that there is a hidden damage process taking place.

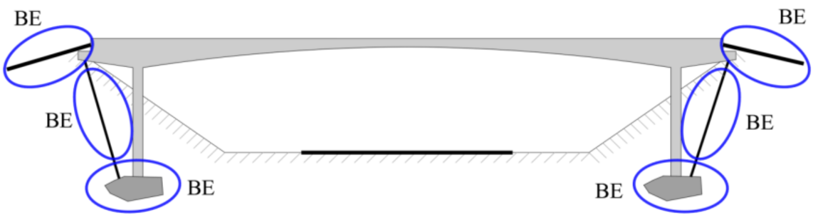

Figure 2.12 - Conceptual weaknesses in abutments and foundation.

Further categorization of the girder and frame bridge elements can be made in relation to their exposure to damage processes and sudden events. For example, middle piers may be exposed to impact from vehicles & vessels depending on the location and type of underpass

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traffic, or local scour if the substructure foundations are not protected from soil erosion due to flooding waters.

Furthermore, elements might be classified in terms of the importance at system level according to different criteria, e.g. structural safety, traffic safety, durability (Strauss & Mandić Ivanković, 2016).

2.3.3.2.2 Vulnerable zones related to the superstructure

Damage processes related to concrete structures might affect any part of a concrete bridge. not all parts of the bridge are equally important with respect to consequences. There are some regions/zones that are highly vulnerable, and that should be treated with special care. It is useful to relate failure modes to structural subgroups. In general, the superstructure might fail in a bending or a shear failure mode. It is also noted that the such classification has its limitations as it is not necessary simple to determine the failure modes for a complex structure as it may depend on many factors including e.g. the utilisation in the different regions.

Segmentation of the superstructure in the longitudinal direction (partitioning of an element into regions with different vulnerability) based on the (NYSDOT, 1997) and (FHWA, 2016) can be as follows:

- *High moment regions*
  - Sagging (label **HMS** region)
  - Hogging (label **HMH** region)
- *High shear regions* (label **HS region)**
- *Construction joint* (rigid type) (label **CJ region)**
- *Shear key* (label **SK region)**
- *Hinges* (label **HG region)**
- *Anchorage zones* (label **AN region)**

An example of such segmentation for a Gerber type bridge girder is given in Figure 2.13.

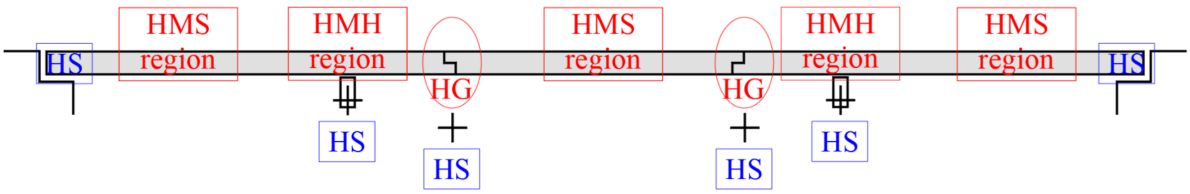

Figure 2.13 - Example of segmentation for a Gerber type girder bridge.

Locations of vulnerable zones in girder and frame bridges are presented in Figure 2.14.

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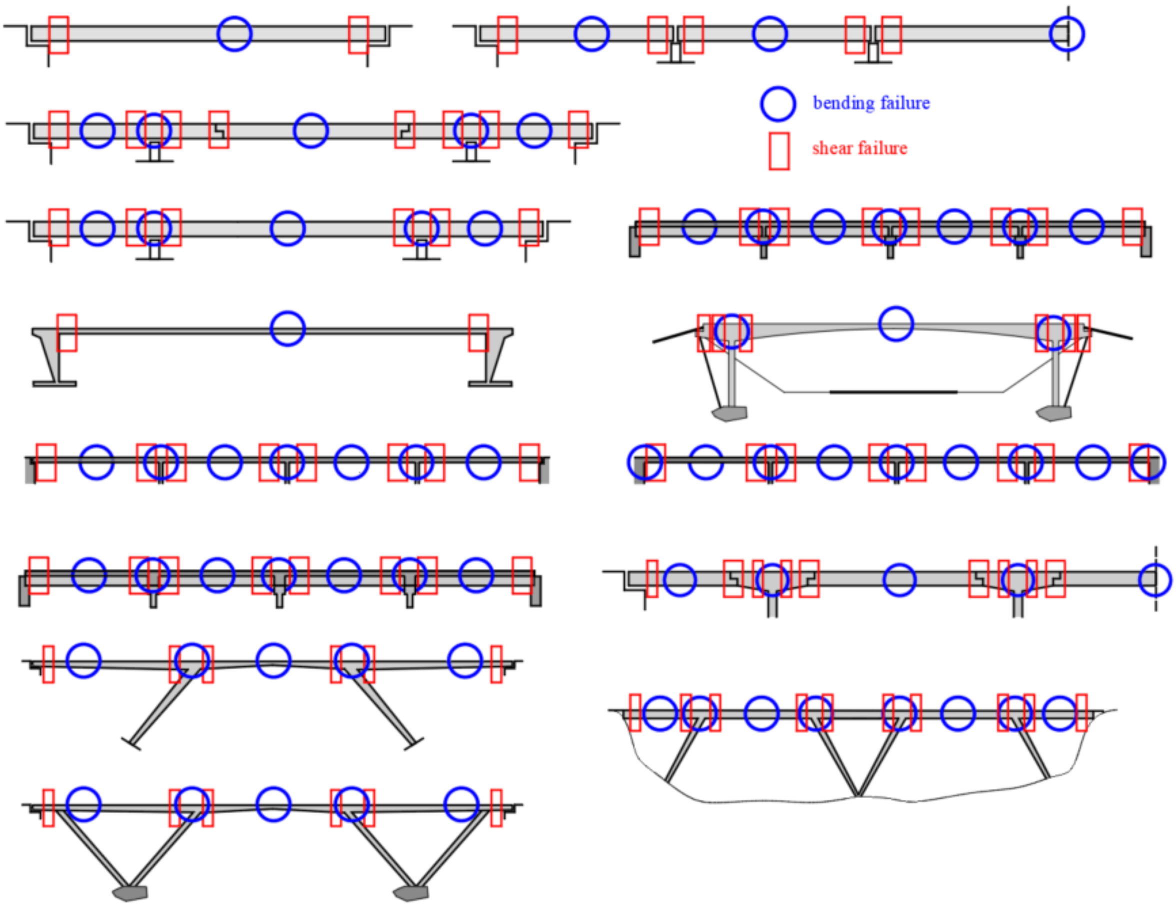

Figure 2.14 - Vulnerable zones for different types of girder & frame bridges.

When damage is observed in those regions, it should be evaluated. A distinction should be made between a structural assessment and a durability assessment. Structural assessment requires the analysis of one or more failure modes that may be associated with one or more vulnerable zones (e.g. hyperstatic systems). The serviceability assessment generally focuses on one vulnerable zone (e.g. a specific crack width, deflection).

a) 

b) 

Figure 2.15 - Examples of observations that affect vulnerable zones a) Chloride induced corrosion in a half-joint (Christodoulou, et al., 2014); b Delamination due to ASR in a deck slab affecting the shear resistance (Larsen, et al., 2009).

Conceptual weaknesses may also be associated with some of the abovementioned vulnerable zones. One such example is poor shear capacity in high shear regions of concrete bridges. This conceptual weakness is due to limited understanding of the shear phenomena, given in old design codes.

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In Figure 2.15 a few examples of observations that affect vulnerable zones are provided. Further examples may be found in various literature incl. inspection manuals, e.g. (DRD, 2014), (Correia, et al., 2012), (Breysse, et al., 2012) and (Ghosn & Yang, 2014).

2.3.3.2.3 Vulnerable zones related to substructure

A collapse of the substructure or its elements that support the superstructure can lead to a complete collapse of the entire system. In general, a substructure might fail in crushing or buckling failure mode. In addition, the bearing areas between the substructure and the superstructure are subject to splitting forces and pier heads (if any) are subject to high shear stresses and are therefore particularly vulnerable. Elements of the substructure are also exposed to sudden events such as impacts, scour and earthquakes. Figure 2.16 shows typical locations of vulnerable zones of piers and their expected failure modes.

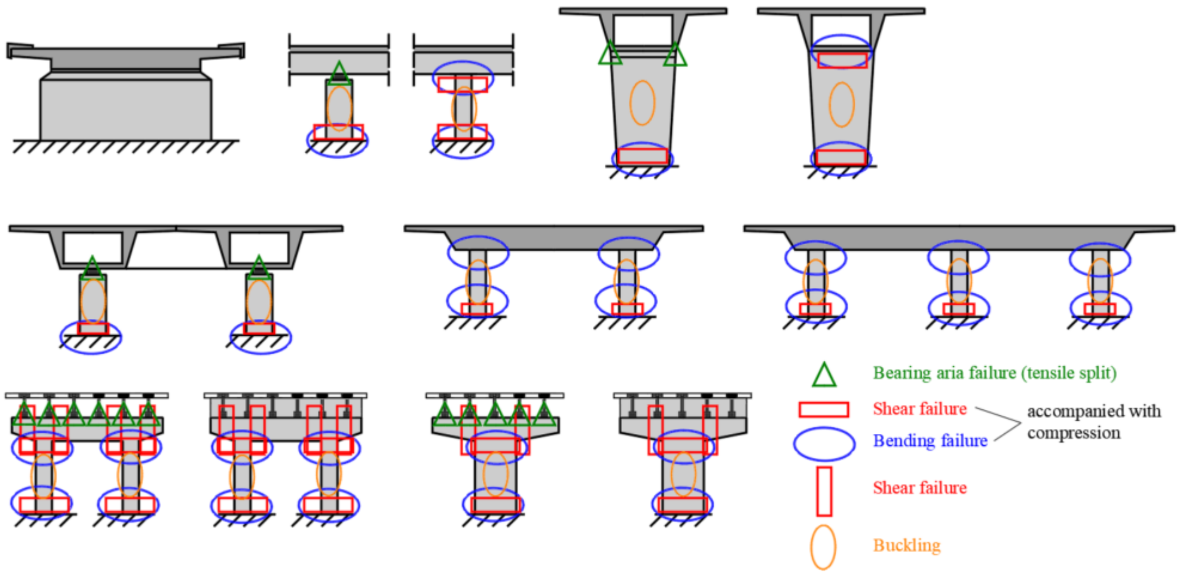

Figure 2.16 - Vulnerable zones related to substructures.

2.3.3.2.4 Damage related to equipment

Elements related to equipment are related to nearly all bridge types. Malfunction of these components might jeopardize load bearing elements of a bridge (serviceability issues) and/or impose severe consequences themselves. Level of service generally depends on adequate function of these elements (traffic safety issues).

Following performance or damages associated with bearing should be inspected, (Ramberger, 2002)*:*

- Sufficient ability to allow movement, taking into account the temperature of the superstructure,
- Correct position of the bearing themselves and parts of the bearing relative to each other,
- Uncontrolled movement of the bearing,
- Fracture, cracks and deformations of parts of the bearings,
- Cracks in the bedding or in adjacent parts of sub- and superstructure,
- Condition of the anchorage,
- Condition of sliding or rolling surfaces,
- Condition of the anticorrosive protection, against dust, and of the sealings.

For more information on bearings refer to specific literature e.g. (SHRP, 2014a), (SHRP, 2014b), and (Austroads, 2012).

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Concrete hinges were introduced more than 100 years ago and when correctly executed they perform very well throughout the world, e.g. (Schacht & Marx, 2015). Concrete hinges are characterised by high load-carrying capacity and a moderate rotational capacity. Concrete cracking in the throat of the Freyssinet (un-reinforced) hinge and risk of shear loading impact in the Mesnager (reinforced) hinge are important considerations when assessing or predicting concrete hinge performance.

Following performance or damages associated with expansion joints should be inspected, (Ramberger, 2002):

- Damage of the anticorrosive protection
- Cracks due to fatigue in steel members
- Damage to seals
- Workability of the linkage (proper function)
- Obstruction or damage of the drainage system

For more information on expansion joints refer to specific literature such as ref. (SHRP, 2014a), (SHRP, 2014b), and (Austroads, 2012).

Drainage, as a sub-component of the equipment category, comprise permanently installed drains and associated piping systems. Inspection should verify proper deck slopes and proper functioning of kerb channels, drainage inlets, pipe, outlets and possible drain holes for drainage of voids. Blockage of drainage may create a serious traffic hazard as well as result in severe deterioration.

Waterproofing is usually not visible, i.e. the condition has to be assessed from possible consequential damage on neighbour components such as:

- Leaking decks/wet spots beneath superstructure,
- Finding of protective concrete wash out,
- Swelling of pavement,
- Cracking of pavement.

2.3.3.2.5 Effect of hidden defects/damages

Weight restrictions and emergency closures on roadway bridges are often required because of a suspicion on hidden defects/damages. These are hidden from sight (i.e. in inspection within touching distance) or not obvious on the first observation/inspection. In the manual (CIRIA, 2017), a guidance for detection and management of hidden defects in bridges has been provided. The three-step procedure is recommended, comprising risk review, risk assessment and risk management. As a part of the risk review, the two key questions are to be asked during a review of existing information: ,,What do the records say?" and ,,What is not recorded?". Also, two questions are to be asked on a site during inspections: ,,What can I see?" and ,,What can I not see?".

The (CIRIA, 2017) manual, provides extensive guidance on identifying key hazards, their consequences, associated deterioration mechanism and their control measures. Typical hidden defects for concrete girder and frame roadway bridges can be related to:

- Superstructure
    - Concrete body of an element,
    - Reinforcement,
    - Prestressing wires/stands and anchorages,
    - Voided and cellular structures,
    - Half-joints,
    - Obscure surfaces,
    - Concrete hinges,
    - Temporary works,

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- Bearings and expansion joints
    - Poor access,
    - Inspection at the 'wrong time',
    - Uninspectable items.
- Drainage
    - Waterproofing,
    - Substructure.

The effect of a possible hidden defect/damage on the bridge performance should be duly considered even if these are not located in vulnerable zones. These damages should be treated in a same way as a natural hazard.

2.3.4 Vulnerability of tunnels

In this first version of D3.1, the assessment of vulnerable zones for tunnels is illustrated through two different approaches: i) the Force and loading based approach; and ii) the Performance-based approach. These two examples generally work well for undamaged structures, however for in-service structures other more sophisticated methods that can detect and locate vulnerable zones taking into account the pathological condition are typically required.

2.3.4.1 Force and loading based vulnerability

In Figure 2.17 to Figure 2.19 the shear force (Q and moment lines (M) are shown for bored and cut and cover tunnels, which can subsequently be used for a first approach to localize the tunnel's vulnerable zones. As can be seen in the right column of the figures, in this approach the vulnerable zones are mostly congruent with the areas of maximum moment stresses or maximum shear force stresses.

The shear and moment lines are essential to have a preliminary vulnerable zones of the structure. Nevertheless, it is essential to analyse and evaluate the specific loading and forces cases by means of the finite element method under the condition of the different local weakening of structural strength. Several studies have demonstrated how the weakening location of soil structural strength has an important effect on the distribution of the lining moment of tunnel. (Li, et al., 2011).

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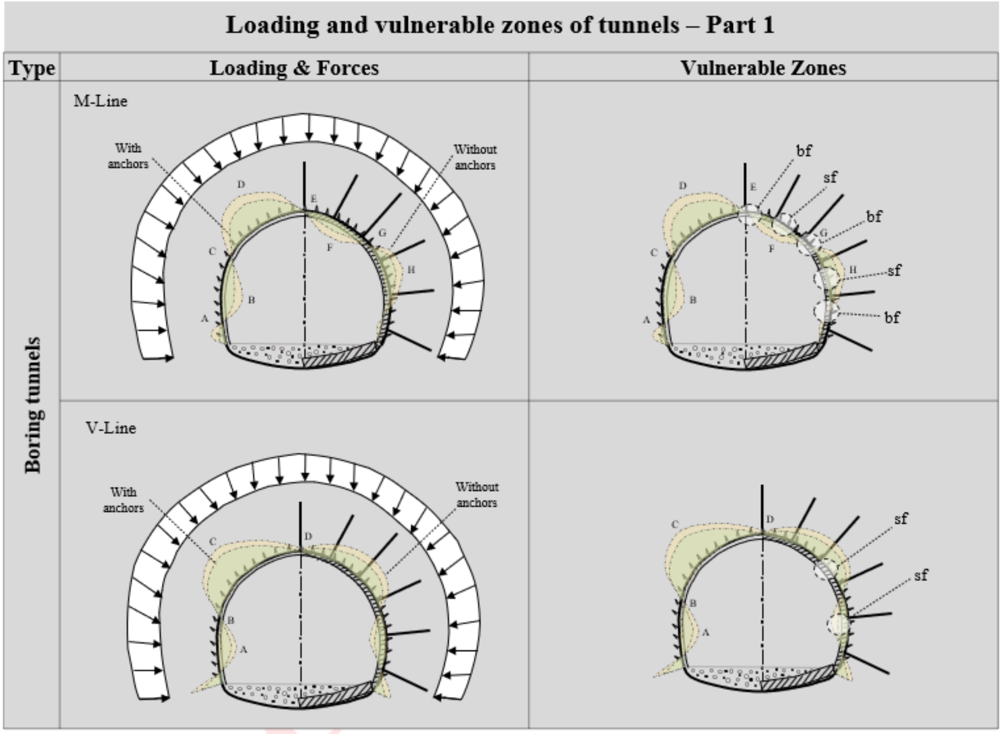

Figure 2.17 - Overview classification of tunnel types based on typology with loading, forces, and vulnerable zones - Part 1.

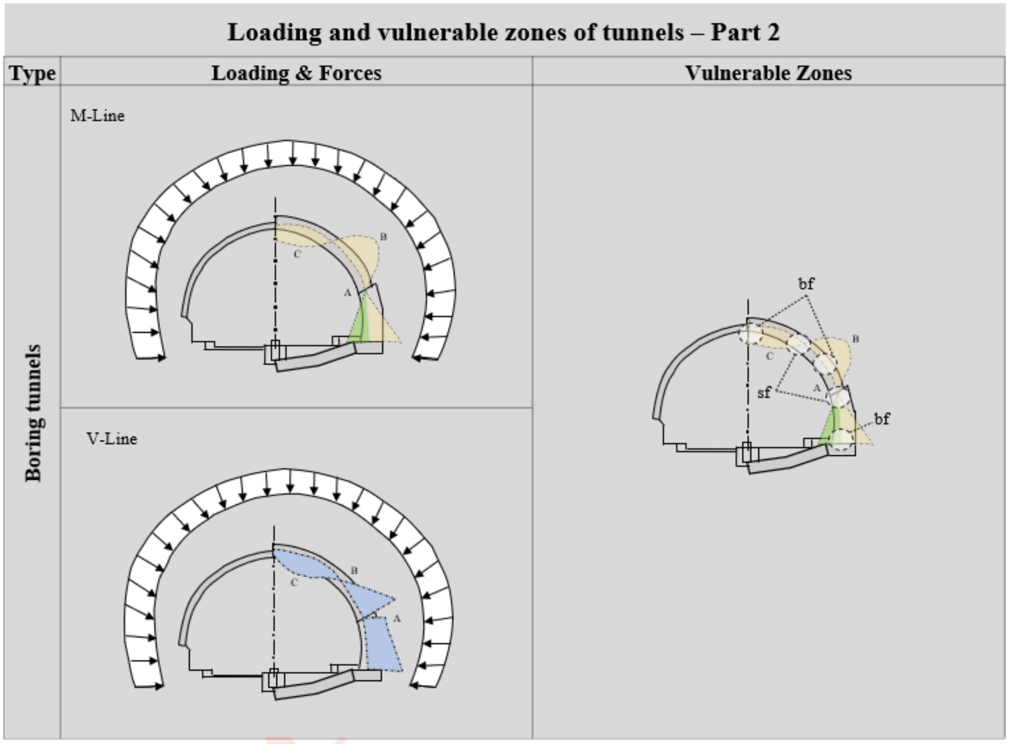

Figure 2.18 - Overview classification of tunnel types based on typology with loading, forces, and vulnerable zones - Part 2.

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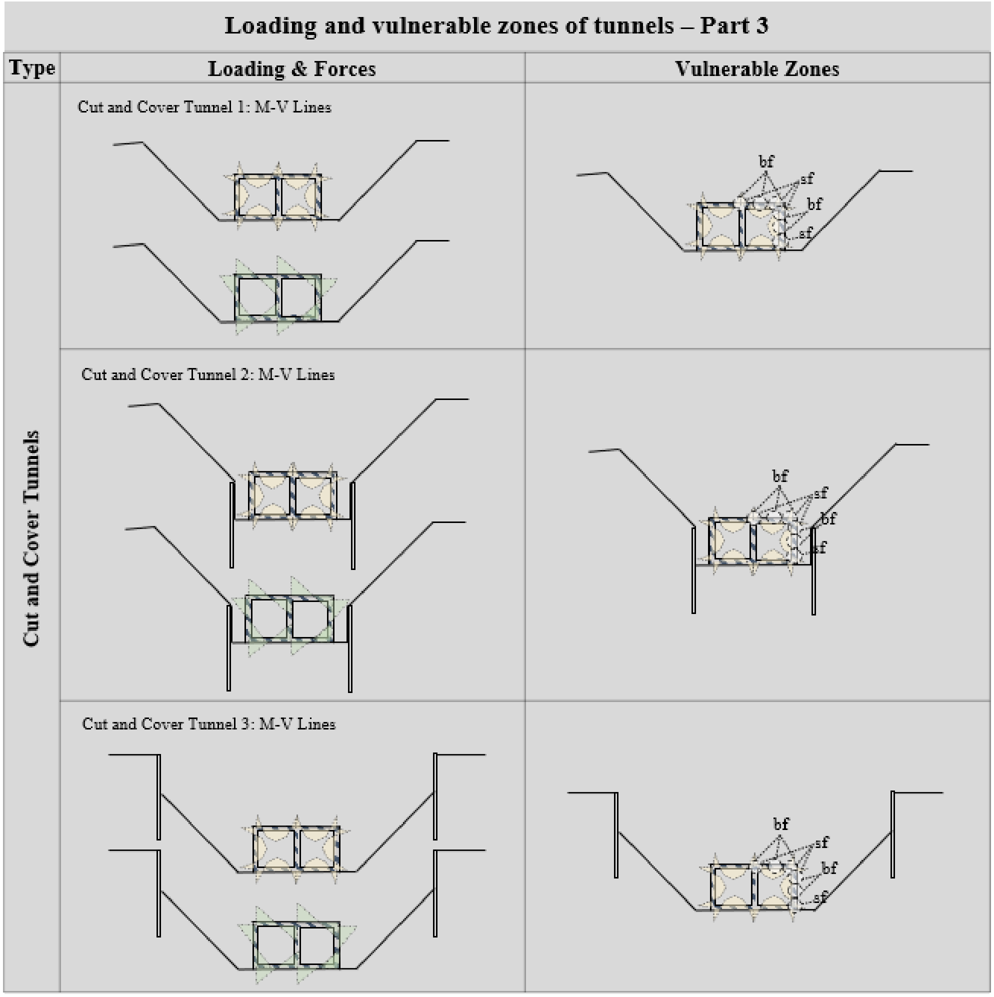

Figure 2.19 - Overview classification of tunnel types based on typology with loading, forces, and vulnerable zones - Part 3.

2.3.4.2 Performance based vulnerability

2.3.4.2.1 Conceptual weaknesses

One of the most important aspects of a tunnel is its cross-section, as this is constant throughout the length of the tunnel. As indicated in section 2.5, it is essential to determine the loads and vulnerable zones. However, it is also important to analyse the tunnel longitudinally, since the tunnel is a flexible structure that is formed by the joining of different segments thus causing longitudinal deformations (Cui, et al., 2015).

Tunnels longitudinally support axial forces, shear forces and bending moments induced, for example, by fault displacement. The most common analytical procedure to analyse a possible cause of longitudinal failures, following the same example of fault displacement, is the finite element method (Hung, et al., 2009).This method can incorporate realistic models of the tunnel and the surrounding geological environment, and is shown in Figure 2.20.

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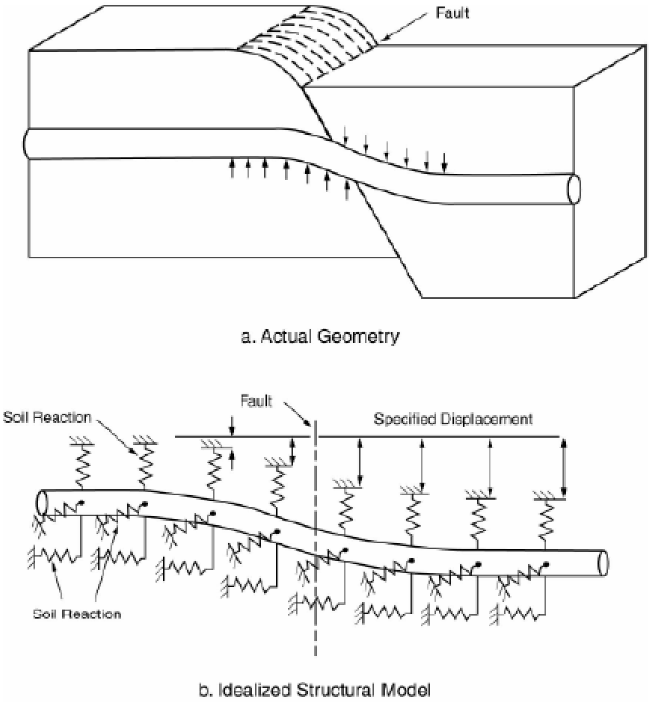

Figure 2.20 - Idealized structural model of a tunnel (Hung, et al., 2009).

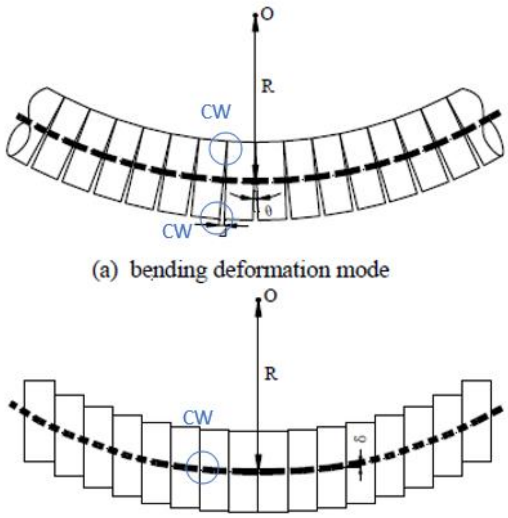

Figure 2.21 - (a) Bending deformation mode (b) Dislocation mode of a tunnel (Hung, et al., 2009).

In this way, the longitudinal aspect of the tunnels is very relevant, being one of the weakest points the junction of the segments that compose it (Cui, et al., 2015).The deformation of these joints causes the longitudinal deformation of the tunnel, identifying two modes of longitudinal deformation: bending deformation Figure 2.21(a) and dislocation Figure 2.21(b).

- Bending deformation: the segments rotate around the centre of the deformation curve, causing compression at the top and tension at the bottom edge.
- Dislocation: the deformation curve builds up due to differential settlement of the rings of adjacent segments, with shear stress between rings predominating.

As in the case of bridges, the same term "Conceptual Weaknesses (CW)" is used to refer to such weak points.

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In bored tunnels, in addition to circumferential connections, longitudinal connections are common Figure 2.22, as the rings of the cross-section usually consist of several elements (Arnau Delgado & Molins i Borrell, 2012). Figure 2.23 shows the CW of the cross-section of bored tunnels. In the case of cut and cover tunnels, the cross-section is usually a single element and therefore there are only circumferential joints.

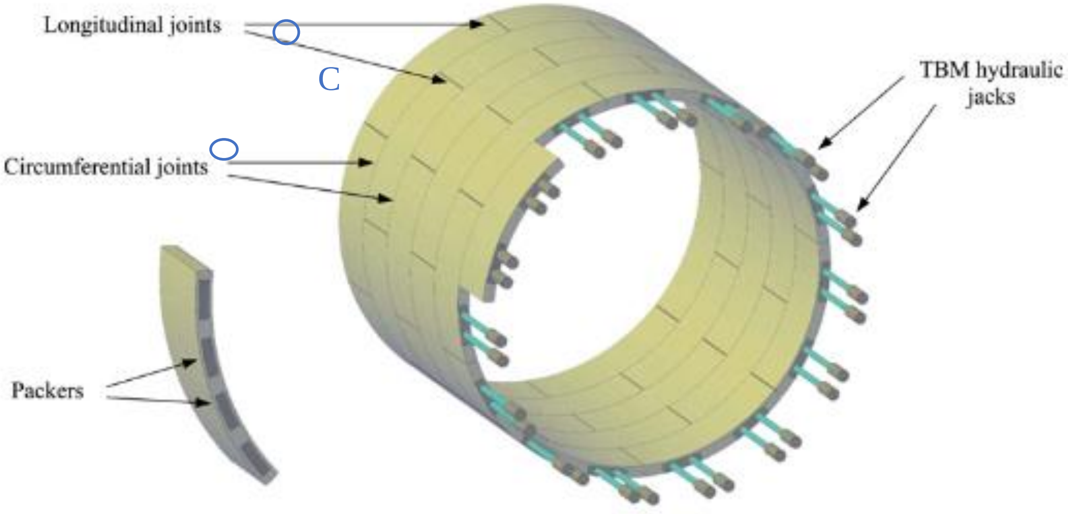

Figure 2.22 - Segmental tunnel lining. From (Arnau Delgado & Molins i Borrell, 2012).

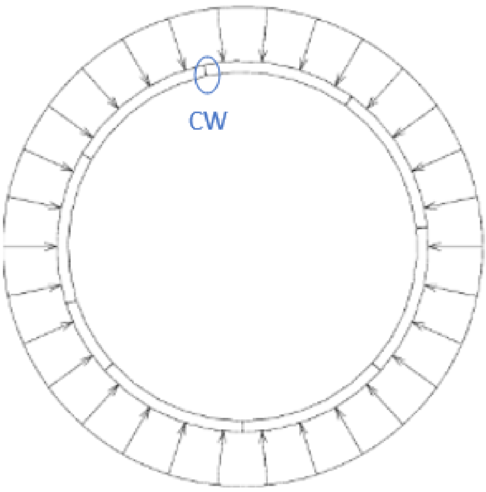

Figure 2.23 - CW of the cross-section of bored tunnels. From (Arnau Delgado & Molins i Borrell, 2012).

Furthermore, in cut and cover tunnels, as in the case of bridges, there are buried elements which are very difficult to inspect. These elements are related to the structural safety of the bridge and are specific to the cut and cover construction method. These elements are the permanent support walls (Hung, et al., 2009), and are identified in Figure 2.24 as BE (buried elements).

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Figure 2.24 - Permanent support walls in a cut and cover tunnel. From (Hung, et al., 2009).

2.3.4.2.2 Specific vulnerable zones

2.3.4.2.2.1 Lining

Tunnels lining are specific zones which are subject to overstress due to several factors. When an explosion occurs in a transportation tunnel, fragmentation of the liner is expected near the detonation point. Then, the peak blast pressures and gas pressures from the explosion may overstress the lining and the initial support systems. The fragments and overstress may induce failure of the liner and support systems. The extent of failure depends on charge weights, charge shapes, detonation points, types and materials of tunnel liner and support systems, thickness of liner, size and shape of tunnel, and type and amount of surrounding ground confinement (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

Tunnel failure modes can start from an overstress in the lining. When this happens, the stress–strain relationship of reinforced concrete is quite different from that under static load. This overstress may lead to failure of the lining if the strength of the lining material is less than the applied stress. The failure of the lining may be restricted to be a local failure such as spalling or local breach. When the tunnel lining is damaged locally or globally, failure of surrounding ground (i.e., collapse) and/or inundation with water (i.e., flooding) may follow. These failures are considered global failures.

Table 2.1 - Typical application of initial support and lining systems, from (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

Ground	Rock Bolts	Rock Bolts with Wire Mesh	Rock Bolts with Shotcrete	Steel Ribs and Lattice Girder	Cast-in-Place Concrete	Concrete Segments
Strong Rock	•	•				
		•	•			
Medium Rock		•	•	•		
			•	•	•	
Soft Rock				•	•	•
				•	•	•
Soil				•	•	•

Typical application of initial support and lining systems for each typical ground surface are shown in Table 2.1 and Figure 2.25 shows the typical arrangement of anchors.

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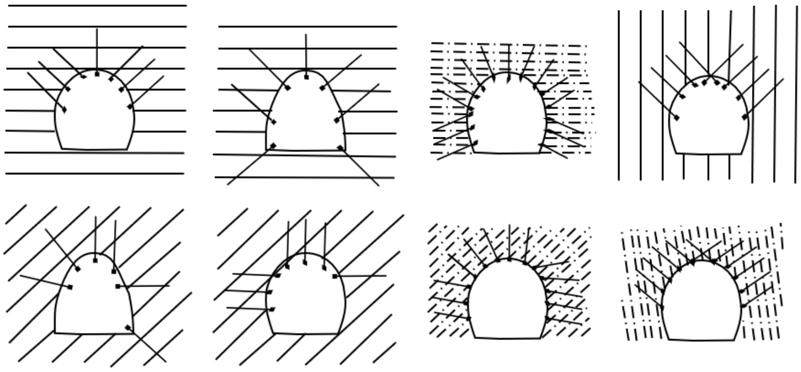

Figure 2.25 - System anchorage: Typical arrangement of anchors, Adapted from (Striegler, 1993).

2.3.4.2.2.2 Joints

Joints are typically vulnerable zones of tunnels. The large joint deformation definitely deteriorates the waterproofing capacity of the tunnel lining, which leads to serious leakage problems (Wang & Huang, 2020). However, the joint, as the main leakage passage, has a complex geometric layout, and complex deformation under external disturbances. Figure 2.26(a) shows the leak along the circumferential joints between the standard segments, and Figure 2.26 -(b) shows the leakage along the longitudinal joints between contiguous and standard segments.

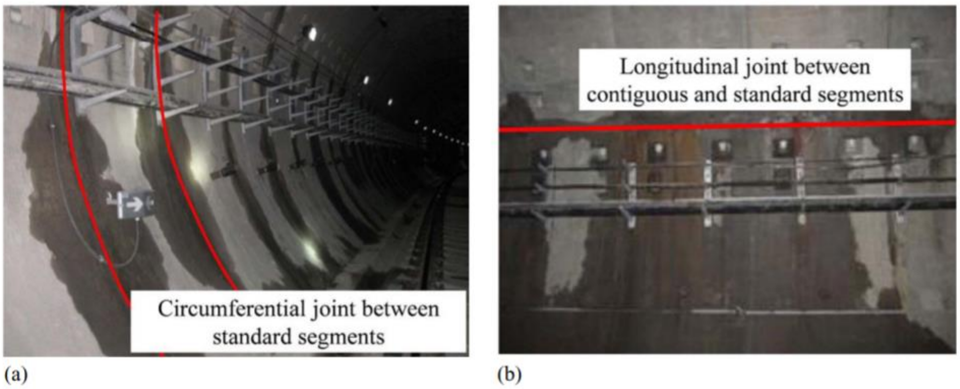

Figure 2.26 - (a) Circumferential joints and (b) longitudinal joints in a tunnel. From (Wang & Huang, 2020).

On the other hand, where a regular tunnel section in soft ground is connected to rigid station end walls or a rigid massive structure such as a ventilation building, special attention should be paid, because the stresses concentrations often occur in abrupt stiffness change conditions. This abrupt stiffness may be potential weak points in the structural system and may be more susceptible to flooding in case of breach, affecting the stability of the tunnel (Wang & Huang, 2020). Figure 2.27 shows that the maximum stresses in a conventional tunnel FEA (Finite element Assessment) are usual in the joint sections.

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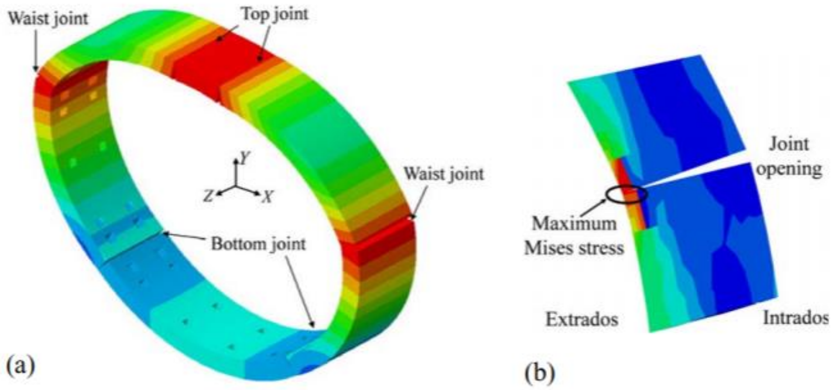

Figure 2.27 - Maximum stresses in a conventional tunnel Finite Element Assessment. From (Wang & Huang, 2020).

Moreover, the join dilatancy takes an important place on the collapse mechanism of a tunnel, considering effects of supporting force and seepage force have quite influence in the collapsing surface. Some studies have demonstrated that in homogeneous soil, the potential collapsing range decreases with the decrease of the dilatancy coefficient (Yang, et al., 2016). While in layered soils, the total height and the width on the layered position of possible collapsing block increase when only the upper soils dilatancy coefficient decrease.

Several types of joints are used in bored tunnels to avoid these problems. Tremie joints which are made from steel formed in soil trenches and rock encased in rock trenches, are as strong as the main body of the tunnel. Particularly in seismic areas, the flexible joints are designed to carry expected shear and tension loads and may sometimes be referred to as seismic joints. This type of joint presents potential weakness for ingress of water and flooding under blast wave conditions resulting from detonation of an explosive. Rigid joints may be designed to have the same section properties as the rest of the tunnel, effectively making the tunnel continuous without joints. The resistance of the joints is therefore the same as the tunnel lining (Hung, et al., 2009).

2.3.4.2.3 Effect of hidden defects/damages

When evaluating vulnerable areas, it is important to take into account that some vulnerable areas can develop only with the aging of the structure or also the occurrence of degradation processes. Some vulnerable areas also cannot be assessed by visual inspection or monitoring because they are not accessible. In such cases, modelling or advanced inspection techniques are recommended for the assessment.

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2.4 Consequence classes for infrastructure

The identification of the type and the properties of the structural system, as presented in the previous paragraphs, is crucial to the assessment of all potential consequences that may affect a 'class' of assets in case of structural failure. Consequences can be referred either to human and economic, environmental or social and political.
In order to take into account the consequences of failure or the malfunction of the structure, (CEN-EN 1990, 1990:2006) defines three **consequence classes**, described in the Table 2-2 below:

- High;
- Medium;
- Low.

For instance, CC3 is usually assigned to structures with essential functions for the communities. These principles can be applied either to the whole structure and to structural members – in the latter case, a particular structural member may be classified independently in a higher or lower class with respect to the rest of the structure; this approach can be very useful in the assessment of existing structures with localized issues.

Table 2.2 - Definition of consequences classes extract from (CEN-EN 1990, 1990:2006).

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

In the Eurocode and other standards consequence classes are generally defined for buildings; however, this approach might result not too accurate as the consequences are associated with the possible failure events and these can have rather different extensions based on the type of structure considered. The aim of future model codes is to introduce quantitative criteria for consequence classes based on the number of persons at risk and to define different classes for different structure and structural members. This differentiation also allows to consider consequences related to damage processes. An example of the indicative quantitative criteria for consequence classes for bridges, referenced in (fib MC2020, 2022), is showed below.

Table 2.3 - Indicative quantitative criteria for consequence classes for bridges (fib MC2020, 2022).

CC	**Description**
CC1	- bridges for pedestrian or cycle use only with traffic less than 50 persons per hour where people are unlikely to gather - road bridges in rural areas with AADT≤ 5000* vehicles per day with **or** with a span length $L < 10$ m - temporary bridges whose failure will lead to no fatalities **and** small economic and environmental impact
CC2	- all other bridges
CC3	- important pedestrian bridges, e.g. with long span(s) and/or where people are likely to gather

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	- road bridges with either AADT ≥50,000 vehicles per day or no possibility of detour (detour length greater than 25 km in rural areas and greater than 10 km in urban areas) or span length greater than 100m - railway bridges on transnational, national and main regional lines and special railway bridges (metro and tram bridges, bridges in important industrial plants) - bridges essential for emergency situations

2.5 Condition state classification for infrastructure in operational and non-operational networks

Infrastructures of the same road network will be evaluated based on their Condition State Classification, ranging between fully operational (new), good (operational/satisfactory), reasonable (operational/needs minor interventions), bad (operational/needs major intervention), poor (non-operational). Safety assessments and maintenance decision-making process will take into consideration the implications of the Condition State Classification of the structures as part of the structural diagnosis process.

2.6 Quality control classes for infrastructures

Quality management of infrastructure comprises the planning, control activities, and improvement work that ensure populations receive high-quality services: the right care, at the right time, responding to the service users' needs and preferences, while minimizing harm and resource waste.

Quality management includes three interlinked concepts, necessary to enhance quality across the system: quality planning, quality control, and quality improvement

- Quality planning includes aims, processes, and goals needed to create an environment for continuous improvement.
- Quality control entails monitoring established processes to ensure their functionality.
- Quality improvement is the action of every person working to implement iterative, measurable changes, to make services more effective, safe, and user-centred.

In general quality control classes can be distinguished in:

- Condition Based
    - Visual inspection
    - NDTesting & SDTesting
    - Monitoring
    - Modelling
- Risk Based
    - Visual inspection
    - NDTesting & SDTesting
    - Monitoring
    - Modelling

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3 Classification of hazards, actions and data-informed characterisation of hazardous events

3.1 Hazard and action systematics

3.1.1 Hazards

Hazard are defined as *a potential sources of undesirable consequences* [*reference to the glossary*]. This definition is consistent with the definition in ISO 13824:2009 and ISO 13824:2020. IEC 31010:2019 provides a more specific definition and clarifies that a hazard is a *potential source of danger, harm, or other undesirable consequences.* However, in EN 1990:2002 and ISO 2394:2015 hazards are more specifically defined with explicit reference to actions: *Hazard is unusual and severe threat, e.g. a possible abnormal action or environmental influence, insufficient strength or stiffness, or excessive detrimental deviation from intended dimensions […]*. It is for this reason that more detailed discussion on classification of actions is included in this report as a part of hazard systematics.

In general, a structural risk-informed verification shall address the following three categories of hazards:

- Category 1: natural hazards and unintentional human-caused hazards
- Category 2: malevolence and vandalism
- Category 3: negligence and human errors

Assigning hazards to a specific category can be based on the categorization provided in Table 3.1. The process of identification and characterization of hazards is explained in the following section of this report.

Table 3.1 - Examples of potentially relevant hazards for bridges and tunnels

Hazard	Category
Internal gas explosion	1
Internal dust explosion	1
Fire (natural or unintentional anthropogenic)	1
Impact by vehicle, aircraft, ship, …	1
Overloading	1
Earthquakes	1
Landslide	1
Mining subsidence	1
Tornado and Typhoons/Hurricanes/Cyclones	1
Avalanche	1
Rock fall	1
Soil and groundwater effect	1
Flood	1
Storm surge	1
Volcanic eruption	1
Physical and chemical attack	1
Tsunami	1
Utilisation	1
Fire (malevolence and vandalism)	2

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Vandalism	2
Bomb explosion	2
Public disorder effects	2
Design or assessment error	3
Material error	3
Construction error	3
User error	3
Lack of maintenance (deterioration)	3
Lack of knowledge	3
Errors in communication.	3

It is important to consider that concurrent and causally dependent hazard events are often more severe than hazards acting individual. Some examples include:

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

It is noted that, depending on the specific nature of the constructed asset / structure and its context, requirements other than those concerning threats to the assets may apply, e.g. in relation to risk of damage to the environment, loss of reputation and finances. Such additional requirements should be identified as part of the hazard identification and risk screening processes.

The representation of hazards should encompass a probabilistic representation and analysis of the temporal and spatial extent of their occurrences and intensities as well as their causal and stochastic dependencies over time and space.

3.1.2 Actions

Actions are a particular source of potential harm and present hazards to structures. Multiple actions may work simultaneously on a structure, which must be dealt with, with due attention to considerable complexity in appropriate representation of actual actions acting on the structure.

Based on their source, actions on structures may be classified in three categories:

- direct actions, i.e. group of forces acting on structures,
- indirect actions, i.e. group of imposed deformations or accelerations acting on structures,
- environmental actions, i.e. physical and chemical climatic loads (wind, snow, temperature, CO_2, chlorides, sulphates etc.) acting on structures.

Based on their variation in time, actions on structures are classified as:

- permanent actions, which are present during the entire duration of the reference period e.g. self-weight, permanent equipment, floor or road finishing, shrinkage, differential settlements, etc.
- variable actions, that are normally not present during the entire reference period, e.g. imposed, traffic loads or climatic (wind, snow, thermal, etc.) loads.
- accidental actions, that have a low probability of occurrence during the reference period, but can have an important influence on structural reliability due to their magnitude (e.g. impact, explosion, fire, impact, flood, avalanches, landslides, etc).
- seismic actions.

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Based on their variation in space, actions on structures are classified as:

- fixed actions, where the assumption is made that the action does not change position in space during the considered reference period, e.g. an example of permanent fixed actions is self-weight,
- free actions, where the considered action is assumed to change position in space during the considered reference period, e.g. an example of a variable free action is the traffic load on a bridge.

In Table 3.2, examples of potentially relevant actions for bridges and tunnels based on their source and variation in time are given.

Table 3.2 - Examples of potentially relevant actions for bridges and tunnels

Actions	**Category**
Self-weight	direct, permanent/variable fixed actions
Weight of installations or road finishing etc.	direct, permanent fixed actions
Imposed deformations due to shrinkage	indirect, permanent free actions
Imposed deformations due to differential settlements	indirect, permanent free actions
Other imposed deformations	indirect, permanent free actions
Imposed accelerations	indirect, free seismic actions
Traffic loads	direct, variable free actions
Wind load	environmental, variable free actions
Snow load	environmental, free variable actions
Chloride attack	environmental, permanent/variable free action
Sulphate attack	environmental, permanent/variable free action
Impact	direct, accidental free actions
Explosion	direct, accidental free actions
Fire	environmental, accidental free actions

In general, actions are considered as random variables with a certain distribution type. However, modelling of variable and accidental actions as a stochastic process may be needed, for instance when combining several variable actions or when combining effects of variable actions with degradation effects.

3.2 Identification of hazards

One of the first steps in executing a performance/safety assessment of a structure or when conducting a risk assessment procedure is the identification of possible hazards. This procedure requires imagination and creativity in order to think of all possible hazards to which a structure is exposed to.

There are several methods available, which aid with the thinking process of recognition of possible hazards (Schneider & Vrouwenvelder, 1997).

- Chronological analysis: a step-by-step procedure to think about what, where, and when can something occur;
- Utilization analysis: a thinking process with regards to the use of the structure in order to examine what will affect a situation, what events will accumulate, what facilities, machines and equipment are planned, what could go wrong in the planned operations, what could break down and thus become hazardous;

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- Sensitivity or influence analysis: to analyse which quantities influence the problem at hand, or to analyse new situations that have previously been considered to have a harmless influence, but might become dangerous;
- Energy analysis: investigating the potential of energy (e.g. gravity, kinetic, chemical, thermal, etc.) to occur in a hazardous way;
- Material analysis: considering the properties of building materials as potential hazards, such as combustibility, explosiveness, toxicity, corrosion, either individually or in combination.

These methods are commonly used and applied as a part of several commonly-used methods for hazards identification, such as Hazard Identification (HAZID) studies, Hazard and Operability (HAZOP) studies, and Failure Mode and Effect (and Criticality) Analysis (FME(C)A). Additionally, there are methodologies that allow critical situations to be highlighted, for example: exploring possible hazards where materials, information or responsibilities are handed over to someone else or where main functions have to be fulfilled. The question "what happens if … fails" is commonly used in this procedure; using logic trees, such as fault and/or event trees by means of the Fault Tree Analysis (FTA) and Event Tree Analysis (ETA) to introduce the logic, order and clarity into the thinking process. A common method for identifying hazards by considering input from several participants is to use brainstorming sessions or structured interviews to search for possible hazards. A further description of methods of hazard identification, such as those described above are described in Chapter 8. Additionally, possible hazards may be elicited from relevant literature, such as codes, regulations, guidelines and recommendations of professional bodies both on the national or international level.

3.3 Probabilistic representation of actions

According to (JCSS, 2001b), "*a complete action model consists in general, of several constituents which describe the magnitude,the position, the direction, the duration etc. of the action. Sometimes there is an interaction between the components. There may in certain cases also be an interaction between the action and the response of the structure*".

An action F can be described by two kinds of variables (constituents): F_O and W (see also (JCSS, 2001a)):

$$F = \varphi\ (F_O, W) \qquad [3\text{-}1]$$

where:

F_O is a basic action variable which is directly associated with the event causing the action and which should be defined so that it is, as far as possible, independent of the structure. For example, for snow load F_O is the snow load on ground, on a flat horizontal surface,

W is a kind of conversion factor or model parameter appearing in the transformation from the basic action to the action F which affects the particular structure. W may depend on the form and size of the structure etc. For the snow load example W is the factor which transforms the snow load on ground to the snow load on roof and which depends on the roof slope, the typeof roof surface etc,

φ (-) is a suitable function, often a simple product.

The time variability is normally included in F_0, whereas *W* can often be considered as time independent. A systematic part of the space variability of an action is in most cases included in *W*, whereas a possible random part may be included in F_0 or in *W*. For one action there may be several variables F_0 and several variables *W*.

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Parameters and variables describing the action model must be evaluated before the model can be used. In probabilistic modelling all action variables are in principle assumed to be random variables or processes while other parameters may be time or spatial co-ordinates, directions etc. Sometimes parameters may themselves be random variables, for example when the model allows for statistical uncertainty due to small sample sizes.

For each variable of Eq. [3-1], a suitable model should be chosen so that the complete action model consists of several models for the individual variables.

These models may be described in terms of:
- stochastic processes or random fields,
- sequences of random variables,
- individual random variables,
- deterministic values or functions.

Since models usually include time-dependent loads, the description of the variations in time is needed. In this respect, (JCSS, 2001b) identifies the following typical process models:
- Continuous and differentiable process,
- Random sequence,
- Point pulse process with random intervals,
- Rectangular wave process with equidistant intervals Δ.

A combination of the models listed above is typically used, such as hierarchical models, in which each term describes a specific and independent part of the time variability.
In the case of a load model containing slowly and rapidly varying parts, F Eq. (3-1) is defined as follows:

$$F = R + Q + S$$

where:
- R are the random variables, constant in time;
- Q is the slow rectangular process;
- S is the fast varying process.

For each process model, both single and combined, (JCSS, 2001b) provides the expression for the calculation of the distribution of the extremes.

These principles can be linked to structural monitoring, in particular parameter F_0 and time variability concept: an updated value of F_0 might be used, evaluated on the basis of the data collected from monitoring systems. The space variability of the action, covered in W, can also be re-evaluated based on the monitoring outcome, as well as the definition of the models probability distributions and the correlation patterns evaluation.

3.3.1 Probabilistic representation of main actions

3.3.1.1 General actions on bridges

Actions on bridges can be categorized as follows, consistently with section 3.1.2:
- Permanent actions
  - Structural,
  - Non-structural,
- Variable actions
  - Snow,

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  - Wind,
  - Traffic,
  - Thermal,
- Accidental actions
- Seismic actions

Actions on bridges are described in (JCSS, 2001b) and in (CEN EN-1991-2, 2005). In (JCSS, 2001b) a description of the probabilistic representation of actions on structures is provided, e.g. self-weight, wind, snow, live actions. In (CEN EN-1991-2, 2005), instead, the focus is mainly on traffic loads and the load models to be used in case of roadway, railway and pedestrian bridges. For a detailed description of the actions on bridges, refer to Chapter 8 of IM-SAFE project report D2.2 (Longo, et al., 2022).

3.3.1.2 General actions on tunnels

Actions on tunnels can be categorized as follows:

- Permanent actions
  - Structural
  - Non-structural
- Soil actions
- Hydraulic actions
- Landslide actions

Tunnels exposure leads in particular to the following action effects:

- Mountain water containing aggressive carbonic acid, which attacks almost all building materials. In addition, the content of sulphates and magnesium compounds is particularly harmful. The chemical reaction with lime and cement leads to an increase in volume.
- In road tunnels, a higher amount of chloride is to be expected due to the winter road salt.
- Unsuitable weather-sensitive building materials and an unsuitable cross-sectional design can lead to additional actions and defects.
- Dynamic effects of rail traffic and the previous effects of smoke gas. The vibrations and shocks caused by rail operations noticeably intensify and accelerate existing damage. Due to individual traffic and the increased number of road users, damage as a result of collisions can certainly not be excluded.
- Fires can cause considerable damage to the tunnel vault. This has to be taken into account for both rail and road operations.
- Wing walls of old tunnel portals are often connected to the preliminary cuts. Thus, the loads from earth and rock pressure, as well as stone fall, must be able to be absorbed. Slope and creeping movements can lead to deformation of the tunnel portals.

3.3.1.3 Effect of climate change on actions

Climate changes have a significant effect on climate loads on structures (wind, snow, temperature, precipitation) used for either design of a new structure or assessment of an existing one. The effect of climate changes on actions is still an open research topic due to the uncertainties of the evaluation of the climate change itself; as such, there is not a clear roadmap or established methods to take into account the climate change into actions definitions. Based on (Nasr, et al., 2019), which analyses the climate change scenarios defined in literature, the Intergovernmental Panel on Climate Change (IPCC) has developed four different scenarios, described in (IPCC AR5, 2014):

- RCP 2.6 (Representative Concentration Pathway 2.6)
- RCP 4.5
- RCP 6.0

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- RCP 8.5

Each scenario is identified by a number, which represents the approximate Radiative Forcing (RF), in W/m2, either at the year 2100, or at stabilization afterward, in comparison to the year 1750.

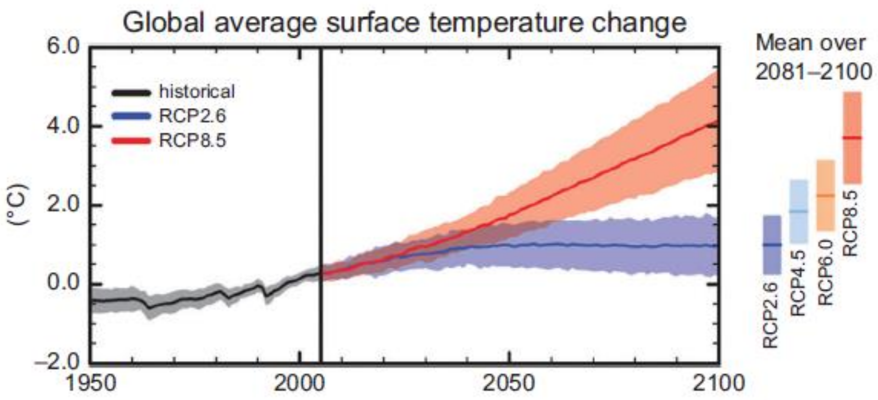

Figure 3.1 - Changes in the global average surface temperature relative to 1986–2005 for the different emission scenarios (extracted from (Nasr, et al., 2019)).

Based on the selected scenario, the magnitude of change and its different parameters can be due to the large degree of uncertainty involved in projecting the future climate and in the effects of climate change itself.

(Nasr, et al., 2019) also provides a review of the potential risks on bridges, though most of the risk can be extended to the other types of assets. A total of 31 risks are grouped into seven categories and summarized below:

- **Durability** (Risk group **D**), which includes
  - **D1**: accelerated degradation of superstructure;
  - **D2**: accelerated degradation of substructure.
- **Serviceability** (Risk group **S**),
  - **S1**: heat-induced damage to pavements and railways;
  - **S2**: risk of increased long-term deformations.
- **Geotechnical** (Risk group **G**), which includes
  - **G1**: higher scour rates;
  - **G2**: higher risk of side-slope failure;
  - **G3**: higher risk of landslides;
  - **G4**: higher risk of foundation settlement;
  - **G5**: higher risk of rockfalls, debris flows and snow avalanches;
  - **G6**: higher risk of soil liquefaction;
  - **G7**: additional loads on piles;
  - **G8**: damage due to clay shrinkage and swelling.
- **Increased demand** (Risk group **I**), which includes
  - **I1**: higher wave impact on piers and abutments;
  - **I2**: higher risk of wind-induced loads;
  - **I3**: additional snow loads on covered bridges;
  - **I4**: higher risk of thermally induced stresses;
  - **I5**: additional demand on drainage capacity;
  - **I6**: higher hydrostatic pressure behind bridge abutments;
  - **I7**: increased load on bridges with control sluice gates;

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- **I8**: increased stresses due to the faster loss of prestressing force;
- **I9**: higher ice-induced loads.

- **Accidental loads** (Risk group **A**), which includes
  - **A1**: higher chance of water vessel collisions;
  - **A2**: higher chance of vehicle-pier collisions;
  - **A3**: higher chance of vehicle accidents;
  - **A4**: higher chance of train-pier collisions.
- **Extreme natural events** (Risk group **E**), which includes
  - **E1**: increase in intensity and/or frequency of floods;
  - **E2**: increase in intensity and/or frequency of storms;
  - **E3**: increase in intensity and/or frequency of wildfires.
- **Operational risks** (Risk group **O**), which includes
  - **O1**: additional operational costs for snow removal;
  - **O2**: more frequent temporary bridge restrictions;
  - **O3**: increased risk of power shortage.

(Nasr, et al., 2019) also provides the agreement between the projections of the four scenarios on the trend of change of many climate parameters and phenomena, summarized in Table 3.3.

Table 3.3 - Projected future trends of different climate parameters/phenomena (Nasr, et al., 2019)

Climate parameter/phenomenon	Trend of change	Reference(s)
Temperature	- Higher global mean (T ↑) - Higher seasonal contrast in some locations (T↔)	IPCC (2013) Imada et al. (2017)
Heatwaves	- Increased intensity and/or frequency (HW ↑)	The World Bank (2012)
Solar radiation	- Possible increase in some regions (SR ↑)	McKenzie et al. (2011) Ohunakin, Adaramola, Oyewola, Matthew, & Fagbenle (2015)
Precipitation	- Increase in intensity and/or frequency in some regions (P ↑) - Decrease in intensity and/or frequency in other regions (P ↓) - Increase in contrast between wet and dry regions and seasons (P↔)	IPCC (2013)
Snowfall	- Increase in intensity and/or frequency in some regions (SF ↑)	IPCC (2013)
Relative humidity	- Decrease in relative humidity over land for most regions. (RH ↓) - Increase in relative humidity over land for some regions under some scenarios (RH ↑)	IPCC (2013)
Wind	- Increase in speed in some regions during some seasons (W ↑) - A decrease in speed during other seasons (W ↓) - Increase in intensity and/or frequency of extreme wind events (W ↑)	IPCC (2013) Cradden, Harrison, &Chick (2006) Bloom, Kotroni, & Lagouvardos (2008)
Soil salinity	- Increase in some regions (SS ↑)	Dasgupta, Hossain, Huq, &Wheeler (2015)
Storms	- Increase in intensity and/or frequency in some regions (S ↑)	IPCC (2013)
Sea level	- Sea level rise (SLR)	IPCC (2013)
Carbon concentrations in the atmosphere and oceans	- An increase in carbon concentrations in the atmosphere and in oceans (CC ↑)	IPCC (2013)
Ocean temperature	- A rise in ocean temperature (OT ↑)	IPCC (2013)
Run-off	- Higher annual mean run-off in some regions (RO ↑)	IPCC (2013)
Near surface permafrost area	- Decrease in near surface permafrost area (PF ↓)	IPCC (Intergovernmental Panel on Climate Change) (2013)
Ocean surface pH	- Decrease in global ocean surface pH (PH ↓)	IPCC (2013)
Fog	- Increase in the in-cloud liquid water content of marine fogs (F ↑)	Kawai, Koshiro, Endo, Arakawa, & Hagihara (2016)
Water level in rivers	- Increased water level fluctuation for some rivers (WL↔)	Úbeda et al. (2013)

Some attempts of quantifying the climate change effect on loads are available in (Croce, et al., 2019), according to which climatic loads should be commonly assumed to be stationary, with characteristic values that are currently evaluated on the basis of observed series of data covering 40-50 years of measurements.

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The characteristic values used in the assessment model can significantly vary due to the climate change influence, with a relevant impact on the structural safety requirements, as structures are designed to withstand all the external actions with the intended level of reliability (see principles of reliability-based approach in section 4.2.5). Also, the assumption of stationarity is becoming debatable, as the frequency of extreme events is continuously evolving. These characteristic values, according to (CEN/TC-250, 2020), should be multiplied for **scaling factors**, which are greater than or equal to 1, to consider climate change effects based on the observed data series.

In (Croce, et al., 2019), the authors suggest to evaluate Factors of Changes (FC) relying on climate projections, resulting from appropriate global or regional climate models: considering forty-year long time windows, FC represent the changes of characteristic values of the variable between the first time-window $c_{k,CM}(n=1)$ and the n-th one $c_{k,CM}(n)$.

LOAD	FC	c_k
Temperature	$FC_{k,CM}(n) = c_{k,CM}(n) - c_{k,CM}(n=1)$	$c_k(n) = \overline{FC_{k,CM}(n)} + c_{k,observed}$
Precipitation, snow, wind	$FC_{k,CM}(n) = \dfrac{c_{k,CM}(n)}{c_{k,CM}(n=1)}$	$c_k(n) = \overline{FC_{k,CM}(n)}\, c_{k,observed}$

where
$c_{k,observed}$ is the characteristic value obtained from the real measurements and $FC_{k,CM}(n)$ are the factors of change provided by the appropriate combination of the climate models.

The authors also describe a methodology to include the effect of climate changes on structural reliability (see section 8.2.3).

To move towards the inclusion of the climate change effects in the harmonized European standards, the proposal may be to set the minimum value for the scaling factors in the National Annexes. As it is recognized that the derivation of the FC is not straightforward, guidance to support NA development would be highly valued.

3.3.1.4 Effect of mobility change on actions

Road traffic in Europe has increased significantly over the last decades, both in terms of traffic volume and intensity. The effect of this trend on load actions should not be overlooked. In this respect, the use of data obtained from monitoring systems could be crucial for the updating of the values to be used in both design and assessment of structures.

Moreover, the continuous evolution of modern technologies has a deep effect on mobility and consequently on the actions that depend on it: automotive driving, alternative fuels (e.g. hydrogen) and electrical vehicles are few examples of the latest technologies which are becoming extremely widespread.

The latest emerging vehicle technology is platooning, which allows digitally tethered convoys of two or more trucks to travel closely together, through connectivity technology and automated driving support systems.

The mobility change may have a considerable impact on the traffic load models magnitude, configuration and corresponding relevant models included in the standardization codes, as well as on dangerous events that may occur, for instance on the risk of fire and explosion.

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3.4 Use of data in quantification of hazards

3.4.1 General

In a performance/safety assessment, once the identification process has taken place, hazards can be evaluated with a quantitative analysis. The quantification process differs according to the nature of hazards identified in the previous phase for the specific scenario under consideration. Data obtained from inspection, testing and monitoring can provide useful information for hazards evaluation.
In the following paragraphs the use of data in the quantification process is described, differentiating on the basis the type of structure and hazards:

- In the case of **bridges** (section 3.4.2), traffic loads are assumed to be the ones that provides the greatest source of potential risk: data are used for the assessment of structure-specific traffic load models. Real traffic loads can be measured using suitable techniques, for instance WIM technologies. Calibrated and cleaned data, then, serve as a basis for the use of probabilistic or simplified methods aimed at assessing load effects/values.
- In the case of **tunnels** (section 3.4.3), uncertainties in subsoil and groundwater are extremely complex and are together with fire assumed to be the ones that provides the greatest source of potential risk: he interactions between the existing structure, structural intervention, subsoil and groundwater are extremely complex, so that mathematical forecasts of the quantity of the expected effects reach their limits. This makes intelligent and adaptive protection and compensation measures based on the principles of the observation method all the more important. A distinction must be made between process-integrated and autonomous safeguards. Autonomous safety measures allow the compensation of expected or already occurred impacts. Process-integrated safety measures prevent incompatible impacts for the affected structures from occurring at all in the course of tunnelling. Both approaches are not mutually exclusive, but complement each other and may even have to be applied in parallel.

3.4.2 Assessment of structure-specific traffic load models for bridges

Traffic is a series of moving loads, therefore internal actions depend on the position of the vehicles. The effect of each vehicle is calculated using the transverse and longitudinal influence lines.

The influence lines are obtained after defining the cross section and the longitudinal static system. In addition, it is necessary to define the configuration of the roadway on the bridge deck, because the lateral position of the traffic lanes depends on it. It should be noted that influence lines are valid only within the elastic field.

The traffic model required to simulate bridge load effects must be consistent with the measured traffic at the site it claims to represent. Yet, it is important that there is the potential for variation from the measured traffic in the model; otherwise, the model would only represent multiple sets of the same traffic. By using parametric statistical distributions, the traffic model may remain sympathetic to the measurements, yet retain the capacity to differ.
For further information, see section 3.4.2.3.

For the probabilistic modelling of uncertain site-specific traffic conditions of a bridge, it is essential to collect data which effectively describe these conditions. In particular, Weigh-In-Motion (WIM) systems have been widely used to obtain various types of data related to traffic environments, such as vehicle characteristics and traffic load patterns (e.g. (Kim & Song, 2019), (Soriano, et al., 2017), (Meystre & Hirt, 2006), (O'Brien, et al., 2012)). For further details on WIM systems, refer to IM-SAFE project report D2.1 (Sánchez Rodríguez, et al., 2022).

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The traffic load models given in codes of practice are intentionally made conservative in order to be valid for a wide range of bridge types and loading conditions, and because the marginal cost of providing additional capacity is low. Load models for bridge assessment tend to be less conservative. However, in most countries the same bridge assessment principles are applied equally to bridges carrying dense traffic with heavily loaded trucks and those carrying sparse traffic with lighter trucks. In some cases, bridges may result to be structurally deficient according to these conservative load models. A more accurate representation of the current loading conditions on the bridge considered can be obtained considering the traffic weight and volume statistics for a specific bridge site.

3.4.2.1 Fundamental traffic load and traffic flow parameters

Fundamental traffic load and traffic flow parameters are described in the following paragraphs.

3.4.2.1.1 Traffic load parameters

The method to establish the desired safety level is by an estimate of the following parameters:

- the characteristic loads,
- α factors,
- maximum loads.

The load and material factors are estimated in such way that a safety level (expressed by β) belonging to the reference consequence class is obtained.

Annex A2 of (CEN-EN 1990, 1990:2006) gives specific rules and methods for establishing combinations of actions for serviceability and ultimate limit state verifications (except fatigue verifications) with the recommended design values of permanent, variable and accidental actions and ψ factors to be used in the design of road bridges, footbridges and railway bridges. It also applies to actions during execution. Methods and rules for verifications relating to some material-independent serviceability limit states are also given.

3.4.2.1.2 Traffic flow parameters

The traffic stream includes a combination of driver and vehicle behaviour, the driver or human behaviour being non-uniform, traffic stream also non-uniform in nature. The individual characteristics of both vehicle and human and the way they interact with each other have a significant influence on traffic. Thus, a flow of traffic through a street of defined characteristics will vary both by location and time corresponding to the changes in the human behaviour.

Thus, the traffic stream itself is characterized by parameters of which the characteristics can be predicted. The parameters can be mainly classified as:

- speed,
- flow,
- density.

The traffic stream parameters can be macroscopic, which characterizes the traffic as a whole, or microscopic, which studies the behaviour of individual vehicle in the stream with respect to each other. The fundamental stream characteristics are discussed below.

3.4.2.1.2.1 Speed

Speed is considered as a quality measurement of travel as the drivers and passengers will be concerned more about the speed of the journey than the design aspects of the traffic. It is defined as the rate of motion in distance per unit of time.

Several types of speed can be defined:

- Spot speed: spot speed is the instantaneous speed of a vehicle at a specified location;

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- Running speed: average speed maintained over a particular course while the vehicle is moving and is found by dividing the length of the course by the time duration the vehicle was in motion;
- Journey speed: effective speed of the vehicle on a journey between two points and is the distance between the two points divided by the total time taken for the vehicle to complete the journey including any stopped time;
- Time mean speed and space mean speed.

3.4.2.1.2.2 Flow

Flow, or volume, is defined as the number of vehicles that pass a point on a highway or a given lane or direction of a highway during a specific time interval. The measurement is carried out by counting the number of vehicles, n_t, passing a particular point in one lane in a defined period t. Then the flow q expressed in vehicles/hour is given by

$$q = \frac{n_t}{t} \qquad [3\text{-}2]$$

The variation of volume with time, i.e. month to month, day to day, hour to hour and within a hour is also as important as volume calculation.

Since there is considerable variation in the volume of traffic, several types of measurements of volume are commonly adopted which will average these variations into a single volume count to be used in many design purposes.

- Average Annual Daily Traffic (AADT): The average 24-hour traffic volume at a given location over a full 365-day year, i.e. the total number of vehicles passing the site in a year divided by 365.
- Average Annual Weekday Traffic (AAWT): The average 24-hour traffic volume occurring on week- days over a full year. It is computed by dividing the total weekday traffic volume for the year by 260.
- Average Daily Traffic (ADT): An average 24-hour traffic volume at a given location for some period of time less than a year. It may be measured for six months, a season, a month, a week, or as little as two days. An ADT is a valid number only for the period over which it was measured.
- Average Weekday Traffic (AWT): An average 24-hour traffic volume occurring on weekdays for some period of time less than one year, such as for a month or a season.

3.4.2.1.2.3 Density

Density is defined as the number of vehicles occupying a given length of highway or lane and is generally expressed as vehicles per km.

3.4.2.2 Measurements of traffic loads and traffic flow

WIM technology is the process of acquiring the real loads on bridges through the use of sensors inbuilt into the road. This allows the load effects on bridges to be determined on a network by network basis. WIM systems allow identification of the Gross Vehicle Weights (GVW), axle loads and axel spacings for each truck as well as the inter-vehicle gap data. WIM data sets also include the vehicle length, vehicle class, passing time and speed, among other factors.

3.4.2.3 Probabilistic methods for assessing traffic load models

The general scheme is illustrated in the following figure:

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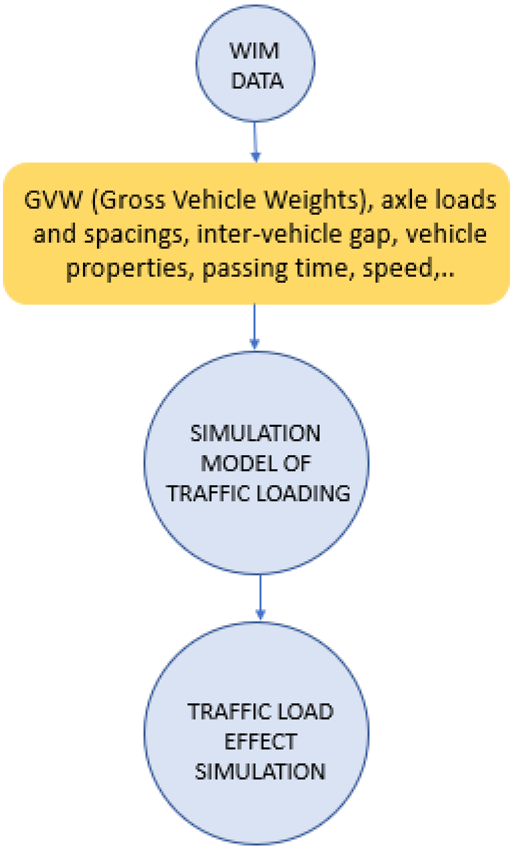

Figure 3.2 - General scheme for assessing traffic load models.

- **WIM DATA**: as the first step of the procedure, the measured WIM data must be calibrated and cleaned, as WIM systems may have some measurements errors and are thus sensitive to environmental conditions such as the variation of temperature, which can cause the subsequent process to produce distorted results.
- **SIMULATION MODEL**: key random variables representing the traffic environments of the structure are identified and fitted to probability distribution models. Then, using the developed simulation model and appropriate parameter values of the key random variables, a Monte Carlo simulation can be performed to generate artificial WIM data for the time period of interest.
- **TRAFFIC LOAD EFFECT SIMULATION**: load effects/values are calculated.

3.4.2.3.1.1 WIM data

The WIM data sets to be used in the assessment are collected in the specific structure-site.

3.4.2.3.1.2 Simulation model

Generally, the Monte Carlo simulation technique is utilized to determine the characteristic values of different traffic load effects.

The main strategy is to use vehicle categories, which can be defined by number of axles. Data analysis and simulation will be carried out within these vehicle categories. It is also assumed that ratio of various vehicle categories in the traffic stays constant in the reference period.
In (Hellebrandt, et al., 2016) the following approach is proposed: vehicle properties are divided to sub-categories within each vehicle category, based on the GVW they had appeared with in the measurements.

The second parameter describing this vehicle, the vehicle property, is then simulated from a sub-category of properties.

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One defines the vehicle property, which represents the ordered distance of axles and the ratio of the gross vehicle weight carried on each axle.

$$R_{n_i} = [d_1, d_{21} \ldots d_{n-1}, a_1, a_2, \ldots a_n] \quad [3\text{-}3]$$

where:

n is the vehicle category;
i is the index of the registered WIM measurement;
d_j is the axle distance between axles j and j + 1;
a_j is the relative load on axle j.

The relative load is defined as that fraction of the total weight that is carried by the axle under consideration. Vehicle properties can directly be determined from WIM measurements. Vehicle properties are sampled randomly from the empirical data and “coupled” to the simulated vehicle weights. This process is carried out separately for vehicle categories.

As described in (Getachew & Obrien, 2007), histograms for traffic characteristics such as GVW are usually fitted with appropriate probability density functions, which are then used for the simulations. The use of the fitted distributions for the simulations allows to reproduce the general trend in the data, taking into account other possible vehicle data, which are not obtained from the traffic records in the period of data collection. There are usually very few data points in the right-hand tails of the GVW histograms. Parametric probability density functions, instead, which are obtained by fitting the entire histogram of GVW, can give a poor description of the histogram tails.

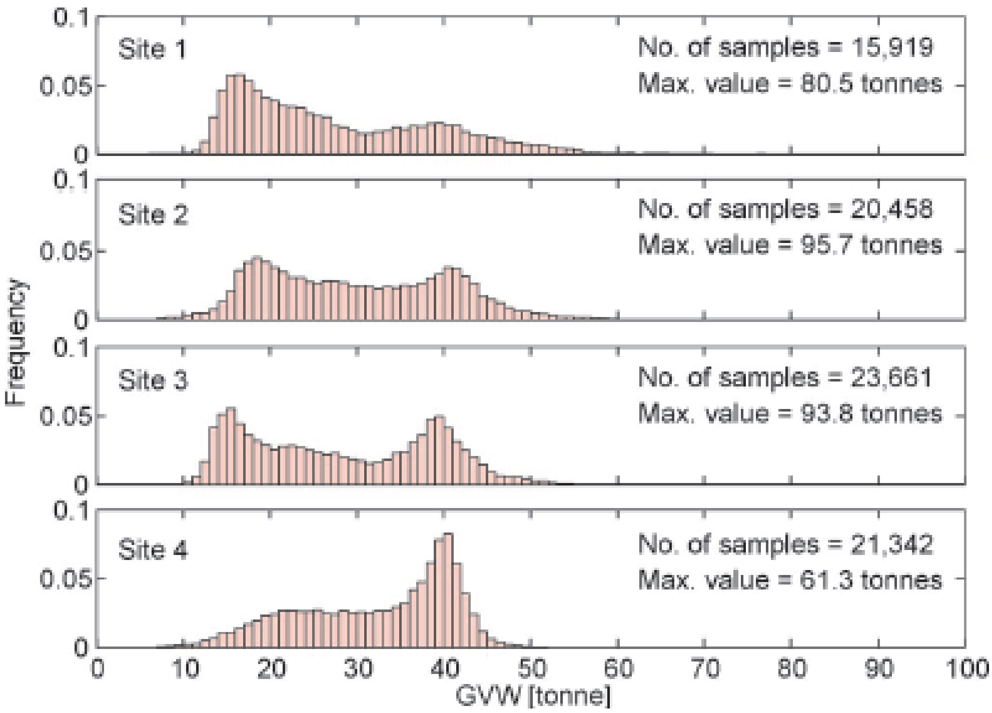

Figure 3.3 - Extract from (Getachew & Obrien, 2007).

The right-hand tail of the histogram is particularly important as it represents the heaviest vehicles and describes their probabilities of occurrence.

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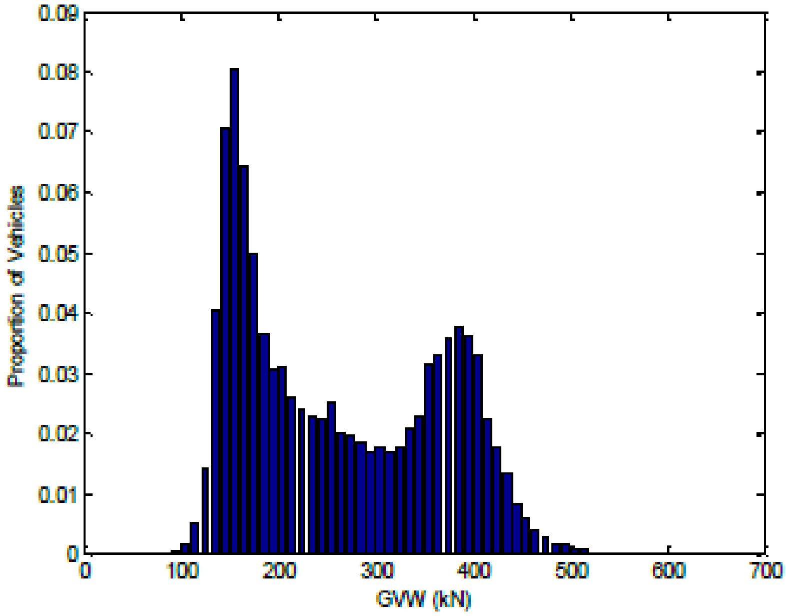

Figure 3.4 - GVW histograms, five axel trucks(extract from (O'Brien, et al., 2012).

The authors of (O'Brien, et al., 2012) favour the fitting of a "semi-parametric" distribution to the histogram of measured GVWs. It involves:

- using bootstrapping to directly simulate from the histogram where there is sufficient data to justify this and,
- fitting the tail of a Normal distribution to the data for extremely heavy vehicles.

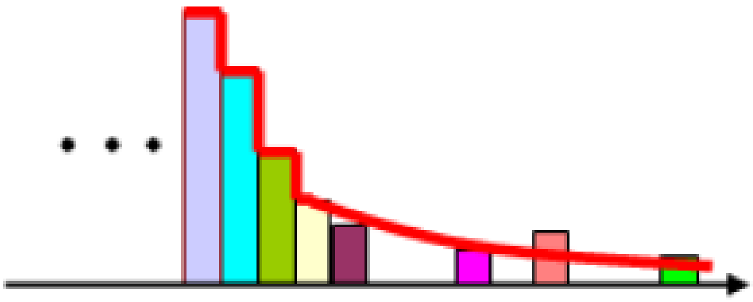

Figure 3.5 - Semi-parametric fitting (extract from (O'Brien, et al., 2012).

Besides the GVW, the axle loads and position of axles have a significant influence on extreme load effects especially for short bridges: the critical load cases for short span bridges are governed by individual axle and axle load groups. The critical load case is sometimes an extreme vehicle ,for instance a typical 5- or 6-axle truck.

The output of a typical traffic simulation model, which will serve for determining load effects on a bridge, is a set of heavy vehicles described by axle loads and distances.

3.4.2.3.1.3 Traffic load effects simulation

The final step of the traffic load modelling is analysis of the simulated load effects. Load effects caused by the simulated traffic are determined by taking the load effect resulting from a unit-weight vehicle with the same property as that of the simulated truck, and multiplying it with the simulated GVW. The main advantage of this approach is the reduction of computation time.

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This process can be used for maximum load, but can also be adjusted for other traffic load effects, such as shear or bending moment.

Other approaches are described in (Kim & Song, 2019) and (Soriano, et al., 2017).

A statistical distribution is fitted to the set of extreme values which were derived from the simulations. The type of the distribution as well as the parameters should be estimated. The type is likely to be an extreme value distribution.

For identification of characteristic load effects using the acquired WIM data, statistical extrapolations can be performed to the required return period (usually 1000 years). A standard Cumulative Distribution Function (CDF) plots the probability of non-exceedance against the load effect as shown in Figure 3.6.

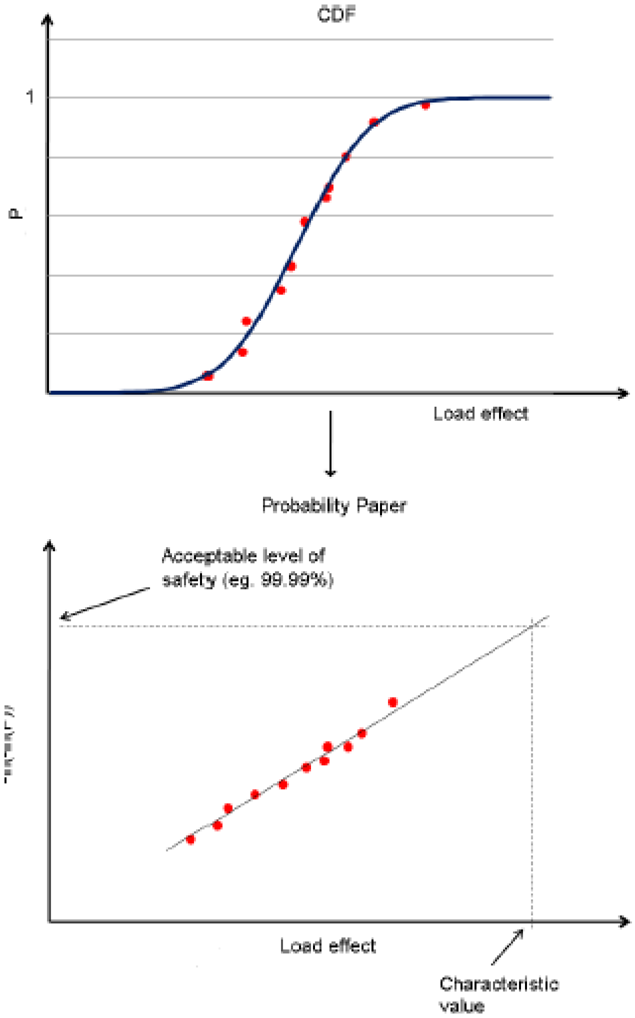

Figure 3.6 - Gumbel probability paper plot (extract from (O'Brien, et al., 2012)).

It can be conservatively assumed that each point represents a maximum-per-day or maximum- per-month load effect. The CDF has been replotted in the lower graph of Figure 3.6 to a Gumbel probability scale. The characteristic maximum load effect can then be found by extrapolating this trend to the predetermined acceptable level of safety. An alternative method can be employed whereby thousands of years of traffic can be simulated using Monte Carlo simulation and the 1000 year maximum found by interpolation. This approach is

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computationally intensive; however, it has the advantage that typical extreme loading scenarios can be identified.

The straight line in Figure 3.6 indicates that the data corresponds to a Gumbel Distribution. More usually bridge load effect data fits a Weibull distribution, which appears as a concave plot on Gumbel probability paper. A Weibull distribution is given by:

$$y = F(x, \lambda, \beta, \delta) = exp\left[-\left(\frac{\lambda - x}{\delta}\right)^{\beta}\right] \qquad [3\text{-}4]$$

where x is the variable in question (i.e. the load effect), λ is the threshold parameter, β is a shape parameter and δ a scale parameter.

The probability of exceedance of the 1000 year load is given by:

$$F(x) = 1 - \frac{1}{R.P} \qquad [3\text{-}5]$$

where $R.P$ is Return Period.

Rearranging equation 1 and substituting for *F*(*x*) allows calculation of the characteristic (i.e., 1000 year) load effect:

$$x = \lambda - \delta - ln\, F(x)^{\frac{1}{\beta}} \qquad [3\text{-}6]$$

3.4.2.4 Simplified approach

Applying the simulation technique described in the previous section requires not only a good knowledge of extreme value theory, but is also time consuming and complex. Simplified model are available in literature. (Getachew & Obrien, 2007), for instance, investigates site traffic dependence of extrapolated load effects, developing a site-specific simplified model whose aim is to reproduce similar critical loading events from knowledge of the site-specific traffic characteristics without having to perform a full Monte Carlo simulation: they assume that, placing pairs of heavy five-axle trucks at critical locations on the bridge, the load effects induced by these trucks are assumed to give a good estimate of the characteristic load effect values obtained from a full simulation.

3.4.2.5 Calibration of Eurocode alpha factors

The techniques outlined previously can be used to determine suitable adjustment factors (i.e. α factors) for Eurocode LM1. EC1 was originally developed considering a series of spans and influence lines. For each influence line, the characteristic load effects are found and compared to the corresponding load effect as calculated from the UDL and tandem axle load of LM1.
For further information see (Meystre & Hirt, 2006) and (O'Brien, et al., 2012)

3.4.3 Assessment of structure-specific hazards for tunnels

3.4.3.1 General

Hazard identification and the management of risk to ensure their reduction to a level ‘as low as reasonably practicable’ (ALARP) shall be integral considerations in the planning, design, procurement and construction of Tunnel Works. So far as it is reasonably practicable, risk should be reduced through appropriate design and construction procedures.

Responsibility for risk management shall be explicitly allocated to relevant parties to a contract so that they are addressed adequately and appropriately in the planning and management of a project and that financial allowances can be made.

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The use of a formalised Risk Management procedure shall be employed as a means of documenting formally the identification, evaluation and allocation of risks.

3.4.3.2 Risk Assessment and Management

Risk assessment is the systematic process of:

- Identifying hazards and associated risks, through Risk Assessments, that impact on a project's outcome in terms of costs and programme, including those to third parties;
- Quantifying risks including their programme and cost implications;
- Identifying pro-active actions planned to eliminate or mitigate the risks;
- Identifying methods to be utilised for the control of risks;
- Allocating risks to the various parties to the Contract.

For the purpose 'Risk' is defined as the combination of the consequence (or severity) of a 'hazard' and its likelihood, that is: Risk is a function of the consequence/severity of a hazard and the likelihood of occurrence of the hazard.

A 'hazard' is defined as an event that has the potential to impact on matters relating to a project which could give rise to consequences associated with:

- health and safety;
- the environment;
- the design;
- the programme for design;
- the cost for the design;
- the construction of the project;
- the cost associated with construction;
- third parties and existing facilities including buildings, bridges, tunnels, roads, surface and subsurface railways, pavements. Waterways, food protection works and all other structures/infrastructure that shall be affected by the carrying out of the works.

Hazards shall be identified and evaluated on a project-specific basis and their consequence risks shall be identified and quantified by Risk Assessments through all stages of a project (Project Development Stage, Construction Contract Procurement Stage, Design Stage, Construction Stage, and operational stage for any stipulated maintenance period).
T
he nature of the hazards (and hence their consequence risks) will be dependent on the stage of a project under consideration.

3.4.3.3 Risk Assessment

Risk Assessment is the formalised process of identifying hazards and evaluating their consequences and probability of occurrence together with strategies as appropriate for preventative and contingent actions.

Risk Assessment required at each stage of a project shall be summarised in appropriate Risk registers. Risk Registers shall clearly indicate the party responsible for the control and hence management of an identified risk (respecting any Contract responsibilities and liabilities), as well as mitigation measures.

The parameters to be used in the assessment of risks, in terms of probability of occurrence of a hazard and its severity of impact (consequence on cost, programme, environment, third parties and existing facilities) shall be both project specific and appropriate to the project stage under consideration.

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3.4.3.4 Risk Registers

The processes of Risk Assessment and the subsequent preparation of Risk Registers are required to identify and clarify ownership of risk and shall detail clearly and concisely how the risks are to be allocated, controlled, mitigated and managed. The systems used to track risks shall enable the management and mitigation of risks through contingency measures and controls to be monitored through all stages of a project.

Risk Registers shall be 'live' documents that are continually reviewed and revised as appropriate and available for security at any time. They shall provide an auditable trail through the life of a project to demonstrate compliance with the Code. They shall identify hazards, consequent risks, mitigation and contingency measures, proposed actions, responsibilities, critical dates for completion of actions and when required actions have been closed out.

3.4.3.5 Monitoring

Monitoring of the construction process shall be carried out by use of Inspection and Test Plans, audits and management reviews.

For any process, the Method Statements and Inspection and Test Plans shall ensure that the critical parameters are clearly identified and monitored in such a way as to be able to be confirmed by audit that they are in compliance with the requirements of the Contract and /or Third parties involved.

With particular regard to Tunnel Works in urban areas and where Third Party equipment or structures are at risk, Method Statements shall clearly identify "trigger levels" at which contingency action shall be taken. The Method Statements shall clearly identify the reporting roles and responsibilities and what actions are to be taken and by whom at each trigger level. Where risks are identified from Construction Stage Project Risk Register that have severity rating but which have been mitigated by the construction methods to an acceptable level, the Contractor shall provide the Client or the Client's Representative with an outline Emergency and Contingency Plan for dealing with the risk in the event that it is realised.

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4 Concepts of condition, performance and risk in structural assessment

4.1 Condition concept

4.1.1 Definitions

4.1.1.1 Condition

Condition is understood to mean state of a system (structure and/or its components) with regard to its appearance, integrity, structural performance and/or functionality, and circumstances and/or factors affecting this state, revealing limitations (such as deficiency or inadequacy in a system including defects) to the ability a structure to meet specified performance requirements (or lack thereof). Condition of a system (structure and/or its components) can be identified by means of condition survey.

The hierarchy of terms related to condition of a system is shown in Figure 4.1. For comprehensive set of definitions reference is made to the IM-SAFE terminology proposal in IM-SAFE online Knowledge Base, https://imsafe.wikixl.nl/).

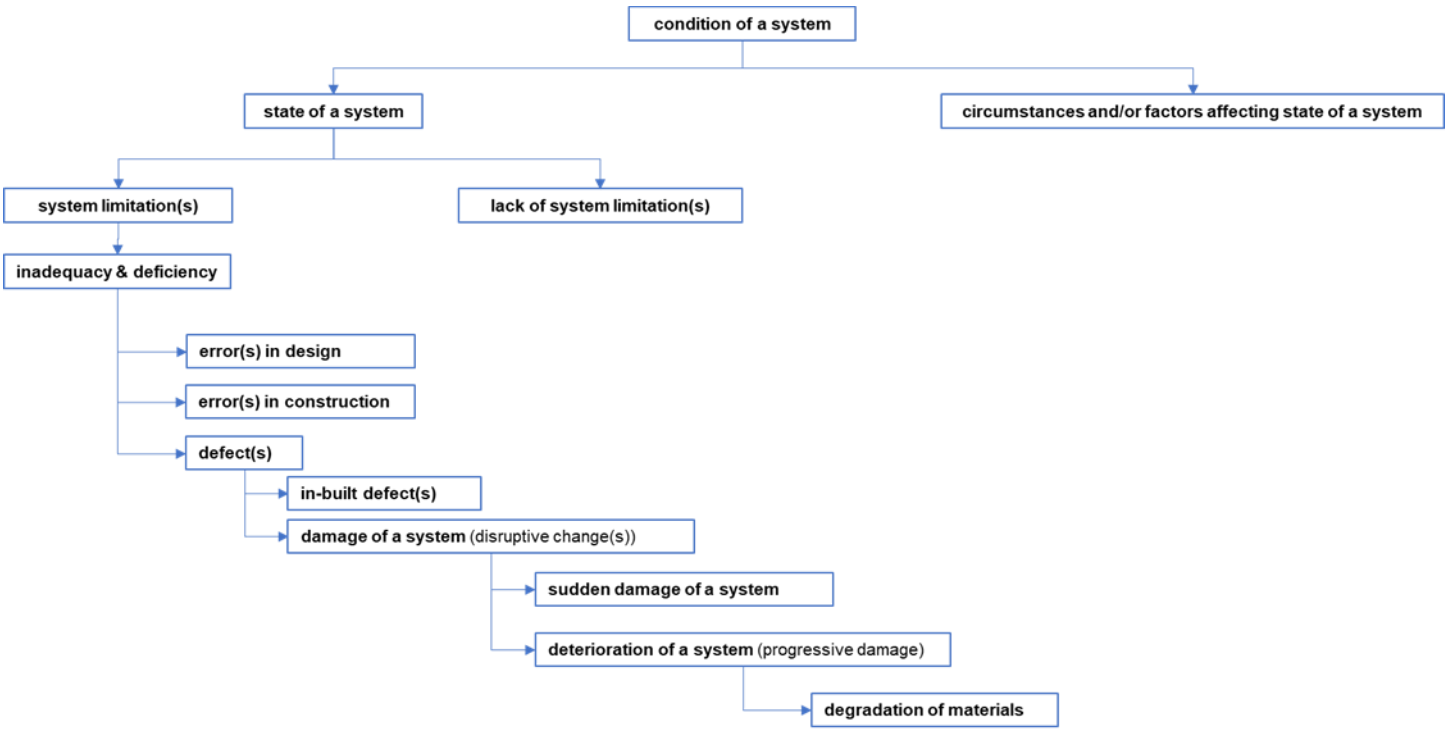

Figure 4.1 - Hierarchy of terms related to condition of a system

4.1.1.2 Condition survey

Condition survey is understood to mean the process of acquiring information related to the current condition of a structure, conducted with the aim of recognizing important limitations, defects and deterioration relevant for the ability of a structure to meet specified performance requirements (e.g. appearance, functionality and structural performance requirements). *Condition survey* would also seek to gain an understanding of the (previous) circumstances which have led to the development of that state, together with the associated mechanisms causing damage or deterioration.

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Condition survey is a part of condition control, which also involves condition assessment and condition evaluation.

4.1.1.3 Condition monitoring

Condition monitoring is understood to mean the process of observing such parameters on a quasi-continuous basis. Significant changes in the data relating to the condition of the structure may be indicative of a developing failure. Surveys, inspections, testing, and condition monitoring activities must be carried out from an early stage in the service life of the structure. Regimes for survey and/or monitoring the condition must be devised on the basis of the requirements of the conservation strategies selected for the structures or their components.

4.1.1.4 Condition assessment

Condition assessment is understood to mean a process of reviewing information gathered about the current condition of a system (structure and/or its components), its service environment and general circumstances, allowing a prognosis to be made of future condition, taking account of active deterioration mechanisms and, if appropriate, predictions of potential future damage.

4.1.1.5 Deficiency

Deficiency is understood to mean an imperfection, possibly arising as a result of an error in design or construction, which affects the ability of the structure to perform according to its intended function, either now or in the future.

4.1.1.6 Defect

Defect is understood to mean a specific deficiency or inadequacy in a system (structure and/or its components) which affects its ability to perform according to their intended function at the required level, either now or at some future time. *Defects* may be in-built or may be the result of deterioration or damage of a structure or its components.

4.1.1.7 Damage

Damage is understood to mean a disruptive change in the state of a system (structure and/or its components), impairing either its current or the future performance. *Damage mechanism* is a (scientifically explainable) (chemical, physical, mechanical, biological or multi-type) root cause of damage.

4.1.1.8 Damage indicator

Damage indicator is understood to mean a robust damage metric, computed based on damage parameters (or features). Damage indicator is usually a measurable and/or testable parameter that serves for quantitative or qualitative damage detection, damage localisation and/or damage characterisation.

4.1.1.9 Damage detection

Damage detection is a process of ascertaining whether the damage to structure exists or not.

4.1.1.10 Damage identification and characterization

Damage identification and characterization is understood to mean localization and assessment, including ascertaining the cause of the damage and its consequences (see damage detection). *Damage identification* involves *damage localization* (i.e. process of ascertaining where the damage to a structure is located) as well as determining the time of occurrence, the size and other features of the damage.

Damage feature/parameter is understood to mean a quantifiable property or pattern sensitive to damage, which can be either directly monitored (e.g., strain) or extracted from monitoring data (e.g., modal characteristics from accelerometer measurements).

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4.1.1.11 Damage assessment

Damage assessment is a process of ascertaining the severity of the damage to a structure.

4.1.1.12 Damage cause

Damage causes can be grouped in the following clusters

– errors in the design
– defective or inadequate material
– errors during fabrication
– failures caused by environmental action
– operating conditions or accidental events, including deterioration mechanisms.

4.1.1.13 Deterioration

Deterioration of a structure or its components is understood to mean a progressive reduction in the ability of a structure or its components to perform according to their intended functional specifications. In a system context, deterioration may also be caused by failures within the system.

Typically, deterioration of a structure or its components will be driven by degradation of materials and will adversely affects the structural performance over time due to:
– naturally occurring chemical, physical or biological actions,
– repeated action such as those causing fatigue
– normal or severe environmental influences
– wear due to use, or
– improper operation and maintenance of the structure.

4.1.1.14 Degradation

Degradation process is understood to mean worsening of the performance of material over time, as the result of (complex interaction between) degradation mechanisms. *Degradation mechanism* is (scientifically explainable) (chemical, physical, mechanical, biological or multi-type) root causes and the evolution phenomena governing development of degradation. *Degradation of materials* is a process of microstructural ageing and subsequent worsening of the performance of the materials over time, controlled by degradation mechanisms.

The main degradation mechanisms of relevance for transport infrastructure are listed in Table 4.2. In some cases, several non-redundant damage process terms are matched to one degradation mechanism.

4.1.1.15 Observation

A datum from a primary source, which may be acquired by human senses or by measuring/recording of some properties via adequate instruments. Observations can be qualitative (i.e. only the absence or presence of a property is noted), or quantitative if a numerical value is coupled to the observed phenomenon by counting or measuring.

4.1.2 Classification of deterioration and damage

4.1.2.1 Damage processes for assessment of bridges

4.1.2.1.1 Gradual observable and non-observable damage processes

Numerous processes can have a detrimental effect on a structure. Those which may act individually or in combination to generate safety and serviceability problems are here referred to as damage processes. The information on damage processes is crucial for a performance prediction, planning of preventive maintenance as well as for planning of eventual rehabilitation. Some damage processes are gradual and observable (e.g. corrosion related to structural steel). These can be detected with a proper inspection strategy. Other damage

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processes are gradual and non-observable (e.g. corrosion of post-tensioning steel). These should be handled by a proper maintenance strategy. By means of reliable information on the processes, inspection and maintenance strategies can be optimized (e.g. reduce Life Cycle Costs and traffic disturbance).

Table 4.1 - Damage processes for reinforced concrete and steel structures, adapted from (Breysse, et al., 2012)

	Chemical	Physical/Mechanical	Biological/Organic
Concrete	Alkali-aggregate reaction (AAR) Internal sulphate attack (ISA) External sulphate attack (ESA) and salt crystallisation Carbonation Chloride contamination Leaching Acid attack	Freeze-thaw Creep Shrinkage Thermal cracking Abrasion/ Erosion Fire Overloading Fatigue	Living organisms' activity Accumulation of dirt or rubbish Oil and fat contamination
Reinforcement and prestressing steel	Reinforcement steel uniform and pitting corrosion Fracture of prestressing steel (Steel dissolution due to) Stray current Fatigue		
Steel Structures	Chloride contamination Leaching Acid attack	Reinforcement steel uniform and pitting corrosion Fracture of prestressing steel (Steel dissolution due to) Stray current Fatigue	Living organisms' activity Accumulation of dirt or rubbish

In order to assess damage processes, damage should be graded with respect to their nature, intensity, extent and location. The gradation should be in accordance with the damage type, the cause of damage, and the material of an affected structural element.

The European Union funded project DURATINET (Correia, et al., 2012) and (Breysse, et al., 2012), which dealt with durability, safety and sustainability of concrete and steel transport infrastructures. DURATINET, making use of a cause-criterion, grouped the damage causes in the following clusters: errors in the design; defective or inadequate material; errors during fabrication; failures caused by environmental action, operating conditions or accidental events, including deterioration mechanisms. This last category encompasses the "interceptable damage processes", which are the focus of this section. Accordingly, proposed damage processes for steel were defined as:

- corrosion,
- fatigue,
- high temperature,
- ponding,
- overloading,
- accidental Impact,
- water retaining and accumulation.

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The damage process for concrete were proposed as shown in Table 4.1. It should be noted that here, some of the processes are catalyzing processes for an actual damage process. For instance, chloride contamination promotes the corrosion of reinforcement, a damage process which eventually causes failures. Other processes/actions such as accidental damages, seismic activity, vibration, scour and vandalism, where considered out of the scope of DuratiNet project (see http://durati.lnec.pt/). For concrete, construction faults are referred to as a cause of early deterioration.

Table 4.2 - Degradation mechanisms of concrete[a] and steel[b] bridges and associated classes of defects (Bień, et al., 2020)

Degradation mechanisms		Class of defects					
		Deformation	Destruction	Loss of material	Discontinuity	Contamination	Displacement
Physical	Accumulation of inorganic dirtiness [a, b]		□	□		■	■
	Cyclic freeze–thaw action[a, b]		■	■	□	□	
	Erosion[a, b]		□	■			
	Crystallization[a]		■	□	□		
	Extreme temperatures[a, b]	□	□	□	□	□	□
	Creep[a, b]	□					
	Relaxation[a, b]	□			□		
	Shrinkage[a]	□	□	□	■		
	Overloading[a, b]	■	□	■	■		■
	Fatigue[a, b]		■		□		
	Geotechnical condition changes[a, b]	■		□	■		
Chemical	Carbonation[a]		■				
	Corrosion[a, b]	□	■	■	□	□	
	Aggressive compounds action[a, b]		■	□		□	
	Chemical dissolving/leaching[a, b]		■	□		■	
	Reactions between material components[a, b]	□	■	■	□		
Biological	Accumulation of organic dirtiness[a, b]		□	□		■	□
	Activity of microbes[a, b]		□	□		■	
	Activity of plants[a, b]	□	□	□	□	■	□
	Activity of animals[a, b]		□			□	

Note: ■-basic degradation mechanism, □-additional degradation mechanism

Specifically (Bień & Gładysz-Bień M., 2016) proposed three main groups of the degradation mechanisms related with physical, chemical and biological phenomena. Table 4-2 shows these groups with respect to correlated class of defects. On the other side Table 4-3 shows the damage processes with respect to the materials and its impact on the structure.

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Table 4.3 - Damage Processes

Nº	Proposed Damage Processes	Material			Impact			
		Concrete	Steel	Masonry	Change in geometry	Change in integrity	Change in material properties	Change in actions
1	abrasion	•	•	•	•	•		
2	aggradation (alluviation)	•	•	•				•
3	erosion	•	•	•	•	•		•
4	pitting corrosion	•	•		•	•	•	
5	changing geotechnical conditions	•	•	•	•	•		•
6	aging of material	•	•	•	•	•	•	
7	alkali aggregate reaction	•			•	•	•	
8	chemical action	•	•	•	•	•	•	
9	corrosion related to prestressing steel	•	•		•	•	•	
10	corrosion related to reinforcement steel	•			•	•	•	
11	corrosion related to structural steel		•		•	•	•	
12	fatigue	•	•			•	•	
13	sulphate reaction	•			•	•	•	
14	corrosion related to equipment made of steel	•	•		•	•	•	
15	corrosion related to fixings, connectors	•	•		•	•	•	
16	overloading of an element	•	•	•	•	•		•
17	biological growth	•	•	•	•	•	•	•
18	freeze-thaw	•		•	•	•	•	
19	high temperature		•		•	•	•	
20								

4.1.2.1.2 Sudden events – natural hazards

The analysis of the impact that natural hazards have on structures and transportation infrastructure is yet to be included in the future Bridge Management System (BMS). The older bridges are often not or not adequately designed for natural hazards, and it is likely that the climate change has an adverse impact on frequency and intensity of gravitational hazards. The risk-based maintenance planning is actually developed for natural hazards and is being used by agencies world-wide, e.g. (ASTRA, 2014). It allows therefore to treat natural hazards in the same way as, for instance, traffic loads. The related failure modes and corresponding probabilities of failure must be defined in the same manner as for traffic loads. This task can be more difficult than the one for traffic loads since some structures were not designed for extreme loadings. Furthermore, the probabilistic characterization for hazards is more difficult. The bridges must be examined for different frequency and intensity of hazard events. Based on this analysis, the probability of failure can be assessed as a function of hazard intensity.

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4.1.2.1.3 Damage indicators for assessment of bridges

Table 4.4 and Table 4.5 show a breakdown of defects into classes, types and categories for bridges, which can be used for a systematic arrangement of information about damage processes in bridges. Since these defects in many cases can also be damage indicators, it makes sense to transfer this systematic to damage indicators as well.

Table 4.4 - Classification of the defects of concrete(a and steel(b bridges (Bień, et al., 2020) – Part I

Class of defect	Type of defect	Category of defect
Deformation	Incorrect geometry of constructed element	Incorrect shape of concrete or steel profiles(a, (b
		Invalid arrangement of reinforcement or of bolts, rivets, welds(a, (b
		Invalid arrangement of prestressing tendons(a
	Change of the geometry of the element axis	Excessive elastic deformation(a, (b
		Permanent deformation(a, (b
	Change of the geometry along the element length	Excessive elastic deformation(a, (b
		Permanent deformation(a, (b
Destruction of material	Change of the chemical characteristics	Change of concrete characteristics(a
		Change of reinforcing steel characteristics(a
		Change of prestressing steel characteristics(a
		Change of protective layer characteristics(a, (b
		Change of profile steel or steel slabs/walls characteristics(b
		Change of bolts, rivets, welds characteristics(b
	Change of the physical characteristics	Change of concrete characteristics(a
		Change of reinforcing steel characteristics(a
		Change of prestressing steel characteristics(a
		Change of protective layer characteristics(a, (b
		Change of profile steel or steel slabs/walls characteristics(b
		Change of bolts, rivets, welds characteristics(b
Loss of material	Loss of structural material	Loss of concrete(a
		Loss of reinforcing steel(a
		Loss of prestressing steel(a
		Loss of profile of steel or steel slabs/walls(b
		Loss of bolts, rivets, welds(b

Table 4.5 - Classification of the defects of concrete(a and steel(b bridges (Bień, et al., 2020) – Part II

Class of defect	Type of defect	Category of defect
Loss of material		
	Loss of the material of the protective layer	Loss of material of concrete protection(a
		Loss of protection of reinforcing steel(a
		Loss of protection of prestressing steel(a
		Loss of protection of profile steel or steel slabs/walls(b
		Loss of protection of bolts, rivets, welds(b
Discontinuity	Crack	Crack of concrete(a
		Crack of reinforcing steel(a
		Crack of prestressing steel(a
		Crack of protective layer(a, (b
		Crack of profile steel or steel slabs/walls(b
		Crack of bolts, rivets, welds(b
	Fracture	Fracture of concrete(a

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		Fracture of reinforcing steel(a
		Fracture of prestressing steel(a
		Fracture of protective layer(a, (b
		Fracture of profile of steel or steel slabs/walls(b
		Fracture of bolts, rivets, welds(b
Contamination	Inorganic	Aggressive(a, (b
		Neutral(a, (b
	Organic	Aggressive(a, (b
		Neutral(a, (b
Displacement	Incorrect linear displacement	Excessive movement(a, (b
		Restricted movement(a, (b
	Incorrect rotation	Excessive movement(a, (b
		Restricted movement(a, (b

4.1.2.2 Damage processes for assessment of tunnels

4.1.2.2.1 Gradual observable and non-observable damage processes

In analogy to 4.1.2.1 there are three main groups of degradation mechanisms in connection with physical, chemical and biological phenomena that can also be applied to tunnels (see also *(Strauss, et al., 2020)*). Table 4-7 shows these groups in detail with regard to the correlated damage classes for tunnels.

*Table 4.6 - Degradation mechanisms of tunnels and associated classes of defects **

Degradation mechanisms		Class of defects					
		Deformation	Destruction	Loss of material	Discontinuity	Contamination	Displacement
Physical	Accumulation of inorganic dirtiness		□	□		■	■
	Cyclic freeze–thaw action		■	■	□	□	
	Erosion		□	■			
	Creep	□					
	Relaxation	□			□		
	Shrinkage	□	□	□	■		
	Overloading	■	□	■	■		■
	Fatigue		■		□		
	Geotechnical condition changes	■		□	■		
	Temperature						
	Wet areas						
Chemical	Carbonation		■				
	Corrosion	□	■	■	□	□	
	Aggressive compounds action		■	□		□	
	Chemical dissolving/leaching		■	□		■	
	Reactions between material components	□	■	■	□		
Biological	Accumulation of organic dirtiness		□	□		■	□
	Activity of microbes		□	□		■	

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	Activity of plants	□	□	□	□	■	□
	Activity of animals		□			□	

Note: ■-basic degradation mechanism, □-additional degradation mechanism

4.1.2.2.2 Damage indicators for assessment tunnels

Defects associated with tunnel structures can be, as for bridges, subdivided into classes, types and categories, which can be as mentioned before for bridges used for a systematic arrangement of information about damage processes. It makes also sense to transfer this systematic to tunnel structure associated damage indicators.

*Table 4.7 - Classification of the defects of tunnels **

Class of defect	Type of defect	Category of defect
Deformation	Incorrect geometry of constructed element	Incorrect shape of concrete component
		Invalid arrangement of reinforcement
		Permanent deformation**
	Change of the geometry along the element length	Permanent deformation
Destruction of material	Change of the chemical characteristics	Change of concrete characteristics
		Change of reinforcing steel characteristics
	Change of the physical characteristics	Change of concrete characteristics
		Change of reinforcing steel characteristics
		Ice formation due to water
		damages on joint tape
		damages on sealing
Loss of material	Loss of structural material	Loss of concrete***
		Loss of reinforcing steel
Discontinuity	Crack	Crack of concrete****
		Crack of reinforcing steel
		Differentiate cracks, if necessary, as different effects/causes are associated with them
	Fracture	Fracture of concrete****
		Fracture of reinforcing steel
		Take into account the void in the inner shell
		Less thick inner shell
		insufficient concrete cover
		spalling/detaching
		wet area (e.g. through cracks, joint tapes in structures that retain pressurized water, access due to damage to the plastic sealing membrane in structures that relieve pressurized water)
		drainage damages
Contamination	Inorganic	Aggressive
		Neutral
		Efflorescence/washout
	Organic	Aggressive
		Neutral
Displacement	Incorrect linear displacement	Excessive movement
		Restricted movement
	Incorrect rotation	Excessive movement
		Restricted movement

* New construction with inner lining, open construction method, single-shell segmental lining method, double-shell segmental lining method old masonry lining tunnel

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** Crushing of cross-section, e.g. at the transition from closed to open construction; profile offset, also bulging possible; ovalization at segments;
*** Grinding, gravel pockets, superficial erosion,
**** Related to component; e.g. crack in inner shell, segment, etc.

4.1.3 Implementation of condition concept in through-life management of structures

The condition survey (see 4.1.1) as a part of condition control, which also involves condition assessment and condition evaluation, is one of the basic elements for the through-life management of structures. The results of the condition survey can be converted into a reliability index view using performance indicators (PIs) or key performance indicators (kPIs), see for instance the quality control plan concept of COST TU1406 WG3 (Hajdin, et al., 2018) and Chapter 5 of this report. For more information on the implementation of condition concept in through-life management of structures reference I made to Chapter 5 on performance indicators in this report and to IM-SAFE deliverable D3.2 (Darò, et al., 2022) for further details.

4.2 Performance concept

4.2.1 Definitions

In the IM-SAFE project context "*performance*" is defined as the efficiency of a system. In the Civil Engineering field, the concept of efficiency can be applied to structures either on network, system and component level, according to the assessment type and the scope of the analysis. The levels relevant for the infrastructures are described below:

- ***Network***: an aggregate of interconnected objects that collectively fulfil a *function*;
- ***System:*** a delimited group of interrelated, interdependent or interacting objects that is assessed for a potential risk. A structural system is specifically an arrangement of interacting structural members offering a potential solution to provide bearing resistance to a specified combination of actions;
- ***Components***: individually identifiable part of an object consisting of one or more elements, designed to provide a specific function for the object; specifically, a structural component is a portion of the structural system to be used as load-bearing part of works designed to provide mechanical resistance and stability to the works and/or fire resistance, including aspects of durability and serviceability.

For all levels under consideration, the goals set for the asset management must be attained. When setting the goals for the asset management of transport infrastructure one must recognise the multiple levels of objectives and multiple tiers posing requirements and creating constrains. The primary objectives of asset management are set at the highest strategic level by the Policy objectives, prevailing legislation, and administrative agreements. Examples of objectives considered for infrastructure include:

- Mobility.
- Sustainability
- Resilience.

These strategic objectives are governing when the primary requirements are set for the function of the infrastructure during its full life cycle life and when the primary requirements are set for the properties that do not affect the basic functionality of the infrastructure but have impact on user expectations. These requirements are listed as the functional requirements and the non-functional requirements.

Both categories of requirements should be specified in terms of aspect requirements. Aspect requirements considered for infrastructure include: reliability, availability, maintainability and safety (**RAMS**), sometimes extended by including security, health, environment, economics and politics (**RAMSSHE€P**). RAMSSHEEP criteria are described in detail in section 4.2.2.

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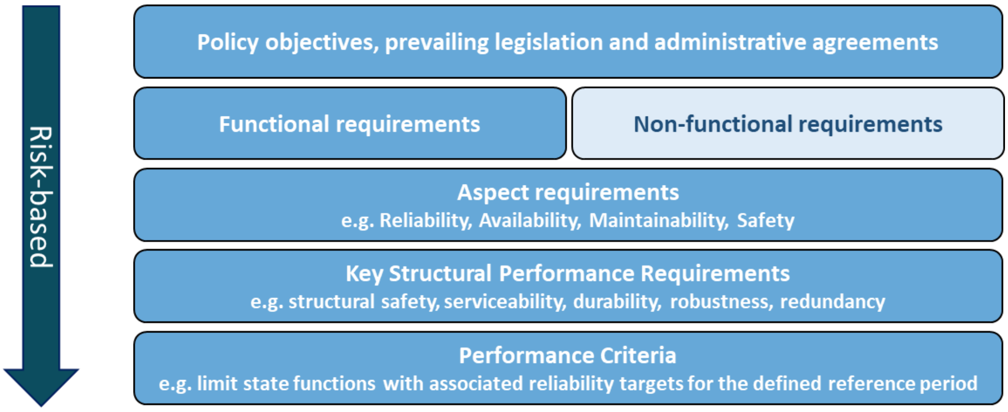

Figure 4.2 - Hierarchy of terms in performance concept

Hierarchy of terms in performance concept is schematically illustrated in Figure 4.2. The aspect requirements are established by means of the (Key) Performance Requirements, whereas the Key Performance Requirements (KPR)[1] are the (main) requirements set for the primary functions or properties for all aspects considered, specified in terms of performance. Examples of KPRs considered for infrastructure include requirements with regard to structural performance, which comprise, for instance, the requirements associated to structural safety, serviceability, durability, robustness or redundancy.

The Key Performance Requirements shall be established by means of the performance criteria, which are the quantitative limits, associated to a performance requirement, defining the border between desired and adverse behaviour.

With regard to structural performances, in context of limit state design, performance criteria are the threshold values that describe for each limit state the conditions to be fulfilled (in the reliability-based approach the performance criteria are established by limit state functions with associated reliability targets for the defined reference period).

It is important to highlight that IM-SAFE project focuses on structural performance, therefore it will only deal with some of the performance requirements listed above, in particular with structural reliability and structural safety.

4.2.2 Categorisation of performance requirements

Performance requirements can be classified into two main groups: technical performance requirements and non-technical performance requirements (Zanini, 2019).

Technical performance requirements are those related to structural safety and serviceability, traffic safety and durability, while non-technical performance indicators are those related to sustainability, allowing an evaluation of the environmental, social and economic performance of a civil engineering work, in this case bridges and tunnels (UNI-EN 15643-5, s.f.). Thus, sustainability encompasses these three dimensions, which are defined as follows (Zanini, 2019) (UNI-EN 15643-5, s.f.):

- Environmental requirements: referring to resource use, waste generation and pollution, among many others.

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[1] Note: in COST TU1406 *key performance requirements (KPRs)* are referred to as *Key Performance Indicators (KPI)*

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- Social requirements: referring to the accessibility and adaptability of infrastructures to society.
- Economic requirements: refers to life cycle cost and external costs.

More globally, technical and non-technical performance requirements can be grouped in the RAMSSHEEP concept (Reliability, Availability, Maintainability, Safety, Security, Health, Environment, Economics and Politics), where the first part of it (RAMS) covers technical aspects, and the second (SHEEP) covers sustainability aspects (Wagner, 2014).
Therefore, the nine terms that make up the RAMSSHEEP concept are requirements that encompass both technical and non-technical performance, thus being considered as Key performance requirement in management systems. Table 4.8 defines these key performance requirements of RAMSSHEEP for both tunnels and bridges.

Table 4.8 - KPRs and definitions. Adapted from: (Dette & Sigrist, 2011); (Hajdin, et al., 2018) and (fib, 2023).

KPI		Definition
Structural integrity (S, R, S)	Safety (structural)	Related to minimizing or eliminating the harm to people during the service life of a structure. The loss of life and limb due to structural failure is not included.
	Reliability (structural)	The probability that a structure will be fit for purpose during its service life.
	Security	Long-term functionality without much intervention.
(A, M)	Availability	Time proportion in which a system is in a functioning condition. Disruption originates from planned maintenance interventions.
	Maintainability	Facility with which a product can be maintained to repair the damage or its cause, repair or replace defective components without having to replace still-working parts, and avoid unforeseen maintenance measures.
Economy	Owner's costs	Adequate costs for construction and operation.
	Social costs	Acceptable and rare detours/accidents related to minimizing long-term costs and maintenance activities over the service life of a structure. Herein the user costs incurred due to detours and delays are not included.
Environment	Greenhouse gas emissions	Associated with minimizing damage to the environment during the lifespan of a bridge and balancing emissions with the utility of the structure.
	Resource consumption	
	Waste generation	
(H, P)	Health	The absence of causes of diseases other than failure (for example, the use of asbestos), which in most cases is regulated.
	Politics	Includes the elimination of the causes of public protest, image protection, etc.

It is important to note that this project will only deal with technical performance requirements.

4.2.3 Structural performance concept

The structural performance of a system or a component refers to the behaviour, or a condition as a consequence of actions, usually classified by means of a quantitative parameters (e.g. reliability index, ratio between resistance capacity and action effect), related to its:

- Safety
- Serviceability
- Durability
- Robustness.

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As described in (ISO-2394, 2015), the performance of a structure relates to the structure as a whole or parts of it. Structures and structural members must be designed, constructed and maintained so that they perform adequately and in an economically reasonable way during construction, service life and dismantlement.

In general, according to (fib MC2010, 2013):

- structures and structural members must remain fit for the use for which they have been designed;
- structures and structural members must withstand extreme and/ or frequently repeated actions and environmental influences occurring during their construction and anticipated use, and must not be damaged by accidental and/or exceptional events to an extent that is disproportional to the triggering event;
- structures and structural members must be able to contribute positively to the needs of humankind with regards to nature, society, economy and well-being.

Accordingly, the four categories of the structural performance that can be characterised by quantitative parameters are the following:

- **serviceability**, that is the ability of a structure or structural members to perform, with appropriate levels of reliability, adequately for normal use under all (combinations of) actions expected during service life;
- **structural safety**, that is the ability of a structure and its structural members to guarantee the overall stability, adequate deformability and ultimate bearing resistance, corresponding to the assumed actions (both extreme and/or frequently repeated actions and accidental and/or exceptional events) with appropriate levels of reliability for the specified reference periods. The structural safety must be analysed for all possible damage states and exposure events relevant to the design situation under consideration.
- **sustainability**, that is the ability of a material, structure or structural member to contribute positively to the fulfilment of the present needs of humankind with respect to nature and human society, without compromising the ability of future generations to meet their needs in a similar manner.
- **robustness**, that is the ability of a structure to withstand adverse and unforeseen events (like fire, explosion, impact) or consequences of human errors without being damaged to an extent disproportionate to the original cause.

In order to assess the performance, one shall select a set of quantitative performance indicators (see Chapter 5) which express physical states that can be used in relation to the performance requirements. Performance indicators can be defined on various levels of abstraction for the following:

- structural characteristics (e.g. stiffness/flexibility, load bearing capacity);
- response parameters (e.g. internal forces, stresses, deflections, accelerations, crack sizes);
- utilization factors;
- functionalities (e.g. safety for people, energy consumption, robustness, usability, availability, failure probabilities).

Models shall be set up to establish the relation between the various levels of abstraction.
The structural performance is assessed by a set of activities to verify the reliability of an existing structure, allowing a prognosis to be made of current and future response, taking account of relevant deterioration mechanisms and, if appropriate, predictions of potential future damage. The levels of the verification used to assess the compliance with the requirements for all design/assessment situations and commonly recognized are: the risk

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level, the probabilistic reliability level, and the semi-probabilistic level. These levels are briefly introduced in the paragraphs below and described in details in Chapter 6.

4.2.4 Principles of the limit state approach

In the contest of the limit state analysis, the structural performance (of a whole structure or part of it) is generally be described with reference to a specified set of limit states, which represent the conditions beyond which a structure no longer satisfies the design requirements.

To assess the structural performance of a structure, two domains consisting of desirable and undesirable states have to be clearly defined. The boundary between these domains is called *limit state* and entering the undesirable domain is defined as *failure*.

In general terms, attainment of a limit state can be expressed as:

$$g(\boldsymbol{e},\boldsymbol{r}) = 0 \qquad [4\text{-}1]$$

where:
g(e, r) is the limit state function, e represents sets of loads (actions) and r represents resistance variables.

Conventionally, failure (i. e. an adverse state) is represented as:

$$g(\boldsymbol{e},\boldsymbol{r}) \leq 0 \qquad [4\text{-}2]$$

Although limit state equations representing different limit state conditions are various, the limit state function g(e, r) can often be subdivided into a resistance function r(r) and a loading (or action effect) function e(e). In such a case, equation [4-1] can be expressed as:

$$r(\boldsymbol{r}) - e(e) = 0 \qquad [4\text{-}3]$$

Consequently, failure can be represented as follows:

$$r(\boldsymbol{r}) \leq e(e) \qquad [4\text{-}4]$$

Structures are analysed for:
- the ultimate limit states
- the serviceability limit states

Details of the limit state analysis is given in Chapter 6.

4.2.5 Principles of reliability-based approach

The reliability is the ability of a structure or structural member to fulfil the specified requirements, during the working life, for which it has been designed (ISO-2394, 2015).
The main objective of a reliability analysis by the probabilistic approach is a probabilistic assessment of the safety of the structure by estimating the failure probability (or the reliability index β) (fib MC2010, 2013).

The verification of a structure with respect to a particular limit state is carried out via estimation of the probability of occurrence of failure in a specified reference period and its verification against reliability requirements. The reference period is the timeframe used as a basis for assessing the statistical parameters of time dependent variables and of the target reliability.

The probability of occurrence of failure can be generally expressed as:

$$P_f = P\{g(e,\boldsymbol{r}) \leq 0\} = P\{M \leq 0\} \qquad [4\text{-}5]$$

where:

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$M = g(\boldsymbol{e}, \boldsymbol{r})$ represents the safety margin.

If in the limit state function parameters characterising actions, environmental influences, material and geometry are represented by the random variables E and R, the probability of occurrence of failure can be expressed as:

$$P_f = P\{r(\boldsymbol{R}) \leq e(\boldsymbol{E})\} = P\{R \leq E\} \quad [4\text{-}6]$$

where:
$E = e(E)$ and $R = r(R)$ are the basic random variables associated with loading and resistance, respectively.

The choice of the target level of reliability is based on the evaluation of all possible consequences of failure, combined with the analysis of the amount of expense and effort required to reduce the risk of failure. The target reliability levels may be based on explicit risk analysis, economic optimization, and/ or on calibration with regard to current best practice.

4.2.6 Principles of performance modelling

The characterisation of the structural system (either a network of structures or a structure) in terms of condition/performance plays a central role in the decision analysis (COST Action TU1402 , 2014-2019). This is explained by the fact that the probability of failure depends on how well this characterization is made. There is a consensual agreement among the engineering community that modelling a structure can be a challenging and complex task. This becomes even more challenging in the context of decision analysis where the objective is to reduce total costs – i.e. take best decisions.

Nevertheless, simplifications are inevitable, and this is mainly dictated by the available information. To minimize (as far as possible) the impact of these simplifications, the following aspects are to be considered thoroughly:
model of the structure (system behaviour),

- description of actions on the structure including loads and environmental factors,
- description of the structural condition (damage identification),
- definition of indicators,
- identification of key members,
- formulation of limit states g($\boldsymbol{X}$) and associated thresholds,
- description of the reliability analysis,
- procedure on the update of PIs,
- assessment of failure consequences and classification,
- risk of future use of the structure,
- required operational life and progress of relevant deterioration mechanisms.

The modelling of the performance requirements addresses all relevant issues concerning the intended use of the structures, the safety of people, as well as the qualities of the environment and economy throughout the entire life cycle of the structure (ISO-2394, 2015). Modelling of the interaction between the structure and its surroundings (i.e. any exposure the structure is subjected to and also the exposures which the structure might influence), dependencies between the structure and, for example, possible mechanical and electrical systems, as well as the influence of human and organizational errors, play a key role. The verification of the performance requirements, therefore, includes the following:

- all relevant failure modes;
- interaction between failure modes;
- interactions between the structure and its environment (wind, water, soil, use);
- non-structural elements (partitions, ceilings, finishing, relevant electric, hydraulic, and mechanical devices);

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- inspection and repair activities, quality management;
- functionality;
- environmental aspect (energy consumption, noise production);
- sustainability aspects (impact on human health, social property, biodiversity).

Different scenarios describing sequences of events which affect the performance, taking into account their likelihood of occurrence and their consequences, are to be accurately modelled, especially in presence of damage and deterioration processes and non-linear responses. The degree of accuracy of models should be adequate for the application at hand; the corresponding uncertainties in the models shall be identified and defined as measurable quantities (see section 6.1.3).

In the identification and description of the scenarios of events which are relevant, the following events shall be differentiated:

- Exposure events; actions, human errors and chemical environment.
- Constituent damage and failure events; direct consequences.
- Loss of function and/or cascading failures (progressive failure); indirect consequences.

Performance models should consider ergodic, as well as non-ergodic variables (random and systematic), time variability of loads and structural properties, such as:

- time variability when analysing load effects due to simultaneous actions;
- dynamic effects when inertia forces are significant;
- degradation mechanisms, which can be of a mechanical nature (like fatigue, load duration effects), a physical/chemical nature (corrosion, chloride ingress,) or a combination thereof (stress corrosion).

In the last case, it might be necessary to include inspection, monitoring, and maintenance in the model.

An adequate level for the assessment procedure has to be selected with proper attention to a correct formulation of the physical and functional boundary conditions. As described in 4.2.4, for each specific limit state model, which describes the behaviour/performance of a structure, a limit state function defined by its relevant basic variables is identified.

4.2.7 Performance models for assessment

4.2.7.1 Performance models at high level of approximation

Design and assessment of existing structures can be carried out with different levels of approximation, on the basis of the level of accuracy required to fully describe the structural response.

In the Level of Approximation (LoA) approach, in fact, a series of parameters and a set of design equations are used in order to characterize the behaviour and the strength of structural members, remembering that, in any case, all analyses performed for the design of structural members are approximations of reality with different levels of accuracy.

Parameters involved can be physical, mechanical and geometrical variables. The more time spent on the analysis, the more accurate the parameters involved and the higher the LoA, as shown in the Figure 4.3 below.

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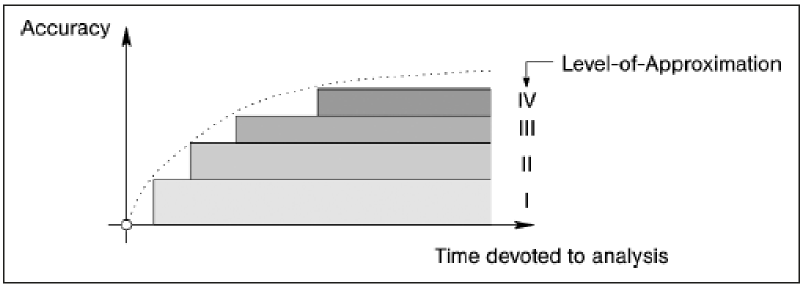

Figure 4.3 - Accuracy on the estimate of the actual behaviour as a function of time devoted to the analysis for various levels-of-approximation (extract from (fib MC2010, 2013).

The higher LoA can be reached using analytical and numerical processes in the analysis. The accuracy can also be enhanced with the integration the design information with the acquired data coming from inspection, monitoring and testing. A detailed analysis may show a 'hidden safety' due to good design, proper execution, inherent robustness or the simplification assumed in the resistance models used for the verification (that provides spare capacity). On the other hand, weaknesses can be caused by omissions in design, errors during execution, or lack of details of the older codes, which meanwhile have been corrected, as well as bad practice not recognized as such at the time of design and construction. Determination of remaining service life is therefore a matter of sufficient knowledge in various fields, like material science and deterioration processes, but as well knowledge of structural engineering, probabilistic and engineering experience.

4.2.7.2 Performance models for existing structures with degradation and damage

Structures are inevitably subject to deterioration that progresses over time, so the real duration of the period of use of a construction is beyond the scope of design forecasts. The level of degradation is determined using models built from information obtained from inspection and monitoring activities, design data, previous maintenance work and environmental conditions. Corrosion, time-dependent deformations and the interaction with the environment are just some of the principal causes of loss of structural safety.

According to (fib MC2020, 2022), it is of crucial importance to:

- evaluate the aggressivity of the environment in order to identify the possible deterioration processes;
- calculate the threshold values for deterioration and the expected rate of deterioration;
- conduct preventive measures to avoid or minimize deterioration and its effects.
-

(fib MC2020, 2022), moreover, highlights that in damaged structures damage may lead to loss of stiffness and a reduction of structural safety, so the bearing resistance of the structure and its structural members has to be assessed in order to determine:

- the loss of load-bearing capacity due to cracking or swelling;
- the reduced cross-section of the concrete due to delamination and spalling;
- the reduced cross-sectional area and ductility of the reinforcement ;
- the residual concrete/steel bond.

For further information about damage processes and corrosion modelling see (fib MC2020, 2022).

The diagnosis process, executed on the basis on monitoring data, should reveal whether the structure suffers from any type of deterioration and determine which state the deterioration has reached. It is to be highlighted that deterioration models are characterized by a number of parameters which are hard to determine and that should be properly calibrated based on site measurements. The standardization process should provide a strong basis for the determination of accurate damage models and the evaluation of the degradation process in

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time, so that the consequences of the damage evolution in space and time could be included in the performance assessment considerations.

4.2.7.3 Performance models for existing structures with non-conforming details

Detailing rules in existing codes often simply give limits to shape and geometry, with the aim to guarantee sufficient safety for the structures to be designed, whilst keeping user-friendly concepts for the designing engineers and reducing the probability of errors through simplification (fib MC2020, 2022).

Poor detailing and the consequences on the failure mechanisms is a key aspect to take into account in the assessment of existing structures. Often those rules have not been adapted to the evolution of building materials. Therefore, there is a need for better modelling of the structural response, including detailing as well as presence of damages or faults.

The use of empirical rules in codes make it often difficult to judge which are the consequences of violation of former code rules, the background of which are not always well known. In this context, the LoA Analysis proves to be a useful tool also for the judgement of details, with the direct evaluation of the flow of forces or the resulting stress fields in the member. The influence of construction errors, defects or poorly conceived alterations should be considered in the structural performance evaluation.

In reinforced concrete structure, for example, bad detailing can influence bond and fatigue strength of reinforcing bars, or on ductility, fire resistance or durability of structural concrete, like e.g.:

- Reinforcing bars with low ductility properties, can be associated with reduced ductility of the structure;
- Smooth or poorly ribbed reinforcement, can be associated with lower bond strength but also with increased ductility of the structure;
- Small concrete cover can be associated with reduced bond strength, but on the other hand with increased ductility of the structure;
- Small bar spacings or a high number of bar layers can be associated with reduced bond
- Strength and a higher risk of concrete cover spalling over large surface areas.

Poor detailing may concern e.g.:

- Indirect supports with lacking or insufficient suspension reinforcement;
- Gerber joints with insufficient anchorage of reinforcement and/or insufficient suspension reinforcement;
- Too pronounced staggering of longitudinal reinforcement, including too little anchorage at end supports;
- Insufficient anchorage of transverse reinforcement in flexural tension zones;
- Transverse reinforcement following the concrete surface, i.e. around concave corners;
- Transverse reinforcement not enclosing the longitudinal reinforcement;
- Smooth or poorly ribbed longitudinal reinforcement, limiting the tensile force increment resulting from an inclined compression field;
- Blocked supports due to unsuitable design, wrong execution or insufficient maintenance

4.2.8 Implementation of performance concept in through-life management of structures

Using a performance-based approach, a structure or a structural component is designed to perform in a required manner during its entire life cycle. In the case of existing structures, by using a performance-based approach we can assess whether the actual performance of an existing structure or structural members and their performance during the residual life satisfies

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the demands of the stakeholders. The choice of performance requirements used in the design depends on the situation that is being modelled.

Differentiating between new and existing structures should be considered:
- **New structures**: the performance-based design of a new structure or a structural component is completed when it has been shown that the performance requirements are satisfied for all relevant aspects of performance related to serviceability, structural safety, sustainability and robustness;
- **Existing structures**: the performance-based assessment of an existing structure or a structural component is completed when it has been identified whether all relevant performance requirements are satisfied or not. In the latter case the performance of a structure or a structural component is qualified as inadequate (failure).

The stakeholders have to give demands for performance of a structure or a structural component and its required service life. Those demands reflect the role(s) that a structure or a structural element should play under the intended conditions of construction, service and dismantlement.

For each aspect of performance that is relevant for a structure or structural component under consideration, the performance requirements must be specified. Performance requirements are established by means of the performance criteria and the associated constraints related to service life and reliability. The performance requirements are satisfied if all relevant performance criteria are met during the service life at the required reliability level. Quantitative limits, associated to a performance indicator (see section 5.1), defining the border between desired and adverse behaviour, are to be used as control parameters.

4.3 Risk concept

4.3.1 General principles

4.3.1.1 Definition of risk

To be able to measure risk, one needs to be able to identify its source – hazards. Risk can be defined as a measure of hazard severity (source?) and includes the information with regards to the likelihood or probability of that source delivering human or material loss, injury, or some other form of damage (Kaplan & Garrick, 1981).

Risk is a concept that is defined differently depending on the field in which it is used. The technical definition typically connects risk with expected consequences that are related to a given activity. When considering an activity with a single event, the risk *R* is the probability of occurrence of the single event *P* having potential consequences *C* (Faber & Stewart, 2003):

$$R = P \times C \qquad [4\text{-}7]$$

If, for example, *n* events with corresponding consequences and occurrence probabilities, the total risk is defined as the expected value of consequence *E*(*C*) (Kaplan & Garrick, 1981). For multiple independent events the expected consequence is:

$$E(C) = \sum_{i=1}^{n} p_i \times C_i \qquad [4\text{-}8]$$

In cases where it is difficult to estimate the probability of occurrence of a consequence directly, it can be estimated by decomposing to hazard and vulnerability. The expected value of consequence is then defined as follows (ISO 13824:2020):

$$E(C) = \sum_{i=1}^{n} p_i \times C_i = \sum_{i=1}^{n} \left(\sum_{j=1}^{m} T_j \times V_{ij} \right) C_i \qquad [4\text{-}9]$$

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where:

m	is the number of identified hazards,
n	is the number of identified consequences,
T_j	is the probability of occurrence of j-th hazard intensity,
V_{ij}	is the vulnerability (probability) of i-th consequence for a given j-th hazard intensity,
C_i	is the i-th consequence.

Risk is often represented with risk curves, which show the probability of exceedance for a certain magnitude of consequences. A popular example of risk curves is the F-N curve, which expresses the probability of exceedance of N fatalities.

4.3.2 Framework for risk management of structures

4.3.2.1 Framework setup

Risk management is defined as coordinated activities to direct and control an organization with regard to risk (ISO Guide 73, 2009). The risk management process systematically applies policies and practices when it comes to managing risks and a risk management framework is set up to provide foundations and organizational arrangements for designing, implementing, monitoring, reviewing, and constantly improving risk management throughout the organization. A generic framework for risk management consists of several components:

- defining risk management goals,
- performing the risk assessment to evaluate risk levels,
- risk treatment based on the evaluated risk levels,
- monitoring and review.

A schematic of a generic risk management procedure based on proposals in (ISO-13824, 2020), (ISO-31000, 2018) and (CUR-190, 1997), is shown in Figure 4.4.

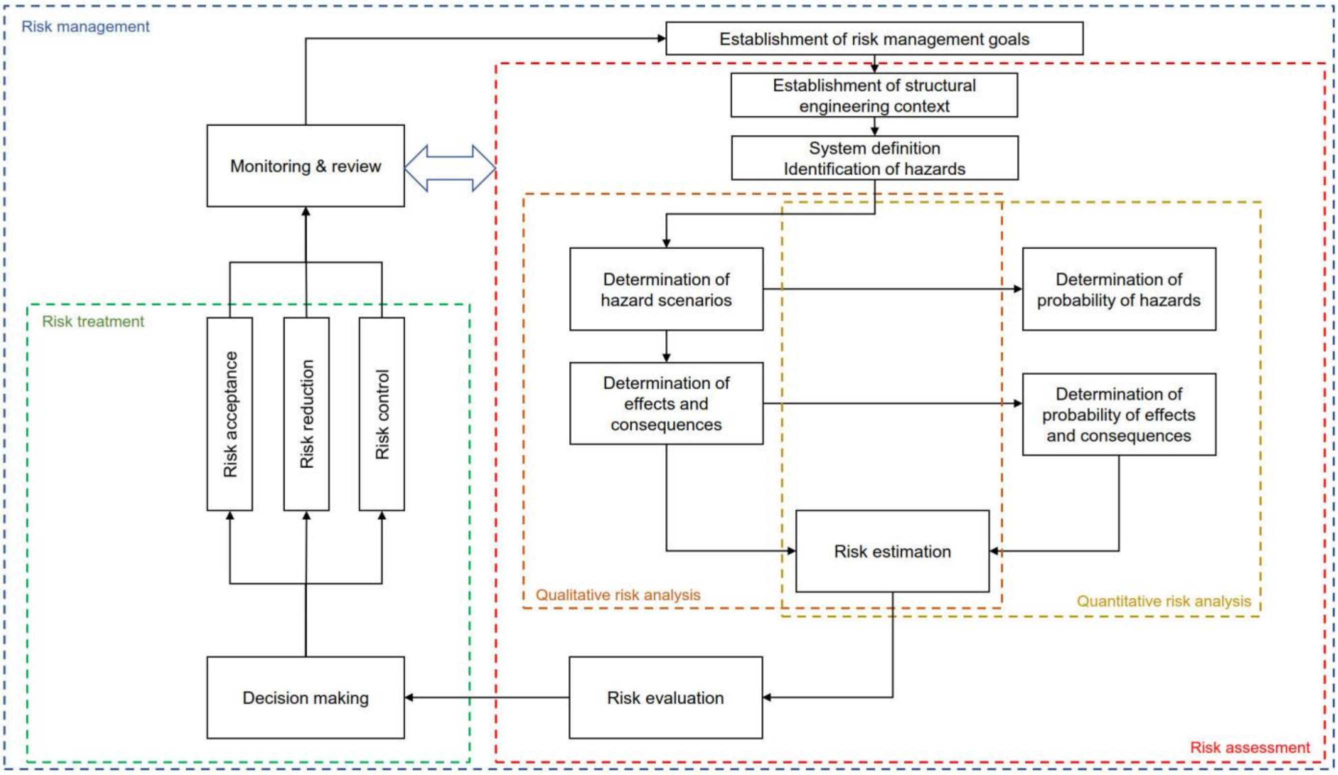

Figure 4.4 - Generic framework and process flow for risk management.

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4.3.2.2 Establishment of risk management goals

The objective of risk management is generally the optimal distribution of resources for the stakeholders, such as organizations, local communities, or society. In terms of risk management of structures, it is expected that the risk-management goals are expressed in terms of protection or optimization of asset use, maintenance of the designed performance of the structure, environmental changes, and regulatory demands. The goals are determined either by the cost-benefit of optional solutions or by various risks, such as those that are known to the society.

4.3.2.2.1 Definition of risk criteria

In this step, risk criteria are also defined, which have to be consistent with the risk management goals. They reflect the values of the decision-maker, the stakeholders and the society in general in terms of specifying the tolerable levels of risk based on their characteristics. Risk criteria are related to performance and consequently the target performance level of the structure. Risk criteria are usually multi-dimensional, as shown in Figure 4.5.

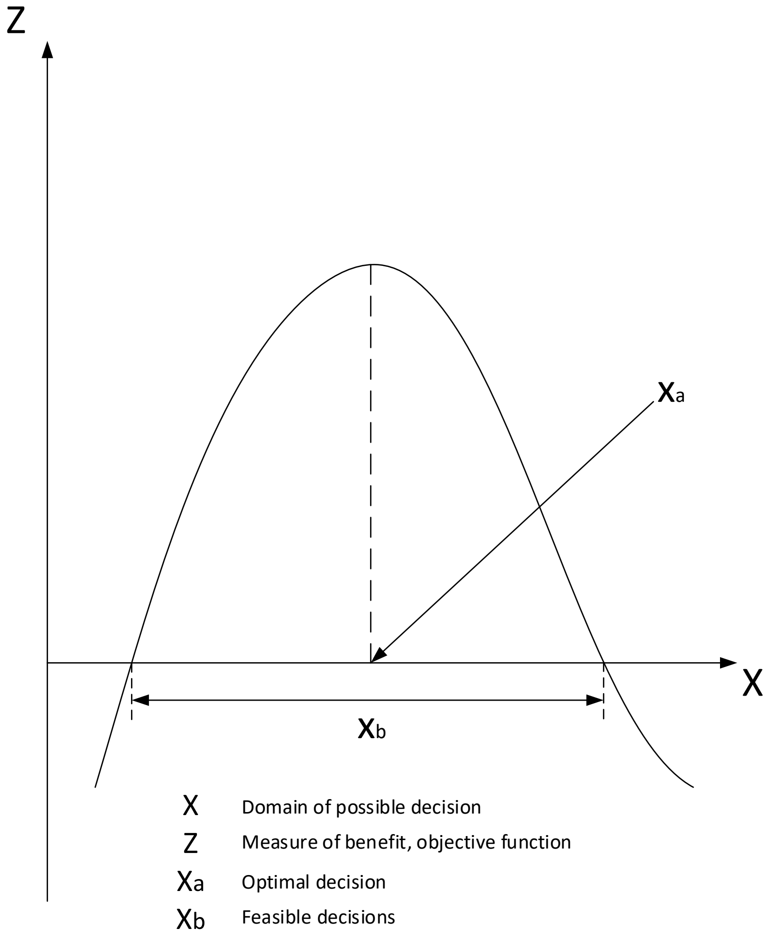

Figure 4.5 - Illustration of risk evaluation considering multiple risk criteria.

4.3.2.3 Establishment of structural engineering context

The establishment of context is the first step of risk assessment and is needed since organization factors can also be a source of risk. Context establishment involves defining external and internal parameters to be taken into account when managing risk and setting the scope, as well as risk criteria for the risk management policy.

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The external context is the external environment in which the organization seeks to achieve its objectives, such as : the current political, societal, or economic climate; key drivers and trends having impact on the objectives of the organization; relationships with and perceptions and values of external stakeholders. Internal context is the internal environment, in which the organization seeks to achieve its objectives, such as: organizational structure, standards and guidelines adopted by the organization, the organization’s culture, etc.

Specifically, when considering systems that include structures the typical structural engineering contexts are the following (ISO-13824:2020, 2020):

- Establishment the basis for design of structures: this criteria should be developed on the basis of target reliability levels;
- Assessment of existing structures: in case an existing structure is damaged or its functionality changes, risk assessment needs to be performed in order to verify that risk is within acceptable levels. If not, subsequent mitigations steps are needed.
- Assessment of exceptional structures and/or extraordinary events: risk assessment of exceptional structures is needed if their failure can have serious consequences.
- Providing support for decisions-making for other contexts: for example to minimize risk given limited financial resources, or to determine the optimal level of investment for risk reduction.

4.3.2.4 System definition

A system is a delimited group of interrelated, interdependent or interacting objects, determined on the basis of established context for the support of decision-making. The system is usually divided into subsystems and components, which together form a configuration that is representative of the total system by means of internal relations.

The definition of the system includes a clear identification of the functions of the system and should express how they are supported by the structural components or subsystems. For each subsystem, its characteristics should then be recorded, such as the type of structure, the codes and norms used in the design, functionality of the structure, location, etc.

4.3.3 Principles of qualitative risk analysis

4.3.3.1 Determination of hazards and hazard scenarios

One of the first and most important steps in the risk assessment phase of managing risk is the identification of all hazards that can cause undesirable events during the service life of a structure. Both subjectively and objectively known hazards should be taken into account, otherwise the risk analysis can lead to incorrect decision-making, which in turn can lead to unacceptable levels of risk and/or cost inefficiency. Possible simultaneous occurrence of multiple hazards also needs to be considered.

Based on the identified hazards, the hazard scenarios can be identified. Scenarios are in (ISO-13824:2020, 2020) defined as the sequence or combinations of events or processes necessary for system failure and resulting undesirable consequences for the system involving structures. The scenarios include possible collapse or damage of the structures, loss of lives or functionality, or other impacts cause to the determined to or by the stakeholders.

Techniques for hazards scenario analysis include:

- Fault trees;
- Event trees;
- Failure, mode, effects and criticality analysis (FMECA);
- Layer of protection analysis (LOPA).

The tools and techniques for hazard scenario identification and representation are further elaborated in Chapter 7.

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4.3.3.2 Estimation of consequences

Based on the identified hazards and hazard scenarios, the (undesirable) consequences following such events can be determined. The consequences can be described in terms of injury to human life, or with other economical/societal impacts caused to the stakeholders. The consequences can be either direct or indirect, which can be defined as any consequences associated with the loss of functionalities of the system or of its components/subsystems.
In terms of qualitative risk analysis, estimation of consequences can be performed by means of hazard scenario analysis from the occurrence of initial event, as specified in Section 4.3.3.1 using tools such as fault tree analysis or event tree analysis.

4.3.3.3 Qualitative risk estimation

In qualitative estimation, risk is estimated in a descriptive manner, where a common approach is to use a risk matrix concept. This provides a simpler basis and supports other aspects of decision-making. The qualitative estimation should therefore be used as: 1) an initial review to identify risks that may require detailed consideration; 2) in such cases that it provides sufficient information for decision-making; 3) when there is insufficient amount of data to perform quantitative risk estimation.

4.3.4 Principles of quantitative risk analysis

4.3.4.1 Determination of the probability of hazards

The probability of hazards is determined based on past data. Otherwise, expert judgment should be used. According to (ISO-13824:2020, 2020), data based on expert judgment can be obtained with the following approaches:

- Weighted-average of the expert-provided data;
- Iterative methods: varying level of interaction among experts where opinions are exchanged. After reviewing and possibly revising the rendered expert opinions, the process is repeated until a consensus is reached;
- Interactive methods: meetings to decide the appropriate data;
- Analytical methods: Bayesian integration of expert-proposed data, based on the confidence of each expert.

4.3.4.2 Determination of probabilities of occurrence of consequences

In this step, the probability of occurrence of undesirable consequences are determined. In quantitative risk analysis, this can be performed based on statistical estimation of data, uncertainty propagation using model analysis tools, and/or expert opinion. A quantitative expression of undesirable consequences should define the number of human casualties and the financial loss and environmental damage related to structural damage and loss of function.

4.3.4.3 Quantitative risk estimation

In a quantitative estimation, both consequences and probability of occurrence of hazardous scenarios are expressed with numerical values based on collected data, in contrast to the qualitative estimation where descriptive scale is used.
The data for risk estimation can be collected from various sources, such as: past records, engineering practice, experiments, expert judgement, etc.

4.3.5 Risk evaluation

In the phase of risk evaluation, the decision whether the estimated risk is acceptable or not is made, and if measures to reduce risk should be undertaken. This is therefore a decision-making step, supported by the results of the quantitative analysis and by risk criteria, consistent with the established risk management goals.

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4.3.5.1 Risk-based decision making

4.3.5.1.1 Risk acceptance criteria

Risk acceptance is often adjudged on the basis of absolute risk criteria: estimated risk is compared to acceptable or tolerable risk level given in tables, developed by governmental agencies or standardization organizations by comparison to encounterable risk in society. If the risk level is too high, appropriate treatment steps must be undertaken in order to reduce the risk to a level that is acceptable for the stakeholder.

One such risk acceptance criteria that are based on absolute risk comparison is the ALARP (As Low As Reasonably Practical) method. The main principles behind this are that: i) there is an upper limit to tolerable risk level, and ii) there is a lower limit, below which risk is of no practical interest. Between the upper and lower border is a region, in which the risk must be reduced to a level as low as reasonably practical in terms of spending money to reduce it.

However, it may often not be appropriate to base decision-making on direct comparison to the values given in tables. For example, when considering risk levels associated with structural failure, the probability of failure needs to be estimated and compared. Here, the problem arises from the subjective definition of failure; this depends in some cases on the consequences, since a failure with large consequences might generate publicity, while failures with minor consequences may remain unreported. Therefore, it may be more appropriate to derive risk acceptance criteria on the basis of (direct and indirect) costs, especially for high-cost construction.

4.3.5.1.2 Cost-benefit analysis

If failure consequences are to be taken into account, a more general criterion of assessing the acceptability of structural failure probability is a cost-benefit analysis, given by the net present-value criterion (Beck & Melchers, 2018):

$$max(B - C_T) \qquad [4\text{-}10]$$

where *B* is the total benefit of the project and C_T is the total cost of the project. This can be rewritten as a minimization problem in terms of costs, and/or it can be used to derive the optimal probability of failure for a structure.

4.3.5.1.3 Risk-based approach

Given the caveats of the absolute criterion for decision-making, (ISO-13824:2020, 2020) proposes the following risk-based or risk-informed approach to the decision-making process:

- i) establishing an understanding of the risk concept, the scope and purpose of the risk analysis, representation and evaluation of risk, and decision-making;
- ii) generating optional solutions;
- iii) assessing the options by performing risk analyses;
- iv) comparing and ranking the options based on criteria such as optimization of utility and cost-effectiveness;
- v) discussing the safety of the selected option, the need for optimization and the effect on other objectives.

4.3.6 Risk treatment

4.3.6.1 General

The risk treatment procedure involves identifying the options for handling risk, assessing these options and preparing and implementing a treatment plan. The approach to risk treatment is decided upon for the purpose of optimizing the expected utility, achieved by the decision making. Risk treatment measures can be implemented at different levels in the system

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representation, with regards to exposure, vulnerability and robustness, as shown below in Figure 4.6.

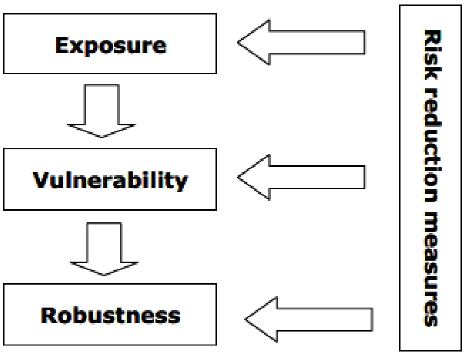

Figure 4.6 - Illustration of how risk treatment might be performed at different levels of the system assessed (extracted from (JCSS, 2001a)).

In the context of a load-carrying structure, the risk reduction can then be achieved by either lowering the exposure (restricting the use of the structure), reducing the vulnerability (strengthening the structure), or by increasing the robustness of the system by increasing the redundancy of the structural system. The feasibility of different options should then be assessed in terms of their risk-reducing effect R_{RE}, which is determined through the reduction of total risks achieved through the measure ΔR divided by the costs of the measure C_R, as described in (ISO-13824:2020, 2020):

$$R_{RE} = \frac{\Delta R}{C_R} \qquad [4\text{-}11]$$

By assessing the efficiency of different risk reduction measures, a prioritization of measures can take place in terms of reducing the exposure and/or vulnerability, or increasing the robustness of the considered system.

4.3.6.2 Risk reduction

Risk can be reduced either by lowering the consequences and/or the probability of occurrence. In practice, this involves physical manipulation of the considered system. For example, probability of failure of a structure can be reduced by relocating the structure to an area with lower hazard or by increasing the capacity. Consequences can be reduced by revising the design plan or by introducing mitigation measures on an existing structure.

4.3.6.3 Risk control

Risk control, in particular risk transfer, may be seen as a special case of risk treatment. The risk is transferred, for example, to a financial institution, and is usually related to costs.

4.3.6.4 Risk acceptance

If other approaches for risk treatment are not effective or feasible, risk acceptance can be chosen as an alternative option.

4.3.7 Monitoring and review

In this step of the risk management process the level of risk is monitored in order to keep it under a target level, regardless of whether the risk is treated. The effect of all elements of the risk management process is continuously reviewed in order to provide ongoing improvement of the process; for each element of the risk management process, records should be kept in order to substantiate undertaken decisions and enable their later review.

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5 Performance indicators

5.1 Performance indicators

Describing desired performance levels and determining how data is interpreted is as important as selecting the measure. It defines good and bad performance, and determines how the data is used. Performance is based on targets, the desired level of performance for a specific reporting period, and thresholds. Thresholds upper and lower limits of desired performance around a target value. Thresholds create the exact points where an indicator displays within a condition assessment e.g., green for good performance, yellow for satisfactory or red for poor.

Thresholds can be expressed in numerical format, in safety/reliability requirements but also in risk formats and should be assigned to performance indicators.

5.1.1 Conceptual remarks

Damage processes initiated on bridges, which are recognizable in form of certain characteristics, must be documented as observations by the bridge inspectors. Inspectors are also responsible for observing and assessing bridges/structures in order to determine those damage processes that are most likely to affect the bridge or its components.

The inspector must make a meaningful diagnosis with the help of the observations and knowledge of damage processes, knowledge of possible consequences is therefore essential. Relevant observations taking place in the course of an investigation can be made through the perception of the human senses or through data measured by instruments. Determining the absence or presence of a property belongs to the group of qualitative observations, and measuring or counting the observed phenomenon belongs to the group of quantitative observations. The fitness of purpose of a bridge or a structure can be measured by a performance indicator (PI). For instance, an observation of a crack width larger than 0.4 mm can be a sign (a) on one side that the reinforcement has been overloaded (at least once) or (b) that there is insufficient strength. This observation in the crack width can indicate two different outcomes regarding reliability and in consequence performance: one with an impact on reliability and one with no impact on reliability.

Successive inspections allow a distinction whether it is (a) a pure observation (e.g. stable crack) or (b) a possible PI (e.g. growing crack). Thus, there is the need of differentiation between observations and PIs. Observations indicates just the fact and PIs already comprises an interpretation of their effects on the structural performance.

For the quality control of bridges and structures, knowledge of the interaction between the observations and the PIs is of highest relevance. The definition of this relationship therefore requires a deep understanding of the underlying damage processes. The correlation of observable symptoms with potential damage processes may reveal what damages can be expected or what observation one might make in the future. Table 5.1 summarizes most common drivers/damage processes and associated selected performance indicators of bridges and Table 5.2 and Table 5.3 those of tunnels.

These tables indicate which performance indicator is related to which damage processes and can be detected with the highest probability by means of visual inspection or by means of extended test and monitoring procedures. In the tunnel-specific Table 5.2 and Table 5.3, the influence of the damage on the load-bearing capacity and on the functionality / operability of the tunnel is also assessed in the last two columns. A significant high level of influence is rated at 10 and no influence is rated at 0.

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Selection of Performance Indicators: In order to select the most important Performance Indicators the following steps should be followed: 1. Define crucial Performance Goals (for example: safety, serviceability, reliability, durability, availability, maintainability, ...) 2. Categorise Performance indicators in relation to Performance Goals (at different levels: component, system, network; taken into account different aspects: technical, sustainability, socio-economic), 3. Answer following questions: Is it measurable? Is it quantifiable? Is target value available? Is it valid for ranking purposes? Does it allow decision with economic implications?

Table 5.1 - Common drivers/damage processes and selected Performance Indicators of bridges

	Observations / Performance Indicator **Damage Process**	Cracks	Crushing	Rupture	Delamination	Scaling	Spalling	Holes	Debonding	Obstruction/impending	Displacement	Deformation	Wire break	Prestressing cable	Reinforcement bar failure/bending	Stirrup rupture	Tensioning force deficiency	Loss of section	Deteriorated mortar joints	Frequency	Vibrations/oscillations
1	Abrasion			O				O				O	O					O	O	O	O
2	Aggradation (alluviation)									O	O	O									
3	Erosion	O		O		O		O			O	O	O		O	O		O	O	O	O
4	Changing geotechnical properties	O	O	O				O			O	O	O	O	O	O	O			O	O
5	Aging of material	O							O		O	O					O	O	O	O	O
6	Alkali aggregate reaction (alkali-silica reaction)	O			O						O	O			O	O	O			O	O
7	Sulphate reaction	O			O	O	O	O			O	O			O	O	O			O	O
8	Chemical attack				O	O						O	O	O	O	O		O	O		
9	Fatigue	O		O				O				O	O	O	O	O			O	O	O
10	Pitt ing corrosion	O				O							O	O	O	O		O		O	
11	Corrosion related toprestressing steel	O	O	O										O				O		O	O
12	Corrosion related to structural steel	O		O		O												O		O	O
13	Corrosion related to reinforcement steel	O		O	O	O	O	O	O						O	O		O		O	O
14	Corrosion related to equipment made of steel	O		O		O												O		O	O
15	Corrosion related to fixings, connectors	O		O		O			O									O		O	O
16	Overloading of an element	O	O	O							O	O	O	O	O	O	O		O		
17	Biological growth	O	O	O				O	O	O	O	O							O	O	
18	Freeze-thaw	O			O	O	O	O	O			O						O	O		
19	High temperature				O						O	O					O		O	O	O

Visual Inspection, O; Visual inspection & testing and monitoring, D

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Table 5.2 - Drivers/damage processes and selected Performance Indicators of tunnels (Part I)

ID associated with Table 5-5	**Performance Indicator** / **Damage Process**	Cracks	Crushing	Rupture	Delamination	Scaling	Spalling	Holes	Debonding	Obstruction/impending	Displacement	Deformation	Wire break	Prestressing cable	Reinforcement bar failure/bending	Stirrup rupture	Tensioning force deficiency	Loss of section	Deteriorated mortar joints	Frequency	Vibrations/oscillations	ULS(U)	Functionality/operational safety(F)
1	Continuous vertical rock movement	O	O	O																		5	7
1	Bending stress	O	O	O																		5	7
2	Local rock movement (punching)	O	O	O							O	O										9	10
3	Higher horizontal actions (underestimation of lateral action)	O	O	O											D			D				9	5
3	Missing reinforcement	O	O	O											D			D				9	5
4	Deformation due to shrinkage, temperature within the shell-blocks	O									O	O										4	4
4	Missing reinforcement	O									O	O										4	4
5	Corrosion of reinforcement	O					O		O						O							6	2
5	debonding	O					O		O						O							6	2
5	Partial spalling of concrete cover	O					O		O						O							6	2
6	Different casting times	O									O	O										4	2
6	Different concrete qualities	O			O						O	O										4	2
6	Delaminations of concrete layers (e.g. spreaded concrete)	O			O						O	O										4	2
7	Overloading (rock movement) of prestressing	O	O	O						O	O											9	10
7	Anchor failure	O	O	O						O	O											9	10
7	Impact due to an accident			O						O	O											9	10
8	Deformation of the ground			O						O	O											6	4
8	Water impact			O						O	O											6	4

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Table 5.3 - Drivers/damage processes and selected Performance Indicators of tunnels (Part II)

ID associated with Table 5-5	**Performance Indicator** / **Damage Process**	Cracks	Crushing	Rupture	Delamination	Scaling	Spalling	Holes	Debonding	Obstruction/impending	Displacement	Deformation	Wire break	Prestressing cable	Reinforcement bar failure/bending	Stirrup rupture	Tensioning force deficiency	Loss of section	Deteriorated mortar joints	Frequency	Vibrations/oscillations	ULS(U)	Functionality/operational safety(F)
8	Soil liquefaction			O						O	O											6	4
9	Thermal reaction	O			O			O	O	O												3	2
9	Lack of concrete curing	O			O			O	O	O												3	2
9	Plastic shrinkage	O			O			O	O	O												3	2
10	Different (partial) shrinkage between layers of concrete	O			O			O	O	O												3	2
11	Higher freeze-thaw cycles (exposure class)	O			O			O	O	O												3	5
11	Insufficient concrete quality	O			O			O	O	O												3	5
11	Freeze-thaw cycles in the first 100 m of a tunnel	O			O			O	O	O												3	5
12	Electrochemical reaction lowers the alkalinity, rebar corrosion														O							3	4
13	Chemical (salt) attack														O							3	5
13	Rebar corrosion (chloride penetration distributed)														O							3	5
14	Lack in the drainage system																					2	4
14	Lack in the waterproofing system																					2	4
15	Lime efflorescence																					1	2
16	Low quality curing		O	O								O										3	3
16	Settlement of the fresh concrete		O	O								O										3	3
17	vibration														O							7	6
17	accident														O							7	6
17	Construction error														O							7	6
17	Lack of anchorage protection														O							7	6

Visual Inspection, O; Visual inspection & testing and monitoring, D

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5.1.2 Symptoms of damage processes

In addition to those observations that may have direct impact on performance, there is the group of observations that are merely symptoms of damage processes, not affecting a current performance (Table 5.4).

Therefore applies, observations can also be symptoms that have no direct influence on the statics or reliability and safety of a structure. On the other side, there are symptoms for interpreting of dynamic time variable relationships. They are e.g. extremely useful for planning purposes and, with a few exceptions, can be used to control performance with regard to Availability and Economy.

Table 5.4 - Observations as Symptoms of a Damage Processes.

Observations/ Symptoms	WG1 Cluster
Staining	Defects
Silting and vegetation	Defects
Efflorescence/crypto-florescence	Defects
Wet spots	Defects
Exposure of element	Defects
Gel exudation	Related to material properties
Hydroxide calcium exudation	Related to material properties
Chloride content	Related to material properties
Chemical parameter	Related to material properties
White colour areas	Related to material properties
Red colour areas	Related to material properties
Cladding damages	Related to equipment & protection
Waterproofing loss	Related to bearing capacity, structural integrity and joints
Accumulated dirt and deposits in joints	Related to bearing capacity, structural integrity and joints
Revealed cable	Related to bearing capacity, structural integrity and joints
Revealed cable anchorage	Related to bearing capacity, structural integrity and joints
Revealed reinforcement	Related to bearing capacity, structural integrity and joints
Sound	Related to dynamic behaviour

5.2 Performance indicators at the component level

Inspections of structures are generally carried out through elements (components). For bridges, there are three main subsystems: substructure, superstructure and roadway. The bridge components associated with these systems, including constitutive materials, are listed in Table 5.5. For tunnel systems, a similar distinction is possible between ridge, callous, abutment and base area, or inner shell, outer shell and sealing level.

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Table 5.5 - Bridge elements for categorization at the component level.

Substructure	Superstructure	Roadway + equipment
Foundations (concrete)	Superstructure (reinforced concrete)	Pavement
Deep foundations, piles (concrete)	Superstructure (prestressed concrete)	Curb & Cornices
Deep foundations, piles (steel)	Superstructure (steel)	Railings & railing anchorage, barriers
Deep foundations, piles (timber)	Superstructure (composite)	Sidewalk (Pedestrian walkway)
Abutments (concrete)	Superstructure (timber)	Bearings
Abutments (masonry)	Superstructure (brick)	Expansion joints
Piers (concrete)	Superstructure (stone)	Drainage
Piers (steel)	Superstructure (concrete)	Lighting
Piers (masonry)	Superstructure (masonry)	Signalization
…	…	…

At the component level, one of the important goals to be reached (or task to be performed) is damage assessment. This implies the detection of damages but also their identification and evaluation within the set thresholds.

Four main approaches in damage detection are visual inspection, non-destructive testing, probing and structural health monitoring, (see WG1 COST Report TU1406 (Strauss, et al., 2016)). In addition to damage detection and characterization, damage identification includes ascertaining the cause of damage and its consequences and damage evaluation comprises the degree or/and extend with respect to the set threshold value. Besides most commonly set up upper limit, an additional threshold in damage assessment may be the duration of damage phase, which will give a clue in which phase of damage progress the element is found: low, moderate or high. The former will request the protection from further progression, the second one will require a routine repair and the last one requests more detailed inspections and testing leading to a routine or special repair (see WG1 COST Report TU1406 (Strauss, et al., 2016)).

The categorisation of damage as a primary performance indicator, requires taking into account related detection methods, performance thresholds and evaluation methods. This categorisation should be done at the level of each bridge component as for example crack detection will be assessed differently depending on where it is found, what is its width, its orientation, and origin (see WG1 COST Report TU1406 (Strauss, et al., 2016)).

For instance, corrosion is a damage process. In addition, it should be established the difference between damage state and damage process, as the former should be evaluated based on extent or degree of damage whereas the latter should be based on the phase of the damage process. Nonetheless, it is evaluated as a damage state based on the extension of damage, such is for example the affected area of a component (in m2) or the percentage of damaged cross section of reinforcement (in %). Upon assessing damages of a particular bridge element, the component functionality level may be evaluated (see WG1 COST Report TU1406 (Strauss, et al., 2016)). Accordingly, the element may be evaluated in best condition when no damage is detected, with unquestionable function when damage is in initial phase, with function not been compromised when damaged is moderate and with questionable function or element is out of function when damage has high degree and/or extend.

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Based on the work of COST TU1406 and the surveys of Work Package 2 of the IM-SAFE project, generic performance indicators for bridges and tunnel systems shown in Table 5.6 can be characterized.

Table 5.6 - Performance Indicators for bridges and tunnel systems

	Structure / Performance - Indicator	Concrete bridges	Steel bridges	Composite bridges	Tunnel systems
1*)	Cracks(	O	O	O	O
2*)	Crushing	O	O	O	O
3	Rupture	O	O	O	
4*)	Delamination	O	O	O	
5	Scaling	O	O	O	
6*)	Spalling	O	O	O	O
7	Holes	O	O	O	O
8	Debonding	O	O	O	O
9*)	Obstruction/impending	O	O	O	O
10*)	Displacement	O	O	O	O
11*)	Deformation	O	O	O	O
12	Wire break	O	O	O	
13	Prestressing cable	O	O	O	
14	Reinforcement bar failure/bending	O	O	O	O
15	Stirrup rupture	O	O	O	
16	Tensioning force deficiency	O	O	O	
17*)	Loss of section	O	O	O	O
18	Deteriorated mortar joints	O	O	O	
19*)	Frequency	O	O	O	
20	Vibrations/oscillations	O	O	O	

*)Key performance indicators, *KPI*

5.3 Performance indicators at the system level

A qualitative scale of values may show how the collapse of a particular element would affect each criteria. Besides technical indicators, at this level sustainability and socio-economic indicators will assume an essential impact to performance requirements. Additionally, indicators related to scientific achievements in, for example, testing and monitoring, dynamic behaviour and reliability of structures, should be included at this level, as well. Structural reliability assessment will require an adequate knowledge level on particular related properties such are for example stiffness changes and local traffic loading which requires investment in additional inspection, testing or monitoring method, advanced modelling techniques and updating data on resistance and loads. Research-based performance indicators, including those that may be put in practice as well as those in whose development is worth investing, have the potential to improve existing structural performance assessment methods and consequently the management of structures.

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5.4 Performance indicators at the network level

At the network level, based on bridge condition assessment gained through standard inspection and evaluation procedures with additional evaluation of bridge importance in the network, the primary goal to be reached is priority repair ranking. Bridge condition assessment is based on four criteria: structural safety and serviceability, durability, traffic safety and general bridge condition. On the other hand, bridge importance in the network is based on five criteria: road category, annual average daily traffic, detour distance, largest span, total length.

Such criteria are reduced to comparable values with the help of preference functions and of an adequate threshold of indifference and preference for each criteria. At this level key performance requirement indicators, as presented in section 5.1 are appropriate for priority repair ranking. Priority repair ranking, at the same time, is an essential indicator for the final goal: optimal management plan of roadway bridges, which is to be evaluated through decision ranking by power and weakness of decisions.

The transfer of performance indicators at the component level via the PI at the system level to the performance indicators on the network level or the key performance requirement indicators should be processes via a quality control plan.

5.5 Performance indicators vs. requirements (tasks) within asset management

At the component level, one of the first tasks is the damage assessment. When assessing the damage to a particular bridge element, the functionality and safety level of the component should be evaluated. In order to assess the impact of the functionality of the damaged element on the whole structure, the importance of the bridge element has to be evaluated according to the following criteria: Load-bearing safety and serviceability, traffic safety and durability.

The weighting of the importance of the element for the assessment at the system level compared to the component level is another essential step to lift the condition assessment from the component level to the system level and to be able to assess the condition of a subsystem and consequently of the whole bridge.

At the network level, the main objective is to prioritise repairs based on the bridge condition assessment, including the assessment of the importance of bridges in the network. At this level, the Key Performance Indicators such as bridge reliability have to be included in the ranking of priority repairs.

The ranking of priority repairs is at the same time essential information for the final goal: an optimal management plan for road bridges. From this overview, it is clear that an interaction of different types of indicators is inevitable, but their categorisation will make it easier to determine methods for their quantification and the degree of their influence on a given structural performance objective.

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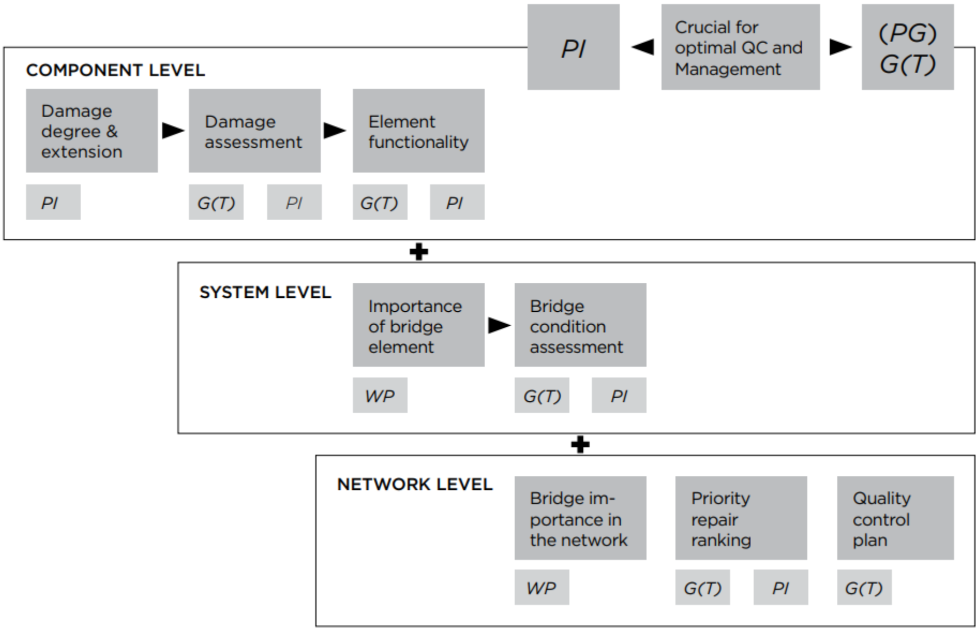

Table 5.7 - Interaction of indicators -PI, goals within bridge management according to WG1 COST Report TU1406 (Strauss, et al., 2016)).

The first goal or task at the component level is to assess the damage that is of relevance for target event that is defined by a limit state e.g. corrosion initiation. Upon damage assessment of a particular element, damage index becomes an indicator for the next goal – evaluation of component functionality level. At the system level, the element functionality as an indicator, together with the importance of a bridge element as weighting parameter are crucial for the following goal – bridge condition assessment. Rising to a network level makes bridge condition assessment an indicator which together with the bridge importance in the network as a weighting parameter will influence the next goal – priority repair ranking.

Finally, priority repair ranking may be considered as an indicator for a Quality Control plan. For more information on this concept reference is made to IM-SAFE deliverable D3.2 (Darò, et al., 2022).

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6 Performance assessment

6.1 Principles of performance verification for existing structures

6.1.1 Performance criteria

As described in 4.2, in the context of performance-based limit state design, (fib MC2010, 2013) identifies the following structural performance criteria for serviceability and structural safety:

- serviceability limit states criteria;
- ultimate limit states criteria;
- robustness criteria.

As a further evolution, in (fib MC2020, 2022) provisions about the needs of both the design of structures and all the activities associated with the through-life management of existing concrete structures are given. The need for a better knowledge and understanding of uncertainties, risk acceptance and risk differentiation for both new and existing structures has led to take advantage of the additional information that can be acquired by inspection, testing and monitoring.

In this regard, (fib MC2020, 2022) adopts a safety philosophy based on reliability concepts and introduces new limit states that can be related to design or assessment situations, which can be persistent, transient or due to extreme and environmental actions, for the through-life management of existing concrete structures including prediction and updating of reliability and durability of deteriorating structures. In particular, Condition Limit States are introduced, where it is possible to distinguish, for instance, condition limit states associated to durability, as well as Partial Damage Limit States. For further information, see (fib MC2020, 2022), (Bigaj-van-Vliet, 2021) and section 8.1.

In the present Chapter, attention will be given to the current performance-based limit state design as per (fib MC2010, 2013).

6.1.1.1 Serviceability limit states

“Serviceability limit states correspond to the states beyond which specified demands for a structure or a structural component related to its normal use or function are no longer met” (fib MC2010, 2013).

The serviceability limit states address fitness-for-use of a structure and can pertain to the following undesirable states (non-exhaustive):

- unacceptable deformations which affect the efficient use or appearance of structural or non- structural elements or the functioning of equipment;
- excessive vibrations which cause discomfort to people or affect non-structural elements or the functioning of equipment;
- local damage affecting the appearance, the efficacy, or functional reliability of the structure;
- local damage (including cracking) which can reduce the durability of the structure or make the structure unsafe for use.

In the cases of permanent local damage or permanent unacceptable deformations, the exceedance of a serviceability limit state is called irreversible and the first time that this occurs causes noncompliance.
In other cases, the exceedance of a serviceability limit state can be reversible and then noncompliance occurs:

- The first time the serviceability limit state is exceeded, if no exceedance is considered as acceptable.

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- If exceedance is acceptable but the time when the structure is in the undesirable state is longer than accepted.
- If exceedance is acceptable but the number of times that the serviceability limit state is exceeded is larger than specified.
- If a combination of the above criteria occurs.

These cases can involve temporary local damage (e.g. temporarily wide cracks or leakage), temporary large deformations, and vibrations. Limit values for the serviceability limit state should be defined on the basis of their consequences.
Accordingly, the serviceability limit states that should be considered can be described as:

- operational limit states;
- immediate use limit states.

The corresponding serviceability limit state criteria are related to:

- functionality of the structure related to its normal use;
- comfort of using the structure.

The limit values that define the serviceability limit state criteria differ, depending on whether it concerns an operational limit state or an immediate use limit state.

6.1.1.2 Ultimate limit states

Ultimate limit states are limit states associated with the various modes of structural collapse or stages close to structural collapse which, for practical purposes, are also considered as ultimate limit states. Ultimate limit states pertain to the following undesirable states (non-exhaustive):

- loss of equilibrium of the structure or part of it considered as a rigid body;
- instantaneous attainment of the maximum capacity of cross sections, members or connections by yielding, rupture or excessive deformations;
- failure of members or connections caused by fracture, fatigue, or other time-dependent accumulation effects;
- instability of the structure or part of it;
- sudden change of the assumed structural system to a new system (e.g. snap through, large crack formation);
- foundation failure.

The exceedance of an ultimate limit state is almost always irreversible and the first time that this occurs causes failure. The ultimate limit state can be the result from a single extreme action event or from a deterioration process over time followed by a (less) extreme action event.

Ultimate limit states can refer to structural elements, as well as to global structural systems.
The ultimate limit states, therefore, address:

- life safety;
- protection of the structure and environment;
- protection of operations.

Accordingly, the ultimate limit states that should be considered can be described as:

- life-safety limit states;
- near-collapse limit states.

The corresponding ultimate limit states criteria are related to:

- resistance of critical regions;
- fatigue;
- stability.

The limit values that define the ultimate limit state criteria vary, depending on whether a life-safety limit or a near-collapse limit applies.

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6.1.1.3 Robustness criteria

Robustness is important for maintaining the ability of the structural system to fulfil its function during events such as accidental loading or due to consequences of human error.
As per the ultimate limit state, robustness of the structural system addresses:

- life safety;
- property and environment protection;
- protection of operations.

Accordingly, the robustness criteria are related to:

- resistance of the structural system;
- special functions (e. g. shelter from climatic phenomena, containment of substances, providing fortification, security, shade etc.).

6.1.2 Performance requirements for existing structures

With reference to the introduction to reliability concepts (see section 4.2.5), the appropriate choice of the target level of reliability should be made taking into account:

- possible consequences of failure in terms of risks for human safety (human life or injury)
- potential economic losses and the degree of societal inconvenience
- amount of expense and effort required to reduce the risk of failure.

Because of large differences in the outcome of such considerations, due attention should be given to differentiating the reliability level of new structures and existing structures (see section \ 8.1).

Reliability requirements for structures to be designed and for existing structures may adequately be expressed in terms of the reliability index β:

$$\beta = -\Phi^{-1}\left(P_f\right)$$

where $\boldsymbol{\Phi^{-1}}$ is the inverse standard normal probability distribution function.

P_f	10^{-1}	10^{-2}	10^{-3}	10^{-4}	10^{-6}
β	1.28	2.32	3.09	3.72	4.75

Table 6.1 - β-values related to the failure probability P_f, according to (CEN-EN 1990, 1990:2006)

Data about the type of structure, its performance over time and degradation processes have to be collected in order to quantify risk, and hence decide on the target reliability values (8.2.3). Moreover, in order to make the right choice for the target β values, the following parameters are to be considered the specific case considered:

- the reference period;
- the consequences of failure;
- the cost of safety measures.

Concerning the reference period, target β values usually refer to the specified design service life for the structures to be designed for serviceability and fatigue. In (CEN-EN 1990, 1990:2006), for instance, the following minimum reliability levels are recommended for a reference period t_R of 50 years based on the relevant consequence of failure.

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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 6.2 - Target β-values related to a reference period of 50 years (examples), according to (CEN-EN 1990, 1990:2006)

It should be noted, however, that service life and target β values are two independent requirements on structural performance: the target reliability is sometimes presented for an equivalent value for different (e. g. 1 year) reference period. In (CEN-EN 1990, 1990:2006) and (JCSS, 2001a) the minimum reliability levels for a reference period t_R of 1 year are given.

Relative costs of safety measures	Consequences of failure			
	small	some	moderate	great
High	2.3	3.0	3.5	4.1
Moderate	2.9	3.5	4.1	4.7
Low	3.5	4.1	4.7	5.1

Table 6.3 - Target β-values related to a reference period of 1 year (examples), according to (CEN-EN 1990, 1990:2006)

1	2	3	4
Relative cost of safety measure	Minor consequences of failure	Moderate consequences of failure	Large consequences of failure
Large (A)	β=3.1 ($p_F \approx 10^{-3}$)	β=3.3 ($p_F \approx 5\ 10^{-4}$)	β=3.7 ($p_F \approx 10^{-4}$)
Normal (B)	β=3.7 ($p_F \approx 10^{-4}$)	β=4.2 ($p_F \approx 10^{-5}$)	β=4.4 ($p_F \approx 5\ 10^{-6}$)
Small (C)	β=4.2 ($p_F \approx 10^{-5}$)	β=4.4 ($p_F \approx 5\ 10^{-6}$)	β=4.7 ($p_F \approx 10^{-6}$)

Table 6.4 - Target β-values related to a reference period of 1 year (examples), according to (JCSS, 2001a)

Concerning the consequences of failure and the cost of safety measures, a differentiation of the reliability level may be done on the basis of well-founded analysis related to formal structural optimisation and risk acceptance criteria. An example is provided in (Kohler, 2021). The maximum acceptable failure probability depends on the type of the limit state and considered consequences of failure for the structure. In table 6-5 target β values for different limit states and reference periods are given.

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Limit states	Target reliability index β	Reference period
Serviceability		
reversible	0.0	Service life
irreversible	1.5	50 years
irreversible	3.0	1 year
Ultimate		
low consequence of failure	3.1	50 years
	4.1	1 year
medium consequence of failure	3.8	50 years
	4.7	1 year
high consequence of failure	4.3	50 years
	5.1	1 year

Table 6.5 - Recommended target reliability indices β for structures to be designed, related do the specified reference periods (CEN-EN 1990, 1990:2006)

The target reliability level for the existing structures may be chosen lower than for new structures, as for existing structures the reduction of β depends on the costs for achieving a higher reliability level, which are usually high compared to structures under design, For more details, see Annex F of (ISO-13822, 2010) and (ISO-2394, 2015). Besides the economic considerations that currently motivate the differentiation of target reliability levels between new and existing structures, also the assumptions on uncertainties play a role in setting those target values. The proper consideration of the effect of the level of knowledge in the target reliability levels (and the corresponding requirements in terms of information to be gathered) is definitely interesting and challenging at the same time. There is an ongoing discussion in the context of fib on the effect of the level of knowledge on the evaluation of target reliability (TG3.1 Group).

In section 8.2.3 the differentiation of target beta values for new and existing structures is described.

The choice of a different target reliability level for existing structures may be taken only on the basis of well-founded analysis of consequences of failure and the cost of safety measures for any specific case. Some suggestions for the reliability index for existing concrete structures are given in Table 6.6 for the specified reference periods and further considerations are given in section 8.1.

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Limit states	Target reliability index β	Reference period
Serviceability	1.5	Residual service life
Ultimate	in the range of 3.1–3.8* in the range of 3.4–4.1* in the range of 4.1–4.7*	50 years 15 years 1 year

* depending on costs of safety measures for upgrading the existing structure

Table 6.6 - Suggested range of reliability indices β for existing structures, related do the specified reference periods (fib MC2010, 2013)

Consequence classes and reliability classes [see par. 2.4] are connected to the levels of reliability differentiation. For further information see Annex B of (CEN-EN 1990, 1990:2006). For instance, the recommended minimum values for β related to reliability classes are given in Table 6.7.

Reliability Class	Minimum values for β	
	1 year reference period	50 years reference period
RC3	5,2	4,3
RC2	4,7	3,8
RC1	4,2	3,3

Table 6.7 - Recommended minimum values for reliability index β (ULS) (CEN-EN 1990, 1990:2006)

The requirements for the reliability of the components of the system will depend on the system characteristics. The target reliability indices given in this Chapter relate to the structural system or in approximation to the dominant failure mode or structural component dominating system failure and are valid for ductile structural components or redundant systems for which a collapse is preceded by some kind of warning, which allows measures to be taken to avoid severe consequences (by explicit requirements or by appropriate detailing it should be assured that brittle failure does not occur). Therefore, structures with multiple, equally important failure modes should be designed for a higher level of reliability per component than recommended.

In (fib MC2010, 2013) the Partial Factor Method for concrete structures is calibrated in such a way that when applying the values of partial factors, the following reliability requirements are satisfied for a defined period of 50 years:

- β = 1.5 for serviceability limit states verification
- β = 3.1 for fatigue verification
- β = 3.8 for ultimate limit states verification.

For other β values (e. g. applied in the assessment of existing structures), the Partial Factor Format can also be applied. However, reconsideration of the partial factors and characteristic values of the fundamental basic variables may be required, following from the consideration of actual uncertainties regarding actions, resistances, geometry, structural modelling and the determination of action effects (see section 8.2.3).

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For steel structures the same considerations on the β values presented in this Chapter are valid.

6.1.3 Uncertainty treatment in performance verification

Decisions concerning structures should account for all uncertainties of relevance for their performances such as:

- **aleatory uncertainty**: inherent natural variability
- **epistemic uncertainty**: lack of knowledge/lack of sufficient samples

Uncertainties are to be represented in the decision process through probabilistic models such as random variables, stochastic processes, and/or random fields (ISO-2394, 2015). The probabilistic modelling addresses the representation of temporal and spatial dependency among the considered uncertainties and events. Moreover, possible non-ergodic phenomena, such as effects of climate changes and demographical developments are to be included in the modelling.

Moreover, the quantification of uncertainties and their probabilistic representation should take into account both subjective information and available evidence, relating, for example, to loads and material properties. For design of structures based on codified load and resistance factor or partial safety factor design, uncertainties are normally taken into consideration through design values and characteristic values together with specified design equations, load cases, and load combination factors.

6.2 Performance verification methods

The detailed assessment of a structure can be carried out using one or more of the following verification method:

- Risk-informed method;
- Reliability-based method;
- Semi-probabilistic method.

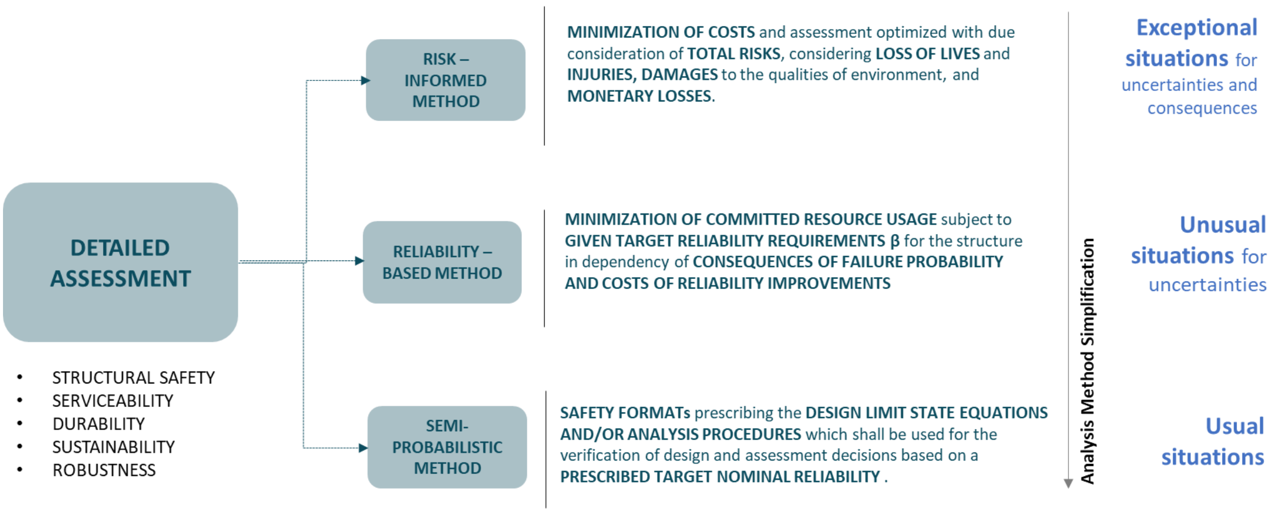

Figure 6.1 - Detailed assessment methods

According to (CEN/TC-250, 2020), the assessment of existing structures should initially be carried out using the semi-probabilistic partial factor method. After the partial factor method has been utilized, the reliability-based method and the risk-informed method can be used for:

- Overcoming the conservatism of partial factor methods;
- Cases of structural failures with serious consequences;

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- Cases of insufficient robustness;
- Evaluating the efficiency of monitoring and maintenance strategies;
- Making fundamental decisions concerning a whole group of structures.

In ISO 2394 (ISO-2394, 2015) it is suggested that design and assessment decisions take basis in information concerning their implied risks. When the consequences of failure and damage are well understood and within normal ranges, reliability-based assessment can be applied instead of full risk assessments.

As shown in Figure 6.2, risk and reliability-based approaches are applied for the calibration of semi-probabilistic approaches, as well as for supporting design and assessment decisions for special structures and projects which are not covered by semi-probabilistic codes. Semi-probabilistic approaches as a further simplification are appropriate when, in addition to consequences, also the failure modes and the uncertainty representation can be categorized and standardized.

	Commonly applied when:	Objective:
Risk-informed decision making: - decisions are taken with due consideration of the decision makers preferences.	Exceptional design situations in regard to uncertainties and consequences.	Maximize the expected utility of the decision maker.
Reliability-based design and assessment: - estimation of the probability of adverse events.	Unusual design situations in regard to uncertainties.	Satisfy reliability requirements.
Semi-probabilistic: - safety format prescribing design criteria in terms of the design equations and the analysis procedures to be used.	Usual design situations in regard to consequences and uncertainties. Default method of most design codes.	Satisfy deterministic design criteria.

Figure 6.2 - Levels of Structural Engineering Decision Making according to (Kohler, 2021)

6.2.1 Risk-informed method

In a risk-informed design and/or assessment, the decisions are optimized with due consideration of total risks, considering loss of lives and injuries, damages to the qualities of environment, and monetary losses. The time horizon to be considered in the assessment of total risks is determined based on the duration of the functionality which the structure has to provide.

The assessment of total risk takes basis in a scenario representation and by probabilistic models of the exposures, the constituent damage, and failure events, as well as the direct and indirect consequences.

The performances of structure related to life safety and qualities of the environment should be assessed with respect to their acceptability. Within these constraints, decisions can be optimized based on a maximization of the expected value of benefits.

Acceptance criteria are considered as constraints to the optimization and should be included in the verification of design and assessment decisions. This principle is illustrated in Figure 4.5: on the x-axis the possible decision alternatives are shown, corresponding to increasing

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reliability, and the benefit is indicated on the y-axis, representative for the expected net benefit associated with the different decisions over the considered period of time. It is shown that the optimal decision may or may not be within the acceptance range.

All anticipated future consequences must be accounted for in the assessment of risks. This includes both the consequences which are associated with the uncertainty and the consequences which are deterministically related to decisions, such as future costs due to planned inspections and maintenance or compensation costs for lives which potentially can be lost during the lifetime of the structure.

In the assessment of the net present value of future costs, the interest rate to be used must be chosen carefully. Considering decisions regarding structures which are made on behalf of society, the default annual discounting rate is the long-term annual economic growth rate and varies by country. The same applies when assessing the net present value of expenditures committed for lifesaving activities.

For structures where failure and damage can imply very serious consequences, a risk-based robustness assessment shall be undertaken as a part of design and/or assessment verification. Annex F of (ISO-2394, 2015) describes the methodology for risk-based robustness assessments. For operational purposes, it might be convenient to introduce a categorization of structures in accordance with their consequences of failure and, on the basis of characterization, decide whether a risk-based robustness assessment is necessary or not. Annex F also contains a suggestion for such a categorization.

A detailed analysis of the risk assessment approaches and risk management methods is presented in Chapter 7.

6.2.2 Reliability-based method

According to (ISO-2394, 2015), a reliability-based approach utilizes an assessment and minimization of costs and/or minimization of committed resource usage subject to given reliability requirements for the structure. The reliability requirements are assessed on the basis of a full risk-informed assessment and thus facilitate reliability differentiation in dependency of consequences of failure and costs of reliability improvements.

Decisions with respect to the design, repair, strengthening, maintenance, operation, and decommissioning of structures have to fulfil given requirements to reliability, or equivalently, requirements to the probability of failure.

A reliability-based method implies that the probability of failure, p_f, does not exceed a specified target value, p_{ft}:

$$p_f \leq p_{ft} \qquad [6\text{-}1]$$

Failure is associated with a limit state and, in the case of a time-invariant reliability problem, the undesired state is defined by the limit state equation presented in 4.2.2.

In the case of time-dependent variables, the problem is time variant and in principle first-excursion or out-crossing approaches should be applied to assess the probability of failure. However, in some cases, the time-variant problem can readily be transformed to time-invariant problems ((ISO-13822, 2010)).
Due to the dependence upon time, p_f shall be referred to a certain *a priori* specified period of time, the *reference period*.

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Such an approach can be applied subject to the availability of:

- uncertainty models,
- reliability methods, and
- expertise in probabilistic analysis.

In many, if not most, cases it is, however, possible to simplify the design process by means of semi- probabilistic design methods.

Figure 6.3 extracted from (CEN-EN 1990, 1990:2006) presents a diagrammatic overview of the various methods available for calibration of partial factor (limit states) design equations and the relation between them.

The probabilistic calibration procedures for partial factors can be subdivided into two main classes:

- full probabilistic methods (Level III), and
- first order reliability methods (FORM) (Level II).

Full probabilistic methods (Level III) give in principle correct answers to the reliability problem as stated. Level III methods are seldom used in the calibration of design codes because of the frequent lack of statistical data.

The level II methods make use of certain well-defined approximations and lead to results which for most structural applications can be considered sufficiently accurate.

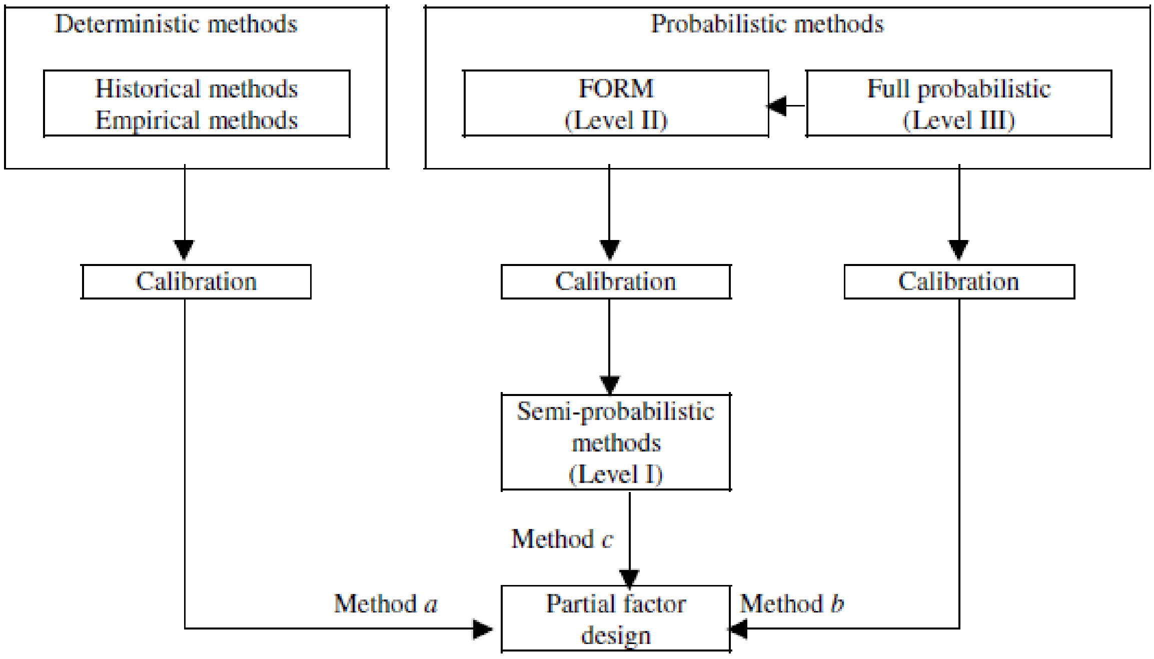

Figure 6.3 - Overview of reliability method (extract from (CEN-EN 1990, 1990:2006))

6.2.2.1 Calculation of the probability of failure

As defined in (ISO-2394, 2015) the calculation of the probability of failure is based on all available knowledge, and the uncertainty representation shall include all relevant causal and stochastic dependencies, as well as temporal and spatial variability. The appropriate choice of method for the calculation of the failure probability depends on the characteristics of the problem at hand and especially on whether the problem can be considered as being time-

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invariant and whether the problem concerns individual failure modes or system, as described below.

6.2.2.1.1 Time-invariant reliability problems

In case the problem does not depend on time (or spatial characteristics), or can be transformed such that it does not (e.g. by use of extreme value considerations), three types of methods can in general be used to compute the failure probability p_f namely the following (JCSS, 2001a):

- First/Second Order Reliability Methods (FORM/SORM).
- Simulation techniques, e.g. crude Monte Carlo simulation, importance sampling, asymptotic sampling, subset simulation, and adaptive sampling.
- Numerical integration, e.g. a family of algorithms for calculating the numerical value of a definite integral or to describe the numerical solution of differential equations.

As with any other analysis, choosing a particular method must be justified through experience and/or verification. 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. Moreover, FORM can handle only linear performance/limit-state functions. As indicated, simulation may be used to verify FORM/SORM results, particularly in situations where multiple design points might be suspected. Simulation results should include the variance of the estimated probability of failure, though good estimates of the variance could increase the computations required. 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.

6.2.2.1.2 Transformation of time-variant into time-invariant problems

Two classes of time-dependent problems are considered:

a) those associated with failures caused by extreme values
b) failures caused by the accumulation and progression of effects over time.

In case a), a single action process can be replaced by a random variable representing the extreme characteristics (minimum or maximum) of the random process over a chosen reference period, typically one year. If there is more than one stochastic process involved, they should be combined, considering the dependencies between the processes.

In case b) (fatigue, corrosion, etc.), the total history of the load up to the point of failure might be of importance. In such cases, the time dependency can be accounted for by subdividing the considered time reference period into intervals and to model and calculate the probability of failure as the probability of failure of the logical series system comprised by the individual time intervals. Examples are provided in (ISO-13822, 2010) and (JCSS, 2001a) annex C.

An exact and general expression for the failure probability of a time varying process on a time interval (0,*t*) can be derived from integration of the conditional failure rate *h*(*τ*) according to:

$$\mathrm{p_f}(0,\mathrm{t}) = 1 - \exp\left[-\int_0^{\mathrm{t}} \mathrm{h}(\tau)\,\mathrm{d}\tau\right] \qquad [6\text{-}2]$$

The conditional failure rate is defined as is the probability that failure occurs in the interval (*τ*, *τ* + *dτ*), given no failure before time *τ*. When the failure threshold is high enough, it can be assumed that the conditional failure rate *h*(*τ*) can be replaced by the average out-crossing intensity *ν* (*τ*):

$$\nu(t) = \lim_{\Delta\to 0} \frac{p\Big(g\big(X(t)\big)\Big)>g(X(t+\Delta)\leq 0)}{\Delta} \qquad [6\text{-}3]$$

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If failure at the start (*t* = 0) explicitly is considered:

$$p_f(0,t) = p_f(0) + \left[1 - exp\left[-\int_0^t v(\tau)\, d\tau\right]\right] \quad [6\text{-}4]$$

in which $p_f(0)$ is the probability of structural failure at t = 0. The mathematical formulation of the out- crossing rate v(τ) depends on the type of loading process, the structural response, and the limit state. For practical application, the formulation might need to be extended to include several processes with different fluctuation scales and/or time invariant random variables.

6.2.2.2 System concepts

Structural design is, at present, primarily concerned with component behaviour (JCSS, 2001a). Each limit state equation is, in most cases, related to a single mode of failure of a single component. However, most structures are an assembly of structural components and even individual components may be susceptible to a number of possible failure modes. In deterministic terms, the former can be tackled through a progressive collapse analysis (particularly appropriate in redundant structures), whereas the latter is usually dealt with by checking a number of limit state equations.

However, the system behaviour of structures is not well quantified in limit state codes and requires considerable innovation and initiative from the engineer. A probabilistic approach provides a better platform from which system behaviour can be explored and utilised.
This can be of benefit in assessment of existing structures where strength reserves due to system effects can alleviate the need for expensive strengthening.

There are two fundamental systems, see Figure 6.4:

- a series system is a system which fails if one or more of its components fail.
- a parallel system is a system which fails when all its components have failed.

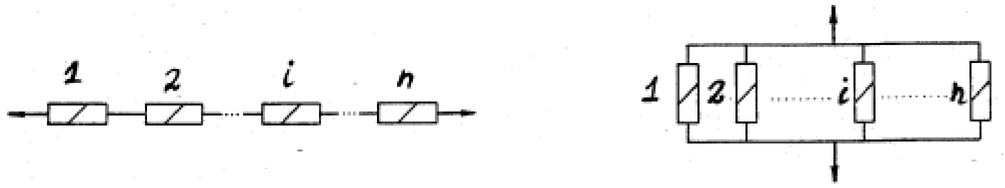

Figure 6.4 - Schematic representation of series and parallel systems (extracted from (JCSS, 2001a))

The probability of system failure is given by:

$$P_{f,sys} = P[E_1 \cup E_2 \cup \cdots \cup E_n] \quad \text{for a series system} \quad [6\text{-}5]$$
$$P_{f,sys} = P[E_1 \cap E_2 \cap \cdots \cap E_n] \quad \text{for a parallel system} \quad [6\text{-}6]$$

where E_i(i=1, …n) is the event corresponding to failure of the i-th component. In the case of parallel systems, which are designed to provide some redundancy, it is important to define the state of the component after failure. In structures, this can be described in terms of a characteristic load-displacement response, for which two convenient idealisations are the 'brittle' and the 'fully ductile' case. Intermediate, often more realistic, cases can also be defined. These can be difficult to evaluate in the case of large systems with stochastically dependent components and, for this reason, upper and lower bounds have been developed, which may be used in practical applications. In order to appreciate the effect of system behaviour on failure probabilities, results for two special systems comprising equally correlated components with the same failure probability for each component are shown in Figure 6.5(a) and (b). Note that in the case of the parallel system, it is assumed that the components are fully ductile.

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More general systems can be constructed by combining the two fundamental types. It is fair to say that system methods are more developed for skeletal rather than continuous structures.

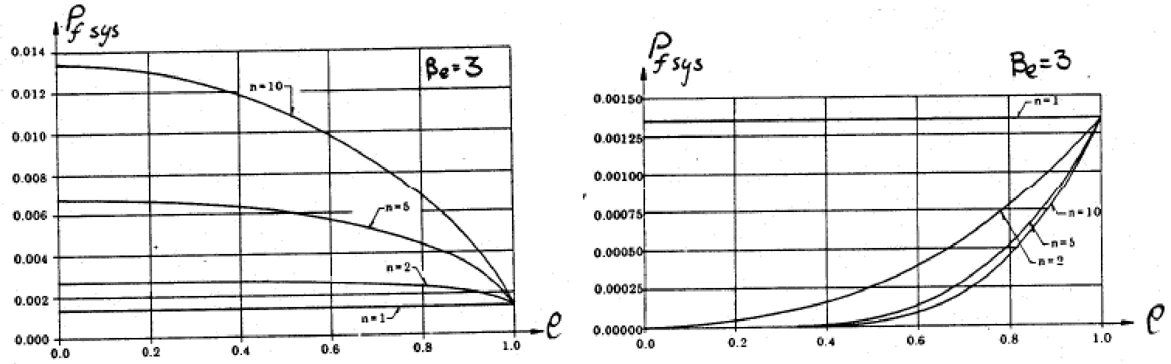

Figure 6.5 - Effect of element correlation and system size on failure probability (JCSS, 2010) : (a) series system (b) parallel system

6.2.2.3 Component reliability analysis

The focus is on the procedure to be followed in assessing the reliability of a critical component with respect to a particular failure mode.
The main steps in a component reliability analysis are the following:

- select appropriate limit state function
- specify appropriate time reference
- identify basic variables and develop appropriate probabilistic models
- compute reliability index and failure probability
- perform sensitivity studies

6.2.2.4 System reliability analysis

6.2.2.4.1 Series systems

As described in section 6.2.2.2, the probability of failure of a series system with m components is defined as

$$P_{fsys} = P\left[\bigcup_{J=1}^{m} E_J\right] \qquad [6\text{-}7]$$

where:

- E_J is the event corresponding to the failure of the jth component. By describing this event in terms of a safety margin M_J

$$P[E_J] = P[M_J \leq 0] \approx \Phi(-\beta_J) \qquad [6\text{-}8]$$

where:

- β_J is its corresponding FORM reliability index, it can be shown that in a first-order approximation

$$P_{fsys} = 1 - \Phi_m[\tilde{\beta}; \tilde{\tilde{\rho}}] \qquad [6\text{-}9]$$

where:

- $\Phi_m[.]$ is the multi-variate standard normal distribution function, β is the (m x 1) vector of component reliability indices and ρ is the (m x m) correlation matrix between safety margins with elements given by

$$\rho_{Jk} = \sum_{i=1}^{n} \alpha_{ij}\alpha_{ik} \quad j,k = 1,2,\ldots,m \qquad [6\text{-}10]$$

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where:

- α_{ij} is the sensitivity factor corresponding to the ith random variable in the jth margin.

Simple first-order linear bounds are given by

$$\max_j[\rho(E_j)] \le P_{fsys} \le \min\left[\left(\sum_{j=1}^{m} P(E_j)\right), 1\right] \quad [6\text{-}11]$$

but these are likely to be rather wide, especially for large m, in which case second-order linear bounds (Ditlevsen bounds) may be needed. These are given by

$$P[E_1] + \sum_{j=2}^{m} max\{[P(E_j) - \textstyle\sum_{k=1}^{j-1} P(E_j \cap E_k)], 0\} \le P_{fsys} \le P[E_1] +$$
$$+\sum_{j=2}^{m}\left\{\left[P(E_j) - \max_{k<j}[P(E_j \cap E_k)]\right], 0\right\} \quad [6\text{-}12]$$

The narrowness of these bounds depends in part on the ordering of the events. The optimal ordering may differ between the lower and the upper bound. In general, these bounds are much narrower than the simple first-order linear bounds. The bisections of events may be calculated using a first-order approximation, which appears below in the presentation of results for parallel systems.

6.2.2.4.2 Parallel systems

Following the same approach and notation as above, the failure probability of a parallel system with m components is given by:

$$P_{fsys} = P[\cap_{j=1}^{m}(E_j)] = P[\cap_{j=1}^{m}(M_j \le 0)] \quad [6\text{-}13]$$

and the corresponding first-order approximation is

$$P_{fsys} = \Phi_m[-\tilde{\beta}; \tilde{\tilde{\rho}}] \quad [6\text{-}14]$$

Simple bounds are given by

$$0 \le P_{fsys} \le \min_j \left[P[E_j]\right] \quad j = 1, \dots, m \quad [6\text{-}15]$$

These are usually too wide for practical applications. An improved upper bound is

$$P_{fsys} \le \min_{j,k}\left[P[E_j \cap E_k]\right] \quad j, k = 1, \dots, m \quad [6\text{-}16]$$

The error involved in the first-order evaluation of the intersections, $P[E_j \cap E_k]$, is, to a large extent, influenced by the non-linearity of the margins at their respective design points.

6.2.3 Semi-probabilistic methods

According to (ISO-2394, 2015), for structures for which the consequences of failure and damage are well understood and the failure modes can be categorized and modelled in a standardized manner, semi-probabilistic codes are appropriate as basis for design and assessment. Standards serve to ensure the quality of analysis, design, materials, production, construction, operation and maintenance, and documentation, and thereby explicitly or implicitly account for the uncertainties which influence the performance of the structures. The

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specifications given in standards should be developed such that they quantify all known uncertainties.

Semi-probabilistic design and assessment codes comprise a safety format prescribing the design equations and/or analysis procedures which shall be used for the verification of design and assessment decisions. The safety format also covers the load combinations which should be considered, as well as the scheme for calculating design values for actions and action effects, material properties, and other parameters associated with uncertainty of relevance for the design.

The design values applied for verification of design and assessment decisions are related explicitly to structural reliability using principles of reliability analysis and calibrated such that the level of reliability achieved for a structure of a certain type and use designed according to the code is close to the prescribed target nominal reliability. For further information see section 4.2.2, section 8.4, and Annex G of (ISO-2394, 2015).

Semi-probabilistic methods usually reduce random variables to a set of design values with a specific use: extreme favourable or unfavourable values, load combination values, serviceability level values, etc. Calibration to higher levels of analysis can take place on the basis of an individual value, as well as for a complete set of design rules.

For semi-probabilistic safety formats, all relevant information concerning possible limitations and assumptions with respect to their validity and application should be specified. This includes specification of:

- legal, temporal and geographical limitations (e.g. project, altitude, and expiry/revision date),types of structures (such as building structures, offshore structures, foundation structures, etc.);
- types of materials (such as concrete, steel, timber, aluminium, composites, and soil);
- types of loads (such as permanent, imposed, wind, snow, traffic, earthquakes, and waves) and relevant load combinations for variable and permanent loads including consideration of stabilizing and destabilizing permanent loads, and
- types of uses (such as hospitals, offices, storage, and energy production/distribution).

Semi-probabilistic safety formats comprise:

- consequence class categorizations,
- design situations,
- design equations, and
- design values.

The principal form of design equations is given as follows:

$$G(X) = R_d(X) - E_d(X) > 0 \qquad [6\text{-}17]$$

where:

- X is a vector of design parameters;
- $R_d(X)$ is the design value for the resistance;
- $E_d(X)$ is the design value for the action effect.

Design equations shall in general be formulated for failure modes involving in principle both failure of individual cross sections of the structures, as well as for failure modes involving the failures of several cross sections of the structures.

The design values for the various actions and materials characteristics entering the design equations should be determined such as to account for the characteristics of the uncertainties

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associated with the loads and resistances which are of relevance for the given design situation (see section 6.1.3).

6.2.3.1 Representative and characteristic values

6.2.3.1.1 Actions

6.2.3.1.1.1 Permanent Actions

A permanent action has often a unique characteristic value. When the action refers to the self-weight of the structure, its value G_k should be obtained from the specified values of geometrical quantities and the mean unit weight of the material. However, in some cases, it might be necessary to define two values, one upper and one lower characteristic value of a permanent action.

6.2.3.1.1.2 Variable Actions

A variable action has often the following representative values Q_{rep}:

- characteristic value Q_k;
- combination value $\psi_0 Q_k$;
- frequent value $\psi_1 Q_k$;
- quasi-permanent value $\psi_2 Q_k$;

The characteristic value for a variable action is chosen so that it can be considered to have a specified probability of being exceeded towards unfavourable values during a chosen reference period.

6.2.3.1.1.3 Combination of Actions

The combination values are chosen so that the probability that the action effect values caused by the combination will be exceeded is approximately the same as when a single action is considered.

- The frequent value is determined so that the total time, within a chosen period of time, during which it is exceeded, is only a small given part of the chosen period of time, and the frequency of its excess is limited to a given small value.
- The quasi-permanent value is determined so that the total time, within a chosen period of time during which it is exceeded, is of the magnitude of half the chosen period.

6.2.3.1.2 Resistances

Properties of materials are defined for some relevant volume of material and are represented by their characteristic values X_k. For a produced material, the characteristic value should in principle be presented as an a priori specified quantile of the statistical distribution of the material property being supplied, produced within the scope of the relevant material standard. For soils and existing structures, the values should be estimated according to the same principle and so that they are representative of the actual volume of soil or the actual part of the existing structure to be considered in the design.

Material properties to be used in nonlinear analyses can either be based on design values, characteristic values, or mean values provided a consistent safety concept is used that results in a design with the required target reliability.

6.2.3.2 Safety formats

Semi-probabilistic safety formats can take basis in different approaches. Commonly, the risk is ensured to be acceptable through adequate choices of design situations, design equations, and representative values.

Other formats can be considered as long as they provide an adequate level of risk and/or level of reliability as the direct use of risk and reliability methods.

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The design values entering into the design equations shall be chosen such as to ensure that an adequate and sufficient level of reliability is achieved for all relevant failure modes of the considered structures.

The partial factor format is generally used as basis for the definition of semi-probabilistic safety formats.

6.2.3.2.1 Partial-factor method

6.2.3.2.1.1 Actions

For a specific load case i, the design values of the effects of actions E_d can be expressed in general terms as:

$$E_{d_i} = \gamma_{E_d} E\left[G_{d_i}; Q_{d_i}; a_d\right] \quad i \geq 1 \qquad [6\text{-}18]$$

where:

- a_d is a vector containing the design values of the geometry;
- γ_{E_d} is a partial factor taking account of uncertainties:
  - in modelling the effects of actions;
  - in some cases, in modelling the actions.

Design values for permanent and variable actions are

$$G_d = \gamma_g G_k \qquad [6\text{-}19]$$

$$Q_d = \gamma_q Q_{rep} \qquad [6\text{-}20]$$

where:

- γ_g, γ_q are the partial factors for permanent and variable actions, respectively.

According to (fib Bulletin 80, 2016), in most cases, the design value in can be approximated as follows:

$$E_{d_i} = E\left[\gamma_{G,i} G_{k_i}; \gamma_{Q,i} Q_{k_i}; a_d\right] \quad i \geq 1 \qquad [6\text{-}21]$$

where

$$\gamma_{G,i} = \gamma_{E_d}\,\gamma_g \qquad [6\text{-}22]$$

$$\gamma_{Q,i} = \gamma_{E_d}\,\gamma_q \qquad [6\text{-}23]$$

For non-linear analysis, this approximation may be unsafe if the action effects are an under-proportional function of the actions. The action effect E_d should be computed by Eq. 6-18 when the action effect increases less than the action (under-proportional structural behaviour).

In the case of over-proportional behaviour, Eq. 6-21 should be used. The two cases are depicted in Figure 6.6.

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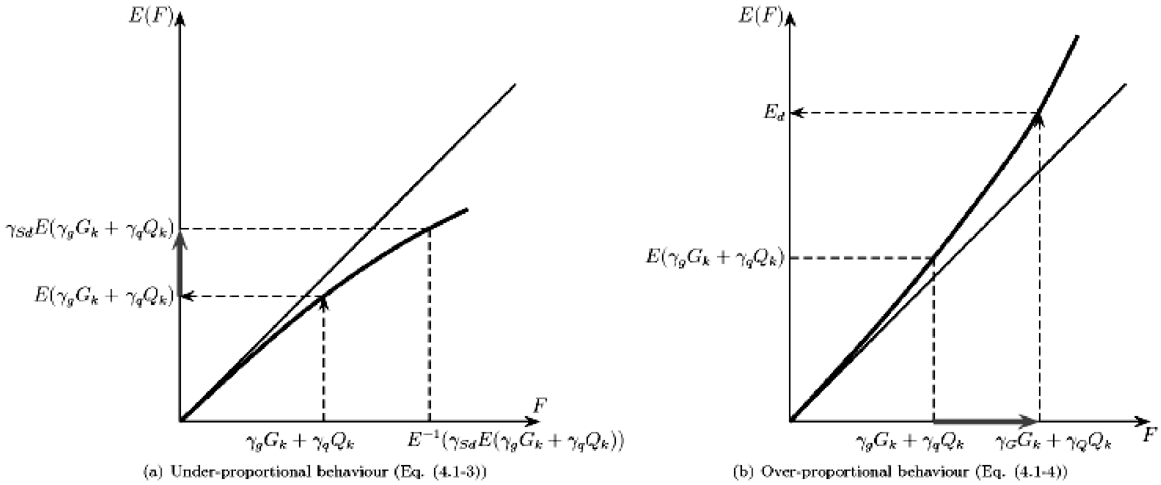

Figure 6.6 - Evaluation of E_d-nonlinear analysis (fib Bulletin 80, 2016)

6.2.3.2.1.2 Resistances

As per (ISO-2394, 2015), the design value of the resistance, R_d, can be determined by different models:

➢ Model 1

Model 1 where a design value of the resistance is determined using design values of the material resistance parameters:

$$R_d = \frac{R(\boldsymbol{X}_d, \boldsymbol{a}_d)}{\gamma_R} \qquad [6\text{-}24]$$

where:

- $\boldsymbol{a}_d$ are design values for the geometry;
- $\boldsymbol{X}_d$ are design values for resistance parameters;
- γ_{R_d} is a partial factor related to the model uncertainty for the resistance model including possible uncertainty related to the transformation from laboratory to real structure and bias in the resistance model.

Design values for material resistance parameters are determined as

$$X_d = \eta \frac{X_k}{\gamma_m} \qquad [6\text{-}25]$$

where:

- η is the conversion factor taking into account load duration effects, moisture, temperature, scale effects, etc.;
- X_k is the characteristic value of the resistance parameter generally defined by the 5 % quantile;
- γ_m is the partial factor for material property.

For geometrical quantities, the design values $_{ad}$ usually corresponds to dimensions specified by the designer.

If more than one resistance parameter is used in the resistance model, then design values are applied for each resistance parameter.

The partial factor γ_m depends on the uncertainty of the resistance parameter(s) and γ_R depends on the uncertainty of the resistance model, including bias:

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$$\gamma_{R_d} = \frac{\gamma_\theta}{b} \qquad [6\text{-}26]$$

where:

- γ_θ is a partial factor depending on the model uncertainty.
- B is the bias in resistance model (can be determined using the method in Annex C - (ISO-2394, 2015).

➢ Model 2

Model 2 where a characteristic value of the resistance is obtained using characteristic values of the material resistance parameters:

$$R_d = \frac{R(\eta X_k, a_k)}{\gamma_M} \qquad [6\text{-}27]$$

where:

- γ_M is a partial factor related to uncertainty of the resistance parameters X through the resistance function $R(\boldsymbol{X}, \boldsymbol{a})$.

The total uncertainty of the resistance depends on the model uncertainty θ and the uncertainty related to the resistance parameters X through the resistance function $R(\boldsymbol{X}, \boldsymbol{a})$. The material partial safety factors are correspondingly obtained from:

$$\gamma_M = \frac{\gamma_\theta \gamma_{R_d}}{b} \qquad [6\text{-}28]$$

➢ Model 3

Model 3 where a design value of the resistance is determined using a characteristic value of the resistance estimated based on tests:

$$R_d = \frac{R_k}{\gamma_M} \qquad [6\text{-}29]$$

where:

- R_k is the characteristic value for the resistance estimated based on tests (is generally defined by the 5 % quantile);
- γ_M is the partial factor related to the uncertainty of the resistance obtained based on tests including statistical uncertainty.

The load partial factors and the material partial factors γ_m, γ_R, and γ_θ should be calibrated such that failure probabilities for the relevant failure modes satisfy the target reliability level.
Whereas the representative values in the design equations are determined in terms of quantile values which are selected according to convention, the partial factors and the load combination values can be determined by means of calibration.

The partial factors and the representative values should take into account both the aleatory and epistemic uncertainties of relevance for the considered design situation and failure modes. Typically, representative values for variables of importance for action effects are selected as upper quantile values whereas representative values of importance for resistances are selected as lower quantile values.

The calibration shall be undertaken by choosing the partial factors and load combination values such that when the semi-probabilistic safety format is applied on a set of structures, the difference between the achieved probability of failure and the maximum acceptable probability of failure is minimized over the entire set of structures. The procedure for the calibration of partial safety factor based codes is outlined in detail in Annex E of (ISO-2394, 2015).

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6.2.3.2.2 Reliability-based framework for the derivation of partial factors

The design values E_d and X_d can be derived considering the reliability index β and the statistical distributions of the actions and material properties. Considering the limit state function described in section 6.2.3, according to the FORM, the design values corresponding to both variables can then be calculated as:

$$R_d = F_R^{-1}[\Phi(-\alpha_R \beta)] \tag{6-30}$$
$$E_d = F_E^{-1}[\Phi(-\alpha_E \beta)] \tag{6-31}$$

With cumulative distribution functions $F_R(r)$ and $F_E(e)$ describing the resistance R and the load effect E, respectively.

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.

The reliability index, sensitivity factors, design values and the partial factors vary based on the description of the reliability problem (type of distribution, statistical characteristics, mechanical model, etc.). α_R and α_E values should be recalculated based on the specific case, since they are associated with the type and importance of the stochastic variable considered.

Nevertheless, for a wide range of civil engineering structures the values of α_R and α_E can be fixed to the following values:

$$\alpha_R = 0{,}8 \tag{6-32}$$
$$\alpha_E = -0{,}7 \tag{6-33}$$

These sensitivity factors can be applied only to dominant variables (in terms of their contribution to the resistance or load effect), assuming that the resistance and load effect are functions of several random variables. (Caspeele, et al., 2013) shows that the sensitivity factors of the nondominant variables are given by Eqs. (6-34) and (6-35), respectively.

$$\alpha_R = 0{,}4\ (0{,}8) = 0{,}32 \tag{6-34}$$
$$\alpha_E = 0{,}4(-0{,}7) = -0{,}28 \tag{6-35}$$

It should be noted, though, that in case of more than two influencing variables this is a conservative approximation.

(Caspeele, et al., 2013) highlights that available measurements may often lead to reduction of uncertainties related to resistance and permanent action effect when assessing existing structures. Then, the sensitivity factors for the resistance and permanent actions generally decrease and the absolute values of the sensitivity factors for the variable actions increase. However, this case-specific effect can only be adequately treated by a full-probabilistic approach (8.2.4).

Based on the definition of partial factors given above, it is possible to derive the reliability-based framework and commonly suggested sensitivity factors, analytical expressions for the partial factors γ_m and γ_f, taking into account their specific distribution type and distributional characteristics.

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6.2.3.2.2.1 Resistance

On the resisting side, the most important variable is the material strength, which can be commonly described by a Gaussian or a lognormal distribution. In this case, suitable expressions for the partial factor γ_m are the following:

$$\gamma_m = \frac{X_k}{X_d} == \frac{\mu_X\,(1-1{,}645\,\delta_X)}{\mu_X\,(1-\alpha_R\,\beta\,\delta_X)} \qquad \textit{Gaussian distribution} \qquad [6\text{-}36]$$

$$\gamma_m = \frac{X_k}{X_d} = \frac{\mu_X\,exp(1-1{,}645\,\delta_X)}{\mu_X\,exp(1-\alpha_R\,\beta\,\delta_X)} \qquad \textit{Lognormal distribution} \qquad [6\text{-}37]$$

where:

- δ_X is the coefficient of variation of the material property under consideration and X_k is assumed to correspond to the 5% fractile of the theoretical distribution of the material property.

6.2.3.2.2.2 Actions

Concerning the load-effect side, permanent and variable actions should be treated separately in most cases. For permanent actions, usually a Gaussian distribution is considered, resulting in the following partial factor in case of an unfavourable effect of the permanent action:

$$\gamma_g = \frac{G_d}{G_k} = \frac{\mu_G\,(1-\alpha_E\,\beta\delta_G)}{\mu_G\,(1+\delta_G)} \qquad [6\text{-}38]$$

where

- δ_G is the coefficient of variation of the permanent action and it is assumed that the mean value is chosen as the representative value.

Similarly, a partial factor γ_f can be derived for variable actions, where considered, taking into account the definition of the characteristic value of the variable action under consideration. Equations for the calculation of the partial factors for variable actions are provided in the following sections.

When model uncertainties are taken into account in an explicit way, the limit state function becomes:

$$g(\boldsymbol{X}) = \theta_R R - \theta_E E \qquad [6\text{-}39]$$

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. If a Gaussian distribution is assumed for both model uncertainties, the partial factors γ_{Rd} and γ_{Ed} can be derived as follows:

$$\gamma_{Rd} = \frac{1}{1-\alpha_R\,0{,}4\,\beta\,\delta_{\theta R}} \qquad [6\text{-}40]$$

$$\gamma_{Ed} = \frac{1}{1-\alpha_E\,0{,}4\,\beta\,\delta_{\theta E}} \qquad [6\text{-}41]$$

where

- $\delta_{\theta R}$ and $\delta_{\theta E}$ are the coefficients of variation of the model uncertainty under consideration and the sensitivity factors correspond to nondominant variables according to Eqs. (6-32) and (6-33).

In order to be able to use the current design format for new structures as well as for the assessment of existing structures, the partial factors for material properties, permanent actions and variable actions (including the associated model uncertainties) should be able to account for:

- a reduced reference period t_{ref} (which should be preferably related to the design working life);

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- an adjusted reliability level, i.e. target reliability index β enabling consideration of the changed economic and human safety considerations in case of existing structures:
- adjusted probabilistic models for basic variables, for example, by modifying their coefficient of variation;
- adjusted model uncertainties.

Examples are given in (IT-Standard-MIT, 2020) and (NEN8700, 2011), (NEN8701, 2011). A more exhaustive analysis of the data-informed performance assessment methods is given in Chapter 8.

6.2.3.2.3 Adjusted partial factor method (APFM)

The basic philosophy of the APFM consists of calculating adjusted partial factors γ_X^{exist} for variables X of an existing structure, considering alternative values for the reference period t_{ref}, the target reliability index β and the coefficient of variation δ_X of the variable under consideration.
For further information see (fib Bulletin 80, 2016) and section 8.2.4.2.

6.2.3.2.4 Design value method

The design value method takes basis in a direct check of the relevant design situations and corresponding design equations using design values for the basic variable which are determined on the basis of reliability assessments. The design values can be determined using simplified methods of direct use of First Order Reliability Methods (FORM) as shown in Annex E of (ISO-2394, 2015). For further information see (fib Bulletin 80, 2016).

6.2.4 Global resistance factor method

Global resistance factor method treats the different sources of uncertainties related to structural behaviour by means of appropriate global safety factors to define the design global resistance of the structure. With this aim, the representative values of the involved resistance variables and of the global safety factors should be selected in order to fulfil the reliability requirements for new and existing structures (i.e., according to a fixed value of the reliability index β).

The representative variable for the global resistance factor method is the structural resistance R. The uncertainty of resistance is expressed by the following values of resistance:

- R_m mean value of resistance;
- R_k characteristic value of resistance (corresponding to a 5% fractile);
- R_d design value of resistance.

The design criterion, in line with the global resistance format, may be expressed in general terms as:

$$E_d \leq R_d \qquad [6\text{-}42]$$

where:

$$R_d = \frac{R_m}{\gamma_R \, \gamma_{R_d}} \qquad [6\text{-}43]$$

E_d is the design value of action;

- γ_R is the global safety factor for mean resistance;
- γ_{R_d} is a model uncertainty factor.

The global safety factor γ_R accounts for random uncertainties of model parameters, namely of material properties. An uncertainty due to model formulation, must be treated by a separate

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safety factor for model uncertainty γ_{R_d}. This can be applied either to the action or to the resistance.

The value of the model uncertainty factor depends on the quality of formulation of the resistance model. (fib MC2010, 2013) recommends the following values:

- $\gamma_{R_d} = 1.0$ for no uncertainties;
- $\gamma_{R_d} = 1.06$ for models with low uncertainties;
- $\gamma_{R_d} = 1.1$ for models with high uncertainties.

The value $\gamma_{R_d} = 1.0$ should be used only in exceptional cases, when evidence of the model validation in the design conditions is available. An example of such a condition is the case of assessment of an existing structure.

The value $\gamma_{R_d} = 1.06$ should be used for models based on a refined numerical analysis, such as non-linear finite element analysis. The model should be objective (low mesh sensitivity) and validated. The factor 1.06 does not cover the errors due to approximations in the numerical model. It covers the other effects not included in the numerical model, such as time effects and environmental effects. An example of such a case is the usual design according to the partial safety factor method.

Nevertheless, recent developments showed that, in case of models with high uncertainties, it should be better to use values of the order of 1.15/1.20.

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7 Risk assessment approaches and risk management methods

7.1 Risk assessment methods

7.1.1 Selection of assessment methods

7.1.2 General

Many of the techniques, described in this section, have been developed in a particular industry with the goal of managing particular types of unwanted outcomes. Their application has over time broadened and developed into similar techniques with different terminologies. Here, the terminology as proposed in (ISO/IEC-31010 , 2019) is used.

In Chapter 9, maintenance and interventions of structures is according to the identified risk level (low, medium, high) in terms of consequence of failure or loss of human life. It is therefore assumed that among the listed methods, the most applicable will be those that pertain to the stage of risk analysis.

7.1.3 Selection criteria

The decision which technique to use and how to apply it should depend on the context of the problem and should provide the stakeholders with desired information. In general, complexity of the method used should increase with the significance of the decision and should take into account other factors, such as time constraints and available financial resources. The choice whether to use a qualitative or quantitative approach depends mostly on the availability and reliability of data: for a quantitative technique, higher quality of data is needed for a meaningful approach. In some cases quantitative techniques can be useful for providing a better understanding of risk, despite an apparent lack of reliable data.

7.1.3.1 Selection of risk assessment methods

According to (ISO/IEC-31010 , 2019):, the choice for a risk assessment technique often depend on the given circumstances: as the degree of uncertainty and complexity of the context, within which the assessment is performed, is increasing, a progressively larger group of stakeholders needs to be consulted. The following needs to be considered when selecting risk assessment techniques:

- Purpose of the assessment;
- Complexity of the situation;
- Available expertise;
- Needs of the stakeholders;
- Potential legal, regulatory or contractual obligations;
- Operating environment;
- Decision importance;
- Any defined decision criteria and their form;
- Available time before decision-making;
- Available or obtainable information for the assessment.

The risk assessment methods are in (ISO/IEC-31010 , 2019) categorized according to their primary application in assessing risk, namely for:

- Eliciting views from stakeholders and experts
- Identifying risk
- Determining sources, causes and drivers of risk;
- Analyzing existing controls;
- Understanding consequences and probability of occurrence;
- Analyzing dependencies and interactions;
- Providing measures of risk;

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- Evaluating the significance of risk;
- Selecting between options;
- Recording and reporting.

Alternatively, the risk assessment methods can be differentiated based on which stage in the risk assessment procedure they are applicable. According to (ISO 31010:2019) the stage of application are:
- Risk identification
- Risk analysis (for evaluating either consequences, probabilities of occurrence or the level of risk)
- Risk evaluation

Subsequently, a table showing the applicability of a number of risk assessment techniques is put forward, where the methods are adjudged to be either strongly applicable (SA), applicable (A) or not applicable (NA). For readability, only the applicable (A) and strongly applicable (SA) cases are shown in Table 7.1. It should be noted that the list is not exhaustive, but includes the most commonly applied methodologies.

Table 7.1 - Applicability of different risk assessment techniques

Risk assessment method	Risk assessment process				
	Risk identification	Risk analysis			Risk evaluation
		Consequence	Probability of occurrence	Risk level estimation	
ALARP, ALARA, SFAIRP	-	-	-	-	SA
Bayesian decision analysis	-	-	SA	-	-
Bayesian network	-	-	SA	-	SA
Bow tie analysis	A	SA	A	A	A
Brainstorming	SA	A	-	-	-
Causal mapping	A	A	-	-	-
Cause-consequence analysis	A	SA	-	-	-
Checklists, classifications, and taxonomies	SA	-	-	-	-
Cindynic approach	SA	-	-	-	-
Consequence/likelihood matrix	-	A	A	SA	A
Cost/benefit analysis	-	SA	-	-	SA
Cross impact analysis	-	-	SA	-	-
Decision tree analysis	-	SA	SA	A	A
Delphi technique	SA	-	-	-	-
Event tree analysis	-	SA	A	A	A
Failure modes and effects analysis (FMEA)	SA	SA	-	-	-
Failure modes and effects and criticality analysis (FMECA)	SA	SA	SA	SA	SA
Fault tree analysis	A	-	SA	A	A
F-N diagrams	A	SA	SA	A	SA
Game theory	A	SA	-	-	SA
Hazard and operability studies (HAZOP)	SA	A	-	-	-

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Hazard analysis and critical control points (HACCP)	SA	SA	-	-	SA
Ishikawa (fishbone)	SA	A	-	-	-
Layer protection analysis (LOPA)	A	SA	A	A	-
Markov analysis	A	A	SA	-	-
Monte Carlo simulation	-	A	A	A	SA
Multi-criteria analysis (MCA)	A	-	-	-	SA
Nominal group method	SA	A	A	-	-
Pareto charts	-	A	A	A	SA
Reliability-centered maintenance	A	A	A	A	SA
Risk indices	-	SA	SA	A	SA
S-curves	-	A	A	SA	SA
Scenario analysis	SA	SA	A	A	A
Structured or semi-structured interviews	SA	-	-	-	-
Structured “what-if?” (SWIFT)	SA	SA	A	A	A
Surveys	SA	-	-	-	-
Value at risk (VaR)	-	A	A	SA	SA

7.1.4 Qualitative risk assessment

7.1.4.1 Brainstorming

7.1.4.1.1 Use

Brainstorming can be applied at any level in an organization in order to identify uncertainties, success or failure modes, causes, consequences, criteria for decisions or options for treatment (ISO/IEC-31010 , 2019). It is also possible to use brainstorming in a quantitative manner, but only in its structured form to ensure that biases are taken into account and addressed. It stimulates and encourages a group of people to develop ideas related to one of more topics of any nature. Brainstorming can either be structured or unstructured; for structured brainstorming, the issue to be discussed is broken down by the facilitator, who also prompts ideas, while less formality is used for unstructured brainstorming. In any case, the goal is to collect as much diverse ideas as possible.

In the context of management of infrastructure objects, brainstorming can be a tool used by the asset manager to facilitate work session where the task is to elicit hazards, which are relevant to be considered in a risk assessment of an object or network of objects.

7.1.4.1.2 Input and output

The input for the brainstorming methods are the opinions, which are elicited from the participants. No other data or external information is usually needed. A skilled facilitator of the brainstorming sessions is needed, and the participants need to have the expertise, experience, and range of viewpoints for the problem at hand.

The output is the list of all the ideas that have been generated during the session and any remarks that have been raise regarding the collected ideas.

7.1.4.1.3 Strengths and weaknesses

Strengths of the brainstorming method are, among others (ISO/IEC-31010 , 2019):

- It is set up easily and quickly
- It encourages creativity and imagination, which helps to identify new risk.

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- It is useful where there is little or no data available, or where a novel approach is needed.
- It aids communication and engagement since it involves key stakeholders

The weaknesses (ISO/IEC-31010 , 2019):

- It is difficult to show that the brainstorming process has been comprehensive in considering all possible ideas.
- It depends on particular group dynamics: people with valuable ideas might not put them forward.

7.1.4.2 Causal mapping

7.1.4.2.1 Use

Causal mapping analyses dependencies and interactions between risks by forming chains of arguments into a graph where events, causes and consequences are depicted. Such maps are usually developed in workshops where participants elicit, structure and analyse the material.

Causal maps for events that have occurred can be developed forensically, which can reveal triggers and consequences and allow causality to be determined. Furthermore, causal maps can be used to proactively comprehensively and systematically capture event scenarios.
Causal mapping can be used by infrastructure asset managers as means of facilitating a workshop where the interactions and dependencies between events, causes and consequences can be made. This can be performed in order to, for example, examine the connection between structural failure of an infrastructure objects and the events that lead to that on the highest (qualitative) level.

7.1.4.2.2 Input and output

The input for performing causal mapping are data, which is used to produce casual maps. This can come from various sources, such as from individual interviews or from documentations such as reports, etc.

The output is information that is relevant to risk management decision along with documentation of the process undertaken to generate this decision.

7.1.4.2.3 Strengths and weaknesses

The strengths of causal mapping are as follows (ISO/IEC-31010 , 2019):

- Visual representation of the risk events and the systematic relationship between these events;
- The open nature of causal mapping workshops allows the participant to express themselves in a free manner, which can reduce the chance of overlooking certain events or relationships.

The weaknesses of causal mapping are, among others (ISO/IEC-31010 , 2019):

- Beside the knowledge of a mapping tool, the process of causal mapping requires skills in managing groups;
- Causal maps results in qualitative results – for situations where quantitative results are expected they can only be used as an input.

7.1.4.3 Checklists, typologies, and taxonomies

7.1.4.3.1 Use

Checklist are used in various ways in the risk assessment process, for example to support the definition of context, for identifying and grouping hazards and consequent risks, to classify controls and treatment of risk, and/or to report and communicate risk. They can be based on past successes and failure, whereas risk typologies and taxonomies are developed more formally to classify risk based on common attributes, where the typologies are conceptually

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derived schemes (the “top-down” approach), while taxonomies are empirically established (the “bottom-up” approach).

Risk taxonomies are intended to be prepared in a way that avoid overlaps and gaps, while risk classifications can focus on isolating a particular category of risk for closer examination. Typologies and taxonomies can be developed hierarchically with several levels of classification. A taxonomy should be hierarchical and should be able to be subdivided to progressively finer level of classification.

Risk can be classified generally by, for example, source of risk, by consequence, aspects or dimensions of objectives or performance, etc.

7.1.4.3.2 Input and output

Input are data for developing checklists, typologies or taxonomies.
The outputs are: a) checklists, prompts or categories and classification schemes; b) an understanding of risk on the basis of the developed lists or groupings of risk.

7.1.4.3.3 Strengths and weaknesses

The strengths of checklists, typologies, taxonomies are, among others, as follows (IEC 31010: 2019):

- A common understanding of risk among stakeholders;
- Once developed they require little specialist expertise.

Their weaknesses, among others, are:

- Limited use in novel situations, for which there is no relevant past information or in situations that differ from those for which checklists, typologies or taxonomies were developed;
- Often generic and might not apply to any particular circumstances;
- Identification of alternative groupings or interconnections of risk can be hindered due to required mutual-exclusivity of checklists, typologies and taxonomies;
- They address what is already known and can therefore discourage exploration of new ideas.

7.1.4.3.4 Examples

Examples of commonly used checklists, typologies or taxonomies at a strategic level include [IEC 31010:2019]:

- SWOT (strengths, weaknesses, opportunities and threats): for identifying factors in the internal and external context to assist in setting objectives and strategies to achieve them while taking risk into account;
- STEEP, STEEPLED, PESTLE: acronyms representing different types of factors to be considered when establishing the context of identifying risks (social, economic, environmental, ethical, political, technological, legal, demographic).
- Within HAZID sessions and PHA hazard checklists are used to identify hazards as a preliminary safety risk assessments.

7.1.4.4 Cindynic approach

7.1.4.4.1 Use

The cindynic approach is used for identifying intangible hazards that might give rise to different consequences. In particular, it identifies: inconsistencies, ambiguities, omissions, ignorance (termed deficits, and differences among stakeholders (termed dissonances) (ISO/IEC-31010 , 2019).

For managing infrastructure, it can be applied at a strategic level in an attempt to understand why unfavorable events happen, even though control measures are implemented.

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7.1.4.4.2 Input and output

The input for the cindynic approach is the information, collected on the system that is the subject of the study, the set of stakeholder networks or groups, as well as on the cindynic (dangerous) situation, defined by a geographical, temporal and chronological space. The information at various times regarding the state of knowledge and the opinion of stakeholders is the collected with semi-structured interviews. The five criteria, in relation to which the information needs to be collected are: 1) goal (primary purpose for performing the survey); 2) values (what is considered important for the stakeholder); 3) rules (for decision-making); 4) data (for substantiating decisions); 5) models (technical, organizational, etc., which use data in decision-making).

The outputs are the tables, showing the deficits of each stakeholder's opinion regarding the five criteria and the discrepancy between them. An example of such tables, is a matrix from (ISO/IEC-31010 , 2019) showing the deficit of each stakeholder (S1-S3) given the criteria of the analysis (Table 7.2) and the discrepancy among the opinion (Table 7.3).

Table 7.2 - The deficit of each stakeholder (S1-S3) given the criteria of the analysis (ISO/IEC-31010 , 2019)

Stakeholder	Criterion for analysis				
	Goals	Values	Rules	Data	Models
S1	-	Focus on a restricted number of values	No reference to procedures	No reference to measurements	No reference to models
S2	Inconsistency between goals and rules	Lack of ranking between values	Lack of ranking between rules	Ignorance of experience and feedback from other countries	Ignorance of specific models
S3	Inconsistency between goals and standards	Focus on a specific values (e.g. employment)	Lack of ranking between rules	No attention paid to specific data (e.g. occupational injuries)	Lack of prioritization in selecting models

Table 7.3 - The discrepancy among the opinion (example) (ISO/IEC-31010 , 2019)

Stakeholder	Stakeholder			
	S1	S2	S3	S4
S1	-	S1 and S2 do not share the same goals	S1 and S3 do not share the same values	S1 and S4 do not share the same measurement systems
S2	-	-	S2 and S3 do not agree on interpretation of procedures	S2 and S4 do not agree on data
S3	-	-	-	S3 and S4 disagree on interpretation of rules
S4	-	-	-	-

7.1.4.4.3 Strengths and weaknesses

The strengths of the cindynic approach are, among others (ISO/IEC-31010 , 2019):

- It is a systemic and multidisciplinary approach, which takes into account human and organizational aspects of risk;
- It produces solutions to reduce risk.

The weaknesses of the approach include (ISO/IEC-31010 , 2019):

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- It does not prioritize risk or its sources;
- A large number of stakeholders can mean a considerable time effort.

7.1.4.5 Delphi technique

7.1.4.5.1 Use

The Delphi method collects and collates judgments on a particular topic based on a set of sequential questionnaires. Through the questionnaires experts are able to express their opinions individually, independently and anonymously while also gaining an insight on the views of other experts after each round of questions. They are afterwards able to reconsider their opinions and put them forward in the next round of questions, until a consensus or quasi-consensus is reached.

The Delphi method is used for complex cases that include uncertainty, which can be dispelled by obtaining expert judgment regarding it. It can be used to identify risks, hazards and opportunities and to gain consensus on the likelihood and consequences.

In the context of management of infrastructure objects, the Delphi technique can be used as an alternative to brainstorming, nominal group technique, or interviews in order to facilitate working sessions to elicit opinions from experts. This can be done for various situations, for example when risks need to be identified, or to gain consensus from experts with regards to probabilities or consequences of future events.

7.1.4.5.2 Input and output

The input for the Delphi method are the questionnaires, which are collected over a variable time scale (days, months, or even years). The questionnaires can be in the written form and filled-out by hand, or can be distributed and collected using electronic communication tools, including emails and the internet. The output is the consensus on the matter under consideration.

7.1.4.5.3 Strengths and weaknesses

The strengths of the Delphi method include (ISO/IEC-31010 , 2019):

- There is less hierarchical bias since all views are anonymous and unpopular opinions can be expressed as well.
- The expressed views have equal weight.
- Flexibility: people do not need to be at the same time at the same place to voice their opinion, they also have time to make a response.

Weaknesses can be, among others, the following [IEC 31010:2019]:

- It is a laborious and time consuming method of eliciting expert views.
- Participants need to be able to express themselves clearly in writing.

7.1.4.6 Failure mode and effect (and criticality) analysis (FME(C)A)

7.1.4.6.1 Use

In FMEA, a system, a process, or a procedure is divided into elements. For each element, the following is then recorded:

- Its function;
- The failure that might occur (failure mode);
- Mechanisms, which could produce the recorded modes of failure;
- The type of consequences if the failure occurs;
- Whether the failure is harmless or damaging;
- How and when the failure can be detected;
- The possible provisions for compensating for the failure.

FMEA can be followed by a criticality analysis, which defines the significance of each failure mode (FMECA). For the criticality analysis, several methods can be used: the most common

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are the consequence/probability matrices (either in qualitative, semi-quantitative, or quantitative description) and the risk index method (RPN).

FMEA or FMECA can be used during design, construction, or operation phase of a structure or network of structure to improve the design, to select between different possible alternatives, or to plan a maintenance program. FMEA can also be used to provide information for other analysis methods, such as for fault tree analysis, and it can be the starting point for a root cause analysis.

7.1.4.6.2 Input and output

Input includes sufficient detail of information about the system and its elements in order that the failure modes and consequences of failure of each element can be determined. This can include technical drawings, details regarding the environment in which the system functions and information regarding any possible past failures.
The output of FMEA/FMECA is:
- a worksheet with failure modes, effects of failure, the causes of failure and existing controls;
- for FMECA, a measure of criticality of each failure mode and the methodology that was used to define it;
- recommended actions, for example for further steps, design changes, etc.

Additionally, FMECA usually provides a qualitative ranking on the importance of failure modes, but it can alternative give a quantitative output if suitable failure rate data and quantitative consequences are used (ISO/IEC-31010 , 2019):.

7.1.4.6.3 Strengths and weaknesses

The strengths of FME(C)A include the following [(ISO/IEC-31010 , 2019):
- General applicability for wide variety of systems, including to both human and technical modes of systems;
- Failure modes, their causes and effects of a system are presented in an easily readable format;
- Provides input for maintenance and monitoring schemes by highlighting critical features that need to be monitored.

Limitations are, among others [IEC 31010:2019]:
- Does not considered combinations of failure modes;
- Unless careful control, FMEA/FMECA studies can be timely and expensive;
- Difficult to apply for complex multi-layered systems.

7.1.4.7 Hazard analysis and critical control points (HACCP)

7.1.4.7.1 Use

The HACCP method provides a structure for identifying sources of risk (hazards) and setting controls at all relevant points in the considered process to protect against them. This methods is most commonly used on the operational level, although its results can also impact the strategic planning of an organization. It is in most countries required for organizations that operate within the food chain in order to control risks of physical, chemical, or biological contamination. Additionally, it has been extended for use in areas where physical, chemical, or biological risks are evident.

HACCAP follows the following protocol (ISO/IEC-31010 , 2019):
- Identification of hazards, factors that influence risk and possible preventive measures;
- Identification of critical control points (CCPs) in the process, i.e. points, where monitoring is possible and the process can be controlled to minimize threats;
- Determination of critical limits for the parameters that are monitored to ensure that risk is controlled;
- Determination of procedures to monitor critical limits for each CCP;

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- Determination of corrective actions that are used when the process is outside of permitted limits;
- Determination of verification procedures;
- Implementation of record-keeping and documentation procedures for each step.

7.1.4.7.2 Input and output

The input for HACCP are:
- A basic flow diagram/process diagram;
- Information regarding hazards that might influence the quality, safety, or reliability of the considered process;
- Information regarding where in the process monitoring and controls of indicators can be implemented.

The outputs include records, including a hazard analysis worksheet and a HACCP.

7.1.4.8 Hazard and operability studies (HAZOP)

7.1.4.8.1 Use

A HAZOP study is a structured and systematic examination of a planned or existing process, procedure, or a system that involves identifying potential decisions, which deviate from the designed, examining their causes and possible consequences.

The procedure for a HAZOP study, which should be executed by a multi-disciplinary and experienced team, is as follows:
- Subdividing the system, process, or procedure into smaller elements;
- Identifying the designed intent for each element along with relevant parameters;
- Applying guidewords to each parameter for each element successively to describe possible deviations from the designed that can have undesirable consequences;
- Determining the cause and consequence in each case suggesting how they might be treated;
- Documenting possible actions regarding identified risks.

Guidewords are used for technical systems to describe parameters such as:
- Physical properties of a material or process;
- Physical parameters such as velocity or temperature;
- Time;

Examples of guide words, taken from (ISO/IEC-31010 , 2019):are shown in Table 7.4:

Table 7.4 - Examples of guide words, taken from (ISO/IEC-31010 , 2019)

Guideword	Definition
No/not	No part of the intended result is achieved or the intended condition is absent
More/higher	Quantitative increase
Less/lower	Quantitative decrease
As well as	Qualitative modification or increase (e.g. addition of a material)
Early	Relative to clock time
Late	Relative to clock time

HAZOP studies for managing infrastructure objects are most often used to improve a design and/or to identify risks that are associated with a design change. It is usually undertaken during the phase of detail design when changes are still practicable. It can also be carried out during operation, however, the required changes can be costly at that stage.

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7.1.4.8.2 Input and output

Input includes information about the system to be reviewed and the intention and performance specification of the design. This can include technical drawings, specifications, as well as operating and maintenance procedures or other documents that describe the functions and elements of the system or procedure.

Output are the minutes of the HAZOP study meetings with deviation of each review point recorded. Records should also include the guideword used, as well as possible causes of deviations, and can include actions to address the identified problems and the person responsible for the action.

7.1.4.8.3 Strengths and weaknesses

Strengths of HAZOP studies are, among others, the following (ISO/IEC-31010 , 2019):
- Systematical examination of a system, process or procedure to determine how it might fail;
- Identifies potential problems at the design stage;
- Applicable to a wide range of system, processes and procedures;
- Allows explicit consideration of the causes and consequences of human error;
- Generates solutions and risk treatment actions;
- Written record, which is an output of HAZOP, can be used to demonstrate due diligence.

Among the weaknesses of HAZOP, the following are most prominent:
- Expensive and time consuming to perform a detailed analysis, for which a high level of documentation is needed;
- Can be hyper-focused on detail issues of design and not on wider or external issues;
- Constrained by the design and design intent and the scope and objectives given to the team.

7.1.4.9 Ishikawa analysis (fishbone diagram)

The Ishikawa analysis identifies possible causes of any desirable or undesirable event, effect, issue or situation. The possible factors for this are organized into broad categories covering human, technical and organizational causes and the information is then depicted in a fishbone diagram. These diagrams are in general used in a qualitative way, but it is also possible to assign probabilities to generic causes and sub-causes on the basis of the degree of belief of their relevance. Quantification is however sometimes invalid since the contributory factors often interact and form a complex contribution to the effects.

The steps for performing the Ishikawa analysis are as follows (ISO/IEC-31010 , 2019):
- The effect (positive or negative) that needs to be analysed is determined and placed in a box as the head of the fishbone diagram;
- An agreement on the main categories of causes is made by the team performing the analysis, for example: materials, methods and processes, environment, equipment, people, etc.;
- The questions “why?” and “how might that occur?” is used iteratively to explore causes and factors influencing each of the category of causes, with each adding to the bones of the diagram;
- The branches are reviewed to verify consistency and completeness and to ensure that the causes actually apply to the effect;
- The most important factors are identified based on the opinion of the team and available evidence.

An example of the fishbone (Ishikawa) diagram is shown in Figure 7.1 (ISO/IEC-31010 , 2019).

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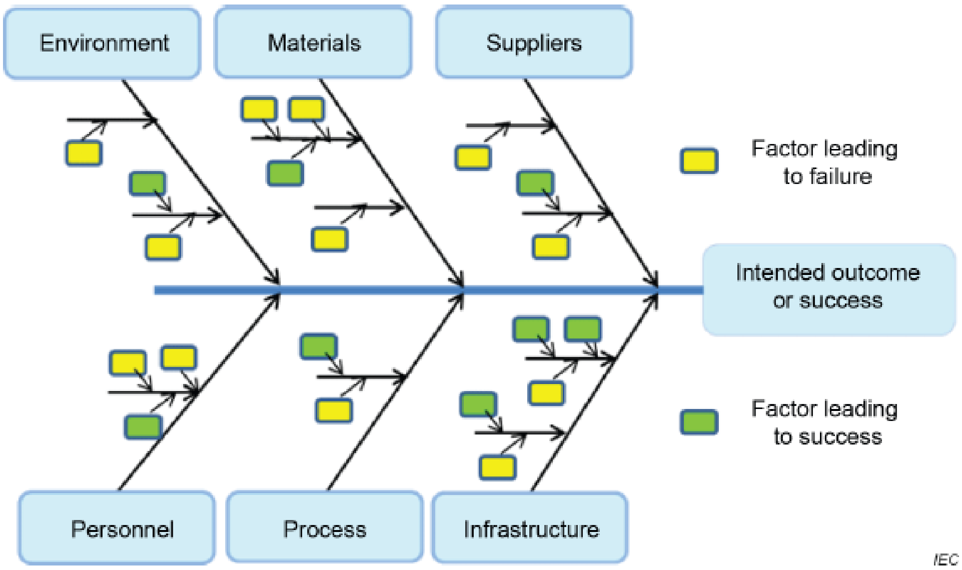

Figure 7.1 - An example of the fishbone (Ishikawa) diagram (ISO/IEC-31010 , 2019)

In infrastructure management domain, Ishikawa analysis can be used to understand the causes of potential events and risk drivers for undesirable events. As such it serves as an alternative to cindynic approach as a method of gaining better understanding of risk.

7.1.4.9.1 Input and output

The input for the Ishikawa analysis is the expertise and experience of participants, as well as their clear understanding of the situation under examination.
The output is the perceive cause of the analysed effect, displayed as a fishbone diagram.

7.1.4.9.2 Strengths and weaknesses

The strengths of the Ishikawa analysis method are, among others, the following [(ISO/IEC-31010 , 2019)::

- It encourages participation of experienced participants and utilizes group knowledge;
- It is applicable to a wide range of situations;
- It provides a graphical output, which is easy to read and understand;
- It identifies causes for both wanted and unwanted effects.

The weaknesses of the method are as follows (ISO/IEC-31010 , 2019):

- It separates causes into major categories during the analysis, which potentially does not consider the interaction between different categories adequately;
- The causes that are not covered by the considered categories are not identified.

7.1.4.10 Nominal group method

7.1.4.10.1 Use

The nominal group method aims to collect ideas and can be used as an alternative to brainstorming sessions, structured interviews, or Delphi technique.
The process of facilitating work according to the nominal group method is (ISO/IEC-31010 , 2019):

- The facilitator provides each group with questions that need to be addressed;
- Participants record their views independently and without interference from others;
- Each member presents their idea without discussion, further clarification can be requested by other participants;
- Views are then discussed within groups to provide a list of agreed-upon views;

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- Groups then vote privately on the ideas and a group decision is made based on the votes.

7.1.4.10.2 Input and output

The input are the ideas, proposed solutions or views, proposed by participants. The output are the ideas, solutions, or decisions that are agreed upon in groups of participants.

7.1.4.10.3 Strengths and weaknesses

The strengths of the nominal group method are, among others, the following (ISO/IEC-31010 , 2019):

- It is a more balanced method than brainstorming, where some members of the group can be more vocal than others.
- It may produce more ideas than brainstorming since it reduces the pressure to conform to a group and have conforming opinions.
- A consensus can be reached in a relatively short time frame.

The weaknesses include the following (ISO/IEC-31010 , 2019):

- The same ideas can be expressed in different ways, making them difficult to compare, or to distinguish between original ideas.

7.1.4.11 Scenario analysis

7.1.4.11.1 Use

Scenario analysis is a colloquial name for a range of methods that involve developing models regarding possible future scenarios through imagination, extrapolation from the present, or modelling. It is applied in the course of management of infrastructure by a group of stakeholders with different interests and expertise (such as the infrastructure management authority, academic experts, civil protection service, etc.), most often to identify risk and explore consequences.

Scenario analysis can be performed either on a short-term or on a long-term scale. Long-term scenario analysis attempts to support planning for major changes in the future, such as in demographics, technology, or physical environment. Short-term scenario analysis can be used to explore consequences of a initiating event, where the scenarios from past events or models are used. If data is not available, expert views can be elicited, however, their opinions need to be substantiated by thorough reasoning. Examples of application of short-term scenario analysis are planning for emergency situations or business interruptions.

7.1.4.11.2 Input and output

For a scenario analysis, data on current changes and trends is needed, along with ideas for future change. For long-term scenario analysis, expertise regarding the used methodology is required.

The output can be a description of the path from the present to the considered scenarios, along with considered effects, both beneficial and detrimental. Alternatively, output can feature an understanding of possible effects of policy or plans for various possible futures, a list of potentially emerging risks if the potential future is realized and, for some applications, a list of indicators of the considered risk.

7.1.4.11.3 Strengths and weaknesses

Strengths of scenario analysis are, among others (ISO/IEC-31010 , 2019):

- It takes into account a range of possible future consequences, which is in contrast to the traditional approach of relying on forecasts that assume future events follow past trends;
- It encourages monitoring of risk indicators.

Weaknesses of scenario analysis include (ISO/IEC-31010 , 2019):

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- The scenarios used might be inadequately based if, for example, data used in the analysis is speculative;
- There is no guarantee that the long-term scenarios actually occur.

7.1.4.12 Structured what-if technique (SWIFT)

7.1.4.12.1 Use

SWIFT is a high-level risk identification method that can be used independently, or as part of methods such as HAZOP or FMEA to make them more efficient. During SWIFT, a structured brainstorming session is used in a workshop where a predetermined set of guidewords (regarding timing, amount, etc.), along with prompts from the participants, which begin with phrases such as “what if?” or “how could?”. Using these prompts, the facilitator of the discussion asks the participants to discuss issues such as:
- Known risks and their existing controls;
- Risk sources;
- Previous experiences, successes and incidents;
- Regulatory requirements and constraints.

In contrast to HAZOP, which focuses on the designer’s intent, SWIFT can be applied to networks, objects, components, procedures, and organizations in general. Specifically, it used to examine the consequences of changes and the associated changes in risk.

7.1.4.12.2 Input and output

An important input for SWIFT is a clear understanding of the system, procedure, and the external/internal contexts in question through interviews, study of documents, plans, and/or drawings by the facilitator.
Output includes a register of risk with actions or tasks that can be used as a basis for a plan for risk treatment.

7.1.4.12.3 Strengths and weaknesses

The strengths of the SWIFT are, among others, as follows (ISO/IEC-31010 , 2019):
- It requires minimal preparation of the team and is relatively quickly performed;
- It can be used to identify options for improvement of processes and systems;
- Reinforces responsibility of those who are accountable for existing controls and for further risk treatment actions through their participation in the SWIFT workshop.

Among other weaknesses, the following are the most obvious (ISO/IEC-31010 , 2019)::
- Risks or hazards might be overlooked if the workshop participants performing the SWIFT analysis are not experienced enough;
- The analysis is prepared on a high-level, which might not reveal complex, detailed or correlated causes, while the recommendations are often generic.

7.1.4.13 Structured or semi-structured interviews

7.1.4.13.1 Use

Structured or semi-structured interviews serve as an alternative to brainstorming sessions, Delphi technique, or nominal group technique as a method of eliciting expert views. These can be with regard to risk sources (hazards), risk, probabilities, or consequences.
In a structured interview, participants are individually asked a set of prepared questions. In a semi-structured interview, more freedom is allowed in order to explore issues that might arise during the interview. The interviews are means of obtaining in-depth and unbiased information and opinions from individuals in a group. If necessary, they can be confidential. Structured and semi-structured interviews are appropriate if it is not possible to gather participants at the same location, or if free-flowing discussion in a group is not appropriate for the situation.
The interview questions should cover a single issue, they should be as simple and open-ended as possible and they should be in a language that is appropriate for the interviewee. Follow-up questions to seek further clarification can be prepared. The questions should not lead the

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interviewee and should be tested beforehand on people with similar background to the interviewees in order to check that they are not ambiguous and cover the intended issue.

7.1.4.13.2 Input and output

The input for the structured/semi-structured interviews is the prepared set of questions, tested on a pilot group of participants.
The output is the required information.

7.1.4.13.3 Strengths and weaknesses

The strengths of structured/semi-structured interviews are, among others, the following (ISO/IEC-31010 , 2019):

- Participants are given time for a considered response.
- Communication is conducted on a one-to-one basis, which allows a more in-depth consideration of issues than a group approach.

The weaknesses include the following (ISO/IEC-31010 , 2019)::

- Designing, performing and analysing interviews is time consuming and demands expertise.
- Bias from participants is tolerated and is not moderated or removed in the discussion.
- Structured and semi-structured interviews tend to produce a considerable amount of information from the interviewee, which can be difficult to group into a unambiguous form for further analysis.

7.1.4.14 Surveys

7.1.4.14.1 Use

Surveys are similar to interviews, except that they engage more people and include more restricted questions, usually in the form of a paper or computer questionnaire. They often contain yes/no questions or questions that require answers on a rating scale. They can be used in situations where the input of a large amount of stakeholders is needed.

7.1.4.14.2 Input and output

Input consists of unambiguous survey questions, tested on a broadly representative sample of people willing to participate. Often, many questionnaires need to be sent out, since the return rates can be low, and a certain amount of responses is needed for statistical validity.
The output is the analysis of the obtained answers, usually in graphical form.

7.1.4.14.3 Strengths and weaknesses

The strengths of the surveys are as follows (ISO/IEC-31010 , 2019)::

- Possibility of interviewing a large group of stakeholders;
- The cost of performing surveys is relatively low, especially if online software is used;
- Statistically valid information can be obtained and the results can be presented in an understandable manner, usually graphical;

Limitations of surveys are, among others, the following (ISO/IEC-31010 , 2019)::

- They are restricted by the need for the questions to be simple and ambiguous;
- Interpretation of results can be difficult and a sufficient number of responses are needed for a statistically-valid analysis;
- It can be difficult to design questions that are not leading;
- It can be difficult to obtain an unbiased response rate.

7.1.5 Semi-quantitative risk assessment

7.1.5.1 Event tree analysis (ETA)

7.1.5.1.1 Use

Event tree analysis provides a graphical representation of possible mutually exclusive sequences of events that can follow an initiating event and result in certain consequences. If

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used quantitatively, it provides probabilities of consequences. For an initiating event lines are drawn for each control to show its success or failure. Conditional probability of success or failure can be assigned to the control, obtained either from expert judgment, statistical data or as an output from individual fault tree analyses.

Event tree analysis can be used to investigate how various control influence the outcomes and to analyse potential scenarios and events following an initiating event. An example of an event tree analysis is shown in Figure 7.2 below (ISO/IEC-31010 , 2019):

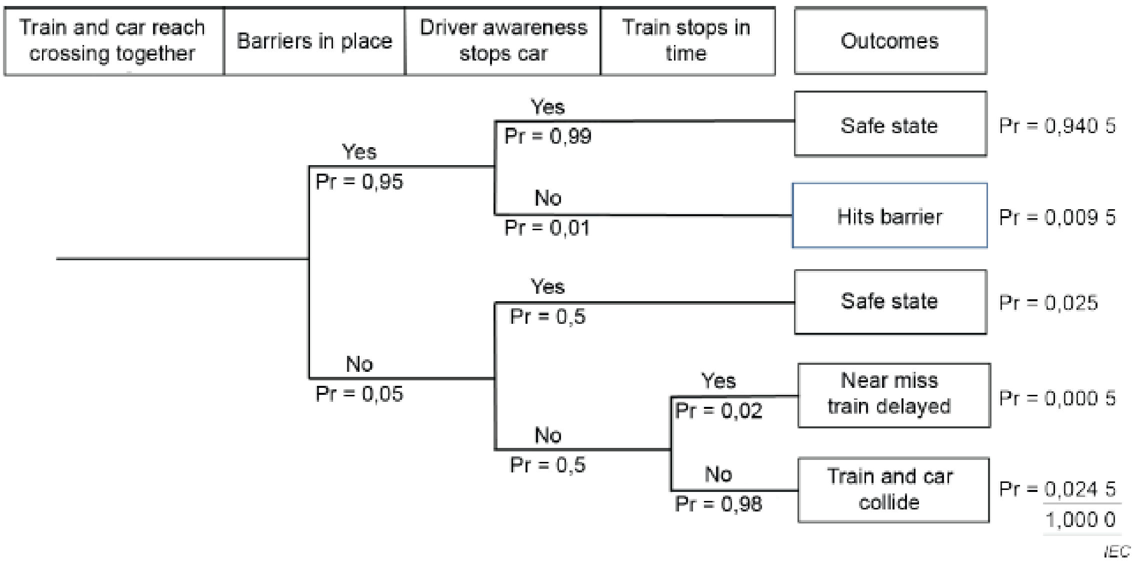

Figure 7.2 - An example of an event tree analysis (ISO/IEC-31010 , 2019)

7.1.5.1.2 Input and output

The inputs for an event tree analysis are:

- An initiating event;
- Information regarding controls, as well as probabilities if a quantitative analysis is performed;

The outputs are:

- Qualitative description of possible outcomes;
- Quantitative estimation of event frequencies or probabilities of occurrence of failure sequences and contributing events.

7.1.5.1.3 Strengths and weaknesses

The strengths of event tree analysis are, among others (ISO/IEC-31010 , 2019):

- Potential outcome scenarios, as well as success or failure of controls of an initiating event are provided in a clear, graphical description;
- Controls and outcomes can be extended to be described in quantitative manner by providing probabilities of their occurrence;

Weaknesses include (ISO/IEC-31010 , 2019):

- Every relevant initiating event needs to be identified to provide a comprehensive analysis;
- Only successes and failure states of a system are considered while partial controls, delayed success or recovery events cannot be taken into account;
- Event tree can be laborious to construct for complex systems

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7.1.5.2 Fault tree analysis (FTA)

7.1.5.2.1 Use

Fault tree is a technique for system representation, which uses Boolean logic to describe combination of faults. Fault trees start at the possible system failure modes, known as top or undesirable events, branching out to (fault) events that may contribute to a failure event. Events, for failure data which cannot be further dissected into more elementary events, are called basic events. If used in a quantitative analysis, a strict logic must be followed based on AND and OR logic gates. Events at the output of AND gates occurs if the input events occur simultaneously, while for OR gates any of the input events occurring causes the output event. An example of a fault tree is shown in Figure 7.3.

Fault tree analysis is mainly used at an operational level, qualitatively to identify potential fault events and pathways that may lead to top events, and quantitatively to calculate the probability of the top event.

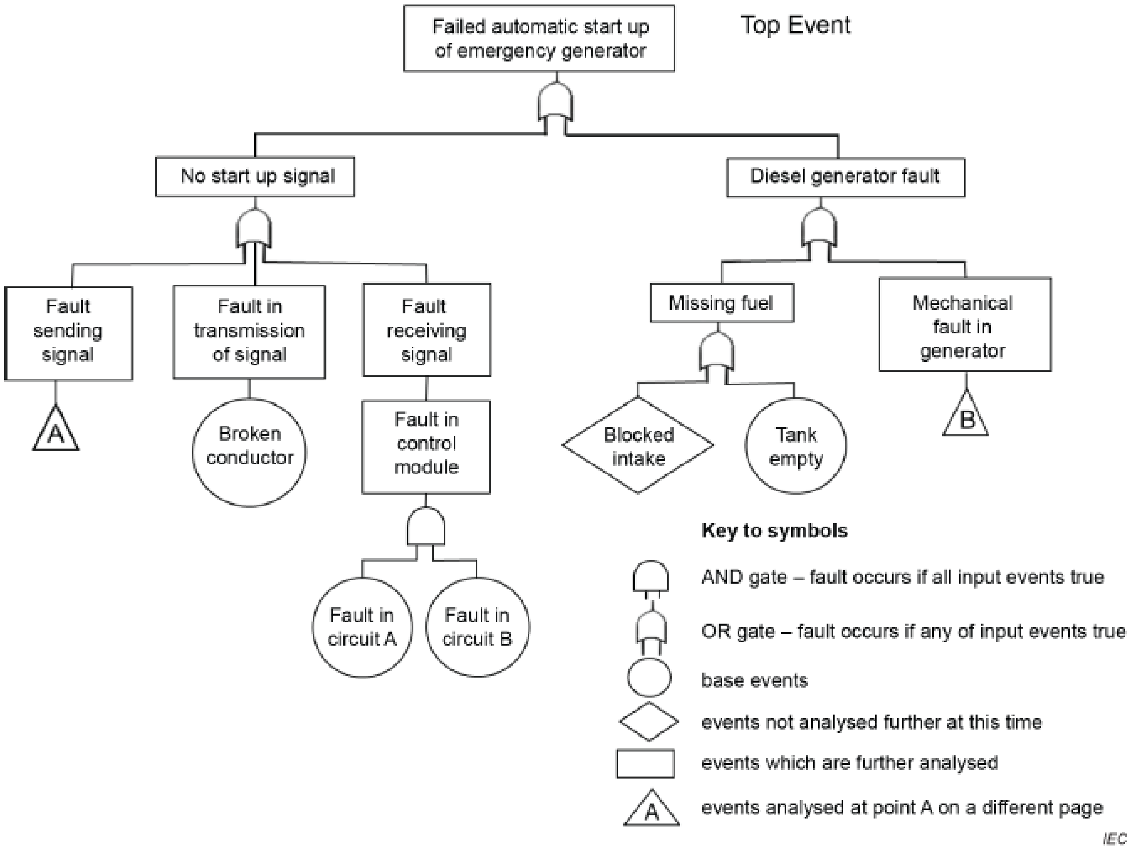

Figure 7.3 - An example of a fault tree (ISO/IEC-31010 , 2019)

7.1.5.2.2 Input and output

The input for the fault tree analysis is:

- Technical understanding of the analyzed system, understanding of ways the system might fail and event that might lead to failure;
- If used quantitatively, data on probability of failure for individual fault event is needed;
- In case complex systems are analyzed, software might be used, which requires clear understanding of the inputs for the computational procedure.

The output of the fault tree analysis is:

- A graphical representation of interacting pathways, involving two or more base events, which might lead to the top event;

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- A list of minimal cuts sets (i.e. combination of events in which the top event definitely occurs) with probability of occurrence for each cut set (if available);
- In a quantitative analysis, probability of the top event and the relative importance of base events is needed.

7.1.5.2.3 Strengths and weaknesses

The strengths of the fault tree analysis, include the following (ISO/IEC-31010 , 2019):
- Systematic approach which allows the analysis of various factors (including human intervention);
- Its graphical output enables the user to better understand the behaviour of the system;
- A fault tree analysis can be performed for simple and complex problems.

The weaknesses, among others, are (ISO/IEC-31010 , 2019).
- It can be difficult to know whether all relevant pathways to the top event have been taken into account;
- It considers only one top event, secondary or incidental failures are not considered;
- For large systems, the fault tree analysis can be very time consuming.

7.1.5.3 Layers of protection analysis (LOPA)

7.1.5.3.1 Use

LOPA is a semi-quantitative method for analysing reduction in risk, which is achieved through controls. It may be considered as a special case of the event tree analysis, and can be used as a follow-up to the HAZOP study (IEC 31010:2019). It can be used qualitatively to review layers of protection between a cause and consequence or quantitatively for analysing the risk reduction, which is produced by each layer of protection.

For the LOPA method, independent protection layers (IPLs) are identified based on the selection of a cause-consequence pair from a list of identified risks. IPLs can be device, systems, or actions, which prevent a scenario from proceeding to its undesired consequence. Among other, IPLS can be design features, physical protection devices, emergency shutdown systems and critical alarms, etc. By then estimating the probability of failure of each IPL and calculating the order of magnitude, it is determined whether the overall protection is adequate to reduce risk to a tolerable level.

7.1.5.3.2 Input and output

The inputs for LOPA are:
- Information regarding sources, causes and consequences of events;
- Information regarding current or proposed controls;
- Frequency of cause event, the probabilities of failure of protection layers, measures of consequence and a definition of tolerable risk.

The output is recommendation for any further treatment of risk and the estimates of residual risk.

7.1.5.3.3 Strengths and weaknesses

The strengths of LOPA are, among others, as follows (ISO/IEC-31010 , 2019).
- It requires less time and resources than event tree analysis or a fully quantitative risk assessment;
- It allows the identification of the most critical layers of protection and it focuses and the most serious consequences;
- It identifies operations, systems and processes for which there are insufficient safeguards;

Its limitations are, among others (ISO/IEC-31010 , 2019).
- It focuses on one cause-consequence pair and scenario at a time while not covering complex interactions between risks or controls. It also does not cover complex

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scenarios where there are interactions between risks and when there is a variety of consequences affecting different stakeholders;
- It might not account for common mode failures when used quantitively.

7.1.5.4 Pareto charts

7.1.5.4.1 Use

Pareto charts are based on the Pareto principle (also known as 80/20 rule), which states that 80% of problems are cause by 20% of causes or that by performing 20% of the work 80% of the benefit can be achieved (ISO/IEC-31010 , 2019). In the context of management of infrastructure objects, Pareto charts can be used on an operational level when prioritization of tasks is needed, e.g. to determine which risk causes should be addressed first.

A typical Pareto chart is a bar chart where the horizontal axis presents categories of interest (such as defects, sources of defects, etc.), and the vertical axis presents frequency. A line of cumulative percentage can be plotted, and the categories where the line intersects the 80% line need to be addressed. An example of a Pareto chart is shown below in Figure 7.4:

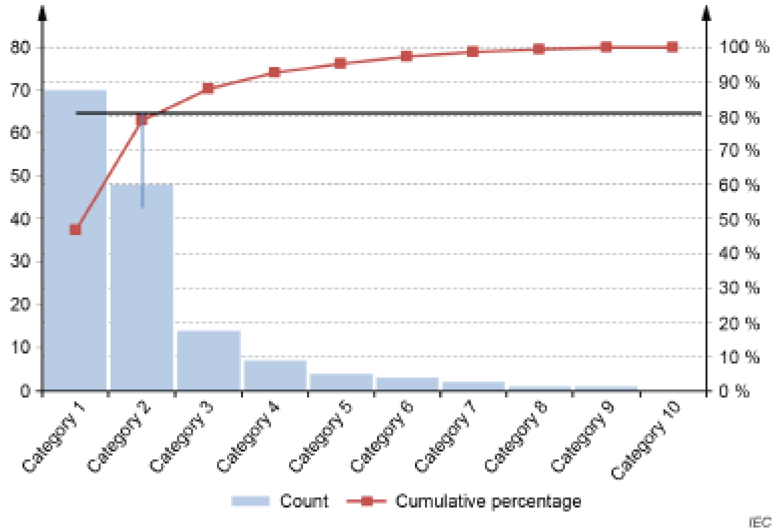

Figure 7.4 - An example of a Pareto chart (ISO/IEC-31010 , 2019)

7.1.5.4.2 Input and output

The input for a Pareto analysis are data that needs to be analysed. This can be data related to past successes or failures along with associated causes.
The output is a Pareto chart, which presents the most significant categories where improvements need to be made.

7.1.5.4.3 Strengths and weaknesses

The strengths of the Pareto analysis, among others, are as follows [IEC 31010:2019]:
- The method requires relatively low effort in order to be performed;
- It presents in a clear, graphical manner which categories need to prioritized;

The weaknesses of Pareto charts, among others are [IEC 31010:2019]:
- Input data is required, otherwise Pareto charts cannot be constructed;
- Data needs to be available for the defined categories in order for the 80/20 rule to be applicable;
- The costs of dealing with underlying causes when addressing problematic categories is not taken into account.

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7.1.5.5 Risk indices

7.1.5.5.1 Use

Risk indices are a qualitative/semi-quantitative technique used for comparing internal or external risks. In the scope of management of infrastructure objects they can be used for particular types of risk to compare different possible situations where that risk might occur. An example of this is a fire hazard ratings system for tunnel.
For risk indices, measure of risk is done with a scoring approach and ordinal scales. The scores for risks are determined using factors, which are perceived to influence the magnitude of risk and the different risks are combined an equation, which attempts to describe the relationship between them.

7.1.5.5.2 Input and output

The inputs for setting up risk indices come from the analysis of the system in question, which requires the understanding of the potential sources of risk and how consequences might develop. Risk indices can be developed based on historical data, or on the basis of results delivered by methods such as fault tree analysis or event tree analysis [IEC 31010:2019].

The output is a series of numbers (indices) for a particular risk, which can be compared to indices developed for other risk relevant for the same system.

7.1.5.5.3 Strengths and weaknesses

The most obvious strength of the risk indices is that [IEC 31010:2019] they provide a tool for simple comparison of different risks while allowing several factors to be included in the formulation of the indices.

The weaknesses of risk indices include the following [IEC 31010:2019]:
- Risk indices result in numerical output, but the method is qualitative, which tends to be overlooked and erroneously used in subsequent analysis;
- If the risk indices model is not validated, the result will not be useful, and it is often hard to provide enough data for validation.

7.1.5.6 S-curves

7.1.5.6.1 Use

S-curve is an alternative name for the cumulative distribution function (CDF). If a risk might result in a range of uncertain consequence values, they can be represented with a probability distribution function (PDF) and consequently with a CDF. These functions can be parametric or non-parametric, they can be set up on theory, or established based on data. S-curves provide the user with the probability that a consequence will not exceed a certain value.
When planning infrastructure management, S-curves can be useful when certain consequence values that represent an acceptable level of risk need to be determined.

7.1.5.6.2 Input and output

The input for S-curves are data, either based on expert judgement or empirical evidence, with which distribution models can be established.
The output are S-curves, which can be used to determine the acceptable level of risk. The decision-makers can also use statistics of the obtained S-curve for determining the acceptable risk.

7.1.5.6.3 Strengths and weaknesses

The strengths of the S-curves are the following [IEC 31010:2019]:
The consequences related to a risk are formulated in a probabilistic distribution, based on which one can make judgments regarding the minimum, maximum, and the most likely value of consequences;

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The weaknesses of the S-curves are the following [IEC 31010:2019]:

- Depending on the level of accuracy of the input data, the establish S-curve can give a false sense of certainty;
- Several assumptions have to be taken into account when establishing an S-curve and obtaining point value or values that represent the distribution of consequences, especially with regards to the form of the probabilistic distribution.
- Probabilistic distribution that are constructed based on past data do not offer valuable information into the probability of event that have a very low chance of occurrence but extreme consequences.

7.1.5.7 Bow tie analysis

7.1.5.7.1 Use

Bow tie is a graphical way of describing the pathways from the causes of an event (sources of risk) to its consequences. The bow tie shows information about risks in situations where an event has a range of possible causes and consequences. As such, it presents controls that modify the probability of the event occurrence and those controls that modify the consequence if the event occurs. A bow tie analysis can be seen as a simplified representation of a fault/success tree and an event tree that can be used as the basis of a means to record information about a risk that does not fit the simple linear representation of a risk register.

The bow tie is drawn according to the following procedure (ISO/IEC-31010 , 2019):

- The considered event is presented by the central point in the bow tie;
- On the left-hand side, the sources of risk (hazards) are listed and joined to the central point (knot) in the bow tie, representing the different mechanisms by which the sources of risk can lead to the event;
- Barriers or controls for each mechanism are shown as vertical bars across the lines depicting the pathways of mechanisms;
- On the right-hand side, lines are drawn from the knot in the bow tie to each potential consequence. Vertical bars on these lines represent control or barriers that modify consequences;
- Factors that might cause the controls to fail (escalation factors) are added, along with the controls for the escalation factors;
- Under the bow tie, the management decision that support controls (such as inspections) can be shown under the bow tie and linked to the respective control.

An example of a bow tie is shown in Figure 7.5 (ISO/IEC-31010 , 2019):

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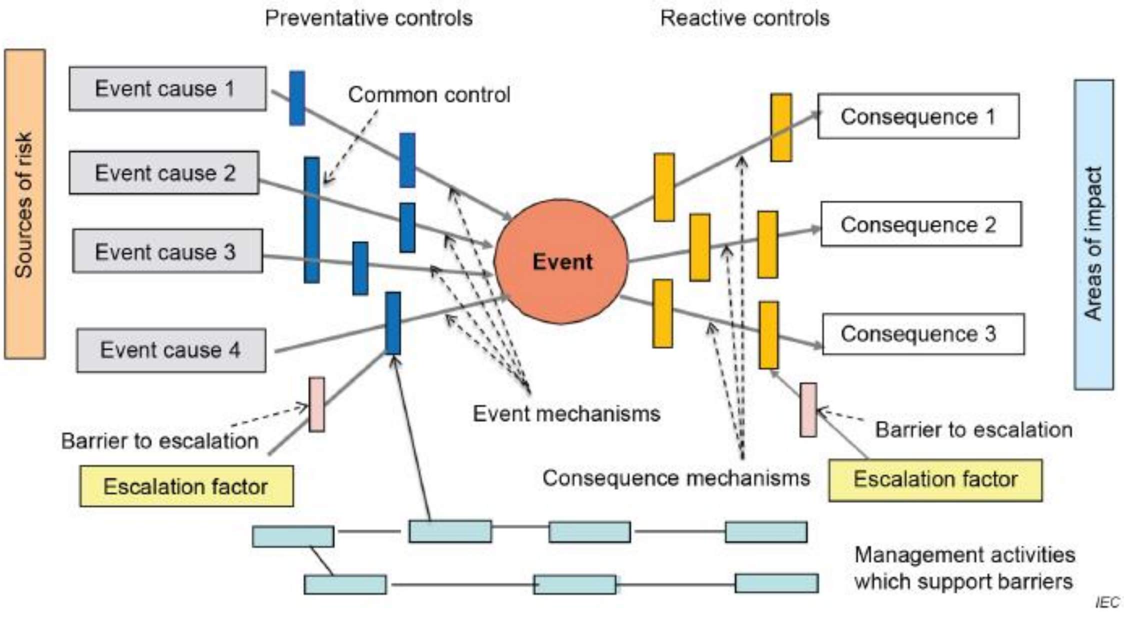

Figure 7.5 - An example of a bow tie (IEC 31010:2019).

7.1.5.7.2 Input and output

The input for the bow tie analysis is the information regarding the considered event, its causes and consequences, and controls that might modify it. This can be collected from the output of the methods for identifying risks and controls (such as FMEA or HAZOP studies) or based on expert judgment of individuals.

The output is the bow tie diagram showing the pathways from risk sources (hazards) to the event and possible consequences, as well as controls that are in place and the factors that might lead to failure of the controls.

7.1.5.7.3 Strengths and weaknesses

Among the strengths of the bow tie analysis, the following are prominent:

- It is a methodology that is simple to understand and gives a clear graphical representation of the event and its causes and consequences;
- It focuses on the controls that are in place and on their effectiveness;
- Both desirable and undesirable consequences can be considered;
- The bow tie analysis does not require high level of expertise to perform.

The weakness of the bow tie analysis is the following:

- Complex situation may be over-simplified: situations where the pathways from event causes to the event are not independent cannot be represented.

7.1.5.8 Consequence/likelihood matrix (risk matrix)

7.1.5.8.1 Use

A consequence/likelihood matrix is a qualitative method of displaying and rating risks according to their consequences and likelihood (probability). It is used for evaluating and communicating relative levels of risk on the basis of a combination of a certain level of consequences and likelihood.

The risk matrix is drawn with consequence on one axis and likelihood (probability) on the other axis, which correspond to defined consequence and probability scales. The scales can be qualitative, semi-quantitative or quantitative and can have an arbitrary number of points. Each cell in a risk matrix is associated with a risk priority and is usually coloured accordingly. The

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following example shows a risk matrix with five risk priority ratings, denoted by roman numerals:

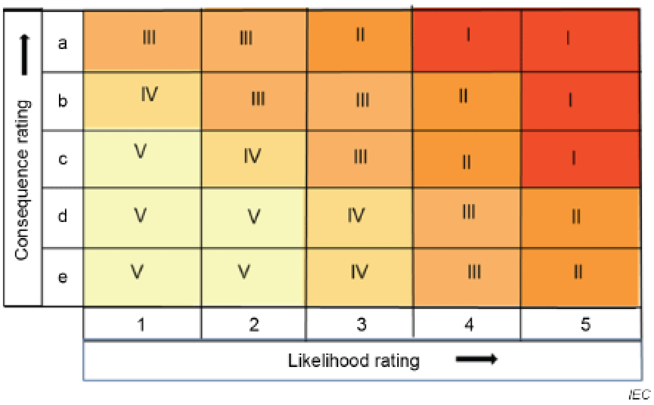

Figure 7.6 - An illustration of a consequence/likelihood (risk) matrix [IEC 31010:2019]

7.1.5.8.2 Input and output

The input needed for establishing consequence/likelihood matrices is the data with which the consequence and likelihood scales are established, as well as the context, according to which the matrices are prepared.
The output is an illustrative display of relative consequences and likelihood and level of risk for a certain risk, as well as a priority rating for each risk.

7.1.5.8.3 Strengths and weaknesses

The strengths of the consequence/likelihood matrices, among others, are the following (ISO/IEC-31010 , 2019):

- A simple, easy to understand ranking of risks;
- Risks are classified according to their significance levels;

Among the weaknesses, the following are the most significant (ISO/IEC-31010 , 2019):

- The validity of risk ratings depends on the method of developing the consequence and likelihood scales;
- Risk matrices tend to be subjective: different people may assign significantly different ratings to the same risk;
- It is hard to compare the level of risk for different categories of consequences.

7.1.6 Quantitative risk assessment

7.1.6.1 F-n diagrams

7.1.6.1.1 Use

The F-n diagram can be seen as a special case of the consequence/likelihood matrix with logarithmically-scaled axes where the horizontal axis present the cumulative number of fatalities, and the vertical axis is the frequency of occurrence. The risk rating is usually presented as straight lines on the graph, where the higher the slope, the lower the tolerance for fatalities (ISO/IEC-31010 , 2019).

F-n diagrams are used either as a historical record showing outcomes involving human fatalities, or as a result of quantitative analysis of the risk of loss of life in comparison with defined acceptability criteria. An example is shown in Figure 7.7. Regions above lines A and B represent regions with intolerable risk level, lines below B-1 or A-1 present regions with broadly acceptable risk levels, while the area between B-1 and B or between A-1 and A denotes the region of acceptable risk if they are as low as reasonably possible (ALARP).

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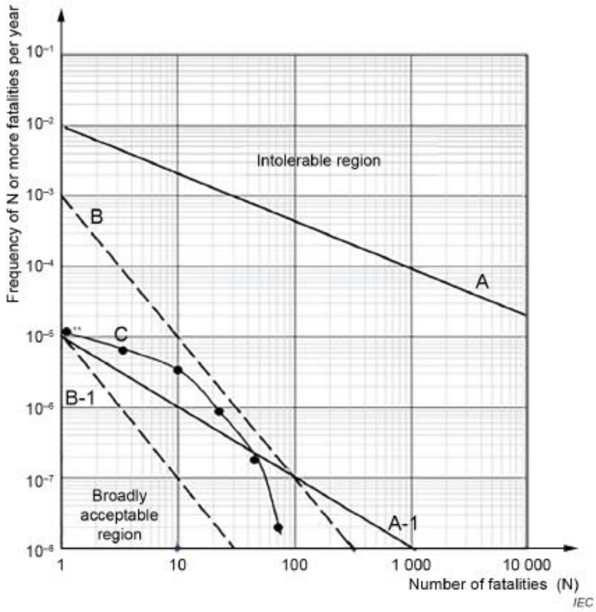

Figure 7.7 - An example of F-n diagrams (ISO/IEC-31010 , 2019)

7.1.6.1.2 Input and output

Input for F-n diagrams is the historical data or output from quantitative risk assessment, which predicts the probability of a certain number fatalities.
Output is the graphical presentation of calculated data from quantitative risk assessment in comparison to a predefined criteria.

7.1.6.1.3 Strengths and weaknesses

The main strength of F-n diagrams is that they result in an simple and understandable presentation of risk assessment results and their comparison to criteria.
The limitations of F-n diagrams include (ISO/IEC-31010 , 2019):

- The quantitative calculations needed for setting up F-n diagrams tend to be complex;
- A detailed analysis requires a high number of scenarios to be evaluated, which is time consuming and requires expert knowledge.

7.1.6.2 Bayesian analysis

7.1.6.2.1 General

In decision analysis, a logical tree (also named decision or event tree) is often used to provide a framework for a systematic analysis of the corresponding consequences and to provide a graphical representation of the different options for decisions. Different options are associated with different information; the decision of whether or not to replace may be associated with different costs, severity of consequences and probabilities of follow-up events from the original decision. For example, the choice of whether or not to undertake maintenance of a damaged bridge, requires both the consideration of the consequences of undertaking maintenance (periodically out of commission, congestions, etc.) and the probabilities associated with such consequences, as well as the consequences and the attached probabilities of occurrence if the maintenance is not performed (continued degradation, loss of life, etc.)
According to Faber (Faber, 2012) decision analysis is differentiated based on the state of information available at the time:

- Prior analysis, where the decision analysis is based on the given information;
- Posterior analysis, where the decision analysis is based on additional information, obtained from experimental, expert judgement or other data;

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- Pre-posterior analysis, where the decision is based on unknown information.

The posterior decision analysis, here referred to as the Bayesian approach is described in detail.

7.1.6.2.2 Use

Bayesian analysis enables the use of both judgmental and empirical data for making decisions. Bayes' theorem allows inclusion of new evidence into prior beliefs to form an updated estimation. A general expression for this theorem is as follows (IEC 31010:2019):

$$P[A|B] = \frac{P[B|A]\,P[A]}{P[B]} \qquad [7\text{-}1]$$

where:

P[A] is the prior assessment of the probability of A;
P[B] is the prior assessment of the probability of B;
P[A|B] is the probability of A given that B has occurred, i.e. the posterior assessment;
P[B|A] is the probability of B given that A has occurred, i.e. the posterior assessment;

More generally, Bayes' theorem can be extended to cover multiple event in a particular sample space. For example, given data D, prior understanding regarding a certain risk needs to be updated. The data needs to be used to asses relative merit of a number N of non-overlapping hypotheses, denoted as H_n for n = 1, 2, …, N. According to the Bayes' theorem, the probability of the j-th hypothesis is (IEC 31010:2019):

$$P[H_j|D] = P[H_j]\,\frac{P[D|H_j]}{\sum_n P[H_n]P[D|H_n]} \qquad [7\text{-}2]$$

According to this, the updated probability for the j-th hypothesis (P[H_j|D]) is obtained by multiplying the prior probability (P[H_j]) with the fraction, which expresses the ratio between the probability of data if the j-th hypothesis is true and the probability of the data if all hypotheses are true.

The Bayesian approach can be used to provide a prior estimate of a parameter of interest on the basis of subjective data or using relevant information from similar situations. A prior estimate can provide probabilistic description of the likelihood of an event. This can be useful for risk assessment when no empirical data is available. Observed event data can then be combined with the prior assessment to provide a posterior estimate of the parameter in question. This can be a point or interval estimate, which captures the uncertainty associated with both variability and state of knowledge.

7.1.6.2.3 Input and output

The input for the Bayesian analysis are the judgmental and empirical data needed to quantify the probability model.
The output can be both single point or interval estimates of the parameter in question.

7.1.6.2.4 Strengths and weaknesses

The strengths of the Bayesian analysis are:

- It provides a mechanism for considering subjective beliefs regarding a problem and for combining the prior beliefs with new data;

The weaknesses of the Bayesian analysis are:

- The posterior estimates depend heavily on the choice of prior estimates;
- Solving complex problems can be computationally expensive.

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7.1.6.3 Bayesian networks/influence diagrams

7.1.6.3.1 Use

Bayesian networks, also called Bayes' nets, are graphical models that present random variables (discreet or continuous) as nodes, which are then connected with arrows, showing direct dependencies between variables. Here, parent nodes (denoted as pa(X), pointing to a certain node X) and child nodes are distinguished: the state of the child nodes depends on the combination of conditional probability distribution of its parent nodes. A simple example is shown in Figure 7.8 (ISO/IEC-31010 , 2019):

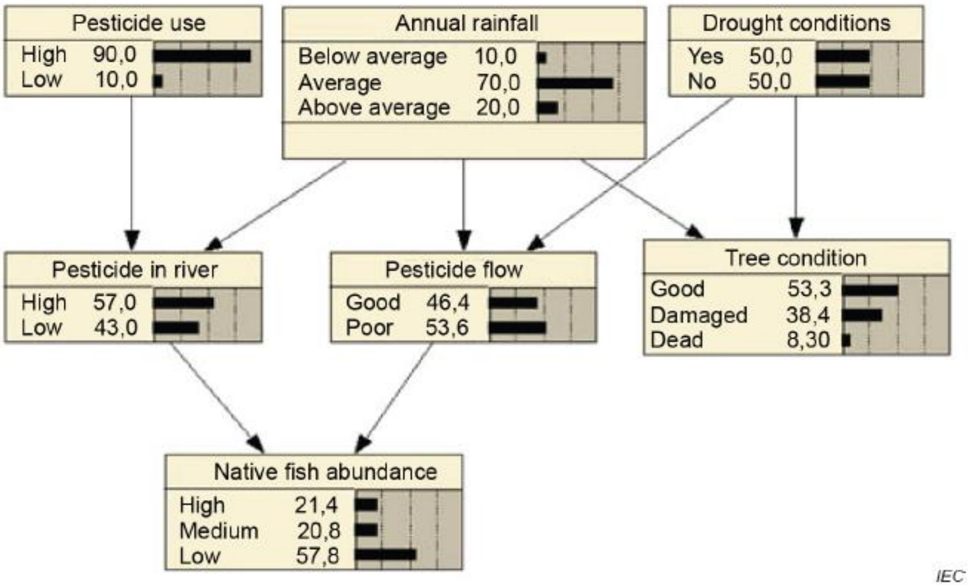

Figure 7.8 - An example of Bayesian networks/influence diagrams (ISO/IEC-31010 , 2019).

Basic Bayesian networks contain variables representing uncertain events and can be used to estimate probability or risk, or to infer key risk drivers leading to certain consequences (IEC 31010:2019). If it is extended to include uncertainties to assess the impact of risk controls or risk mitigation, it is known as an influence diagram. Bayesian networks can be constructed as qualitative models by the stakeholders and then quantified using relevant judgmental and/or empirical data. They provide visual models that can be used for gaining stakeholder input and for reaching agreement on decisions when there is too much divergence in stakeholder opinion. Constructing the qualitative Bayesian network structure can be done with casual mapping and a Bayesian network can be used in tandem with scenario analysis and cross-impact analysis (ISO/IEC-31010 , 2019):

7.1.6.3.2 Input and output

The input or Bayesian networks is the understanding of system variables and the relationships between them, as well as the prior and conditional probabilities for the relationships. If used in the form of an influence diagram, valuations and uncertainties are also needed.
The output contains graphically-represented conditional and marginal distribution.

7.1.6.3.3 Strengths and weaknesses

The strengths of Bayesian networks are:

- Subjective beliefs regarding the problem at hand can be combined with empirical data;
- Readily available software means that Bayesian networks are relatively easy to use, while also easy to understand due to graphical representation of the problem.

The limitations can be:

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- Bayesian networks require conditional probability distributions, which are often provided by expert judgment;
- Solving complex problems can be computationally intensive, and representing all interactions can be difficult.

7.1.6.4 Cause-consequence analysis (CCA)

7.1.6.4.1 Use

A cause-consequence analysis (CCA) is a combination of fault- and event- tree analysis, considering both the causes and consequences of initiating events, as well allowing time sequential failure and time delays to be considered. It considers various paths a system can follow after a critical event on the basis of behaviour of particular subsystems and, if quantified, it provides an estimation of the probability of different possible consequence following a critical event.

The result of each sequence in a cause-consequence diagram is a combination of fault trees, as shown in Figure 7.9 (ISO/IEC-31010 , 2019):

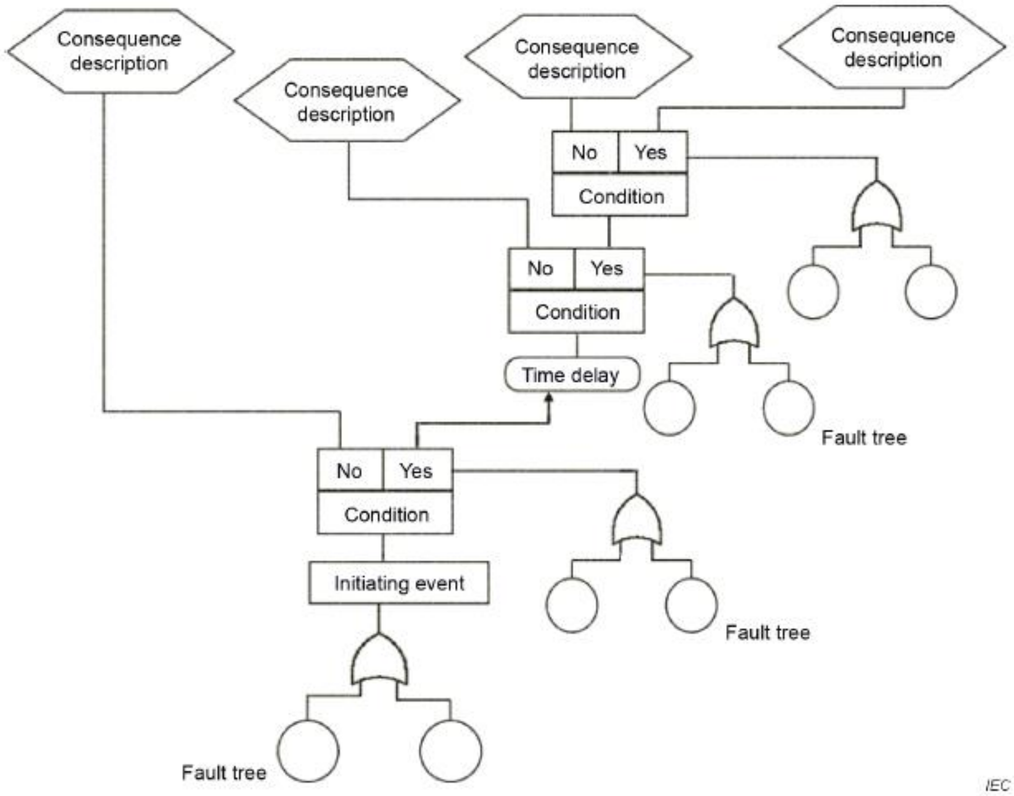

Figure 7.9 - An example of cause-consequence diagram (IEC 31010:2019):

7.1.6.4.2 Input and output

The input for the CCA is the understanding of the system, its failure modes and possible paths it could follow after a critical event.
The outputs of the CCA are: a diagram showing how a system might fail, as well as the causes and consequences of failure; a probability of occurrence estimate for each potential consequence.

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7.1.6.4.3 Strengths and weaknesses

Its advantages in comparison to a fault-tree analysis, namely in situations where (ISO/IEC-31010 , 2019):
- it is easier to determine event sequences than causal relationships;
- the complexity of the considered problem might cause the fault trees to become very large;
- there are separate teams working on different parts of the analysis.

The limitation of the CCA is that it is harder to construct compared to fault- or event-tree analysis due to its complexity.

7.1.6.5 Cross impact analysis

7.1.6.5.1 Use

Cross impact analysis is a family of techniques used for evaluating changes in the probabilities of occurrence given a set of events that impact the actual occurrence of one of the events.
A cross impact analysis is performed by constructing a matrix that show mutual dependencies of events: rows list the events that might occur and columns represent the events that might be affected by the row events. For each event, the following needs to be estimated: the probability of occurrence at a given time point; the conditional probability P(*i* | *j*) of event *i* if event *j* occurs simultaneously. Usually, a Monte Carlo simulation is performed to calculate probabilities of one event while taking into account other events.

Cross impact analysis is used for predicting how factors impact future decisions (up to 50 years in advance), for example for complex projects where there are interacting risk.

7.1.6.5.2 Input and output

The input for the cross impact analysis is the knowledge and experience of experts who are able to envisage future developments and provide a realistic estimate of probabilities. Suitable software is needed for the calculation of conditional probabilities, as well as knowledge and time for developing the models.

The output provides a list of possible future scenarios based on the specified cross impact model.

7.1.6.5.3 Strengths and weaknesses

The strengths of cross impact analysis include the following (ISO/IEC-31010 , 2019):
- It provides knowledge regarding future developments;
- Highlights the chains of causality of events, i.e. which event has an influence on other events.

The weaknesses of the cross impact analysis are as follows (ISO/IEC-31010 , 2019):
- In practice, the number of events that can be covered is limited by software and by time required to include them;
- Since this method is based on expert judgment, a certain level of expertise is required.

7.1.6.6 Markov analysis

7.1.6.6.1 Use

Markov analysis is used for calculating the long-term probability of the system being in a specific state, as well as for estimating the expected time for the first failure of a system to occur (the first passage time) or that the expected time for the system to return to a specified state (the recurrence time). Markov analysis is a quantitative technique, which is applicable to systems that can be described with discrete states and discrete transitions between the states. This approach assumes that transitions between the states occur at specified intervals with corresponding transition probabilities, a principle called discrete time Markov chain.

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The states and their transitions can be represented with Markov diagrams, as shown in Figure 7.10:

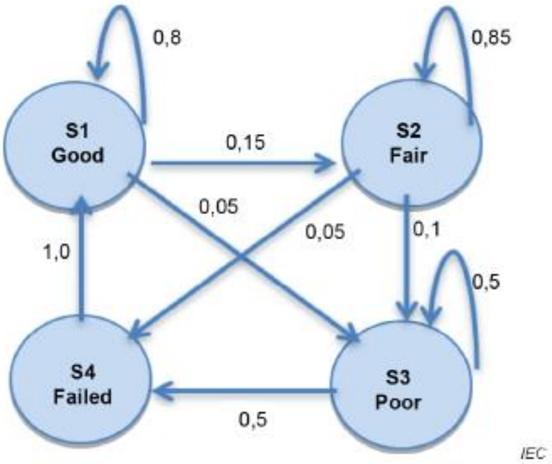

Figure 7.10 - An example of Markov diagram [IEC 31010:2019].

In this cases, the system has four discrete states, represented with circles: S1, S2, S3, and S4. The arrows represent the transition and are accompanied by their associated transition probabilities. A Markov matrix (see Table 7.5) can also be used to show transitional probabilities, where the sum of each row is 1 since the values in the row represent all possible transitions from each state.

Table 7.5 - An example of Markov matrix [IEC 31010:2019].

		Next state after transition			
		S1, Good	S2, Fair	S3, Poor	S4, Failed
Current state	S1, Good	0,8	0,15	0,05	0
	S2, Fair	0	0,85	0,1	0,05
	S3, Poor	0	0	0,5	0,5
	S4, Failed	1	0	0	0

7.1.6.6.2 Input and output

The input for the Markov analysis are discrete states, which the system can occupy, the understanding of all possible transitions that need to be modelled and the transitional probabilities.
Output of the Markov analysis are estimations regarding the probability of the system being in any of the specified states.

7.1.6.6.3 Strengths and weaknesses

The strengths of Markov analysis include (ISO/IEC-31010 , 2019):

- It enables modelling complex, dynamic systems with multiple states;
- The transitions between states are clearly represented with diagrams.

The weaknesses of Markov analysis are (ISO/IEC-31010 , 2019):

- In order for the modelling to be accurate, a large amount of input data needs to be collected;
- Assumptions made during the set-up of the Markov analysis may not always be applicable, especially if the probabilities or transition rates tend to change with time.

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7.1.6.7 Monte Carlo simulation

7.1.6.7.1 Use

As a part of risk assessment, Monte Carlo simulations can be used to take into account uncertainties in conventional analytical models or for probabilistic calculations when analytical techniques are not applicable. The simulation usually involves taking random sample values from the distributions of each input variable, performing a calculation to obtain a result value and then repeating this process in order to obtain a distribution of the results. The result can be a probability distribution of the result or a statistic, such as the mean value.\

7.1.6.7.2 Input and output

The input for the Monte Carlo simulation is the model of the system, which describes the relationship between the input variables, as well as information on the type of input variables and of the sources of uncertainty that need to be taken into account. The input variables with uncertainty are considered as random variables, described with probabilistic distributions such as normal, log-normal, extreme value distribution, etc.

The output of the Monte Carlo simulation will in general be the entire probabilistic distribution of the calculated result variables, or key measures from a distribution, such as a value of the outcome for which one can have a certain level of confidence that it will not be exceeded.

7.1.6.7.3 Strengths and weaknesses

The strengths of the Monte Carlo simulation are as follows:

- The models can be relatively easily developed and extended if needed;
- The technique is widely used and there exists readily available software and documentation for performing simulations;
- Any relationships between variables, including effects such as conditional dependencies, can be represented.

The weaknesses of the Monte Carlo simulation are:

- The accuracy of the technique relies on the representation of uncertainties and on the number of simulations that are performed;
- It cannot be used to accurately simulate low probability and high consequence events since the weight of such outcomes cannot be included in the simulation, thus potentially removing extreme events from consideration and rendering unjustified high confidence for the decision maker.

7.2 Risk treatment methods

In the reference frame of managing risks, after the process of risk assessment has been performed, i.e. all the relevant risks have been identified and described in a qualitative and/or quantitative manner and evaluated to determine whether further steps need to be undertaken, risk treatment can take place – a process of selecting and implementing options for dealing with risk.

According to (ISO/IEC-31010 , 2019), risk treatment is an iterative process consisting of the following steps:

- Choice of risk treatment method(s) to be used;
- Implementation of risk treatment
- Assessment of effectiveness of the implemented risk treatment
- Decision whether the remaining risk is acceptable or not;
- If the remaining risk is not acceptable – continuation of risk treatment.

Risk treatment can lead to both expected and unexpected consequences - one of which can be the introduction of new risks. The process of risk treatment is therefore closely connected with monitoring and review.

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7.2.1 Risk treatment options

If the estimated risk is higher than the set risk criteria, or if it is evaluated that it in any case requires treatment, several risk treatment options can be undertaken (Stewart & Melchers, 1997):

- Risk avoidance: this refers to not proceeding with the use of the system. In the context of infrastructure objects this could, for example, mean closures of or limitations to the use of bridges;
- Risk reduction: achieved either through reducing the probability of occurrence of an event or limiting the severity of the consequence.
- Risk transfer: applicable only where the primary consequences are financial since financial or insurance mechanisms are put in place to share or to transfer the financial risk to other parties. An example is a contractor submitting a bank guarantee to obtain a tender for construction of an infrastructure object in order to share the risk of financial bankruptcy with the bank.;
- Risk acceptance: accepting the risk even if it might exceed the given criteria, usually for a limited amount of time, in order to allow other measures to take place first.

7.2.2 Selection of the risk treatment options

7.2.2.1 General principles

As described in section 4.3.2, the process of risk treatment requires decision-making on behalf of the person responsible for conducting risk management since it involves balancing the potential benefits of achieving the risk management goals with the drawback of additional cost and effort associated with implementing the risk treatment. However, as defined in ISO 31000:2018, when selecting the relevant risk treatment methods a broader justification should be considered, additionally taking into account the values and ideals of the stakeholders.
An example of such consideration is the process of managing the risk of corrosion of reinforcing steel, exposed to atmospheric influences due to insufficient concrete cover, of a RC bridge in a densely built settlement. This risk can be treated (mitigated) by performing additional concrete casting, which can, however, lead to annoyance for the local residents due to e.g. additional noise. In such cases, other options can be selected, which meet the requirements and needs of the residents.

7.2.2.2 Methods for selecting risk treatment options

7.2.2.2.1 Cost-benefit analysis (CBA)

7.2.2.2.1.1 Use

Cost-benefit analysis is used on an operational level to help decide on the best option for risk treatment. It weighs the total expected financial costs of risk treatment options against the total expected benefits to help decide on the most cost-effective option. It can be performed on both qualitative and quantitative level: on the quantitative level, a monetary value is assigned to tangible costs and benefits, while for the qualitative analysis it provides a descriptive analysis summarizing the costs and benefits, relationships and trade-offs between different costs and benefits (ISO/IEC-31010 , 2019).

7.2.2.2.1.2 Input and output

Inputs include information regarding costs and benefits (both tangible and intangible, direct and indirect) as well the uncertainties that are associated with those costs and benefits. This can be provided on a basic level using simple spreadsheets or a qualitative analysis, or in a more detailed manner by collecting necessary data and by estimating intangible costs.
The output of CBA is information regarding costs and benefits of different options – expressed either as a net present value, a best ratio, or as the ratio between the value of benefits and costs [(ISO/IEC-31010 , 2019).

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7.2.2.2.1.3 Strengths and weaknesses

The strengths of the cost-benefit analysis are the following (ISO/IEC-31010 , 2019):

- Costs and benefits can be compared in terms of a single, monetary value;
- Decision-making is based on collected information, providing transparency to the process;

The weaknesses of the cost-benefit analysis are as follows (ISO/IEC-31010 , 2019):

- It requires a high level of knowledge of likely benefits and is thus not suited to new situation with relatively high uncertainty;
- It is a relatively rigid in terms of estimating when the benefits and cost will since it does not consider the time-dependency of uncertainties;
- A quantitative analysis can be sensitive in terms of the value it outputs depending on the assumptions and methods used to assign values to intangible benefits.

7.2.2.2.2 Decision tree analysis

7.2.2.2.2.1 Use

A decision tree analysis can be used to structure and solve decision problems by quantifying the best possible decision on the basis of the possible (uncertain) outcomes. It models possible paths from an initial decision that might lead to a range of events and for which certain predictable decision need to be made. These are graphically represented as decision trees, which are similar to event trees. Decision tree analysis often includes subjective estimation of event probabilities and can aid decision-making by overcoming subjective biases with regards to success or failure (ISO/IEC-31010 , 2019)

7.2.2.2.2.2 Input and output

The input to a decision tree analysis are decision points, information regarding possible outcomes of decisions and probabilities of uncertain events that might influence decisions.
The output of a decision tree analysis is the graphical representation of the decision problem, calculation of expected value for each possible path, and a recommendation regarding the best possible pathway to be followed.

7.2.2.2.2.3 Strengths and weaknesses

The most significant strengths of the decision tree analysis are (ISO/IEC-31010 , 2019):

- A decision problem is represented in a graphical, understandable manner;
- By developing the tree, one might develop a better understanding of the problem;
- The best course of action (pathway) is highlighted and quantitatively substantiated.

Among the weaknesses of the decision tree analysis, the following are prominent (ISO/IEC-31010 , 2019):

- As the complexity of the problem grows, so does it graphical representation;
- The decision problem is often oversimplified in order for it to be represented as a decision tree, which can lead to the elimination of extreme values;
- It is based on past data, which might not be applicable to decisions being modelled.

7.2.2.2.3 Game theory

7.2.2.2.3.1 Use

Game theory is used to evaluate risk where the outcome of decision depends on the action of another decision-maker (in game theory called player) or on possible outcomes.
The following example from (ISO/IEC-31010 , 2019) presents a situation where the game analysis supports the decision between three different technologies given different actions that can be taken by three different decision-makers (competitors). Which technology a decision-maker will choose is not known, but the probabilities can be estimated, along with the resulting profits (shown in million monetary units), and presented in a game matrix:

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	Competitor			Expected profit	Guaranteed profit	Maximum regret
	Action 1	Action 2	Action 3			
Probability	0,4	0,5	0,1			
Technology 1	0,10	0,50	0,90	0,38	0,10	0,50
Technology 2	0,50	0,50	0,50	0,50	0,50	0,40
Technology 3	0,60	0,60	0,30	0,57	0,30	0,60

Figure 7.11 - Example of a game matrix as resulting from a game analysis [IEC 31010:2019]

From this example the technology 3 delivers the most profitable outcome (0.57), however, it is also sensitive to actions of decision-makers. Technology 2 on the other hand, results in the same profit regardless of the actions taken (0.50). From this game analysis it is clear that it should be considered if the risk associated with choosing technology 3 is worth the marginally larger profit.

7.2.2.2.3.2 Input and output

The input for a game analysis must at least contain: the decision-makers (players) or alternatives of the game; information and actions, which decision-makers can take at any decision point.

7.2.2.2.3.3 Strengths and weaknesses

The strengths of the game theory approach include the following [IEC 31010:2019]:

- It proposes a framework for analysing decision problems where several interdepended decision are taken into account and where outcomes depends on other decision-makers or outcomes;
- In certain situations game theory can outlines can serve as a scientific technique, which the user can apply for developing an optimal strategy.

The most prominent weaknesses of the game theory are [IEC 31010:2019]:

- In some cases discretizing problems into recognizable outcomes based on decisions that were taken is not possible;
- It is assumed that only the closed-set of considered actions and their pay-offs is applicable and/or practical.

7.2.2.2.4 Multi-criteria analysis

7.2.2.2.4.1 Use

Multi-criteria analysis (MCA) or multi-criteria decision analysis uses multiple criteria as part of the decision-making process. MCA can be used as an extension of the cost-benefit analysis, which is limited to monetary values when making judgments with regards to suitability of decisions. MCA is therefore particularly useful in such situations where single-criterion approaches cannot be used to assign monetary values, for example to social and environmental criteria.

7.2.2.2.4.2 Input and output

MCA is set up by defining the objectives of the analysis, the options for achieving the objectives, criteria that are used for evaluating the options.

The output of MCA is either a single most preferred option, options ranked according to suitability, a short-list of options for a further appraisal, or a classification of options as acceptable or unacceptable.

7.2.2.2.4.3 Strengths and weaknesses

Strengths:

- Allows the decision-maker to include a full range of criteria (financial, environmental, technical, social, etc.);
- Broad scope of use, can be applied in all sectors;

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- Accessibility, there is a wide variety of MCA tools available with wide-ranging difficulty of use and complexity.

Weaknesses:

- Some expert judgement is needed for discerning among the possible options.

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8 Data-informed approaches in assessment

8.1 Principles of use of data in performance verification for existing structures

The evolution of fib Model Codes for structural concrete throughout the years has contributed significantly to the development of improved design methods. Over the last decades they have been updated on the basis of the growing awareness of the need to take into account the phenomena of degradation that occur in structures during their lifetime.

The first complete version of the fib Model Code 2020, (fib MC2020, 2022), will be released in 2022 and will be based on the following basic principles:

- dealing with new and existing concrete structures, and removing constraints for novel types of materials;
- reflecting the importance of sustainability and through-life management of structures;
- implementing fundamental principles and a safety philosophy based on reliability concepts;
- implementing consistent treatment of safety, serviceability, durability by performance-based approach.

Therefore, the major changes are related to the use of a consistent approach with differentiation to new and existing structures. The key differentiation aspects that characterize the performance assessment of existing structures with respect to new structures are highlighted in Figure 8.1.

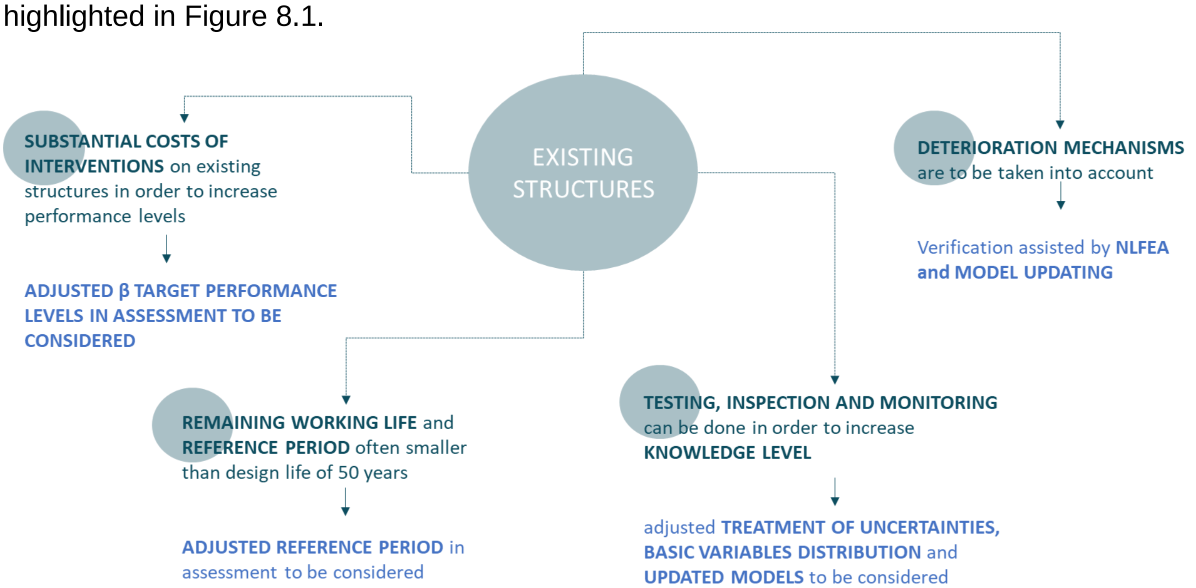

Figure 8.1 - Key differentiation aspects that characterize the performance assessment of existing structures with respect to new structures.

Existing structures often have a remaining working life and reference period smaller than design life of 50 years and their condition must be assessed based on available information and of data gathered from testing, inspection and monitoring. These data can contribute to the creation of adequate structural models for existing (e.g. deteriorated) structures, given also the substantial costs of interventions in order to increase performance levels.

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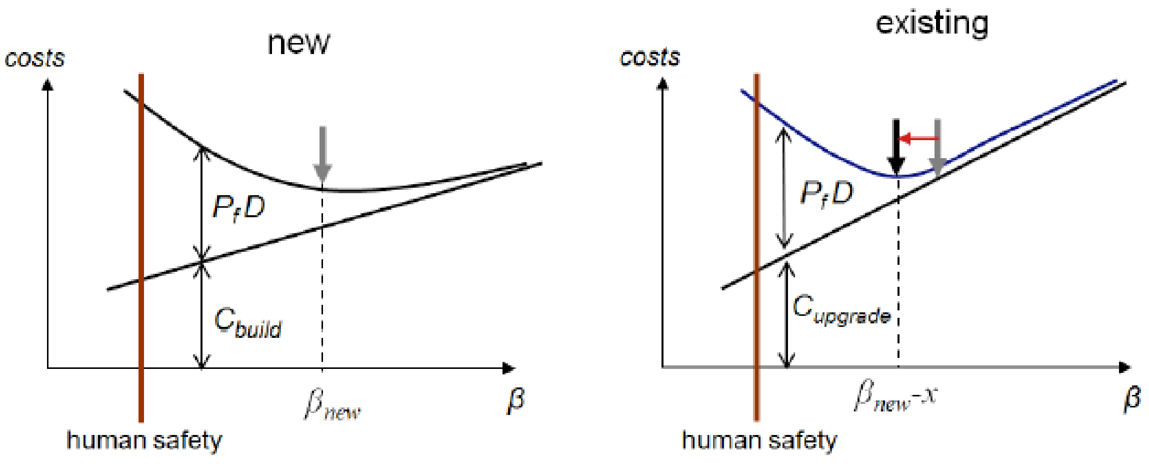

Figure 8.2 - Illustration of the difference in cost optimisation for the design of new structures versus upgrading of existing structures (fib Bulletin 80, 2016)

Figure 8.2 shows, for instance, how target reliability levels for existing structures decrease compared with new structures, due to difference in terms of the needed effort and investment to achieve functionality requirements for the remaining working life.
Performance verification using a data-informed approach based on the information collected from inspections, testing and monitoring is still an open research topic under development as more advanced knowledge is gained in the fields of data processing, monitoring and maintenance planning.

On the basis of the above, the aim of (fib MC2020, 2022) for existing structures is to consider:
- adjusted target performance levels in assessment;
- adjusted reference period in assessment;
- adjusted treatment of uncertainties.

One of the major challenges is how to deal with degradation phenomena and its impact on structural resistance and load effect in time. In this respect, (fib MC2020, 2022) purpose is to introduce an improved approach to the assessment of “actual” capacity.
It recommends to consider in the performance-based assessment of the remaining service life the following aspects:
- **ultimate limit state or serviceability limit state verification** (e.g. in case damage is already existing) using **annual target reliability values**, corresponding to the target reliability levels. Such verifications take basis in a coupled modelling of the initiation time and propagation time and in the present condition of the structure should take into account:
- the uncertainties typically considered in structural reliability calculations for ULS/SLS (as done for non-degrading structures) as well as those related to the degradation modelling and data-informed analysis:
    - the uncertainties in relation to the initiation phase modelling;
    - the uncertainties in relation to the propagation phase modelling;
    the uncertainties associated with monitoring, inspections and the interpretation of such data, if applied.
- **condition limit states verification** (e.g. corrosion initiation phase not yet reached). In the case of performance-based assessment of existing structures with respect to durability on the basis of full-probabilistic methods, the same target reliability level for condition limit state associate to durability can be applied as for new structures, unless ULS and/ or SLS verifications or the application of risk-based methods justify a different β level. Alternative values for target reliability levels of condition limit states compared

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to new structures may be considered applicable when accounting for e.g. altered service life, altered requirements on structural behaviour over the remaining service life (i.e. cracking, spalling,...) and the actual progress of the deterioration.

The advantages of implementing annual target reliability values for existing structures are shown in Figure 8.3 - .

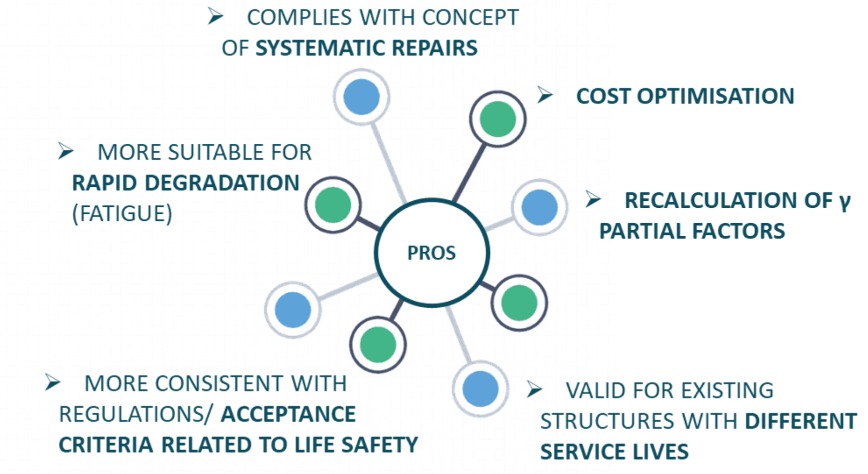

Figure 8.3 - Advantages of implementing annual target reliability values for existing structures.

As per most of the standards analysed (e.g. (CEN/TC-250, 2020) (ISO-13822, 2010) (ISO-2394, 2015) (ISO-13822, 2010)), updating information of properties and performance modelling of a structure is an essential part of the assessment of existing structures. In assessment, an existing structure can be inspected/tested so that load, resistance, environmental parameters and global static and dynamic response can be measured on-site. The need for assessment, its type (preliminary and detailed) and its level of application (network, system or component) may be originated by different causes (external actions, damages, planned assessment) and could be based on various available information.

[CEN/TS 17440]

NEED FOR ASSESSMENT IN TIME EXTERNAL CAUSE	STRUCTURAL ISSUES	N – S – C LEVEL*	ASSESSMENT TYPE	AVAILABLE / REQUIRED INFOs
	CONSTRUCTION ERRORS	S | C	DETAILED	• ORIGINAL DESIGN DOCUMENTS • AS-BUILT & CONSTRUCTION DETAILS (BIM)
SCHEDULED ASSESSMENT for ASSET MANAGEMENT PROGRAMME		N | S | C	PRELIMINARY | DETAILED	• PERIODIC/DETAILED INSPECTION, SURVEYS OUTCOMES
	DETERIORATION PROCESSES	N | S | C	PRELIMINARY | DETAILED	• DEFECTS, DETERIORATION CHARACTERIZATION
CHANGE OF DESIGN LOADS		N | S | C	PRELIMINARY | DETAILED	INSPECTION AND TESTING RESULTS ON: • MATERIAL PROPERTIES • HAZARDS DISCRETE/CONTINUOUS (IN SPACE AND TIME) DATA FROM: • NDT/DT • MONITORING SYSTEMS
CHANGE OF HAZARDS (e.g. landslide, accidental actions)*		S | C	DETAILED	
RETROFITTING		S | C	DETAILED	
NEED FOR EXTENSION OF WORKING LIFE		S | C	DETAILED	

Working life — Incremental level of knowledge

*[IM-SAFE integration to CEN/TS 17440]

[*N=Network S=System C=Component]

Figure 8.4 - Need for assessment in time (CEN/TC-250, 2020) revised by IM-SAFE.

The collection of data can be used to update prior information as follows (see 8.2.5):

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- *Direct information*:
  - **Basic variables**: updating of probability distributions, mean values or assessment values of basic variables (resistance, actions, degradation process evolution in time,..)
- *Indirect information*:
  - **Probability of failure**: updating of the probability of the structural failure by using information from load testing or about the past performance
  - **Model updating**: deterministic or probabilistic methods to update numerical structural models

Data can be differently used in the structural assessment based on the verification method selected (risk-based, reliability-based and semi-probabilistic approach). A framework to move towards a data-informed verification process is outlined in 8.2. In such analysis, particular attention has to be given to the definition and modelling of the deterioration processes over time with the aim to move towards a provisional assessment of the evolution of the structural performance during the residual service life.

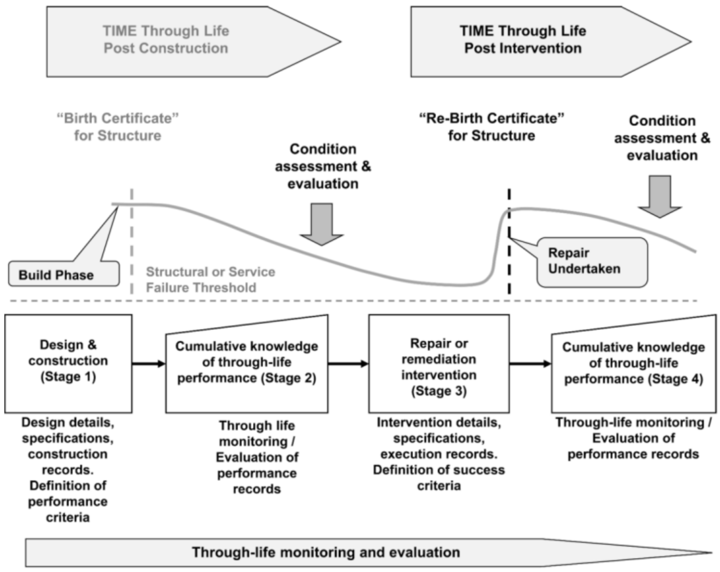

Figure 8.5 - Idealized representation of the through-life performance of a structure requiring a remedial intervention to meet its intended design service life (fib MC2010, 2013)

It is essential to remember that inspection methods have a limited resolution; thus, the uncertainties associated with the inspection, testing and monitoring procedures are to be properly addressed and quantified in the evaluation of the indicators of the estimated condition of a structure (see section 8.1.1). The key issue is to which degree the indication of a certain behaviour is related to the real condition state and consequently to the Condition State Classification definition. For this purpose, the concept of the Probability of Detection (PoD) is very useful. PoD provides a quantification of the quality of inspection methods through the probability of detection of a defect of a given size or extent.

A conservative design does not usually lead to a significant increase in structural cost, while a conservative assessment can result in unnecessary and costly repairs or replacement. Therefore, in order to reduce the model uncertainty, more refined and calibrated structural models (e.g. finite element models) can be used for the assessment of existing structures.
The process of collecting information, assessing structural performance through the data analysis and planning repair and strengthening activities is a decision procedure which aims to identify the most effective investigations and interventions required to satisfy the target

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reliability requirements to the use of the structure and/or to reduce the uncertainties regarding its current condition and future performance. It is important that this process is optimized with due consideration of the total service life costs of the structure.

8.1.1 Uncertainty classification and modelling in data-informed performance verification

8.1.1.1 Types of uncertainties

As mentioned in 6.1.3, decisions concerning structures should account for all uncertainties of relevance for their performances such as (fib MC2010, 2013):

- measurement error,
- aleatory uncertainties: inherent variability of a measured parameter (direct information),
- model uncertainty: when a parameter of interest cannot be measured directly so that a relationship between it and the corresponding measured parameter is needed (indirect information)
- statistical uncertainty: due to a limited number of measurements
- other epistemic uncertainties: lack of knowledge on the structural system (as-built), numerical modelling uncertainties

Uncertainties can be reduced or mitigated by updating the available information on the basis of measurements and inspections.

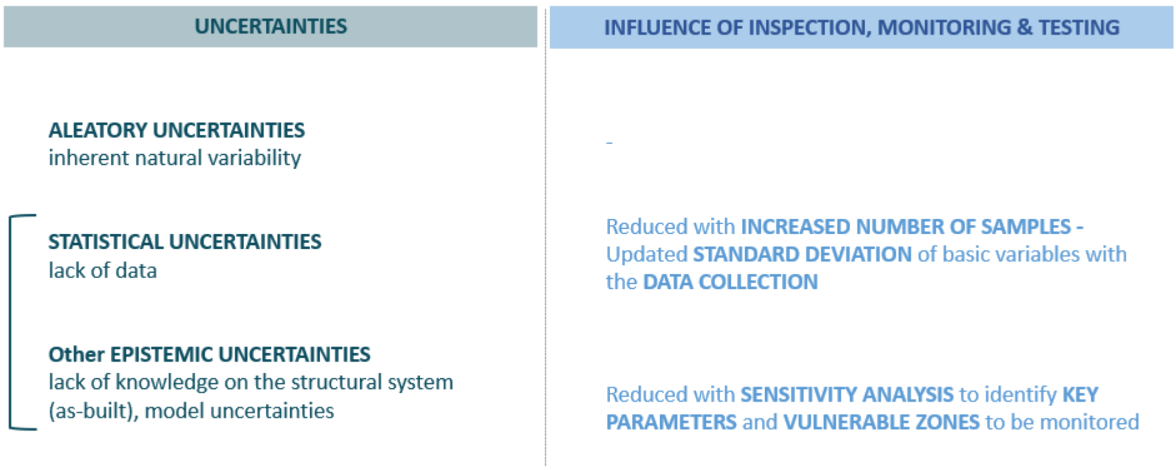

Figure 8.6 - Uncertainties description and possible mitigation actions.

Uncertainties associated with the data collection processes should be taken into account in a structural assessment using the theory of probability (ISO-2394, 2015). This can be done using probabilistic methods either explicitly in reliability analysis or for updating the characteristic and design values of basic variables. Because of the different nature of uncertainties associated with the design and assessment situations, applicability of the characteristic values of basic variables and the partial safety factors from design codes shall be carefully investigated.

In the structural reliability analysis, no distinction should be made between the treatment of aleatory and epistemic uncertainties. This differentiation is introduced only for the purpose of setting focus on how uncertainty can be reduced by additional testing or more detailed research. Depending on the nature of the reliability problem, basic variables can be represented as random variables, random processes, and random fields, discrete as well as continuous.

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The probabilistic models should describe the characteristics of the uncertainties of individual random basic variables, but also accommodate the consideration of the dependencies among them.

8.1.1.2 Definition of populations

The random quantities within a reliability analysis should always be related to a meaningful and consistent set of populations. The description of the random quantities should correspond to this set and the resulting failure probability is only valid for the same set.

The basis for the definition of a population is in most cases the physical background of the variable. Factors which may define the population are:

- the nature and origin of a random quantity
- the spatial conditions (e.g. the geographical region considered)
- the temporal conditions (e.g. the intended time of use of the structure considered)

The choice of a population is to some extent a free choice of the designer. It may depend on the objective of the analysis, the amount and nature of the available data and the amount of work that can be afforded.

In connection with theoretical treatment of data and with the evaluation of observations it is often convenient to divide the largest population into sub-populations which in turn are further divided in smaller sub-populations etc. Then it is possible to study and distinguish variability within a population and variability between different populations. In an analysis for a specific structure, it may be efficient to define a population as small as possible as far as use, shape and location of the structure are concerned (microzonation).

8.1.1.3 Hierarchical modelling of uncertainties

A hierarchical modelling is recommended for modelling different kinds of actions and materials, where applicable. The hierarchical model assumes that a random quantity X can be written as a function of several variables, each one representing a specific type of variability:

$$X_{ijk} = f\left(Y_i, Y_{ij}, Y_{ijk}\right) \tag{8-1}$$

The variables Y_i, Y_{ij}, Y_{ijk} represent various origins, time scales of fluctuation, or spatial scales of fluctuation.

8.1.2 Data-informed performance verification

Similar to the design of new structures, when assessing existing structures, it should be verified that with an appropriate reliability level no limit state is exceeded for all relevant assessment situations. The assessment can be performed following different methods of progressively decreasing complexity (risk-based, reliability based and semi-probabilistic methods) as described in section 6.2.

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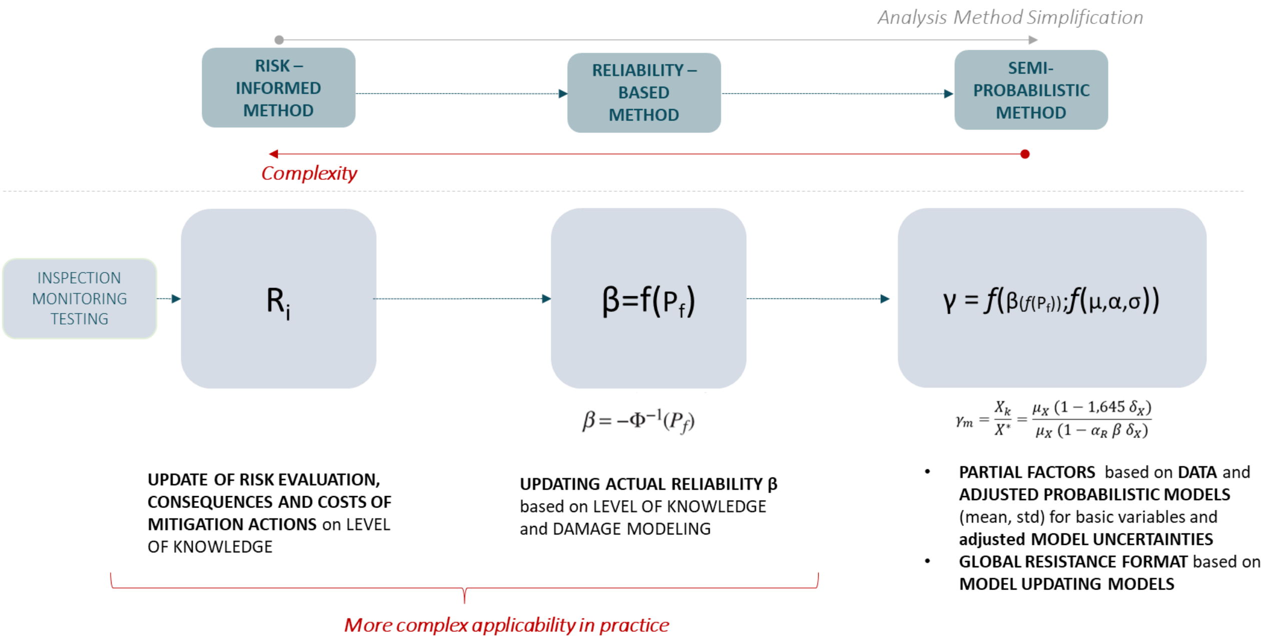

Figure 8.7 - Data-informed safety assessment methods

The most accurate way of assessment would be to explicitly consider updated load and strength variables applying reliability methods or risk-based decision procedures. However, such methods and procedures are time-consuming, calling for a specific operational knowledge of probabilistic methods, and are preferably used in special cases, such as:

- for strategic structures
- in case of uncertainties outside the usual ranges;
- in cases of severe failure consequences or insufficient robustness;
- for decisions regarding a whole group of similar structures (e.g. calibration of partial factors);

To verify if existing structures fulfil the reliability requirements for all assessment situations, the semi-probabilistic partial factor method is normally used via updating the characteristic values of the basic variables and partial factors based on updated information or referring to updated FE nonlinear models in global factor approaches.
As part of the updating procedure, deterioration due to environmental influences, repeated actions or use-induced wear has to be taken into account, as well as its cumulative process that can adversely affect the reliability of existing structures. When assessing existing structures, the simplifications of neglecting the influence of deterioration (assumed for the design of new structures) is not appropriate as these are already affected by damage mechanisms. Reliability requirements should be verified for the combined effects of cumulative deterioration and the relevant actions likely to occur during the remaining service life. Models should explicitly take into account the effects of deterioration on the resistance, including a quantification of all relevant material-specific associated uncertainties. Models should also be developed to describe the propagation of deterioration as a function of time, with the aim of predicting the condition of an existing structure over the remaining service life, going out from its actual condition at the time of assessment. Depending on the conditions to which the structure is exposed (e.g. environmental influences, repeated actions), these models should describe the onset and the rate of the cumulative processes that affect the parameters influencing the remaining structural resistance. The spatial distribution of the processes should be accounted for if relevant. The uncertainties associated with the models that describe the propagation of deterioration as a function of time should be taken into

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account and may be reduced by implementing structural health monitoring techniques to provide information about environmental influences on the structure, degradation processes or structural performance and their variation over time.

8.2 Use of data in performance verifications

8.2.1 Types of data and their relationship with the parameters used in performance verification models

Locations where inspection, testing and condition monitoring activities are to be undertaken must be carefully selected so that the desired information about the deterioration of materials and/or structural performance can be obtained, keeping in mind factors such as:

- the likely mechanism(s) and rate of deterioration;
- the environmental conditions;
- the conservation strategy and tactics;
- the inspection testing and monitoring regimes defined at the time of design or redesign.

The identification of vulnerable elements/zones for different structural typologies, or for specific structures, provides a good guidance to select the inspections locations or the monitoring areas of major interest, so that the structural adverse response during the occurrence of a hazardous event can be efficiently analysed (see section 2.3).

Data may be gathered from the structure for condition control purposes by a variety of techniques which are used for undertaking inspections, measurements, testing and monitoring activities. Tools and techniques for inspection, testing and monitoring could involve a wide range of procedures. Typically, they are likely to include a combination of visual observations, material sampling and possibly selected non-destructive and non-invasive testing methods as well as the installation of IoT sensors for continuous monitoring. Gathering the data necessary to establish the form and current condition of a structure may involve numerous techniques including inspections, measurements, sampling, and various forms of local condition and global response testing.

Information acquired by inspection, testing and monitoring can greatly improve the accuracy of performance prediction by more precisely assessing the variability of the input parameters, which are typically assumed to be random variables.

The aim of the verification assisted by testing is to obtain design values for the parameters governing the response of structural members and structural systems under specified load conditions with respect to a certain limit state. Verification assisted by testing is considered as a procedure where loading tests on limited series of representative specimens are used for the determination of the response of structural members or structural systems.

8.2.2 Use of data in risk-based methods

Risk management, and by assessment risk assessment methods is dependent on data. All methods as described in Chapter 7 require data in order to identify risks, hazards, causes, consequences and/or to quantify probabilities and consequences. As a consequence, all risk assessments are ‘data-informed’. Thus it is important to be more specific about the use of data in risk management and risk assessment methods. One way of doing so is to make a distinction between:

- The purpose of the data, what is the data use for?
- The type of data being used, what does the data represent?
- Sources of data, where does the data come from?

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8.2.2.1 Purposes of data

When a quantitative approach to risk management (and assessment by extension) is undertaken, this requires quantitative description (data) of the performance of elements of the system, which is being modelled. The main variables of the performance of system and its elements are according to (Stewart & Melchers, 1997):

- Reliability of system elements;
- Resistance capacity of the system elements;
- Loads and stresses, which arise from hazards,
- Undesirable consequences of failure

Specifically, reliability (and failure) data are of interest because they provide the analyst information with regards to (target) reliability levels used in (risk-based) performance assessment methods. Increased level of knowledge additionally provides a better understanding of risk and costs related to risk management.

8.2.2.2 Types of reliability and failure data

The data used for the risk assessment depends on the characteristics of the system in consideration. (Stewart & Melchers, 1997) specify that at least on one of the following reliability data is needed:

- Overall failure rates;
- Failure rates in individual failure modes;
- Variation of failure rates with time;
- Unavailability in terms of demand;
- Repair times.

8.2.2.3 Source of failure data

Failure data for system elements and their components are generally of two sources (Bello, 1987):

- Experimental data;
- Expert opinion.

Based on experimentally collected failure data, a statistical analysis can be performed in order to calculate the average failure rate λ(t), which is usually the reliability measure most interesting to the analyst (Stewart & Melchers, 1997)

$$\lambda(t) = \frac{n}{t_L S} \quad [8\text{-}2]$$

where:

- n number of element failures observed in a given time interval;
- t_L operational time;
- S number of elements at the start of a given time interval.

In order to describe the average failure rate, experimental data for n, t_L, and S need to be assembled, which can be done based on laboratory testing, field data or from (historical) incident data.

Data based on expert opinion can be of qualitative or quantitative nature and can be obtained from infrastructure operators, maintenance staff, management, and others. Typically, the identified experts are asked to express their opinion with regards to the average failure rate and/or to estimate the range of failure rates (Stewart & Melchers, 1997). Since such data depends on the experience, knowledge, and ability to make judgment and convey opinions of people consulted, data based on expert opinion can be heavily subjective. Therefore, a statistical analysis may be performed to derive the point estimates or probability distributions of, for example, individual failure rates. Eventual qualitative data can be processed into

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quantitative information for later calculations by aggregating the expert opinions in order to achieve a sufficient level of consensus – this can be done, for example, by using the Delphi technique. This, along with other methods for collecting expert opinions, is described in detail in Chapter 7.

8.2.3 Use of data in reliability-based methods

As already introduced in section 8.1, (fib MC2020, 2022) recommends the principles of probabilistic structural limit state design with a possibility for differentiating the target reliability levels for new and existing structures promoting the **annual approach** and suggesting three levels of target reliability:

- β_{new} - level indicating desired reliability for design of new structures;
- β_0 - level below which the existing structure is considered unreliable and should be upgraded;
- β_{up} - level indicating an optimum upgrade strategy while upgrading of existing structures.

Recommended target reliability levels for structural design (ULS)			
Annual target β-values for structures to be designed, based on economic optimisation			
Relative cost of safety measure	Consequence Class		
	CC1	CC2	CC3
Large (A)	3.1	3.3	3.7
Normal (B)	3.7	**4.2**	4.4
Small (C)	4.2	4.4	4.7
Informative target reliability indices β for structures to be designed, related to a 50-year reference period			
Relative cost of safety measure	CC1	CC2	CC3
Normal (B)	3.3	**3.8**	4.3
Recommended annual target reliability levels for assessment of existing structures (ULS)			
Relative cost of safety measure	CC1	CC2	CC3
Large (A)	3.1	3.3	3.7
Recommended target reliability levels for upgrade of existing structures (ULS)			
While slightly lower values can be normally justified for β_{up}-levels in comparison to design target levels, it is common and reasonable to require the compliance with the design levels when upgrading the structure.			

Figure 8.8 - Recommended target reliability levels for concrete structures (Bigaj-van-Vliet, 2021)

The use of β_1 instead of β_{50} should be accompanied by the change of sensitivity factors, but, in most design situations, the design values obtained considering either β_1 or β_{50} values recommended differ insignificantly provided that the appropriate values of the sensitivity factors α_R and α_E are taken into account. Hence, though values given in 6.2.3.2.2 may not be accurate, it is possible to conservatively assume that the small differences do not affect the average reliability level achieved by the Level II method (updated partial factors) or Level III method (probabilistic approach).

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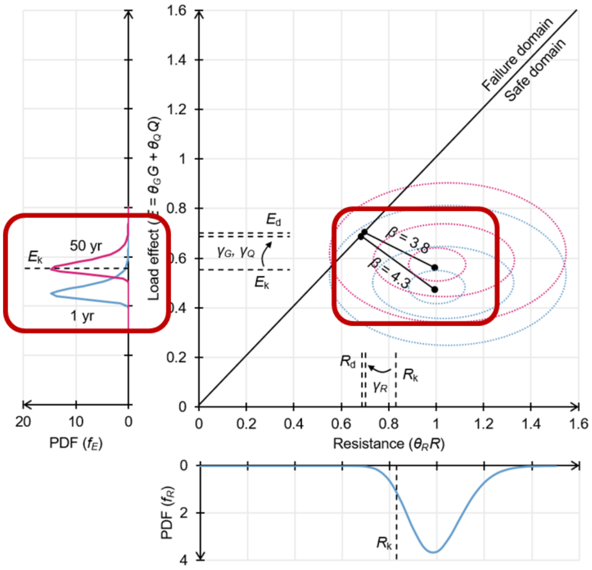

Parameter type	Influence	Coefficient
Resistance	Leading	0.8
	Accompanying	0.4 · 0.8 = 0.[illegible]2
Load effect	Leading	-0.7
	Accompanying	-0.4 · 0.7 = -0.28

Parameter type	Influence	Coefficient
Resistance	Leading	0.7
	Accompanying	0.4 · 0.7 = 0.28
Load effect	Leading	-0.8
	Accompanying	-0.4 · 0.8 = -0.32

Figure 8.9 - Example of update of sensitivity factors in case of change from β_{50} to β_1., extracted from (de Vries & Steenbergen, 2019).

The default case for structural design is CC2 and Normal (B) relative cost of safety measure. Normal (B) relative cost of safety measure is associated with medium variabilities of the yearly extreme of the total loads and resistances and normal service life duration of 30-70 years.
β-values depend on many parameters affected by the economy of a country including failure consequences (e.g. societal consequences and impact to the economy of a region affected by the failure), relative costs of safety measure, obsolescence and interest rates. All consequences related to structural collapse, failure or malfunction should be considered including for instance persons affected indirectly in the case of emergency situations. β-levels are, thus, commonly adjusted considering the level of development of a country. The target reliability indices β provided in Figure 8.9 are indicative for developed countries. For countries under development, lower β-values may be justified considering relevant socio-economic parameters, using either a cost-benefit optimization or, for instance, the Life Quality Index approach provided in (ISO-2394, 2015).

The assessment of existing structures essentially requires specifying a minimum target reliability β_0 below which the structure should be upgraded (or another safety measure should be taken) and the target level for structural upgrade β_{up}.

β_0 values may be based on the informative annual target reliability indices given in Figure 8.10 for Large (A) relative cost of safety measure. For existing structures with multiple, equally important failure modes, higher level of reliability per structural member should be considered.

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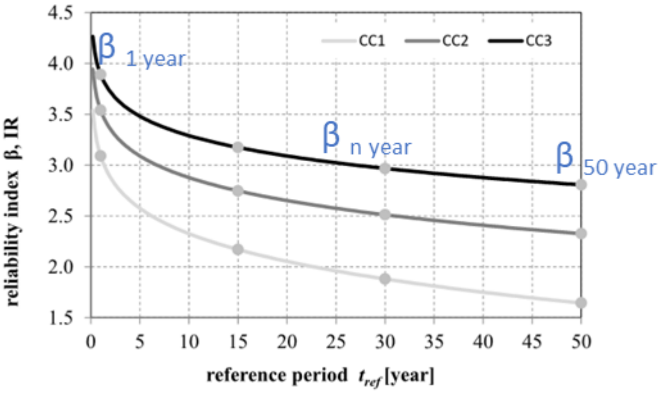

Figure 8.10 - Reliability index resulting from individual risk criterion for buildings (fib Bulletin 80, 2016)

In general terms, according to (Holicky, 2009), the reliability index for a period of n years may be then calculated from the following approximate equation:

$$\Phi\left(\beta_{t,n}\right) = \left[\Phi(\beta_{t,1})\right]^n \quad [8\text{-}3]$$

where:

- n indicates the number of years in which the failure events are supposed to be independent.

For instance, if the target reliability level of a structure is specified by $\beta_{50} = 3.8$ for the design working life $T = 50$ years, then it could be conservatively verified using the reference period $T = 1$ year and $\beta_1 = 4.7$. The higher the period of years n considered, the more conservative the approach is, as the required reliability index β increases with low t_{ref}.

It should be emphasised that both values β_{50} and β_1 correspond to the same reliability level, but to different reference periods considered for the assessment of the design values of some actions (1 and 50 years). Based on the information from the design phase or from in-depth investigation, the consequence class can be identified and therefore the target reliability value β can be determined.

The effect of climate change can also be included in the evaluation of the reliability. A general methodology to evaluate the climate change effects on relevant climatic actions and the associated impact on structural reliability is presented in (Croce, et al., 2019), based on the analysis of observed data series and climate models' projections. By taking into account the non-stationary nature of climatic actions linked to climate change, the time-dependent structural reliability is evaluated based on the predicted changes in mean load intensity and standard deviation of annual maxima of the relevant loads.

Climate change, as a result, could cause a more significant reduction over time of the structural reliability than what expected in case of stationary climate conditions, due to resistance degradation effects. To maintain the required reliability level, climatic actions would need some adaptation to take into account the effect of climate change.

8.2.4 Use of data in semi-probabilistic methods

As introduced in (8.1), the data-informed semi-probabilistic methods favour from the additional level of approximation via:

- updating the characteristic values of the basic variables (standard deviation and mean values of the variables distribution) and partial factors based on updated information,

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- performing a model updating procedure of an FE nonlinear model based on the results of monitoring and diagnostics load testing and verifying with global factor approach.

Details regarding the updating of variables distribution or FE models are given in 8.2.6. The verification should follow a "Levels of Approximation Approach" (LoAA) combined with a "Levels of Knowledge Approach" (LoAKA), where the upper level should be selected depending on the significance of the uncertainties and on the ratio between costs of interventions and investigations. In this context, the choice of the parameters to be refined by site investigations or analysis should also be based on an estimate of the relationship between the uncertainties and the sensitivity of these parameters to the final result. The verification of an existing structure should start from a global condition assessment, where the site investigation should play a major role.

Example:

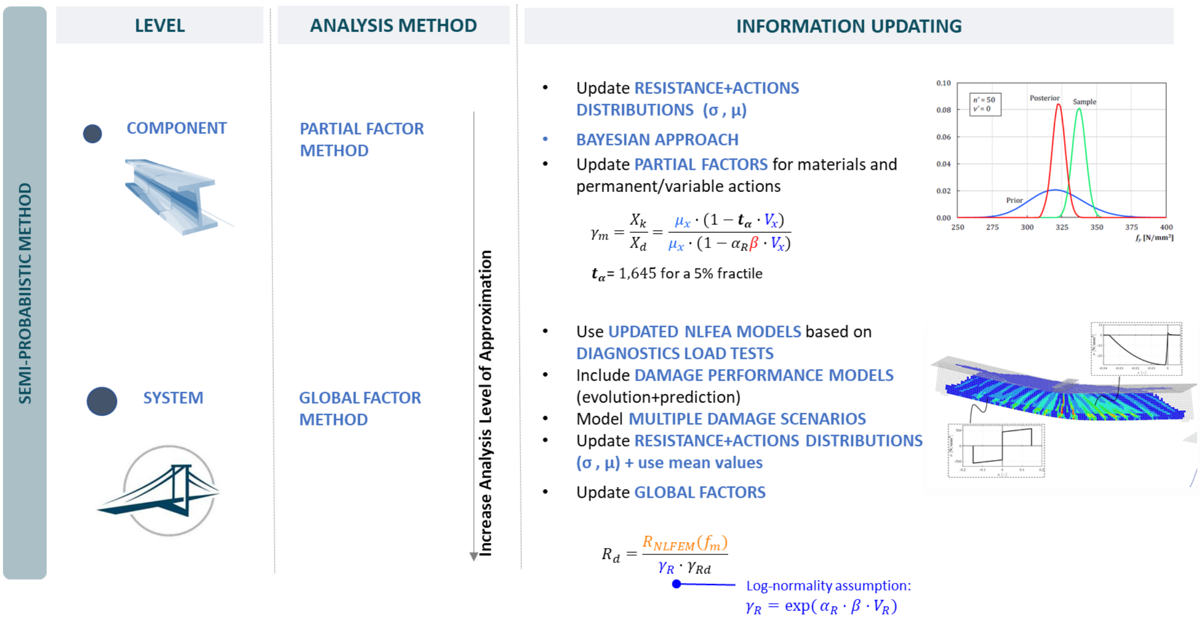

Figure 8.11 - Example of assessment methods and information updating procedures.

8.2.4.1.1 Design value approach for assessment

As outlined in section 6.2.3.2.2, the design value method takes basis in a direct check of the relevant design situations and corresponding design equations using design values for the basic variable which are determined on the basis of reliability assessments.

Partial factors for materials, resistance and loads must be updated for existing structures: general procedures for the assessment of design values and assigned target reliability levels and remaining working lives are described. These procedures, also described in (fib Bulletin 80, 2016), are based on the Bayesian approach: the evaluation of new factors relies on the updating of the resistance and actions distributions on the basis of gathered data and the related uncertainties, as described in the following paragraphs.

8.2.4.1.2 Material factor γ_M

Assuming a lognormal distribution for the material property, the variation of the partial factor γ_m obtained from Eq. (6-37) with the coefficient of variation δm is shown in Figure 8.12 for α_R = 0,8 and target reliabilities β = 2,3; 3,1; 3,8 or 4,3 (*very low, low, medium* and *high failure* consequences in ULS (Ultimate Limit State), respectively, according to (ISO-13822, 2010) for

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the assessment of existing structures). Further, the partial factor γ_{Rd} can be obtained from the following relationship based on a lognormal distribution:

$$\gamma_{Rd} = \frac{1}{\left[\left(\frac{\mu_{\theta R}}{\theta_{Rk}}\right) exp(-\alpha_R\, \beta\, \delta_{\theta R})\right]} \tag{8-4}$$

where:

- $\left(\frac{\mu_{\theta R}}{\theta_{Rk}}\right)$ denotes the ratio of the mean to the characteristic (nominal) value of the model uncertainty in resistance θ_R (bias) and $\delta_{\theta R}$ is the coefficient of variation of θ_R . For instance, when assuming $\mu_{\theta R} = 1$ and $\delta_\theta = 0{,}05$, $\gamma_{Rd} = 1{,}05$ might be accepted for a range of β from 2,3 to 4,3.

A significant bias and/or larger coefficients of variation may apply e.g. for resistances based on nonlinear analyses or shear resistances assessed by simplified EN 1992-1-1 models. Calculating γ_m according to Eq. (6-37) (or Figure 8.12 -) and γ_{Rd} according to Eq. (8-4), the partial factor for material properties when assessing existing structures is obtained as $\gamma_M = \gamma_{Rd}\, \gamma_m$.

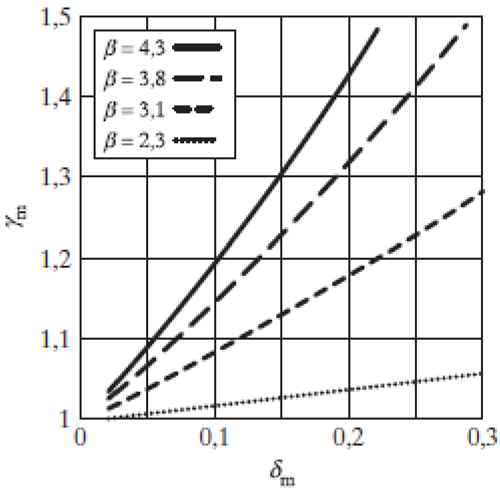

Figure 8.12 - Variation of the partial factor γ_m with the coefficient of variation δ_m for α_R = 0,8 and various target reliabilities (Caspeele, et al., 2013)

8.2.4.1.3 Permanent action factor γ_G

The partial factor for permanent actions can be written as:

$$\gamma_G = \gamma_{Ed}\, \gamma_g \tag{8-5}$$

where:

- γ_g is the reliability-based partial factor accounting for variability of the permanent action and statistical uncertainty. The model uncertainty factor γ_{Ed} can be obtained from the following relationship based on a lognormal distribution:

$$\gamma_{Ed} = exp(-\alpha_E\, \beta\, \delta_{\theta E}) \tag{8-6}$$

where:

- $\delta_{\theta E}$ denotes the coefficient of variation of the model uncertainty θ_E of the load effect. Usually, no bias is assumed in this case. For a range of β from 2,3 to 4,3 and unfavourable effects, the following model uncertainty factors might be accepted:
- $\gamma_{Ed} \approx 1{,}05$ for $\delta_\theta = 0{,}05$ (axial forces),
- $\gamma_{Ed} \approx 1{,}10$ for $\delta_\theta = 0{,}10$ (bending moments, shear forces).

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Assuming a normal distribution for the effect of the permanent action, variation of the partial factor γ_g obtained from Eq. (6-38) with the coefficient of variation δ_G is shown in Figure 8.13 for $\alpha_E = -0{,}7$ (unfavourable action) and different specified target reliabilities.

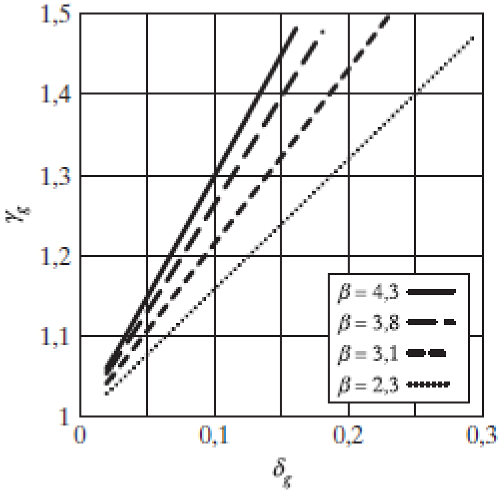

Figure 8.13 - Variation of the partial factor γ_g with the coefficient of variation δ_g for α_R = -0,7 and various target reliabilities (Caspeele, et al., 2013)

8.2.4.1.4 Variable action factors γ_Q

When site- or structure-specific data on variable loads can be gathered and a detailed assessment needs to be made, partial factors for variable loads may be derived using the design value approach. Similarly, as for permanent actions, the partial factor for variable actions can be obtained as follows:

$$\gamma_Q = \gamma_{Ed}\,\gamma_q \qquad [8\text{-}7]$$

where:

- γ_q is the reliability-based partial factor accounting for variability of the variable action and statistical uncertainty. The model uncertainty factor γ_{Ed} is obtained in the same way as for the permanent actions.

The partial factor γ_q should be derived based on maxima of the variable load Q_{tref} during the reference period t_{ref}, considering the sensitivity factor α_E and target reliability index β. In accordance with (CEN-EN 1990, 1990:2006)the load maxima Q_{tref} should be based on the same reference period as used for the reliability index β. In general, the variable load depends on the time-variant component $Q_0(t)$ and on the time-invariant component C_0.
In most cases the maxima of the variable load related to t_{ref} can be obtained as a product of both components:

$$Q_{tref} = C_0 \max_{tref}[Q_0(t)] = C_0\, Q_{0,tref} \qquad [8\text{-}8]$$

Assuming a Gumbel distribution for the time-variant component, the mean of $Q_{0,tref}$ is obtained as:

$$\mu_{Q0,tref} = \mu_{Q0} + 0{,}78\,\sigma_{Q0}\, ln\left(\frac{t_{ref}}{t_0}\right) \qquad [8\text{-}9]$$

where:

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

$$\mu_{Q,tref} \approx \mu_{C0}\, \mu_{Q0,tref}\, \delta_{Q,tref} = \sqrt{\left(\delta_{C0}^2 + \delta_{Q0,tref}^2 + \delta_{C0}^2 \delta_{Q0,tref}^2\right)} \qquad [8\text{-}10]$$

where:

$$\delta_{Q0,tref} = \sigma_{Q0} / \mu_{Q0,tref}. \qquad [8\text{-}11]$$

Consequently, the partial factor is approximated as:

$$\gamma_q \approx \left(\frac{\mu_{Q,tref}}{Q_k}\right) \left[1 - \delta_{Q,tref}\left(0{,}45 + 0{,}78\, ln\left(-ln(-\alpha_E\, \beta)\right)\right)\right] \qquad [8\text{-}12]$$

where:

- Q_k is the characteristic value applied in the assessment.

For further information see (fib Bulletin 80, 2016).

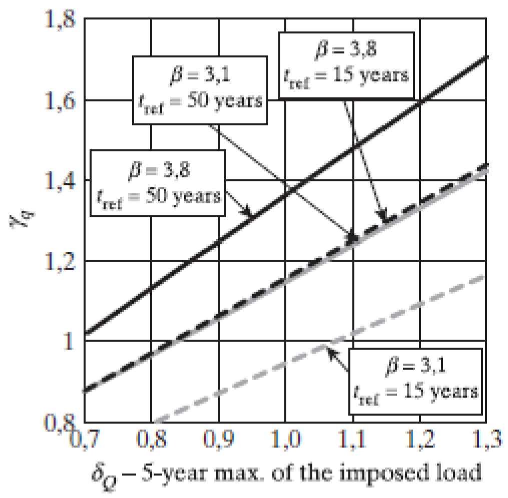

Figure 8.14 - Variation of the partial factor γ_q for the imposed load with the coefficient of variation of 5-year maxima (Caspeele, et al., 2013)

8.2.4.2 Adjusted partial factors for assessment

As outlined in section 6.2.3.2.3 the basic philosophy of the APFM consists of calculating adjusted partial factors γ_X^{exist} for variables X of an existing structure.

$$\boxed{\gamma_X = \omega_\gamma \cdot \gamma_{X,New}} \qquad [8\text{-}13]$$

PARTIAL FACTOR FOR EXISTING STRUCTURES
ADJUSTMENT FACTOR
PARTIAL FACTOR FOR NEW STRUCTURES

Partial factors for materials, resistance and loads must be updated for existing structures. The procedures are described in (fib Bulletin 80, 2016) and are based on the Bayesian approach: the evaluation of new factors and characteristic values relies on the updating of the resistance and actions distributions on the basis of gathered data and the related uncertainties.

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8.2.4.3 Global safety factors for assessment

As outlined in section 6.2.4, Global Resistance Factor Method treats the different sources of uncertainties related to structural behaviour by means of appropriate global safety factors to define the design global resistance of the structure. In the Global Factor Method, the design capacity is obtained as the global capacity, estimated by performing a NLFEA (Non Linear Finite Element Analysis) with the mean values of the material parameters affecting the local behaviour, divided by the product of the global factor and the global factor for modelling uncertainty.

The design criterion is expressed in general terms as:

$$E_d \leq R_d \qquad [8\text{-}14]$$

where:

$$R_d = \frac{R_m}{\gamma_R \, \gamma_{R_d}} \qquad [8\text{-}15]$$

E_d is the design value of action;

- γ_R is the global safety factor for mean resistance;
- γ_{R_d} is a model uncertainty factor.

where it is assumed that the NLFEA run with mean values of the basic variables gives the mean value of the capacity. Mean values R_m and global factors γ_R can be evaluated on the basis of the updated model with resistance and actions distributions.

In assessment of existing structures by means of a global safety factor approach, updated NLFEA models based on diagnostic tests are used (see section 8.2.6.3).

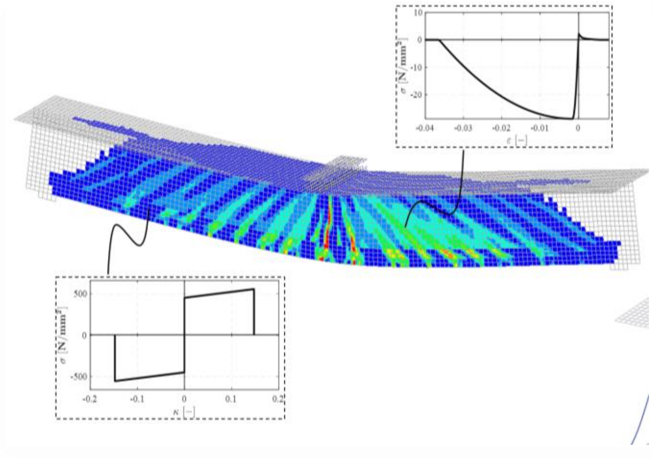

Figure 8.15 - Example of NLFEA models evaluation (Bigaj-van-Vliet, 2021)

In this respect, it is possible to distinguish between complex and simple structural schemes. Based on the complexity of the structural scheme, it is possible to have two different approaches, which are characterized by different Levels of Approximation (LoA): design and assessment of existing structures can be carried out with different levels of detail, on the basis of the level of accuracy required to fully describe the structural response. In case of complex structural schemes, performance monitoring and a higher level of approximation of the analysis are needed, implying numerical modelling, model updating processes and deeper analysis of the presence and localization of damages. Simple structural schemes, or cases

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where a detailed NLFEA analysis process is not applicable, may comprise monitoring of the structural performance.

This approach allows to include damage performance models and to consider multiple damage scenarios, as well as calibrate the structural condition at the beginning of the monitoring period in order to evaluate thresholds levels for damage progression control (UNI/TR11634, 2016). The model, calibrated and updated, using sensors measurement, can be used to explore several critical scenarios that are representative of the limit conditions for which the thresholds must be defined. In the practical applications, it is envisaged to define at least two categories of thresholds; a first class corresponding to a serviceability limit, within the required safety levels and a second corresponding to an unacceptable risk for human life (fib Bulletin TG3.3, 2021).

The attainment of the global limit state of interest can be governed either by multiple concurrent local failure mechanisms or by a single one. For the case of multiple concurrent local failure mechanisms, a distinction is made between the limiting cases of non-correlated and fully correlated mechanisms.
Typical application areas of NLFEA for concrete structures are those where simplified analytical models fail to accurately predict structural resistances, e.g. where non-linear behaviour related to material properties and/or geometry significantly affects structural reliability or where load effects are difficult to assess due to interaction of structural members or dynamic nature of the phenomenon. Specific examples include:

- Quantifying the inelastic deformation capacity of a concrete structure for quasi-static push-over loading,
- Determination of the distribution of action effects in a structure for further capacity control of cross-sections, when significant concrete cracking and/or reinforcement yielding influences the load paths,
- Assessing the seismic capacity of an existing structure under earthquake action,
- Assessing the shear capacity of an existing structure with outdated reinforcement detailing,
- Assessing the activation of a second bearing path in a design of a bridge,
- Assessing the risk of thermal cracks in massive concrete structures such as dams, foundations and tunnels, during the early hardening process in concrete, due to restraints and uneven temperatures,
- Assessing the bearing capacity of a tunnel section subject to fire and temperature dependent material deterioration,
- Assessing the failure mode and capacity of disturbed regions with complex detailing and boundary conditions, such as anchorage zones, subjected to concentrated loads,
- Assessing the residual capacity of corroded structures with deteriorated steel and concrete properties due to carbonation and/or chloride penetration.

8.2.5 Use of data in updating of performance models and model parameters

In civil engineering, numerical or mathematical models are used to simulate the behaviour of real systems, with the purpose of performing analysis, prediction and design. In case of use of performance models for structural health monitoring purposes, it is essential to refer to calibrated models through parameter identification or estimation using model updating techniques. Model updating generally aims to calibrate unknown system properties which appear as parameters in numerical models, based on actually observed behaviour of the system of interest.

Observations collected from inspection, monitoring and testing can be subdivided into two categories:

- **Direct information**: are those relevant properties used in the assessment verification that can be directly measured

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- **Indirect information**: are those relevant to properties that cannot be directly measured, for which it is necessary to define a specific indicator and a model is necessary to connect the indicator to the relevant property ('Performance Model - (COST Action TU1406 , 2015-2019).

The information updating process may refer to classical statistical methods or to a Bayesian approach:

- **Classical statistical methods**: The classical methods are based on estimating some statistics of the parameters (e.g. mean values and/or covariance matrices) so that the statistics of the output of the model correspond in some optimal way to the statistics of the observed data. This can be done analytically (i.e. classical statistical inference methods) or numerically (i.e. stochastic model inversion) (Y. Govers, April 2010) (Beck & Arnold, 1977) .
- **Bayesian approach**: The Bayesian inference approach was introduced in structural dynamics by Beck et al. (Beck & Katafygiotis, 1998) as a method of statistical inference in which Bayes' theorem is used to update the probability for a hypothesis as more evidence or information becomes available. Using the Bayes' theorem, prior probabilistic models, which are constructed based on the a priori available information, can be transformed into posterior models, using the available experimental data and the probabilistic prediction error model. The Bayesian approach is particularly suited for inverse problems.

Type of information, moreover, can be differentiated between:

- **Equality type information,** corresponding to measured variables;
- **Inequality type information**, which is carried with a measurement that some variable is greater than or less than some predefined limit.

	direct	indirect
equality	load measurment	chloride content
inequality	proof-loading	status inspections

Figure 8.16 - Examples for different types of information (Extract from D1.1)

Some examples are given in Figure 8.16.

The information updating approaches and the use of data in updating performance models and relevant parameters are detailed below.

Observations acquired by inspection and monitoring inform about the safety of the structure and can be explicitly utilized to update the reliability analysis.

8.2.6 Information updating approaches

Given the result of an investigation, there is a requirement to update the properties and the reliability estimates of the structures.

Three different methods can be distinguished:

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a) Updating of the (multivariate) probability distribution of the basic random variables
b) Direct updating of the structural failure probability;
c) Model updating.

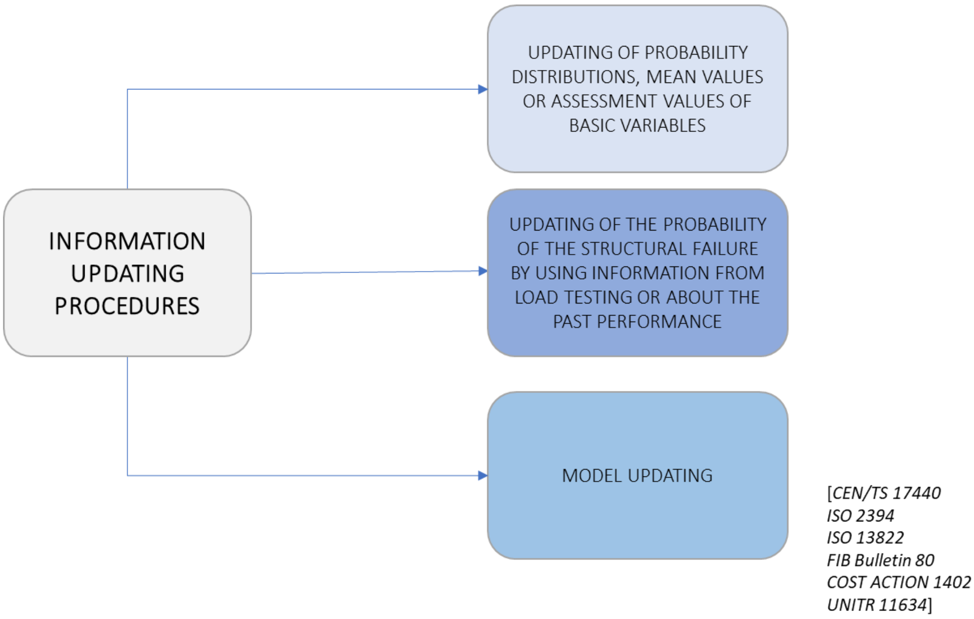

Figure 8.17 - Information updating procedures.

8.2.6.1 Basic variables updating

The calculation model for each limit state considered contain a specified set of basic variables, i.e. physical quantities which characterize actions and environmental influences, material and soil properties and geometrical quantities. The model also contains model parameters which characterize the model itself and which are treated as basic variables.

Finally, there are also parameters which describe the requirements (e.g. serviceability constraints) and which may be treated as basic variables. The basic variables are assumed to carry the entire input information to the calculation model.

The basic variables may be random variables (including the special case deterministic variables) or stochastic processes or random fields. Each basic variable is defined by a number of parameters such as mean, standard deviation, parameters determining the correlation structure etc.

The basic variables needed for an assessment include:

- geometrical data;
- actions and environmental influences;
- material and product properties.

The values for basic variables may in the first instance be based on the available prior information derived from:

- the original design documents;
- the codes and standards from the time of the construction;
- manufacturers' data and product literature.

The values for basic variables should take account of the observed condition of the structure and other information obtained in the condition survey (Bayesian approach). The values and

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distributions for basic variables may be updated from the prior information based on site-specific data obtained from:

- samples of measured values for basic variables for the structure;
- measurements of the performance of the structure.

Structure-specific characteristic values for basic variables may be based on either:

- structure-specific sample data;
- structure-specific sample data combined with known statistical parameters describing the distribution and / or variability of basic variables derived from a population of comparable structures.

Where data are obtained from the structure by sampling, the methodology of the data acquisition should be developed based on:

- investigation of variables that are significant for the assessment;
- sampling from locations that are representative for the members being assessed (e.g. vulnerable zones - see section 2.3);
- a sample size that provides a statistically significant and representative basis for updating parameters;
- sampling methods that can be carried out safely;
- repair of the structure following any removal of materials for sampling;
- testing arrangements that provide representative data for the assessment.

The values for basic variables should be consistent with the reference period for the assessment.

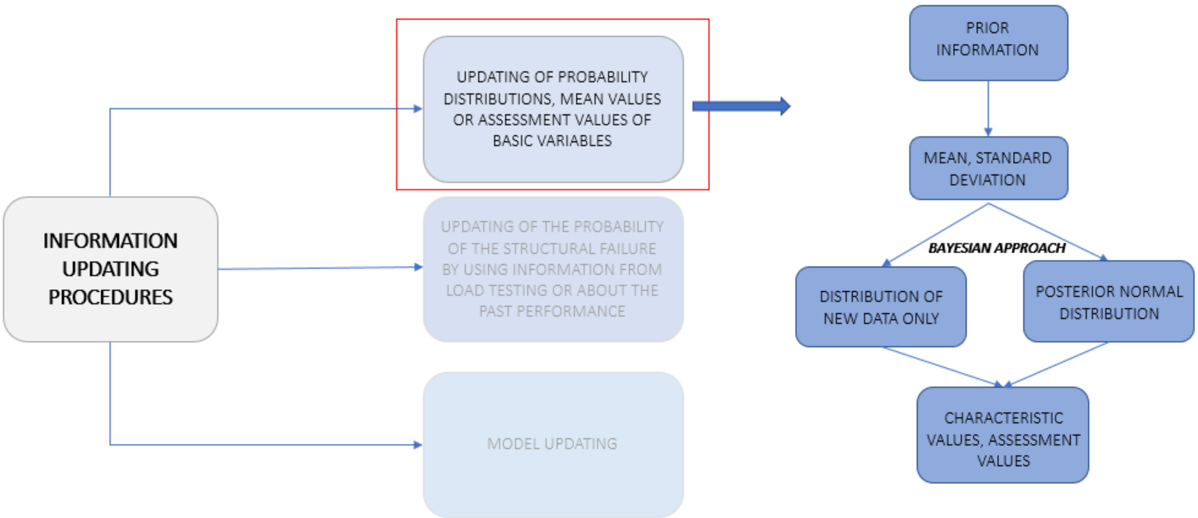

Figure 8.18 - Basic variables updating

8.2.6.1.1 Prior information

When updating the probability distribution function of a basic variable X, its distribution parameters (the mean, standard deviation, coefficient of skewness, lower bound, etc.) may be considered as random variables. Prior distribution functions for the unknown parameters of the investigated variable should reflect all the available prior information.

Given such prior distributions and statistical data from observations, posterior distributions can be derived. The prior information about the standard deviation σ should be characterized by the parameters s' (prior standard deviation) and ν' (prior number of degrees of freedom for

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s'). Where ν' is large, the expectation E and the coefficient of variation V of the standard deviation σ may be expressed by the following expressions:

$$E(\sigma) = s' \quad [8\text{-}16]$$
$$V(\sigma) = \frac{1}{\sqrt{2\nu'}} \quad [8\text{-}17]$$

The prior information can be interpreted as the result of hypothetical prior test series for the mean and standard deviation, respectively. In that case, the information about the standard deviation is given by:

- s' hypothetical sample value;
- ν' hypothetical number of degrees of freedom for s'.

The prior parameter s' represents the best estimate for the standard deviation. Through the choice of ν', the uncertainty with respect to these estimates can be expressed.

The prior information about the mean value μ should be characterized by the parameters m' (prior mean), n' (prior number of observations) and s'. For large n', the expectation E and the coefficient of variation V of the mean value μ may be expressed by the following expressions:

$$E(\mu) = m' \quad [8\text{-}18]$$
$$V(\mu) = \frac{s'}{m'\sqrt{n'}} \quad [8\text{-}19]$$

Compared to the standard deviation, the information about the mean value requires two additional parameters:

- m' hypothetical sample average;
- n' hypothetical number of observations for m'.

The prior parameter m' represents the best estimates for the mean value. Through the choice of n', the uncertainty with respect to these estimates can be expressed. The hypothetical prior parameter n' can be chosen independently from ν' (in general, $\nu' \neq n'$ -1).

Well known estimates for the prior parameters (i.e. low values for the coefficients of variation of the mean value and the standard deviation, respectively) should be chosen only if they are justified.

8.2.6.1.2 Posterior normal distribution: Bayesian Approach

A posterior distribution can be generally determined based on the prior distribution and statistical test data with the following equation:

$$f''(\boldsymbol{q}) = C\, L(\bar{x}|\boldsymbol{q})\, f'(\boldsymbol{q})$$

where

- $f''(\boldsymbol{q})$ is the posterior distribution of $\boldsymbol{q}$;
- $f'(\boldsymbol{q})$ is the prior distribution of $\boldsymbol{q}$;
- $L(\bar{x}|\boldsymbol{q})$ is the likelihood function for the parameters $\boldsymbol{q}$ given the observations $\bar{x}$;
- $\boldsymbol{q}$ is the vector of distribution parameters;
- C is the normalizing constant.

Then, the updated distribution of the resistance (R) is:

$$f_R''(r) = \int f_R(R|\boldsymbol{q})\, f''(\boldsymbol{q})\, d\boldsymbol{q}$$

where

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- $f_R(R|\boldsymbol{q})$ is the distribution of R for given values of $\boldsymbol{q}$
- $f_R''(r)$ is the updated distribution of R. (ISO-2394, 2015) highlights that this distribution for R can be directly used in a probabilistic design procedure.

Combining the prior information characterized in 8.2.6.1.1 and new data constituted by *n* observations with sample mean *m* and sample standard deviation *s*, the posterior normal distribution for the unknown mean value and standard deviation of a variable may be obtained by the following updating rules:

$$n'' = n' + n \qquad [8\text{-}20]$$
$$\nu'' = \nu' + \nu + \delta(n') \qquad [8\text{-}21]$$
$$n''m'' = n'm' + n\,m \qquad [8\text{-}22]$$
$$\nu''(s'')^2 + n''(m'')^2 += \nu'(s')^2 + n'(m')^2 + \nu\, s^2 + nm^2 \qquad [8\text{-}23]$$

where

- m', m, m'' denote respectively the prior, sample and posterior mean values;
- s', s, s'' denote respectively the prior, sample and posterior standard deviations;
- n', n, n'' denote respectively the prior, sample and posterior numbers of observations;
- ν', ν, ν'' denote respectively the prior, sample and posterior numbers of degrees of freedom;

$$\delta(n') = 0\; for\; n' = 0 \qquad [8\text{-}24]$$
$$\delta(n') = 1\; for\; n' > 0 \qquad [8\text{-}25]$$
$$\nu = n - 1 \qquad [8\text{-}26]$$

These parameters can be used for the calculation of the prior distribution, assuming that R has a normal distribution:

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

The predictive value of R can be then found with the following equation:

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

where $t_{\nu''}$ has a central t-distribution. In this respect, (ISO-2394, 2015) also provides values of $t_{\nu''}$ for given probabilities of exceeding the limits.

8.2.6.1.3 Assessment values

Posterior (i.e. updated) distribution parameters for variables or the new data distribution may be used to determine the corresponding updated assessment values.

In general, the following assumptions are appropriate for most applications:

- a Gaussian distribution for permanent action effects,
- an extreme value distribution to represent a maximum value within a chosen reference time (e.g. effects due to variable or accidental actions),
- for dimensions and material properties, a Gaussian distribution or a log-normal distribution.

In the case of dimensions and material properties, the log-normal distribution can be used to avoid the misleading possibility of negative values associated with a Gaussian distribution.

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The choice of probability distribution functions should be made with caution, considering possible bias and skewness.

The coefficients of asymmetry and kurtosis can provide valuable information for determining the appropriate theoretical model (probability distribution function). If the actual distribution shows a multimodal character, the choice of one single distribution can lead to considerable error. The assessment values of normally, log-normally and Gumbel distributed random variables may be determined according to EN 1990, taking into account updated mean values and standard deviations of the corresponding variables, as well as appropriate sensitivity factors α and target reliability indices β_t.

8.2.6.1.4 Characteristic values

Alternatively, to the determination of assessment values, posterior distributions for variables X may be used to determine the corresponding updated characteristic values from which the assessment values can be obtained by applying the appropriate partial factors.

The updated characteristic values of normally, log-normally and Gumbel distributed random variables may be determined according to EN 1990, taking into account updated mean values and standard deviations of the corresponding variables and considering appropriate distribution fractiles.

8.2.6.1.5 Structural failure probability updating

The updating of the failure probability may be performed by method, using three approaches:

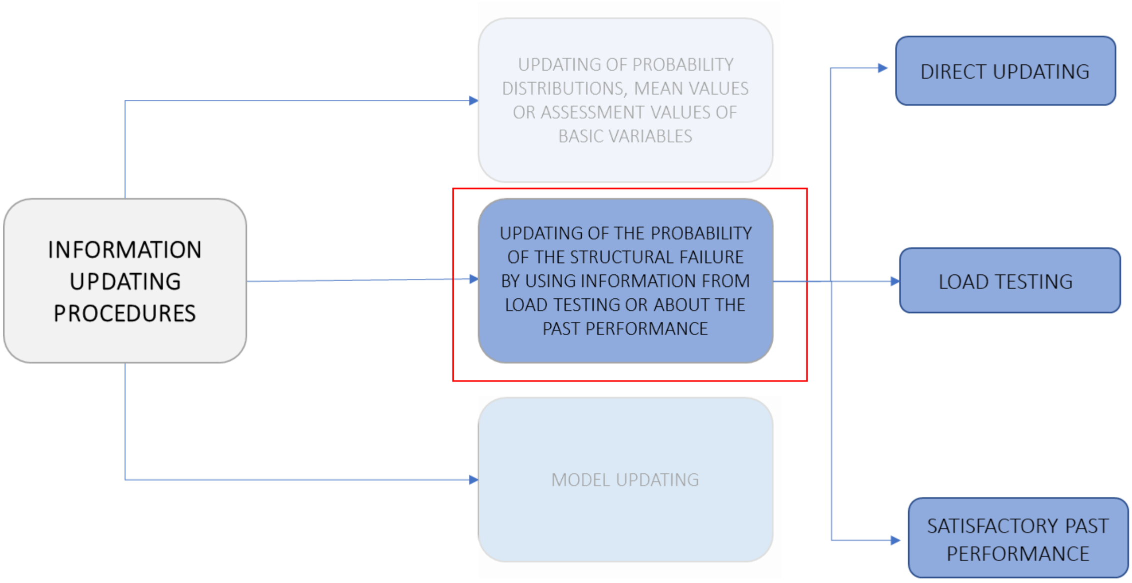

Figure 8.19 - Approaches to updating of the failure probability

8.2.6.1.6 Direct updating

Direct update of the structural failure probability by using new data may formally be carried out by using the basic relationship from probability theory:

$$P(F|I) = \frac{P(F \cap I)}{P(I)} \qquad [8\text{-}27]$$

where:

- F denotes a local or global structural failure;
- I denotes the inspection information;
- $\cap$ indicates the intersection of two events;

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- | indicates "conditional upon".

This procedure can be applied, for example, after the execution of a proof load test.

8.2.6.1.7 Load testing

(fib MC2020, 2022) recommends verification assisted by testing, inspection and monitoring with due consideration of load testing of structures as means of conformity evaluation.

According to (fib TG3.3, 2021), load test is an efficient and robust approach: it can prove that the structure's load bearing capacity is actually adequate, i.e. it can reveal its hidden capacity, and it always enables a check if the response of the structure is according to the objective of its design.

In this respect, it is possible to distinguish two types of load testing:

- **Proof loading**: focuses on improving the analytical assessment of an existing structure revealing the potential hidden safety reserve. It can give important information about the effective structural performance and its actual level of safety. It is defined as the assessment of a structure under a given limit state by applying an equivalent load,
- **Diagnostic load testing**: focuses on confirming the response of the structure against the service loads. Results from the model and the observed behaviour of the structure under a certain percentage of the design live loads are examined in order to verify the suitability of the design/analytical model. It can be static and/or dynamic.

(fib TG3.3, 2021) also highlights that comparison between results of two consecutive load tests offers additional information about the structure deterioration rate.

Main stages of load testing are represented in the figure below:

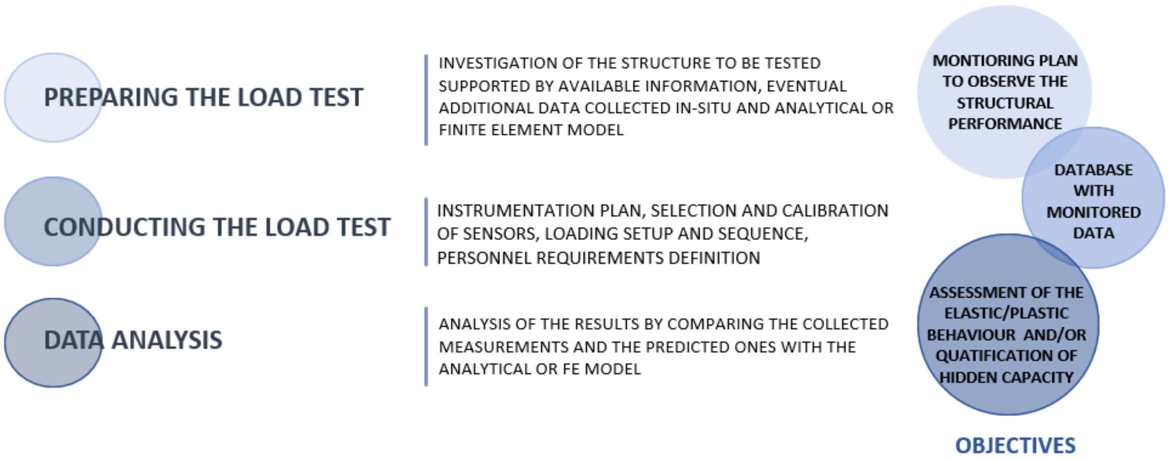

Figure 8.20 - Load testing stages and relative objectives

Proof loading and diagnostic load testing are described in the following paragraphs.

A load test may be performed in order to update the failure probability of a structure.
It should be noted, however, that the survival of a load test does not reveal the actual resistance of the tested structure or component, nor does it provide a direct measure of structural reliability, however, as well as study of track records, it may reveal implicitly a part of the hidden safety.

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For the updating of the failure probability of a structure on the basis of a known load, the original cumulative distribution function for structural resistance, used for that purpose, may be truncated at the level of the known load effect.

When applied in context of model updating (see section 8.2.5) the diagnostics load test aims primarily to verify the compliance of the numerical model with the real structural response by examining, on a comparative basis, the results from the model and the observed behaviour of the structure under a certain percentage of the design live loads.

The information obtained from load testing offers valuable insights regarding some of the parameters that influence the global behaviour of the structure, such as:

- Young's modulus of concrete based on the global indicators, mainly from measured deflections at mid-span sections and rotations of supporting sections, complementing the information collected from other methods to assess this parameter.
- Boundary conditions based on measurements collected by transducers installed near the bearing devices. Boundary conditions are usually simulated in an ideal scenario (e.g. simply supported, fixed), which might not be entirely accurate (e.g. malfunctioning of the bearings).
- Elastic/Plastic behaviour of the structure based on the analysis of any meaningful residual modification of a reference measurement before and after the load test that might be a sign of the existence of plastic behaviour of the structure.
- Crack opening patterns based on measurements collected from transducers installed directly on a set of critical cracks and the correlation between the applied load and the crack opening, to better model the effective cross-section area in fractured areas.

8.2.6.1.7.1 Proof loading

Proof loading is performed to check the ability of the structure to carry a specified variable load (e.g. design loads given in the codes). In a proof load testing, a load representative of the factored live load is applied to the structure. Since the load requirements are quite high, the desired effects are accomplished with special loading systems, such as heavy vehicles in the case of bridges, or specially designed load buckling etc. In any case, the test must be stopped if some abnormal event is observed and not identified in advance in the planning document.

(fib TG3.2, 2023) presents the approach that can be applied to reinforced or prestressed concrete structures or parts of those structures.

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A schematic representation of the strategy to be used during proof loading is given in Figure 8.21.

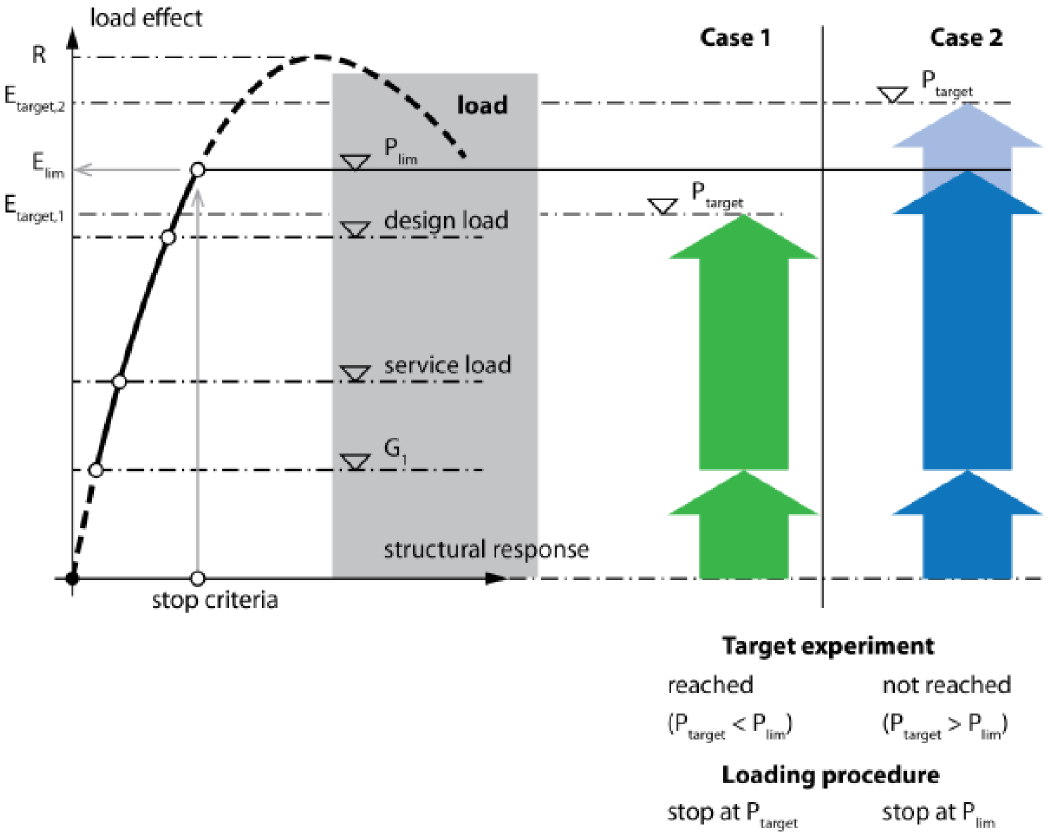

Figure 8.21 - Proof loading strategy (Extract from (fib TG3.2, 2023))

R is the unknown bearing resistance of the structure considered, which is subjected to a preceding judgement of the actual condition and then to an additional load. The effects, e.g. deformations, of the latter are measured. At the start of the test, the permanent loads are active with their real magnitude. During the test an additional external load P_{target} is applied on the structure. The sum of the load contributions G_1 (permanent load during the test), $G_{d\,j}$ (design value of the permanent load in addition to G_1 during the life time of the structure) and Q_d (design value of variable loads), which represent in combination the total load P_{target} may not exceed the limit load P_{lim} , for which the criteria are a function of the effect of the load P. For further information about proof loading and the definition of the loads mentioned above, see (fib TG3.2, 2023).

Proof loading reduces the model uncertainty of the resistance of the structure, and therefore provides the most robust evaluation of the safety of a structure. Moreover, results of proof load tests can be used in reliability-based approach to safety assessment, in particular for:

- updating of uncertainty on the resistance of the structure based on attained proof load level;
- updating of failure probability.

8.2.6.1.7.2 (Static / dynamic) diagnostic load testing

In diagnostic load testing a percentage of the ULS load effect is applied, approximately from 50 % to 60%, on the basis of the load combination defined in the codes.

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While live load models usually include distributed lane loads and design vehicle loads, for diagnostic load testing dead weights or loading vehicles, for instance water tanks or trucks/trains for building bridges, are commonly used.

(fib TG3.3, 2021) includes the following recommendations: *"the total load should remain in the structure until the measured parameters become stable in time. For simply supported structures the load should be applied span per span. For continuously structures, the loading arrangement should consider the overall structure behaviour. In this case, the loading will be carried out by placing trucks in contingent spans (aiming at maximum negative moments and maximum shear forces) and by loading alternate spans (aiming for maximum positive moments). Loading cases to study the transverse distribution (torsion) of the applied load can also be included in the load test protocol."*

Static diagnostic load testing
In the static diagnostic load testing, the load is positioned in each intermediate stage and the final stage long enough for the measures to stabilize. The load is applied following application systems (e.g. loading frames and hydraulic jacks). Test should be considered successful when the structure carries the applied load without signs of distress and therefore fulfils the code requirements.

Structural behaviour is controlled by the measurement parameters (e.g. deflections, strains etc.), which have to be constantly monitored and evaluated.
In case of nonlinear behaviour or signs of structural distress, (e.g. cracking of the concrete, yielding, signs of a loading scheme and held for a certain period. The duration of the test and the accuracy of the results will depend on the time the load should be held until stabilization of the outputs. It is important to have some criteria to decide when the stabilized value of the displacements is obtained.

The most usual load configuration for simply supported structures is that to obtain maximum static vertical deflection of the spans. Sometimes there are intermediate stages for obtaining intermediate results. In the case of continuous structures, the most usual load configurations are those to obtain maximum static deflection (loading alternate spans) and those to obtain maximum negative moments and maximum shear forces in the supports line.
In static load tests the measures most common are:
- Vertical deflections and longitudinal strains at midspan,
- Vertical deflections in supports (when the supports have displacements),
- Cracks evolution (if they appear),
- Temperature and RH.

Dynamic diagnostic load testing
The dynamic test is performed to assess the structure's response to real dynamic load and to assess its dynamic coefficient - the coefficient amplifying the structural response when changing the acting load from static to dynamic mode. This type of test is especially important in the case of bridges, which are carrying moving traffic loads. In this specific case, the loads used in this test are equal to those used in the static test (heavy trucks or some type of railway vehicle, depending on the type of the bridge). The loading protocol comprises a set of vehicles equal to the number of lanes driving from one end of the bridge to the other side by side at a certain speed. Usually, a few speed levels are used which are determined according to the speeds that can appear in real traffic. Since it is quite challenging for multiple vehicles to be driving side by side at higher speeds, this test can also be performed by using one vehicle which can then safely drive at quite high speeds, even higher than those that can appear in real traffic. This situation is quite simpler in the case of railway bridges. Another subtype of the dynamic test is the braking test. In this type of test, moving vehicle brakes on the bridge (at

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different speeds and with different braking power) to examine the horizontal movements of the structure.

During the dynamic test, the speed of the moving load should be recorded and at least the vertical deflection in the midspans should be continuously measured. Additionally, strains and longitudinal (important for the braking test) and transversal displacements can be measured.

Dynamic test also comprises the determination of basic structural dynamic parameters - natural frequencies, damping ratios and modal shapes. This is especially important for structures that are sensitive to dynamic actions (e.g. cable bridges that are sensitive to wind action and pedestrian bridges that are sensitive to pedestrians moving at marching and running speeds). The goal of this type of test is to verify if the structure is designed in such a manner that its behaviour under dynamic action does not put its load-bearing capacity in danger nor does it cause unpleasant feelings to people using it. Besides making the comparison of the obtained results with the design results, it is also preferred to compare the results with criteria indicated in the relevant codes.

8.2.6.2 Satisfactory past performance

For the estimation of the failure probability of a structure on the basis of a satisfactory past performance during T years, the distribution function for structural resistance may be updated considering the cumulative distribution of the maximum load effect over the same period of T years.

Satisfactory performance of a structure during T years of service indicates that, in the absence of any significant deterioration, its minimum resistance is greater than the maximum load effect applied over this period of time.

8.2.6.3 Model updating

Model updating techniques, widely used in the SHM field, allow for the calibration of the system properties (parameters in numerical models), based on actually observed behaviour of the system of interest. Structural FE model updating serves for design verification and validation, to obtain improved predictions of structural response quantities, or simply to identify unknown system characteristics (Worden, et al., 2007). The basic assumption is that localized structural damage results in a local reduction of stiffness, thus, updating stiffness parameters of the FE model in several damage states provides a (non-destructive) means to thoroughly and accurately investigate the condition of the structure (Simoen, et al., 2015).

Both data acquired from vibration analysis (i.e. acceleration time, histories, frequency response functions, natural frequencies and mode shape displacements, modal strains or curvatures, modal flexibilities, etc.) and static diagnostics load tests (deflections, rotations, cracking, etc.) are used for updating purposes, as they provide detailed information regarding the global and local behaviour of the structure of interest, and can be measured in an operational state of the structure.

Model updating problems are inverse problems, as they reverse the standard relationship between parameters and output of a model; the objective is to obtain the parameters that produce a certain given output. As such, these techniques are often prone to ill-posedness and ill-conditioning, as the existence, uniqueness and stability of a solution of inverse problems cannot be guaranteed. Accounting for uncertainties due to measurement and modelling errors is therefore essential and may be accomplished within a Bayesian approach. The calibration procedure requires the following logical steps (see Figure 8.22):

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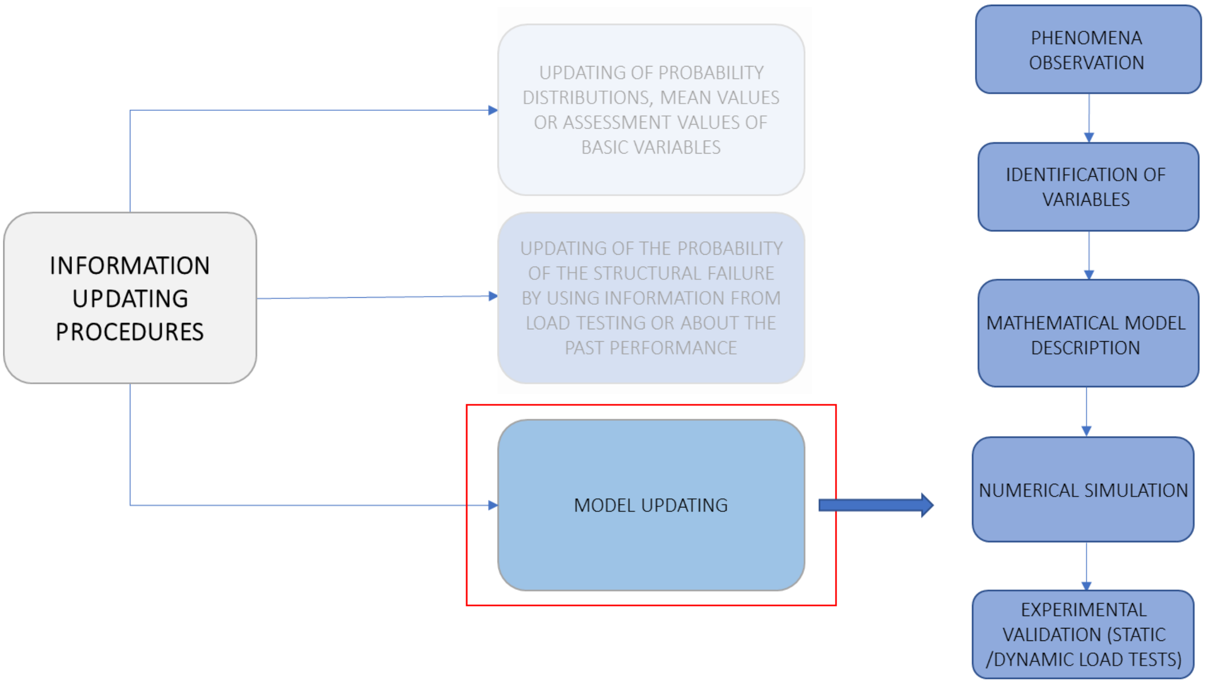

Figure 8.22 - Schematisation of the steps in calibration procedure

The identification of the condition of the structures and the presence of damage is based in principle on the minimization of an error function built from experimental results that photograph the real state of the structure at a certain instant of its life and the results of a calibrated numerical model. The construction and calibration of the reference model can be very complex operations.

For new structures, the numerical model used for design verification may account for a higher degree of simplification. More complex is the case of existing structures, for which the information on materials, the history of the loads, the history of interventions and modifications to which the structure has been subjected are not known or partially known and only the use of in-depth experimental investigations to reconstruct the structural organization, the materials used, and their characteristics and the possible state of degradation allow the development of a reliable numerical model.

Among the methods of calibration of models, we distinguish (UNI/TR11634, 2016):

- **Deterministic models**: it is possible to calibrate a model by trying to minimize an objective function that can express the residuals between numerical and experimental data. In this broad class of methods, it is possible to recognize two large groups, direct methods and parametric methods. The first are non-iterative methods (one-step), which aim to reach the solution by applying the variation of the parameters directly on the matrices describing the system. The others (multi-step) iteratively modify the values of the parameters until the convergence is reached;
- **Probabilistic models**: statistical approach to update the probability distribution of the parameters.

The algebraic differences between the experimental and numerical values constitute the "**residuals**". The **objective function** is, instead, used to compare numerical and experimental responses; it is usually a synthetic and cumulative representation of the residuals, often in the form of mean square value or Euclidean distance.

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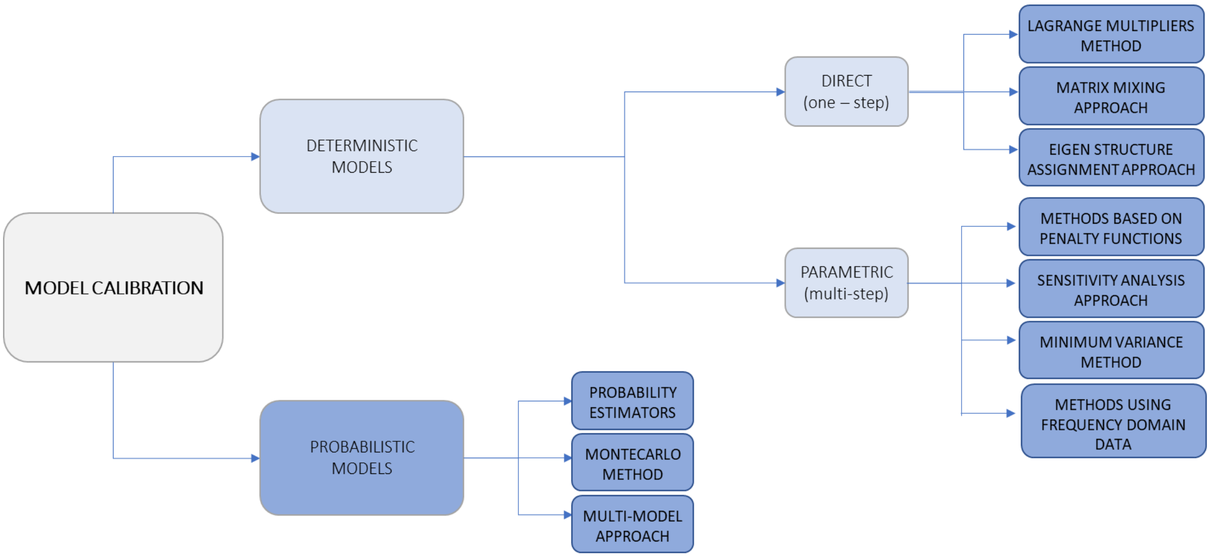

Figure 8.23 - Model calibration deterministic and probabilistic techniques.

8.2.6.3.1 Deterministic models

Deterministic models, both direct and parametric, are listed in Figure 8.23. The experimental data that are most typically used for the process of FE model updating are modal characteristics, which are extracted from measured response time histories using modal analysis techniques. Several types of modal data can be used; the simplest case pertains to exclusive use of natural frequencies or eigenvalues, which are known to be affected by changes in structural stiffness and can be measured fairly accurately.

Parameters extracted from measurements and computed using the FE model are confronted in a cost function, which is often expressed as weighted least squares fit between predictions - expressed in parametric form - and measurements. The optimal values of the model parameters is determined through minimization of the cost function. Quantification is in this case achieved via the identified parameters, which typically pertain to structural properties, e.g. stiffnesses. Their identification thus leads to a direct quantification of expected damage.
In the direct model updating, the modal parameters are directly equated to the measured modal data. Model updating is then characterized by the direct updating of mass or stiffness matrix elements. This effectively constrains the modal properties and frees the system matrices for updating. This approach often results in unrealistic elements in the system matrices e.g. large and physically impossible mass elements. In the indirect or iterative model updating approach the updating problem is formulated as a 'relaxed' optimization problem, often approached by the use of maximum likelihood methods.

In parametric model updating, selected few parameters are varied in model elements and the resulting model is optimized to minimize its difference from the measured data.

Dynamic structures are often analysed from bottom to systems (component) level. At the bottom most level the structure is discretized into constituent elements, for mathematical analysis and computational feasibility, to a system of a second-order matrix of the form:

$$M\ddot{x}(t) + C\dot{x}(t) + Kx(t) = 0 \qquad [8\text{-}28]$$

where M, C and K are of equal size and are called the mass, damping and stiffness matrices respectively or system matrices.

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Given a set of measured real system modal data, the updating problem is then for the model to realistically approximate the mass and stiffness matrices that will produce modal data that is as close to the real systems' as possible. If the model does not result in matching modal properties, the error vector is non-zero and some model parameters will need to be updated.

8.2.6.3.2 Probabilistic models

The most broadly applied approach for fusing models with monitoring data is Bayesian Model updating, which takes advantage of Bayesian statistics to treat parameters as random variables, thereby assigning probability distributions to these. Bayesian statistics can handle complex problems by properly combining information even from different sources. The Bayes' rule, the basis of Bayesian statistics, is able to incorporate the effect of information conveyed by new data (e.g. monitoring data collected by SHM systems) with prior knowledge (e.g. based on computation based on design reports) through the so-called likelihood function and the prior probabilities of a parameter of interest (e.g. mid-span span deflection, prestress loss, bearing displacement), respectively. The Bayesian updating leads to updated probability distributions of the parameters to be used in the updated numerical model.

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9 Appraisal with regards to standardization

9.1 Proposal of the data-informed framework for assessment of existing structures

The IM-SAFE project proposes a framework for the data-informed safety assessment highly interrelated with the through-life management of the transport infrastructure. The need for the clarification of the relationship between the available data, the need of assessment and the maintenance strategies has led to the development of the data-informed performance assessment flow presented in Figure 9.1 and described below.

The proposed flow describes the different stages of the assessment of new and existing structures, focusing on various levels (network, system and component) and taking into account the available data during the lifespan and maintenance process of the structure: it allows to consider both input information and data collection and storage, as well as the possibility to have a BIM and/or Digital Twin model to support the analysis.
The aims of this flow are:

- Rationalize the data-informed safety assessment flow,
- Rationalize the decision-making process with regard to interventions,
- Provide an efficient asset management with a guidance on the use of data.

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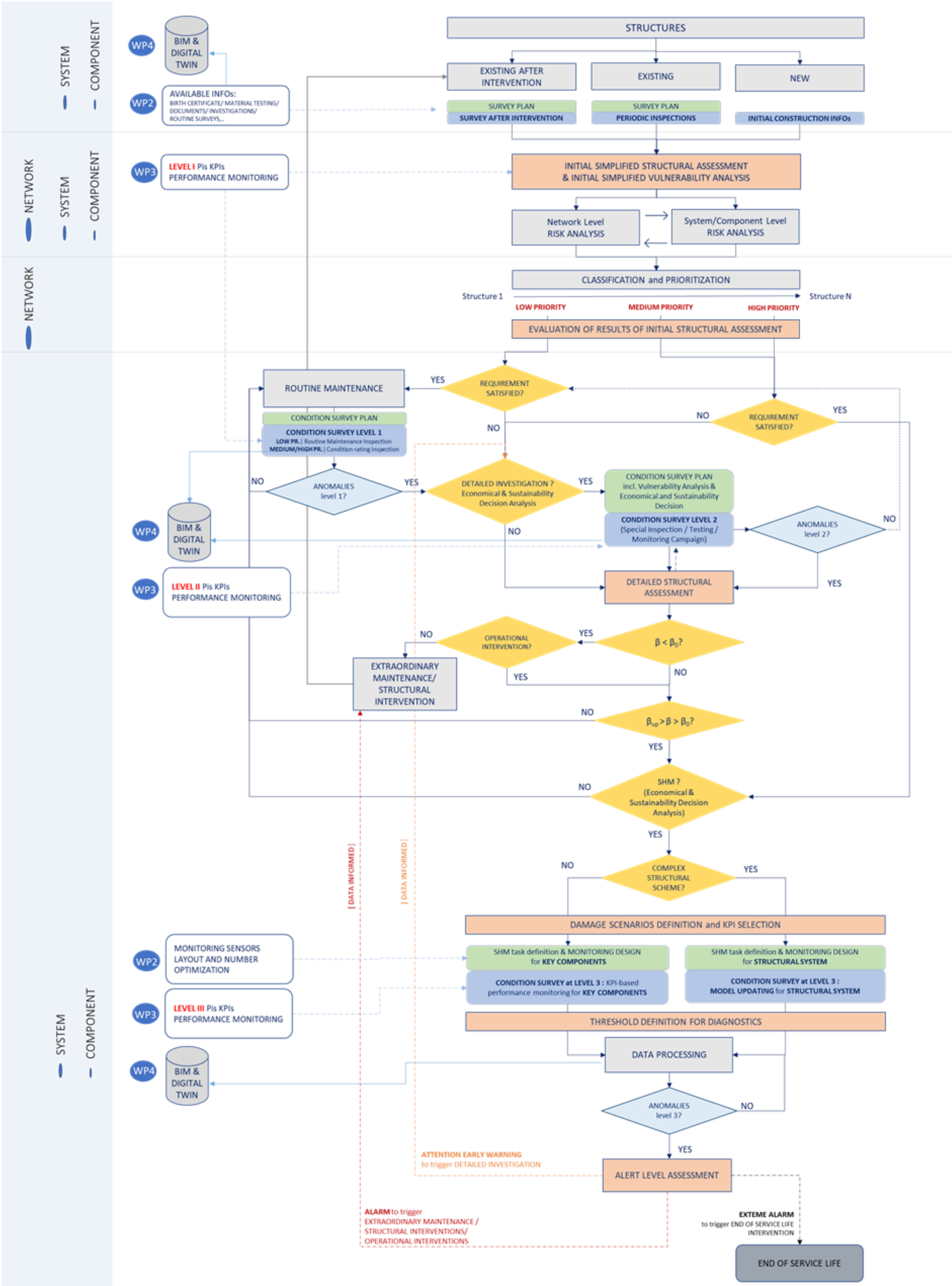

Figure 9.1 - Data informed performance assessment flow

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The first level of analysis includes a simplified assessment of the structures that are part of a road network, based on the review of relevant documentation, condition surveys, structural investigations and material testing, as well as supported by a vulnerability analysis aiming to identify the critical elements of each asset (see section 2.3). The simplified assessment is performed based on the Level I PIs and KPIs.

In line with the reliability differentiation presented in 8.2.3, the assets have been subdivided in 3 relevant classes:

- New structures
- Existing structures
- Existing structures after intervention

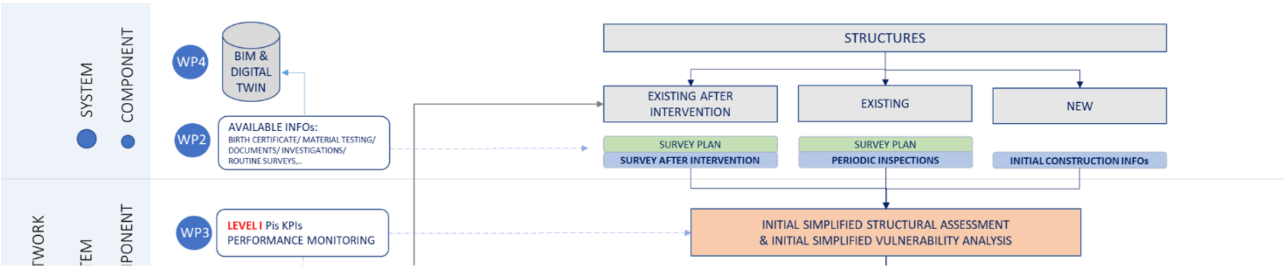

Figure 9.2 - Data informed performance assessment flow (part 1)

The simplified analysis is required as a prerequisite to perform a classification and prioritization of the assets that are part of the same road network based on a risk analysis (see section 4.3), which can be performed either at the network level or at the system/component level. This classification may be based on a simplified assessment (based on review of relevant documentation, condition surveys, structural investigations and material testing) and a vulnerability analysis, aiming to identify the critical elements of a network of infrastructure assets and the assets within the network. Risk analyses at the network level and at the system and component level are suggested for the purpose of define the priority list. Risk analyses shall be performed consistent with the risk assessment framework presented in 7 and making use of the available information, as discussed in section 8.2.2.

In case of a single structure analysis, instead of a prioritization of intervention requirement, the need for assessment, its type (preliminary and detailed) and its level of application (network, system or component) may be originated by different causes (external actions, damages, planned assessment) and could be based on various available information (see Figure 8.4).

This risk analysis performed to support the structures classification aims to identify the consequences of failures and the risks for human life. Structures could be assigned to one of the following classes with respect to the need of maintenance and interventions:

- **Low priority**;
- **Medium priority**;
- **High priority**.

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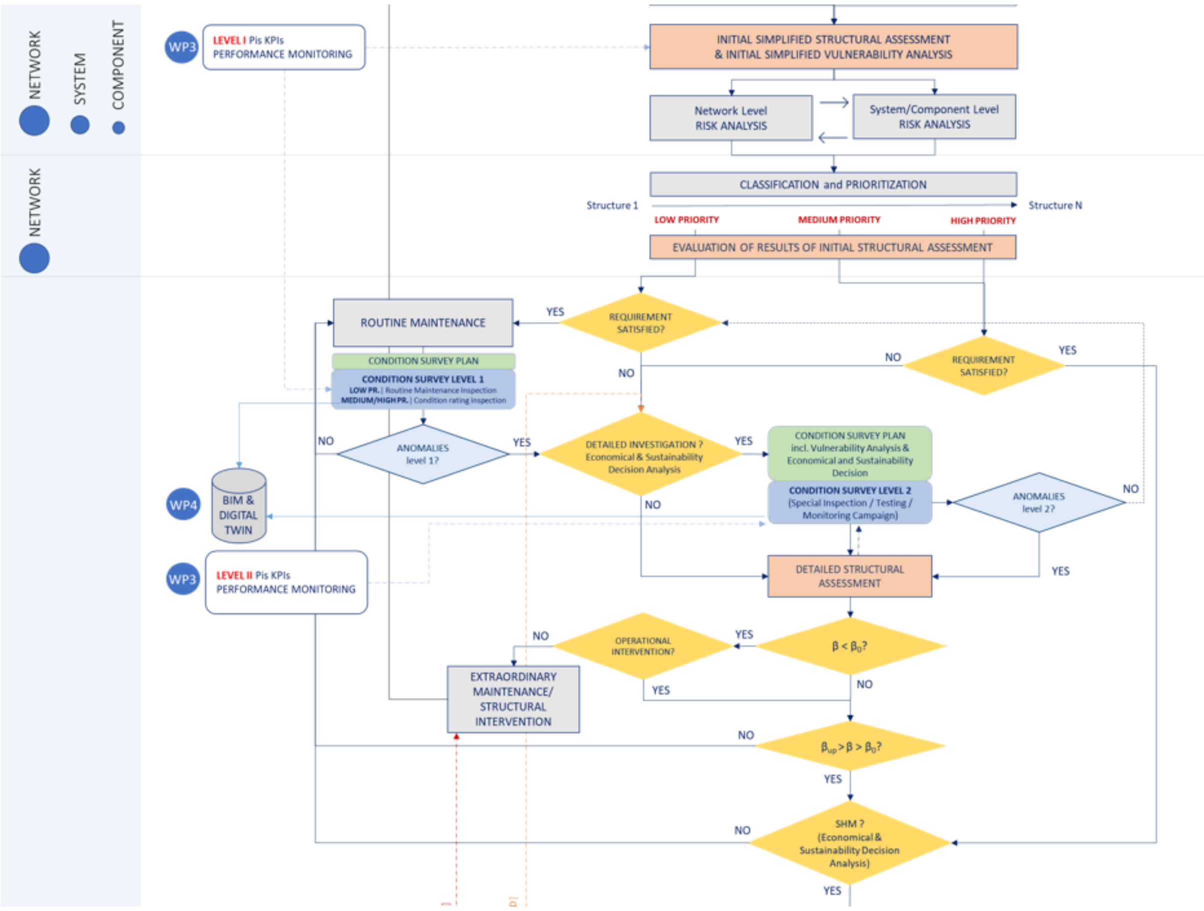

Figure 9.3 - Data informed performance assessment flow (part 2)

Low priority structures undergo routine maintenance inspection plans. Structures could change class based on the outcomes of the ordinary maintenance or eventually light monitoring. A detailed investigation plans as well as a detailed assessment might be considered in case of detected or suspected anomalies. In this case, further condition survey plans can include special inspections, testing and monitoring campaigns that constitute a Level II PIs and KPIs information degree.

Structures in the “Medium priority” and “High priority” classes may be subjected to detailed assessment (see section 6.2), triggered by the reasons outlined in (see section 8.1). Based on the gathered information about the condition of the structure, decisions regarding the use of inspections, monitoring and testing are evaluated. In particular, with respect to the reliability levels differentiation proposed for the assessment of existing structures, actions may vary based on the target levels:

- β_0 - level below which the existing structure is considered unreliable and should be upgraded;
- β_{up} - level indicating an optimum upgrade strategy while upgrading of existing structures.

If the reliability is lower than the minimum accepted β_0, the outcome of detailed assessment can result in extraordinary maintenance or interventions directly. In case of operational interventions (e.g. traffic limitation) are needed before any structural intervention, it may be considered to apply monitoring strategies as a step to prevent undesired events before the structural upgrade.

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If the reliability is between β_0 and β_{up}, the evaluation of the most suitable monitoring strategy is suggested, based on an optimization approach to either select increased visual inspection schedules, further additional testing and/or application of periodic, frequent, or continuous monitoring. The identification of the monitoring strategy can be made based on economical and sustainability decision making processes. Cost-benefits optimization analyses can guide the selection of monitoring sensor layouts or key parameters to be detected. Permanent monitoring system might be coupled with dedicated inspections and destructive/non-destructive testing (see also IM-SAFE project report D2.2 (Longo, et al., 2022)).

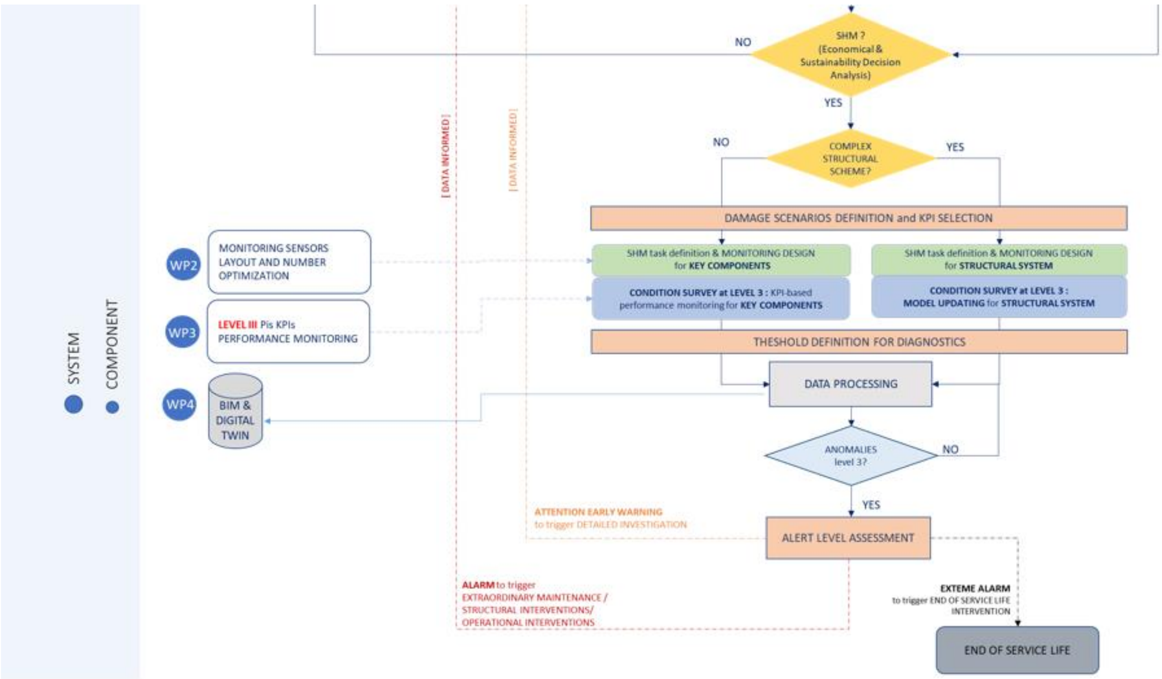

Figure 9.4 - Data informed performance assessment flow (part 3)

In case continuous SHM systems are used, the approach to monitoring differs depending on the complexity of the structural scheme following an increasing level of approximation in the analysis approach. Monitoring of the structural performance of key structural components is suggested for simple structural schemes, whilst model updating is suggested for more complex structures. Complex structural schemes, as such, require a higher complexity of the analysis and performance monitoring, which implies numerical modelling, model updating processes and deeper analysis of the presence and localisation of damages. In both cases, the use of Level III PIs to control the structural performance over time is suggested as well as the definition of relevant damage scenarios. The definition of thresholds for structural diagnostics purposes is essential to promptly identify potential anomalies or identify a sudden change of the behaviour of the structures. Structural diagnostics has to be supported by a thorough data processing. For more information on the thresholds definition reference is made to IM-SAFE deliverable D3.2 (Darò, et al., 2022).

Regardless of the complexity of the structural system, the gathered data should feed appropriate key-performance indicators and should serve as input for structural diagnostics procedures. The detection of anomalies may be considered as a trigger for decisions regarding the end of service life of the structure. In case of very extreme events or exceedance of the alarm levels, the end of the service life of the structures could potentially be reached. For less severe thresholds levels exceedance, the monitoring system would be a trigger for

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further investigation levels or, eventually, for another detailed structural assessment to evaluate if a structural intervention or an upgrade is needed.

9.2 Proposal for implementation of performance indicators and Key Performance indicators in assessment phases

Experience shows that breaking down the performance assessment of a structure or any other facility into a minimum of three phases is reasonable. Figure 9.5 presents these phases in a flow chart. Each of these phases should be completed in itself. During the assessments to be performed over the time period, a change in the phases can occur, e.g. a SHM based reliability assessment can lead back to phase 1 of the visual inspection in case of a good performance assessment.

Phase I: Preliminary assessment (Visual inspection)
The purpose of Phase I is to remove existing doubts about the performance using fairly simple methods, which must, however, be adequate. The preliminary evaluation consists of a rough assessment based on inspection, an accompanying study of the available documents, and a rough check on the structural safety. In this first phase, it makes sense to distinguish between key performance indicators, which allow a rough assessment, and performance indicators for detailed inspection and to define performance targets and performance thresholds.

The information gained in Phase I must be summarised in a report for the owner and must result in Key Performance Requirement Indexes (KPRs) for the strategic asset management and budget allocation. The performance evaluation shown in Figure 9.5 is based on anomaly detection and leads to the KPR level or, in case of an insufficient condition, to phase II.

Phase II: Detailed investigations (Detailed Inspection, testing and monitoring)
Structural investigations and updating of information are typical of Phase II – it comprises detailed inspection, testing and monitoring among others. The additional information gained e.g. from the performance indicators of these investigations can be introduced into confirmatory calculations with the aim of finally dispelling or confirming any doubts as to whether the structure is safe. A detailed inspection of the structure or structural part in question is extremely important, especially the recognition of typical hazard scenarios that could endanger the structure's residual service life. Further any defects and damage process due to excessive loading must be detected e.g. by using the visual inspection associated observations or performance indicators, as shown in Table 5.1 (see also section 5.1).
Performance indicators or observations in these phases are mainly received from detailed inspection, testing and monitoring, see Table 5.1 and Figure 9.5. As in the first phase, it is recommended to distinguish between key performance indicators, which allow an assessment based on a minimum of detailed inspection, testing or monitoring related observations, and performance indicators associated with detailed testing and monitoring.
The results of Phase II must be summarised in a report. In particular, the report must give information on the structural safety based on the performance indicator findings of phase I and II, and must contain the information about the Performance Requirement Indexes (RPIs) for e.g. network assessment purposes. As in Phase I (see Figure 9.5), in this II phase the performance evaluation is based on the detection of anomalies and leads to the CPR stage or, in the case of an inadequate condition, to Phase III.

Phase III: Assessment and prediction by advanced analysis (Structural Health Monitoring (SHM) und Modelling)
For problems with substantial consequences, an advanced analysis for performance assessment and performance prediction should be planned to check carefully the proposal for the pending decision that results from Phase I and II. In assessing an existing structure, such an analysis (see Figure 9.5) acts to a certain extent as a substitute for the codes of practice,

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which for new structures constitute the rules to follow in a well-balanced and safe design. In particular, the acceptance of increased risks should be left to the assessment team of experts. In this phase, extended surveys such as continuous monitoring or SHM are usually necessary for the in-depth analyses with regard to Phases I and II for the determination of the analysis input variables. Some of the observations in these surveys of the input variables as well as some of the analysis responses are suitable for the performance assessment and can therefore be assigned to the class of performance indicators. In this phase III, the distinction between key performance and performance indicators can be applied analogously to Phases I and II.

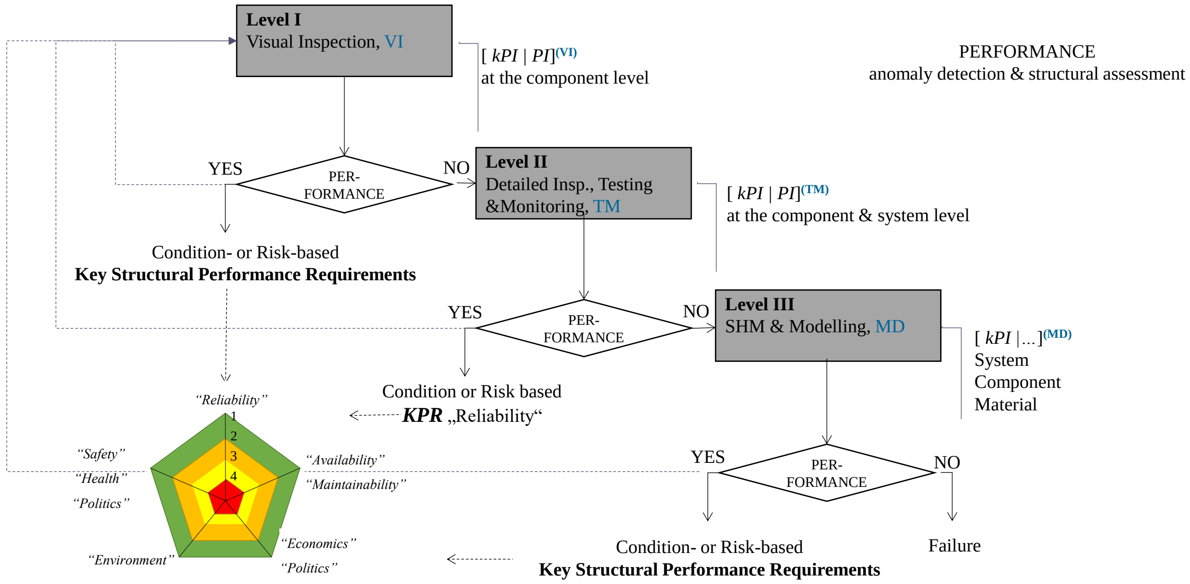

Figure 9.5 - Performance assessment according to the different inspection levels (Phase I Visual Inspections; Phase II Detailed Inspections, Testing and Monitoring; Phase III SHM and Modelling) for the comparison with the Key Performance Requirements

9.3 Proposal of the scope of standardization for data-informed approaches

Review of the concepts and methods provided in the preceding Chapters is the basis for the proposal of the scope of future standardization for data-informed approaches for optimal maintenance and safety of transport infrastructure enabled by inspection, testing and monitoring. Taking into account maturity of the knowledge and general consensus with regard to the approaches, the following items are proposed for consideration for the future extension of the existing EU standards on safety assessment:

- harmonisation of terminology for data-informed safety assessment, to set the basis for a common understanding of the glossary within Europe and to respond to the need of resolving conflicting definitions that may cause misleading interpretation (see IM-SAFE glossary),
- standardisation of a framework for the data-informed safety assessment, interrelated with the risk-based through-life management and maintenance of the transport infrastructure, to set the basis for use of data from inspection, testing and monitoring (see section 9.1),
- standardisation of implementation of performance indicators and Key Performance indicators in assessment phases (see section 9.2),
- standardisation of the minimum reliability requirements, to enable differentiation between the minimum reliability requirements in assessment of existing structures by including the "fitness-for-use" in operation level and the "structural upgrade" level (see section 6.1.2),

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- standardisation of the methodologies for safety assessment at the semi-probabilistic level, considering the 1-year reference period for the evaluation of the residual service life of existing structures undergoing time-dependent deterioration processes (see sections 8.1 and 8.2),
- standardisation of the methodologies for considering damage and deterioration in the safety assessment at the semi-probabilistic level of bridges and tunnels (see section 4.2.7),
- extending the standardised methodologies for assessment of structures to enable advanced use of structure-specific information (see Chapter 8), incl.:
    - improving standardisation of the methodologies for the evaluating the structural reliability by assessing the failure probability conditional on the observations
    - improving standardisation of the methodologies for the evaluating the structural reliability by updating the random variables involved in the limit state function
    - extending the standardised methodologies for safety assessment at the semi-probabilistic level considering information not directly related to the parameters of the limit state functions
    - standardisation of the methodologies for adjusting the partial factors on the load variables based on monitoring of the structural response
    - standardisation of the methodologies for use of proof-load testing in the safety assessment of bridges at the semi-probabilistic level
    - standardisation of the methodologies for updating of traffic load models for bridges based on load monitoring
    - standardisation of the methodologies for updating the model uncertainty of the load effect model based on monitoring of the structural response during load tests
    - standardisation of the formulation of condition limit states to enable the use of monitoring in combination with threshold values of the structural response for assessing safety during operation

With regard to the formulation on the new standard on preventive maintenance and risk-based maintenance management of transport infrastructure the following items are proposed for consideration in order to enable development of a practical framework for management of structures accounting for the classification of structures based on risks:

- standardisation of the methodologies for identification of vulnerable zones of bridges and tunnels (see section 2.3)
- standardisation of the methodologies for identification and characterisation of the hazardous events (see Chapter 3)
- standardisation of the methodologies for risk assessment (see section 4.3 and Chapter 7)
- standardisation of the KPI used for the management of infrastructure assets (see Chapter 5)

In the appraisal above, reference is made to the specific section of this report where the review of the methodologies for all items identified as suitable for consideration for the future extension of the existing EU standards is given.

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10 Summary and conclusions

Aiming to ensure the safety of the transport infrastructure during operation through the improvement of maintenance policies across Europe, the European Commission opened in 2019 the call for the Coordination and Support Action (CSA) “Monitoring and safety of transport infrastructure”. The main goal of this CSA is to support the preparation of a mandate for a CEN standard for the maintenance and control of the European transport infrastructure. In 2020, the CSA was granted to the IM-SAFE project consortium (H2020 CSA IM-SAFE, 2020).

This report provides extensive information that serves as the technical background for proposing further amendments to the existing EU standards on safety assessment taking into account inspections, monitoring and testing and for proposing a new standard for preventive maintenance of transport infrastructure. Activities in the scope of Work Package 3 (WP3) of the IM-SAFE project, which are reported in this document, aim to identify the normative gaps and evaluate opportunities to close these gaps with regard to data-informed safety assessment and maintenance decision-making based on review of the current state-of-the-art as represented by standards, guidelines, other regulations as well as current practice and research.

The report is based on the activities for Task 3.1 of IM-SAFE WP3, which include:

- the review of the methodologies for reliability assessment at risk-informed, reliability-based and the semi-probabilistic level, including data-informed approaches accounting for prior information, information from inspection, testing, and monitoring including principles of :
    - probabilistic characterization of the hazardous events,
    - evaluation of the probability of occurrence of adverse events,
    - principles of predictive performance models accounting for deterioration and damage,
    - cost models for inspections, monitoring, maintenance, direct and indirect consequences of failures;
- the review of the damage indicators (Dis) and performance indicators (Pis) used for the management of infrastructure networks and infrastructure objects;
- the review of the methodologies for risk assessment and risk management of infrastructure networks and infrastructure objects the evaluation of the current approaches to the use of information from inspection, testing and monitoring in the assessment and management of bridges and tunnels including:
    - the proposal of the framework for data-informed assessment,
    - the appraisal of concepts and methods for assessment of existing structures with regard to future standardization.

The report provides information that explains and proposes:

- classification of types of infrastructure based on typology, vulnerable zones and vulnerability (in terms of both functionality and structural loading for bridges) (see Chapter 2),
- classification of hazards, actions related to hazards is given and the probabilistic and data-informed representation of hazardous events (see Chapter 3),
- concepts of condition, performance and risk (see Chapter 4), including:
    - the classification of deterioration and damage, the damage indicators for assessment and the outline of the implementation of the condition concept in through-life management,

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    - the principles of the limit-state and reliability-based approach to performance modelling,
    - the general principles of risk representation and a generic framework for risk management of infrastructure objects.
- concept of performance indicators, categorized for different levels of assessment (network, object, and component level) and concept of key performance requirements for bridge and tunnel management systems (see Chapter 5),
- principles of performance verification of existing structures and performance verification methods (see Chapter 6),
- methods for assessment and management of risk for infrastructure networks, objects and components (see Chapter 7),
- principles of data-informed approached in assessment and principles of the use of data in performance verification (see Chapter 8),
- appraisal with regards to standardization including a proposal for a framework for data-informed assessment of transport infrastructure is included. (see Chapter 9).

Taking into account maturity of the knowledge and general consensus with regard to the approaches, the following items are proposed for consideration for the future extension of the existing EU standards on safety assessment:

- harmonisation of terminology for data-informed safety assessment, to set the basis for a common understanding of the glossary within Europe and to respond to the need of resolving conflicting definitions that may cause misleading interpretation (see IM-SAFE terminology proposal in IM-SAFE online Knowledge Base, https://imsafe.wikixl.nl/),
- standardisation of a framework for the data-informed safety assessment, interrelated with the risk-based through-life management and maintenance of the transport infrastructure, to set the basis for use of data from inspection, testing and monitoring,
- standardisation of implementation of performance indicators and Key Performance indicators in assessment phases,
- standardisation of the minimum reliability requirements, to enable differentiation between the minimum reliability requirements in assessment of existing structures by including the “fitness-for-use” in operation level and the “structural upgrade” level,
- standardisation of the methodologies for safety assessment at the semi-probabilistic level, considering the 1-year reference period for the evaluation of the residual service life of existing structures undergoing time-dependent deterioration processes,
- standardisation of the methodologies for considering damage and deterioration in the safety assessment at the semi-probabilistic level of bridges and tunnels,
- extending the standardised methodologies for assessment of structures to enable advanced use of structure-specific information, including:
    - improving standardisation of the methodologies for the evaluating the structural reliability by assessing the failure probability conditional on the observations,
    - improving standardisation of the methodologies for the evaluating the structural reliability by updating the random variables involved in the limit state function,
    - extending the standardised methodologies for safety assessment at the semi-probabilistic level considering information not directly related to the parameters of the limit state functions,
    - standardisation of the methodologies for adjusting the partial factors on the load variables based on monitoring of the structural response,
    - standardisation of the methodologies for use of proof-load testing in the safety assessment of bridges at the semi-probabilistic level,
    - standardisation of the methodologies for updating of traffic load models for bridges based on load monitoring,
    - standardisation of the methodologies for updating the model uncertainty of the load effect model based on monitoring of the structural response during load tests,

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- standardisation of the formulation of condition limit states to enable the use of monitoring in combination with threshold values of the structural response for assessing safety during operation.

With regard to the formulation on the new standard on preventive maintenance and risk-based maintenance management of transport infrastructure the following items are proposed for consideration in order to enable development of a practical framework for management of structures accounting for the classification of structures based on risks:

- standardisation of the methodologies for identification of vulnerable zones of bridges and tunnels,
- standardisation of the methodologies for identification and characterisation of the hazardous events,
- standardisation of the methodologies for risk assessment,
- standardisation of the KPI used for the management of infrastructure assets.

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Appendix 1 - Typology of bridges

1. Beam Bridges

1.1. Static system

Bridges with bridge spans supported by an abutment or pier at each end are the most simple and common bridge forms. The modern beam bridges include girder, plate girder, and box girder bridges. The girders bend under vertical loads resulting in horizontal tension and compression on the top and bottom sides of the section.

Single Beam: Single beam bridges with one beam have two hinged bearings or foundations. At both bearings, no vertical movements are allowed and a horizontal movement is allowed at one bearing. The single-beam bridges are statically determined and can also form a row of several single beam bridges. Due to the horizontal movements especially longer beam bridges require expansion joints for the transition. As a rule, three to four fields are combined seamlessly. Only one bearing of the single beam can be fixed – the other must be able to move horizontally.

Single Beam + Cantilever + Gerber: These types of bridges combine several beams with flexible joints allowing the transfer of vertical and horizontal loads but no moments and are statically determined. The distribution of the bending moments on the span and support area can be influenced favourably by the positioning of the joints and changing the moment of inertia (changing beam height of the cantilever girder). Due to greater width between supports, these types of bridges have historic importance. However, as these joints reduce stability and induce points of weakness regarding service life these systems are considered to have many disadvantages compared to seamless continuous beams.

Multi Beam, Continuous Beam: Continuous beams over two or more fields have already been built continuously over multiple fields without joints. If possible, the length of the end fields should be around 20% shorter compared to the intermediate fields so that field moments are approximately the same allowing optimal static use of the beam. In contrast to single beam bridges, the bending moment is distributed to field, allowing greater slenderness of the bridge. Statically indeterminate bearings increase system safety as there is no collapse if a part of the beam fails. Uneven column subsidence causes constraining moments which should not be seen as a disadvantage. Constraint moments due to small settlement differences are reduced in the prestressed concrete beam by creeping. Larger settlement differences can be compensated for by lifting the beam with hydraulic presses and lining the bearings. With a suitable construction, continuous prestressed concrete beams are less sensitive to temporary uneven settlement than steel bridges. Cracks close again when readjusted thanks to the high elastic spring force of the steel tendons with a high elastic limit. The great advantage of the continuous beams is the seamlessness of the roadway over large bridges (800 to 1000 m). In contrast, flexible road joints are expensive, interfere with the driving and are a weak point in every bridge construction regarding service life. Therefore, if possible, only one longitudinally movable joint should be arranged at one end of the bridge. The fixed bearing is often arranged at one end to get by there with a small joint without a movable part. The other bearings must allow horizontal movement. If the spans differ greatly, it may be better to place the fixed bearing at the point of the greatest bearing pressure. Very long bridge beams can be subdivided with suspension brackets or Gerber's joints.

1.2. Bridge shape

Single Beam: Are best designed parallel chorded, which means that the lower edge runs parallel to the driving line, the construction height is constant. This also applies if the driving line is inclined (downhill) or in a curve.

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Single Beam + Cantilever + Gerber: Cantilever and Gerber beams are usually placed on the fields with the longest width at the end of a cantilever support beam in the middle of the field. These suspended beams are statically similar to single beams and have been constructed accordingly.

Multi Beam, Continuous Beam: Multi beams are usually designed parallel chorded as well if the span length of the fields is about the same. This also applies if the road runs through a hollow in its longitudinal profile, i.e. the lower edge is curved downwards and “sags”. Experience has shown that this “sagging” looks quite natural.

2. Frame Bridges

2.1. Static system

In bridge construction, frames are created through rigid connections of the bridge beam with the supporting walls or abutments or with supporting pillars. The end of the beam is clamped into the abutment wall so that part of the bending moment is reduced by negative clamping moments, which reduces the required overall beam height in the field. The beam moment in the field can be greatly reduced with stiff support pillars.

Single Frame: The single frame bridge consists of a support pillar or foundation with stiff frame corners being directly connected to the main horizontal beam. Compared to a similar single beam bridge the field width can be extended by 40 to 60%.

Multi Jointed Frames: Are continuous beams over two or more fields with a fixed connection to vertical or inclined support pillars allowing favourable moment distribution and greater field width compared to cantilever + Gerber or continuous beam bridges.

2.2. Bridge shape

By choosing the relation of stiffnesses between the frame members and frame corners, the distribution of the bending moments can be favourably influenced in the design stage allowing smaller construction heights and cross-section dimensions compared to single beam bridges with the same length. This is also the reason why converting existing single beam bridges to frame bridges in rehabilitation has become increasingly popular. In addition, the typical weaknesses coming with transition areas and joints are greatly reduced.

Single Frame: The single frame bridges with short to medium length are usually designed parallel chorded with increased stiffness and dimensions in the corner areas.

Multi Jointed Frames: In the case of medium lengths and vertical frame pillars the dimensions of the frame are usually uniform over the entire length. In the case of inclined support frames and pillars and greater lengths, the stiffness and dimensions are adjusted according to the moment distribution allowing elegant and efficient cross-section dimensions.

3. Arch Bridges

3.1. Static system

The arch, shaped like a vault according to the support line of the dead weight loads, is the most suitable type of structure for solid building material (stones, concrete) with its high compressive strength if the foundation is solid and can absorb the arch thrust. That makes it possible for an arch bridge to handle more loads compared to horizontal support designs. Because there is a reduced need for reinforced vertical support, it is usually cheaper to construct this type of bridge compared to other options. Arch bridges made of strong natural

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stone generally do not need any expansion joints. In the case of concrete, however, the deformations due to shrinkage, temperature and creep must be considered, which affects the arch shape and makes joints necessary. Unreinforced concrete, therefore, requires, for example, the choice of a statically determined three-hinged arch, which allows deformation free from constraint through crown sag. As a rule, however, arch bridges are now made of reinforced concrete, and their deck is often made of prestressed concrete. Arch bridges are usually too expensive for small spans (up to ~ 50m). Arches are particularly suitable for bridging mountain valleys with rocky slopes but are not comparable in terms of cost and efficiency on flat land with frame bridges.

Tied Arch Bridge: This bridge type consists of a combination of an arch and a horizontal beam or horizontal cables tying together the outward horizontal pressure of the arch at each end. The vertical forces on the horizontal beam are redirected into the arch by vertical hangers. It is also possible to use two parallel and/or several consecutive arches. The main advantage of these bridge types is the possibility of spanning great lengths without the need for massive abutments or foundations.

Truss Arch Bridge: Truss arch bridges are somewhat similar to tied arch bridges with the main difference being that a massive arch and small hangers are replaced by an arch-shaped truss with diagonal members being able to carry either tension.

Sickle Arch Bridge: For reasons of design or limited height (above/below) the bridge deck can be partly suspended from the arch (middle area) and partly supported by columns (left/right). This integration of the arch is considered elegant by some and can also be combined into a series of smaller or similar arches.

Deck Arch Bridge: A majority of historical bridges have been using consecutive arches with a continuous bridge deck above. Each arch supports the neighbouring arch towards the foundations at both bridges ends. Modern deck arch bridges are usually constructed over steep valleys with usually one arch covering the distance and the entire bridge deck being supported by pillars above.

3.2. Bridge shape

In almost all cases the arch is the defining structural and aesthetic element of arch bridges being visible from large distances due to their dimensions. As the bridge deck is either suspended or supported by pillars with limited loading it is possible to design these elements as slender, elegant structures.

Tied Arch Bridge: The modern interpretation of this bridge type mainly uses pre-casted steel elements that are assembled on-site or are transported as already assembled structures consisting of the arch and horizontal ties or frame. The foundations and abutments are constructed beforehand and the bridge deck is constructed on-site after placement of the arch. The main benefits are short construction periods and high-cost efficiency especially in cases of large river crossings using vessels or rafts.

Truss Arch Bridge: This bridge type has been used in many historic cases with modern interpretations also being based on various forms of aesthetic dominant precast steel arches. As this bridge type requires a high amount of labour, they are used less frequently nowadays.

Sickle Arch Bridge: The arch in this bridge type has been constructed as a dissolved truss arch or two steel arches on the left and right with horizontal bar connections or as a single arch in the middle with the bridge decks on both sides covering the traffic in different directions.

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Deck Arch Bridge: Historic consecutive stone arches with a bridge deck have been built for centuries providing very long service lives. In the case of deeper rifts, the many arches have been replaced in the 18th and 19th centuries by wood or steel truss arches supporting the bridge deck above (mainly rail bridges). Modern deck arch bridges for highways are mainly built using reinforced concrete arches being constructed on a wooden shuttering (Cruciani Arch), as a folding arch with climbing formwork or cantilever. In these cases, the supporting pillars and the bridge deck are also in concrete.

4. Suspension Bridges

4.1. Static systems

These types of bridges are defined by large cables anchored on the ground, steep slopes and/or between towers with the bridge deck being tied to these cables by vertical suspenders. Special emphasis has to be given to the cables and anchoring with the loads being transformed into tension and anchoring forces.

Simple Suspension: These bridges consist of two ropes or cables being anchored on both sides on the ground, steep slopes or other anchoring points. The bridge deck is suspended by vertical hangers acting as a stabilizing element in case of uneven loading.

Standard Suspension: These bridges consist of one or two pylons with one or two ropes being anchored on both ends into the foundation or other anchoring points. The bridge deck is suspended by vertical hangers acting as a stabilizing element in case of uneven loading.

4.2. Bridge shape

The combination of anchored ropes and a suspended bridge deck has been used for centuries. More than 600 years ago the Incas already used rope bridges using ichu grass woven into bundles as part of their road system between gorges and ravines. At around the same time iron chain bridges have been built in Bhutan and China with the first chain bridges in the Western world being built 400 years later. Around the same time instead of chains wire cables have been introduced being commonly used until now. Modern suspension bridges are among the longest bridges covering distances up to 2.000 m.

Simple Suspension: The origin of these bridge types are the historic rope and chain bridges with a suspended lightweight bridge deck. Modern versions are mainly used for lightweight travel (pedestrian, cycling) over greater distances with the goal of a lightweight construction. Although these bridge types are robust they are susceptible to uneven loading and dynamics resulting in movement and temporary deformations.

Standard Suspension: Although the principal static system of suspension bridges has remained the bridge form has changed with the development of construction materials and demands. Starting with stone towers and chains the development included truss pylons, trussed bridge decks and wire-cables. Modern suspension bridges consist of one, two or several consecutive towers/pillars with two or more suspension cables and bridge decks consisting of lightweight steel frames. The construction always starts with the foundations, anchoring and pillars followed by the suspension cables. The bridge deck elements are lifted, fixed with their hangers and connected together.

5. Cable Stay Bridges

5.1. Static Systems

Cable stayed bridges consist of pylons with inclined cables and a suspended roadway. In contrast to suspension bridges the towers are the main load-bearing structure and the inclined cables are directly supporting the bridge deck. If only a few cables with large distances

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between the suspension points are used, the bridge is to be regarded as a girder bridge with intermediate supports (suspension points) and the beam must have a structural height and flexural rigidity corresponding to the span length. To enable lightweight bridge decks the development went to many cables with correspondingly small distances between the suspension points. These bridges are viewed more as a cantilever bridge with the deck forming the lower chord, while the cables as cantilever tension chords transfer the loads to the pylon towers. The towers have to be anchored strongly depending on the ratio of the main opening to the side opening. The roadway deck can have a very small overall height in the longitudinal direction, but has to provide sufficient buckling resistance of the compression chord when deformed by traffic loads. The view of the cables can be arranged like a fan or harp, as a bundle in rays or in parallel. With large ratios l: b = span to bridge width, A-shaped pylons can offer aesthetic and technical advantages. Cable-stayed bridges have proven to be technically particularly suitable and also economical for large spans (500 – 1100 m).

Single Pylon Fan Cable Bridges: These bridges have foundations and bearings on both ends with a single pylon being placed centrically or eccentrically. The inclined fan-shaped cables are anchored on top of the pylon one below the other and on the edges of the bridge deck outside the railings.

Single Pylon Harp Cable Bridges: These bridges also have foundations and bearings on both ends with a single pylon. Instead of fan shaped cables the cables are placed parallel to each other in form of a harp. Although the fan shape is technically more effective and more economical than the harp shape, the latter may provide a more aesthetic appearance with a few cables.

Double/Many Pylon Cable Bridges: In these cases two or more pylons are situated in order to cover larger distances. In principle the inclined cables can be designed as harp or fan as well although the fan design is commonly used in cases of more than one pylon due to higher efficiency.

5.2. Bridge shape

Cable stayed bridges are considered optimal for spanning lengths between the typical range of cantilever bridges and suspension cable bridges. In contrast to cantilever constructions the cables allow for lighter bridge deck constructions and therefore longer spans.

Single Pylon Cable Bridges: These bridges are aesthetically defined by the design and position of the pylon (eccentric, between directional lines or on the side), the selection and number of fan or harp cables as well as the construction and form of the bridge deck and railings. The harmonic integration of structural requirements and aesthetic design are key for great design.

Double/Many Pylon Cable Bridges: Cable bridges with more than one pylon usually cover longer distances (rivers, lakes, shallow sea) and are usually designed with focus on structural requirements and efficiency. Nonetheless, the same static and design principles apply as already mentioned in case of single pylon cable bridges.

6. Truss Bridges

6.1. Static Systems

Truss bridges are defined by the dissolution of the load-bearing elements into a truss system of members arranged in a triangular shape. The articulated rods transmit the forces as tension or compression beams towards the foundations. The main advantage of truss bridges was their static stability and the economical use of materials making it a standard construction for rail and bridges in the 18th and 19th century. As truss bridges are constructed by a large number

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of shorter truss components/members, the transport of the necessary construction materials through rough terrain is much easier compared to other bridge types. The simple and quick construction of truss bridges makes them an ideal solution for military use and as emergency bridges with limited spans (e.g. Bailey truss). The disadvantage of static determinacy is the failure of the structure in case of failure of a beam as well as the limited durability and maintainability in case of corrosion due to winter maintenance. Thus, truss bridges are still dominant in rail networks and are less used in road and highway networks.

Queenpost Truss Bridge: These truss bridges are the most simple form with the arrangement of three triangles in the form of a trapezium. With a limited length of the beams this truss type can only cover limited span widths.

Supporter Truss Bridge: This bridge shape corresponds to a mirrored queenpost truss with the truss as supporting structure below the bridge deck. These under-span bridges allow for efficient design of the bridge cross-section with limited span length, but are also vulnerable in the event of an under-span failure.

Warren Truss Bridge: This bridge type consists of longitudinal beams connected by diagonal cross members alternating direction. All beams are hinged being subjected to tension or compression only. The static design combines an economic use of materials with high loading capacity and variable span length also allowing a high extend of standardization due to similar lengths of the beams.

Howe Truss Bridge: The howe truss bridge consists of horizontal longitudinal beams connected by vertical and diagonal members sloping from both sides towards the centre. The diagonal members are compressed and the vertical members carry tension forces in contrast to pratt truss bridges with the diagonal members sloping towards both ends.

6.2. Bridge shape

Truss bridges and trussed beams and supporting structures are key elements in bridge design allowing lightweight and efficient constructions covering a large range of span distances depending on the selected truss type or combination with other bridge types. The historic truss bridges have been constructed using wooden beams. During the industrial revolution this has changed towards the use of steel beams as greater distances and loads had to be covered. With the recent development of modern timber connections and laminated beams, the construction of timber truss bridges has become more again popular.

In contrast to the simplified static depiction of truss bridges most designs consist of two parallel longitudinal trusses being connected by a trusses bridge deck stabilized by an orthotropic roadway plate. With rare exceptions the parallel longitudinal trusses are connected by struts and lateral bracings stabilizing the entire construction against uneven loading and torsion. The critical areas regarding durability are the design and maintenance of the nodes connecting the beams.

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Appendix 2 - Typology of tunnels

1. Cut and cover tunnels

Cut and cover tunnels are quite used in urban areas and are often constructed at the approaches to subaqueous vehicular tunnels due to the depth required. This type of tunnel is usually under the groundwater table and typically consists of massive reinforced concrete structures (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

According to the tunnel construction methods, the cut and cover method involves braced, trench type excavation (“cut”) and placement of fill materials over the finished structure (“cover”). The excavation is typically rectangular in cross section as shown in Figure A.1, and only for relatively shallow tunnels. The typical cut and cover tunnels were constructed using a steel frame structure with reinforces or unreinforced concrete between the frames. This method is referred to as jack arch construction (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

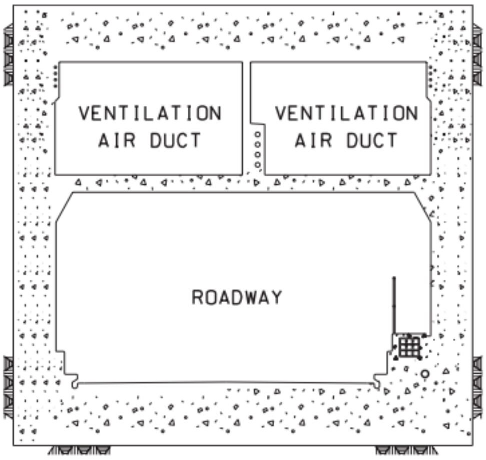

Figure A2.1 - Typical cut and cover road tunnel. From (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006)

2. Bored tube tunnels

Bored or mined underground tunnel construction is usually preferred when a tunnel is located at significant depth or when overlying structures exist above the tunnel alignment. Bored tunnels are often excavated using usually circular mechanical equipment, which gives the tunnel a circular cross-section (Figure A2.1) They can also be excavated using manual or mechanical methods and may be rectangular or horseshoe shaped. They can be separated in two groups based on the type of surrounding ground: soft ground tunnels and rock tunnels (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

In soft ground, the main concerns during excavation are associated with groundwater conditions and stability characteristics of the soil along the alignment. The control of groundwater is of outmost importance in soft ground tunnelling. On the other hand, for tunnels in rock the stability problems in blocky jointed rocks are generally associated with gravity falls of rock wedges from the roof and sidewalls (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006).

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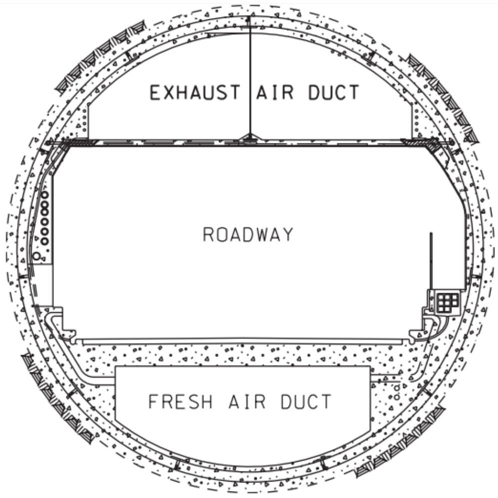

Figure A2.2 - Typical bored tube tunnel. From (Parsons Brinckerhoff Quade Douglas Inc.; Science Applictions International Corporation; Interactive Elements Incorporated, 2006)

3. Submerged Floating Tunnels

Submerged Floating Tunnels (SFT) are constructed in deep water, where conventional bridges or tunnels are technically difficult or prohibitively expensive. As any solid submerged into the water, the SFT has its own weight and buoyancy, which is according to its immersion depth volume. SFT working principle generally depends of their buoyancy, that means that the tunnel is designed for the buoyancy covers the structural weight, and the tunnel is then only subjected to an upward force, which is exerted by a fluid. The SFT internal arrangement and all related the cross-section size is determined by static and functional requirements such as strength properties, water tightening, stiffness properties, buoyancy ratio and size of infrastructures.

Most SFY cross sections that have been proposed are single circular, multi circular, rectangular, elliptical, oval, and polygonal cross sections. Several authors have investigated the influence of the cross section in the tunnel performance and some of them assure that "elongated shapes guarantee the best behaviour against hydrodynamic actions induced by waves and currents" (Ostlid, 2010). Nevertheless, many of cross sections that were mentioned have been constructed in the SFT projects. Nevertheless, many of cross sections that were mentioned have been constructed in the SFT projects due their good structural performance.

The SFT can be differentiated according to their support system, all of them are shown in Figure A2.3.

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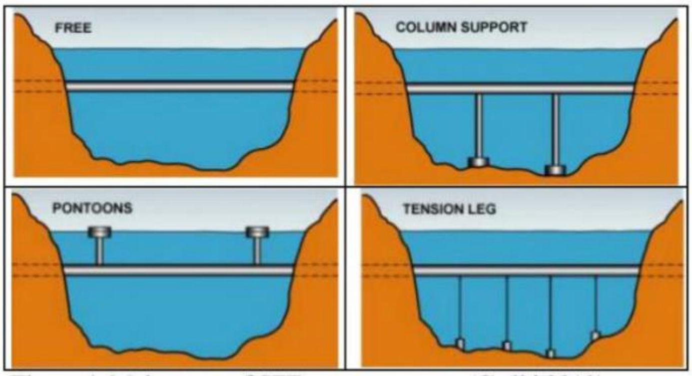

Figure A2.3 - Types of Submerged Floating Tunnels according their support system. Extracted from (Ostlid, 2010).

<u>Free Submerged Floating Tunnel:</u> This is characterized because has no anchoring at all except at landfalls. It have a free support system which is limited by the length of the tunnel, and it is independent of water depth. Support soil capacity need to be taken into account in the structural design of free SFT.

<u>Column Support Submerged Floating Tunnel:</u> Column support Submerged Floating Tunnels are usually employed for negative residual buoyancy (when the self-weight is greater than the buoyancy force). This type of tunnels are supported in columns which need to be in a seabed with good geological conditions in order to guarantee the pears position.

In deep waters the height of the column could generate instability in the tunnel structure, for this reason the water level plays an important role in these types of tunnels. The columns are usually fabricated using composite materials due the higher durability that is needed in submerged structures.

<u>Pontoon Support Submerged Floating Tunnel:</u> Pontoon support Submerged Floating Tunnels are independent of eater depth. Its design need to take in consideration wind and swell waves loads and the structure need to survive in case of a pontoon lost.

<u>Tension Leg Submerged Floating Tunnels:</u> In a tension leg Submerged Floating Tunnels, support system is a group of cables which are distributed along the axis support the structure. Difference to column support system, tension legs are usually used for positive residual buoyancy (when the self-weight is lower than the buoyancy force).

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Appendix 3 - Performance based vulnerability of arch bridges (example)

1. Introduction

Masonry, concrete or steel, arches transmit the loads from the superstructure diagonally to the substructure - an exception must be made to tied-arches. True arches are in pure compression, although most arch bridges resist a load combination of axial compression, bending moment, and shear.

Load paths vary depending on whether it is an open or close deck arch, a through arch or a tied arch. Load path determines regions with different vulnerability. Floor systems are a conjugation of beams, stringers, deck and eventually bracing, as well as bridge equipment. They shall be analysed according to section 2.3.

2. Close Spandrel Deck Arch

2.1. Masonry close spandrel deck arch

Vulnerable zones in masonry arch bridges are those locations/areas where the existence of certain damages has the highest probability to cause an undesired behaviour of the structure. The main deficiencies in masonry arch bridges are damages to foundations and superstructure. Observations related to foundations are local undermining, differential settlements and masonry dislocations due to loss of mortar joints. The main problem in identifying damage at foundations is the difficulty of inspecting.

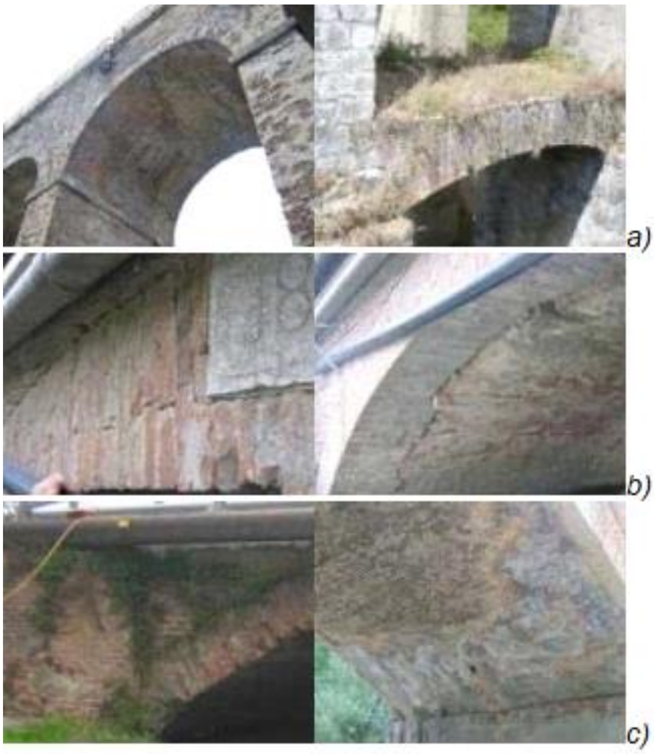

Figure A3.1 - Typical defects of masonry arch bridges, from (Zampieri, 2014) a) Degradation and loss of bricks, loss of mortar joints and salt efflorescence; b) longitudinal cracking; penetration by vegetation, fractures in piers and wing walls (taken from COST TU1406 WG3 report (Hajdin, et al., 2018).)

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Superstructure damages as presented in Figure A3.1 can be a result of:

- Processes causing loss of bricks, loss of mortar Joints and salt efflorescence in bricks. These mostly occur due to inadequate rainwater drainage, freeze-thaw cycles and penetrating vegetation (Figure A3.1a)
- Arch barrel deformations with longitudinal or transverse cracking; opening of arch joints and separation between brick rings in multi-barrel vaults (Figure A3.1b)
- Spandrel wall movements: sliding and bulging or detachment from the barrel - due to the low resistance of spandrel walls to out-of-plane displacement caused by orthogonal pressures due to the weight of infill, traffic loads and horizontal transverse seismic action
- Fractures/cracking in piers and wing walls (Figure A3.1c)

Damage assessment associated with the identification of causes should comprise the investigation of following:

Cracks in the arch transverse joints associated with the development of hinge mechanisms in the arch (Figure A3.2). Can be a sign of low material strength, excessive loading or occurrence of foundation settlements/rotations.

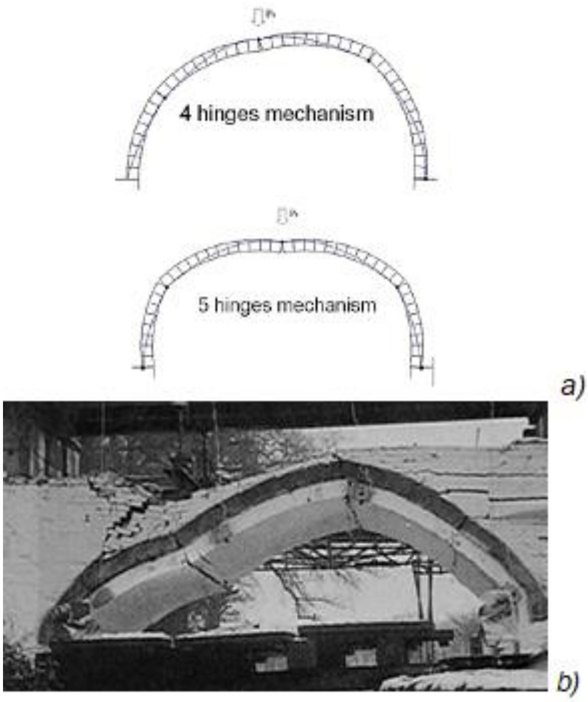

Figure A3.2 - Arch hinge mechanisms a) Four and five hinges mechanisms (Costa, 2009); b) Experimental testing (Page, 1987) (taken from COST TU1406 WG3 report).

Longitudinal cracks in the arch, along the spandrel-arch connection, likely related with the structural response in the transverse direction, typically influenced by the interaction between arches, spandrels walls and infill material (Figure A.5b).

Leaning and bulging of the spandrel wall as well as detachment between spandrel-arch connect ion (opening and slipping), likely related with the structural response in the transverse direction (Figure A.5b).

Longitudinal cracks in the arch, distributed along the arch - associated with the structural response in the transverse direction and foundation settlements.

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Diagonal cracks in the arch intrados - frequently associated with settlements (Figure A3.3a). Cracks in the intrados of piers and abutments - associated with foundation settlements and with its different configuration representing the different cracks' orientation (horizontal, vertical and diagonal cracks are likely related with larger vertical settlements in the central zone, in both external zones or in one external zone involving rotation, respectively) (Figure A3.3b).

Detachment between cutwaters and piers, as well as loss of stones and total or partial collapse of cutwaters - caused by the mechanical action of vegetation growth and by the river flow (with eventual solid material transported). Shallower foundations of cutwaters causing differential settlements between elements and the pier itself (Figure A3.3c).

Localized cracking in masonry units (blocks) - related with settlements and crushing due to (local) overstress.

Material deterioration of blocks and loss of mortar in the joints - frequently related to aging and/or caused by water flowing through the structure (Figure A3.3c).

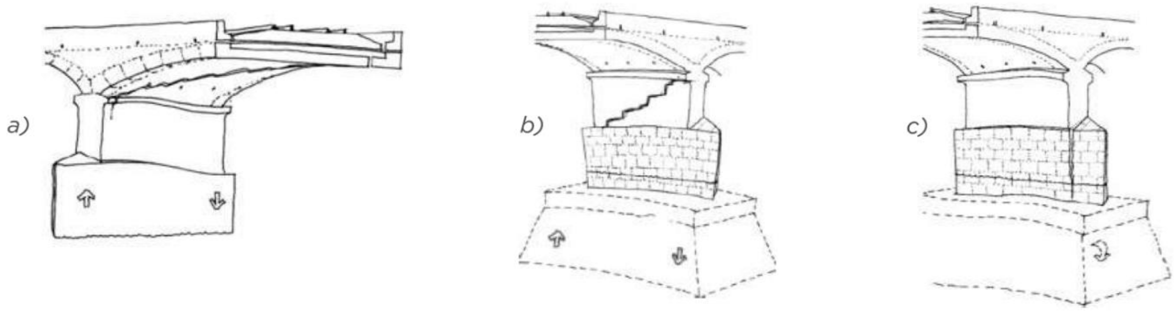

Figure A3.3 - Typical cracks in masonry arch bridges a) Diagonal cracks in the arch intrados; b) Cracks in the intrados of piers and abutments settlements; c) Detachment between cutwaters and piers, adapted from (García-Catalán & Álamo, 2006) (Extracted from COST TU1406 WG3 report (Hajdin, et al., 2018).

2.2. Concrete close spandrel arch

Concrete close spandrel arch bridges have to be assessed in a similar way to masonry close spandrel arches.

3. Open Spandrel Deck Arches

3.1. Concrete open spandrel deck arch

A segmentation should be made regarding regions with different vulnerability include *Bearing Areas, Shear Zones, Tension Zones* and *Compression Zones, Equipment, Foundation Settlement:*

Bearing areas can be potential vulnerable zones according to their load magnitude. Longitudinal cracks at the arch indicate an overstress condition and spalling of concrete would indicate loss of cross section. The second greatest bearing load magnitude occurs at the arch/spandrel column interface. The third greatest bearing load magnitude belongs to the spandrel column/cap interface. Existence of diagonal cracks in columns, beginning at the inside corner and propagate upward, with possible cause in differential arch rib deflections (Hartle, et al., 2002). The superstructure supports with thin sheets of another material (e.g. rubber, cooper) between the concrete elements (Figure A3.4), can caused heavy cracking both in the anchor zones (i.e. hinge zones) of the girders and in the pier cross-beams supporting them (Šavor, et al., 2009). At the ends of the spandrel bent caps, as well as other high shear zones in the floor system shear cracks can be formed due to high shear forces in these regions. Struts connecting arch ribs are also subject to torsional shear stress . Cracks

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in this region indicate excessive differential deflection in the arch ribs (Hartle, et al., 2002). Tension Zones exists in spandrel bent caps, spandrel columns and in the floor system. At spandrel bent caps maximum tension generally occur in the midspan at the bottom and, if exist, at the cantilevered ends at the top. Transverse cracks in the arch (oriented perpendicular to the arch member) may indicate tension and therefore an overstress condition. Spandrel walls are also vulnerable due to tension therefore the arch/spandrel wall interface can experience cracks, movement, and general deterioration of the concrete (Hartle, et al., 2002). Compression Zone exists throughout the arches and spandrel columns where buckling forces and bending moments can cause excessive surface stresses resulting in cracks (Hartle, et al., 2002).

Position of all equipment enables the proper functioning of the bridge and should be regularly inspected and replaced when it is necessary. Defects or damages of a drainage systems, pavement or railings reduce traffic safety on a bridge. Inadequate drainage of roadway and/or inside the arch box, but also damaged waterproofing or deteriorated expansion joints can cause deterioration of the structure due to combination of water retention and several processes e.g. carbonization, chloride induced corrosion, freeze/thaw actions, etc. Decayed bearings and expansion joints can cause unplanned structural movements and additional strain in concrete (Šavor, et al., 2009), (Kušter Marić, et al., 2016). Except conditions of an aggressive maritime environment or high seismic activity of the area, one of the most vulnerable zone of arch bridge as well as girder and frame bridges is bridge deck: due to direct exposure of the traffic and aggressive substances (deicing salts, water, CO, etc.).

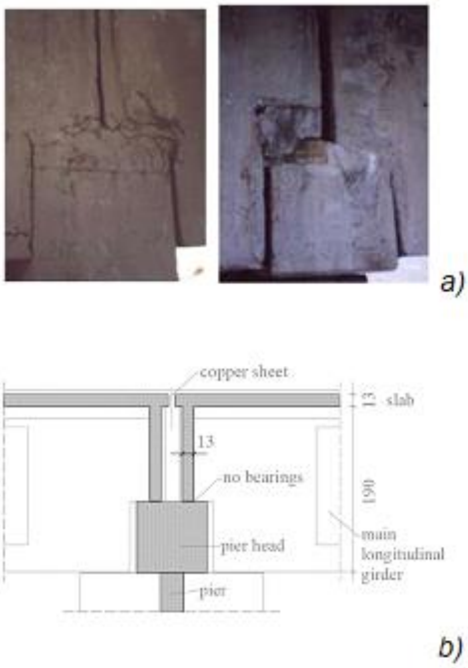

Figure A3.4 - Details of superstructure resting on the pier a) without bearings (Šavor, et al., 2009) and b) cracking of girder and pier head (HIMK, 2002) (extracted from COST TU1406 (Hajdin, et al., 2018)).

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3.2. Steel open spandrel deck arch

As the arch is the superstructure main load-carrying part, all its members can be considered as vulnerable. In addition to the arch members, vulnerable zones can be considered as the spandrel members in the deck arch.

Regarding *arch members* global alignment should always be checked for any movement, deflection or distortion. Alignment of bearings can denote global movements of the arch, as well as arch ribs buckling and crippling, and arch rib splice plates. Shear stress and overloading should be dismissed through the observation of defects/damages in bearings on each of the supports. On flexural zones, tension and compression flanges can denote signs of overstress, as well as the existence of cracks near connections in tension members. Web areas over the supports, including bearing stiffeners, jacking stiffeners and diaphragms, should be checked for buckling, crippling and loss of section.

In deck arches the end connection of spandrel columns and spandrel girders should be examined for cracks and loose fasteners. Signs of section loss and buckling damage are also important if observed in spandrel girders, spandrel caps or spandrel columns. In these elements signs of flexure overstress should also be examined.

Orthotropic steel plates, which are lighter than concrete decks, are often used for long-span bridges and viaducts in urban area which are subjected to topographical conditions or other factors. On bridges where an extremely large number of heavy vehicles travel, however, in recent years there have been several observed cases of deck damage due to fatigue. Fatigue of orthotropic steel decks may occur in locations that do not allow easy discovery of the precursors or progression thereof.

The Figure A3.5 shows an example of fatigue crack in an orthotropic steel deck found by a check after a deformation was spotted in the pavement This is s a vulnerable zone of a steel arch bridge.

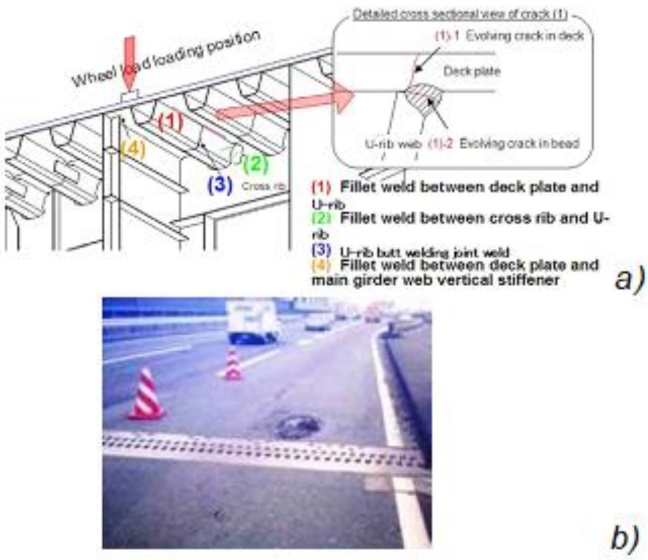

Figure A3.5 - Fatigue of orthotropic steel deck Example of damage to pavement caused by fatigue of orthotropic steel deck; b) Types of orthotropic steel deck fatigue crack (PWRI, 2022), (extracted from COST TU1406 (Hajdin, et al., 2018)).

Another vulnerable location is Stringer-to-Floor-Beam connections. In many existing bridges, mechanically fastened stringer-to-floor-beam connections are made using double angles riveted or bolted to the web plates of both members. Connections of this type have displayed high vulnerability to fatigue cracking, see Figure A3.6. Like stringer-to-floor-beam connections,

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connections between floor beams and main load-carrying elements or system (main girder, main trusses, arch ties, etc.) show also numerous fatigue problems.

a) b)

Figure A3.6 - Example of fatigue damage in stringer-to-floor-beam connections (Al-Emrani, 2002) a) at the junction between the rivet head and shank; b) at the external leg of the connection angle (extracted from COST TU1406 WG3 report (Hajdin, et al., 2018).)

4. Steel Through Arch and Tied Arches

For through and-tied arches *bracing* might be subject to tension or compression and should be inspected as any other truss. Top rib chord should be inspected as a compression member.

Hangers are subjected to large tensions therefore be critical. Alignment of the hangers, collision damage due traffic, corrosion and cracks near both ends are signs of distress. Any welded connection between hangers and attachments should be observed for cracks. Fatigue cracking at the connections of bridge hangers. There are two different mechanisms for fatigue cracking in these details:

Vibration : The slender hangers usually have very low bending stiffness, which makes them very sensitive to resonance. The cables can be excited by traffic loads on the bridge and/or wind loads.

Secondary stresses due to connection stiffness: Ideal moment-free pin connections do not exist in reality. Even when designed as such, a connection will always acquire some rotational stiffness inherent to detailing or gradually during the service life of the bridge, due to corrosion (so-called freezing)

Figure A3.7 shows an example of an arch bridge in which the hangers have developed fatigue cracks at their connections to the steel arch.

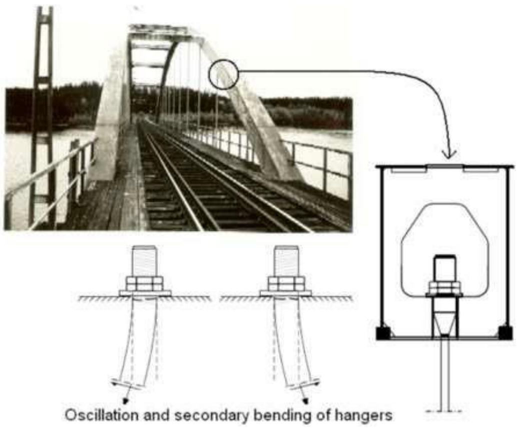

Figure A3.7 - Fatigue cracking of hangers, from (Kesson, 1991).

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