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REVIEW OF METHODOLOGIES AND INSTRUMENTS FOR DIAGNOSTICS OF TRANSPORT INFRASTRUCTURE

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 WP2 (Diagnostics of structures based on inspection, monitoring and testing) Task 2.2 (Review of methodologies and instruments for diagnostics of transport infrastructure) activities, listed as delivery D2.2. 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 monitoring and maintenance strategies, taking into account inspection, testing and monitoring data in safety assessment and maintenance approaches.

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WP2 contributes to the identification of the normative gaps with regard to standards in monitoring and maintenance, damage detection 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 2.2 is on diagnostics of structures based on survey data.

This report includes the review of the methodologies for inspection, the review of the damage detection indicators (DIs), procedures used for determining DIs based on condition survey data and KPIs from diagnostic testing and actions evaluation procedures. The conclusions from the review are used to formulate conclusion with regard to developments in standardization of concepts and methods for diagnostics of existing structures with includes a proposal for the development of a Damage Classification database.

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1 Problem statement

1.1 Introduction

In recent years, we have faced rapid growth of testing, inspection, and monitoring technologies in various sectors. In the domain of transport infrastructure, intensive research has been carried out to enable the use of these technologies to support asset management of bridges and tunnels.

Accurate information from monitoring of structures is crucial to take the right decisions on maintenance and safety; unfortunately, there are gaps in the existing European standards and the monitoring practice at national level: accepted and harmonised approaches to condition survey and diagnostics of structures are lacking until now. This hinders asset owners and public authorities in charge of maintenance of the transport infrastructure to apply the latest development in maintenance strategies.

Moreover, structural monitoring is not addressed in the current Eurocodes (CEN/TC 250), and the existing standards on monitoring are not consistently interpreted and implemented in different European countries due to a lack of coherent policies and the gaps in knowhow.

The current standards do not embed the full extent of knowledge on analysis of extensive measurement and monitoring data to provide input for optimal maintenance strategies and safe operation of the infrastructure and on the adoption of digital technologies beyond the conventional inspection methods. The high diversity of transport infrastructure assets and their environments add to the complexity for standardised monitoring.

Specifically, on Structural Health Monitoring (SHM) only a few technical guidelines and standards available, which are only valid at the national level and very largely diverse in their approaches. Some examples are the Italian standard “UNI/TR 11634:2016. Linee guida per il monitoraggio strutturale”; the Austrian standard “RVS13.03.01. Monitoring von Brücken und anderen Ingenieurbauwerken”; the SAMCO “Guideline for Structural Health Monitoring”; the Chinese standard “GB 50982-2014 Technical code for monitoring of building and bridge structures”; and the Canadian “Guidelines for structural health monitoring”. The guidelines provide the classification of the monitoring systems, goals of the monitoring activities, treatment of the monitoring data and damage identification techniques. However, the guidelines for SHM are often used incorrectly.

Foroptimal safety, availability, and cost-effectiveness of transport infrastructure, IM-SAFE envisions a paradigm shift from the time-based/corrective maintenance towards risk-based/predictive maintenance through data-informed decision-making enabled by a new and harmonised European standard for monitoring, including a standardised digitalisation approach. The new standard should be supported and implemented coherently by the public authorities and the industrial stakeholders across Europe.

The gathered data will be used for assessing the actual safety and the risk levels of the structure as well as for predicting the future safety and risks. This is currently possible by using the latest digital innovations that integrate structural models, predictive degradation models and data analytics techniques. The combined human expert and artificial intelligence will result in the transformation of measured data into knowledge about the performance and safety of the structure.

In this context, the advantages of performance monitoring will be fully exploited and compared to the conventional approaches based on visual inspections and non-destructive testing (NDT).

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The expected result of IM-SAFE is, among others, to realise this vision by filling-in the gaps in the current standards and closing the gap between the standard and the practice. IM-SAFE will lead to new European standards for monitoring and maintenance of the structures, and the rules in the structural design codes (Eurocodes). The essential aspects that will be included are:

- Guidance about the physical parameters to be monitored and the performance and predictive analysis
- Guidance on how the monitoring data can inform the safety assessment and maintenance approaches
- Requirements on the algorithms used for damage identification
- Requirements on reliability, robustness, installation, operation, maintenance of sensors/monitoring systems

1.2 Diagnostics of Structures based on survey data

Condition survey seeks to gain an understanding of the current condition of the structure with the purpose of diagnostics. Diagnostics of a structure is a process of reviewing the structure and/or network-specific data gathered from inspection, monitoring and testing that allows the identification of the current condition of the structure., as well as the prediction of damage and prognosis of future performance. Diagnostics of structure comprises damage detection, actions evaluation and identification of system limitations. Data collection can be performed either manually or (semi-)automatically using a wide variety of technologies and techniques. The method and frequency of data collection depend on the type and size of the infrastructure assets owned/operated by a public authority or a company.
Structural Health Monitoring (SHM) to support maintenance strategies has been proposed during the last decades: embedded or external SHM encompasses Non-Destructive Testing (NDT) in order to provide damage detection and condition assessment throughout sensors and the analysis of the current condition of the structure.
Shortcomings of the actual practice and the gaps between current standards and practice to be tackled by IM-SAFE are:

- Many existing data collection, monitoring and inspection protocols are insufficiently covered in standards and the fragmentation in responsibilities; for instance, procedures of bridge maintenance lie within national legislations, but no specific standard on bridge monitoring is in place beyond some common practices.
- There is still an urgent necessity for the harmonisation of the developed techniques for SHM-based condition assessment and risk evaluation. Moreover, risk-based inspection and infrastructure monitoring standards are not yet unified through the European Union. Although there are international standards, they are not directly applicable in the EU countries because they do not address European-specific issues. Therefore, the development of standards and guidelines that guide the SHM of transport infrastructure is crucial.
- Consolidated Key Performance Indicators (KPIs) and common agreements of the methods for quantification of the relevant KPIs are needed in order to validate the monitoring and inspection actions and results.

In contrast to the traditional maintenance at predefined intervals, predictive maintenance responds more effectively to the changes of condition of the asset. In the IM-SAFE approach such a strategy can be supported by real-time monitoring of the conditions of a transport infrastructure asset in combination with predictive structural and degradation models. Combining a model-driven approach (using data in a predefined structural model) and a data-

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driven approach (deriving a model from gathered data) will ensure optimal maintenance and safety.
Aligned with these technologies, IM-SAFE evaluates the methods (such as Modal Analysis (MA), statistical analysis methods, Big Data Analytics and Artificial Intelligence.) for the analysis of data obtained from monitoring and NDT technologies. To this end, new algorithms and relevant utilisation of Artificial Intelligence / Machine Learning for diagnostics incl. damage detection and condition monitoring need to be explored and standardised. Techniques such as anomaly detection and forecasting can be applied to time series dataset, such as measures collected by sensors (accelerometer, inclinometer, etc.) installed on civil infrastructures. In the case of anomaly detection, a machine learning (or deep learning) AI model can be trained to learn the "normal" (intended as reference) state of an infrastructure and then be able to detect "anomalies" (intended as deviations from the reference state) that might not be easily observable from a direct analysis of the data. In some cases, if the engineer has observed and annotated specific anomalies in previous time series, the AI model can be trained to detect specifically those anomalies, turning the problem to a classification exercise. For a large number of assets, the model creation for each specific asset can be automated, thus enabling large scale monitoring.

This new approach has been examined through a preliminary **S**trengths-**W**eaknesses-**O**pportunities-**T**hreats (**SWOT**) analysis, which showed that:

- The greatest strength lies in the quantitative recording of inventory and status data, which allows better condition predictions and savings through the efficient use of means in the life cycle.
- The biggest weakness lies in the correct gradual implementation with targeted investments in research, software and IT as well as training and integration into the company processes.
- The greatest opportunity lies in building up efficient, transparent decision-making systems as a modern operator and thereby leveraging considerable efficiency potential.
- The greatest threats are poor strategies and implementation, which means that despite the investment, neither the desired efficiency gains nor the necessary acceptance in the company can be achieved.

Within this context, the objectives of task T2.2 are:

- To evaluate the methodologies and instruments currently available for diagnostics of structures, incl. damage detection.
- To identify the required technical guidance to incorporate the available data collection technologies and data analysis methods in diagnostics of structures.

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 diagnostics of structures with a focus on the methodologies and instruments for diagnostics of transport infrastructure. 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

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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 diagnostics of structures based on survey data and for a new standard for monitoring strategies. The activities for Task 2.2 in the context of WP2, which are documented are as follows:

- the review of the damage detection indicators and procedures, including principles of:
  - Identification of Damage Indicators (DIs), related to the road infrastructure Key Performance Indicators (KPI), to be measured and evaluated for the purpose of damage classification.
  - Procedures for determining DIs based on condition survey data.
  - Procedures for damage classification, accounting for type, size and location of defects or other relevant issues depending on the type of structure, the actions on structure, and the risks that may potentially affect the structure in the future, such as the one following from changes in traffic loads or service life demand, and from resilience issues related to climate change and increased use (ref. to EU projects like SAFEWAY).
  - Procedures for including DIs in evaluating the KPIs.
- Actions evaluation procedures, including principles of:
  - Procedures for determining actions on structures based on condition survey data.
  - Procedures for including actions determined from condition survey in risk and safety analysis.
- Identification of system limitations, including:
  - Procedures for determining deviations from design specifications based on condition survey.
  - Procedures for assessing KPIs from diagnostic testing.

1.5 Report contents

Chapter 2 provides an overview of damage and degradation mechanisms, with a focus on damage causes and effects and the proposal of a damage detection procedure based on existing damage evaluation approaches. Principles on damage and performance indicators and the description of survey techniques and data analysis methods for damage identification, localization, quantification and prediction are also provided.

In Chapter 3 a classification of inspection methodologies is given. Each type of inspection is described with respect to its objectives, frequency of execution, data collection methods and outcomes.

Chapter 4 introduces the objectives and requirements of monitoring systems and their classification based on the period of execution and on the purpose. Monitoring system architecture and design process are described. Additionally, guidance on monitoring system installation and management and data acquisition, processing and treatment is given.

Chapter 5 is focused on damage and degradation processes. For each damage process the description, the identification of related DIs and PIs and of the survey techniques used for the detection is given. Survey techniques are classified based on a colours rating scale with respect to selected parameters (e.g., availability, cost, efficiency). Data analysis methods related to each damage process are also given.

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Chapter 6 describes the damage detection procedure introduced in chapter 2, providing for each phase (damage identification and localisation, quantification, prognosis and monitoring) the description of the decision-making procedure.

In Chapter 7 a procedure for damage classification and for the development of a damage classification database is given.

Chapter 8 focuses on actions evaluation, including general information on each type of action, the surveying technologies used for their evaluation and principles on how to use data for actions modelling.

Lastly, in Chapter 9 a description of the decision-making process for model updating purposes, model calibration methods and the procedures for assessing KPIs from diagnostic testing is provided.

Summary and conclusions of the report are given in Chapter 10.

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2 Overview of damage and degradation mechanisms

2.1 General

In the context of the IM-SAFE project, “**damage**” is defined as a disruptive change in the condition of a structure, structural components or structural members that can unfavourably affect its current of future structural performance. Aiming to classify and describe damage and degradation processes, the following distinction has been made ([1]).

- **Degradation**, defined as the worsening of the performance of material over time, as the result of (complex interaction between) degradation mechanisms, which are (chemical, physical, mechanical, biological or multi-type) root cause of damage.
- **Deterioration**, which is defined as the progressive reduction in the ability of a structure or its components to perform according to their intended functional specifications. Typically, deterioration of a structure or its components will be driven by degradation of materials.

Structures, indeed, are inevitably affected by damage processes, due to which deterioration processes may arise and, therefore, lead to durability, safety, and serviceability problems. of both single and/or multiple elements. Hence, it is crucial to optimize inspection and maintenance strategies based on reliable information on the processes.

According to [2] and [3] damage processes can be classified as follows:

- They may affect structures and structural components at the local or global scale.
- They may act singly or in combination.
- They may be gradual and observable or gradual and non-observable.
- They may have a chemical, physical, biological, or mechanical origin or even a combination.

Basically, a damage assessment should consider the nature, the intensity, the extent, and the location of each damage process. In this respect, since deterioration processes have different origins and acting timespan, a damage detection procedure is proposed in [2.5], in order to analyse data for defining indicators that are capable of evaluating the effects of acting identified deterioration processes. Some examples for damage processes are provided in Figure 2.1.

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a) Corrosion
b) Alkali-aggregate reactions
c) Fatigue
d) Freeze-thaw

Figure 2.1 – Examples for damage processes

2.2 Damage and degradation mechanisms

This section overviews the principal damage and degradation mechanisms that affect structures, resuming the classification of damage processes and the distinction between concrete and steel structures made in [4].

Table 2.1 – Damage processes for reinforced concrete and steel structures

, for instance, shows a general overview of chemical, physical, and biological processes considered in the context of the IM-SAFE Project.

	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		

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		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

Table 2.1 – Damage processes for reinforced concrete and steel structures [4]

Table 2.2 shows the damage processes with respect to the materials and its impact on the structure.

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		•		•	•	•	

Table 2.2 – Damage processes [4]

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2.2.1 Bridges

There are three main groups of degradation mechanisms in connection with physical, chemical and biological phenomena. Table 2.3 shows these groups in detail with regard to the correlated damage classes for bridges.

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]		□			□	

Table 2.3 – Degradation mechanisms of concrete (a and steel (b bridges and associated classes of defects [4]

Defects associated with bridges can be subdivided into classes, types and categories. Table 2.4 shows 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.

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	Change of the chemical characteristics	Change of concrete characteristics[a]

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of material		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
	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 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

Table 2.4 – Classification of the defects of concrete and steel bridges [4]

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2.2.2 Tunnels

There are three main groups of degradation mechanisms in connection with physical, chemical and biological phenomena that can also be applied to tunnels. Table 2.5 shows these groups in detail with regard to the correlated damage classes for tunnels.

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

Table 2.5 – Degradation mechanisms of tunnels and associated classes of defects [4]

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

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 (Table 2.6).

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

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	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

Table 2.6 – Classification of the defects of tunnels [4]

2.3 Damage causes and effects

2.3.1 Damage causes

Damage of structures may arise for several reasons, which can be grouped in the following clusters, represented in Figure 2.2.

- **Actions**:
  - Accidental actions;
  - Environmental actions;
  - Operating conditions.
- **Deterioration processes**;
- **Construction and design errors**:
  - Construction errors;
  - Design errors.
- **Change of use**.

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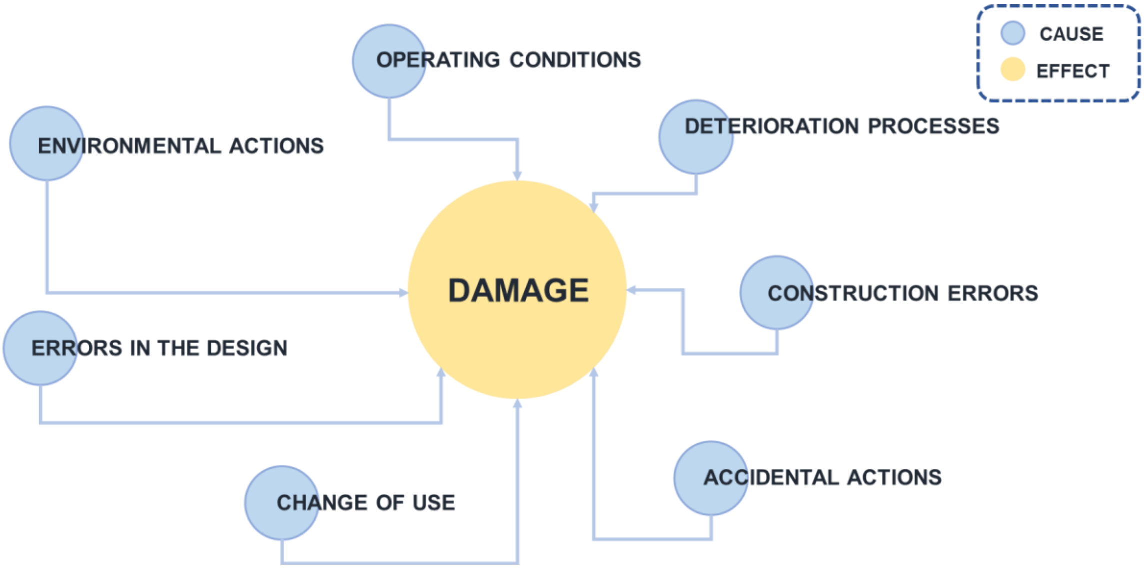

Figure 2.2 – Damage causes

2.3.1.1 Actions

The three main types of actions are represented in Figure 2.3:

Figure 2.3 – Main types of actions

Some examples for each type are given in Table 2.7.

ACCIDENTAL ACTIONS	• Accidental external or internal gas explosions • Accidental impact by various forms of vehicle, including road vehicles, aeroplanes, trains, etc.
ENVIRONMENTAL ACTIONS	• Snow actions • Wind actions • Thermal actions
OPERATING CONDITIONS	• Overloading due to misuse • Overloading caused by exceptionally strong winds or heavy snow • Fatigue

Table 2.7 – Examples for each type of action

2.3.1.2 Degradation/deterioration processes

As outlined in paragraph 2.2, structures are affected by degradation and damage processes, which can lead to durability, safety, and serviceability issues. Degradation may occur due to

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chemical, physical, mechanical, and biological processes, which are summarized in 2.2.1 and 2.2.2 for bridges and tunnels, respectively.

2.3.1.3 Construction and design errors

Errors can occur during the design and/or construction stages. Moreover, it should be noted that design errors are not always detected before construction has begun, and this leads to significant consequences. According to [5], most of these types of errors are due to lack of understanding of basic engineering methods, inadequate development of details, or sometimes to last-minute changes without proper assessment of the consequences of these changes. Construction errors may be related to inadequate understanding of the design concept or to last-minute changes in the construction sequence.

2.3.1.4 Change of use

Every structure is designed for a specific use and must meet specific requirements for serviceability, structural safety, durability, sustainability, and reliability during the entire required service life. If that use changes, the structure may not meet the new requirements.

This may concern in particular bridges, which, for instance, when subjected to a change of the road category, may not bear the increase of variable loads (e.g., traffic loads) and therefore damage may occur.

2.3.2 Elements affected by damage

In section 2.3.1 damage causes are categorized. Consequently, a categorization of the levels on which damage may act is given here (Figure 2.4):

- Material;
- Section;
- Structural component;
- Structural system.

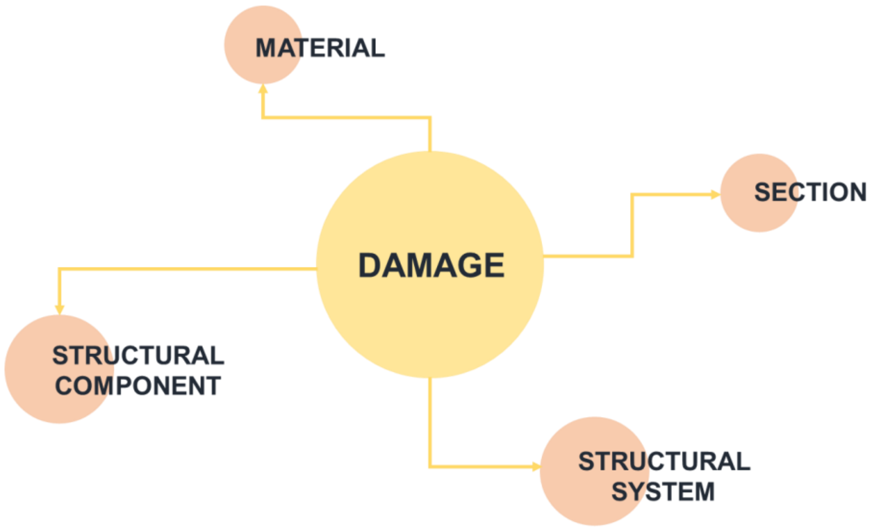

Figure 2.4 – Damage causes

Each of the causes identified above may have different consequences with respect to their level on which the damage acts:

- Actions (accidental, environmental, operating) can affect both material, section structural components and system.

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- Degradation/deterioration processes (chemical, physical, biological) can affect both material, section and structural component.
- Construction and design errors can affect both material, section, structural components and system.
- Change of use can affect both material, section structural components and system.

The relationship between each damage cause and the relative effects is summarized in Figure 2.5.

CAUSE		EFFECT			
		MATERIAL	SECTION	STRUCTURAL COMPONENT	STRUCTURAL SYSTEM
ACTIONS	ACCIDENTAL ACTIONS ENVIRONMENTAL ACTIONS OPERATING CONDITIONS	×	×	×	×
DEGRADATION/ DETERIORATION PROCESSES	CHEMICAL PHYSICAL BIOLOGICAL/ORGANIC	×	×	×	
CONSTRUCTION & DESIGN ERRORS	CONSTRUCTION ERRORS DESIGN ERRORS	×	×	×	×
CHANGE OF USE	-	×	×	×	×

Figure 2.5 – Correlation between damage causes and effects

2.4 Basis for damage evaluation

Indicators allowing for assessment of structural integrity, state of deterioration or condition assessment can be derived in various ways from experimental data: significant changes in the data relating to the condition of the structure may be indicative of a developing failure. However, such deductions can only be established through an adequate process of data assessment and evaluation, which enables an appropriate context-based engineering interpretation to be made of the indications obtained.

Damage evaluation, therefore, is based on a condition assessment, which relies on the potential to capture deviation from a reference condition, usually represented by a specified set of limit states, which separate desired states of the structure from adverse states. Beyond serviceability and limit states criteria, in fact, it is possible to define limit states based on the condition on the structure, such as the partial damage limit states [6]. The main system properties or a model that reproduces the system response are determined based on a **system identification** process, which, according to [3], often refers to the estimation of physical model parameters, such that the existing numerical model best reflects the structural behaviour observed experimentally.

The Damage Indicators (DIs) may be used to capture deviations from the reference condition, considering directly the monitored variables (e.g. strain) measured by means of dedicated sensors or tests, or more indirect properties, for instance continuous response functions or modal parameters.

Depending on selected Dis, further damage measurements may be performed with visual and/or in-depth inspections. The choice of the indicators and their combined evaluation aims to provide a univocal, exhaustive, and as much as possible unambiguous interpretation of the actual structural behaviour.

Damage identification methods can be classified in the following categories:

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- **Data-driven approaches;**
- **Model-based approaches.**

Data-driven and model-based methods for damage detection, localization, characterization and prediction are illustrated in 2.4.1.1 and 2.4.1.2.

The identification of damage generally is described by the indication of a four Level process:

1. **Damage identification and localization**;
2. **Damage quantification;**
3. **Damage modelling and prediction**;
4. **Damage monitoring**.

Damage identification usually requires the analysis of responses recorded at several locations, i.e., a higher spatial resolution in the description of the structural condition but at a certain extent can still be performed with a data driven approach provided spatially distributed damage features, for example modal shapes, are used.

The evaluation of the type and severity of damage, typically requires a model-based approach to map the monitored response to different damage types and scenarios.

2.4.1 Damage evaluation approaches

The observation of the structure over time is obtained by measurements, which should be analysed to determine the current state of the structure and to reflect the capability of a structure to continue to perform its intended function after being exposed to aging and the accumulation of damage. Since damage significantly alters the properties of the structure, information gathered from measurements and SHM monitoring systems are key factors in damage evaluation. Model-based methods, as well, can be used for real-time damage identification, updating significant parameters of a numerical or finite element model.

Hence, damage evaluation can be performed with both data-driven approaches and model-based approaches, which are described in the following paragraphs.

2.4.1.1 Data-driven approaches

Data-driven methods extract damage features relying solely on the recorded response and rely on picking up shifts of or differentiation in damage sensitive features, with respect to those corresponding to regular operating conditions. These methods are hence attractive for adoption within real-time damage identification but the absence of a numerical model of the system often hampers the estimation of damage severity.

On the purely data-driven front, the advent of advanced and increasingly affordable monitoring technologies has allowed for continuous monitoring of structures based on use of heterogeneous sensors, which are able to deliver a "Big Data" stream of monitored information. Naturally, the treatment of this data requires suited processing tools, which are typically tied to data-driven time-series analysis, statistical methods and Machine Learning (ML) schemes.

2.4.1.2 Model-based approaches

Model-based methods most typically rely on the use of the damage features to update significant parameters of a numerical, often finite element (FE), model. They are usually less attractive for real-time damage identification, due to the computational cost of the updating process, but the availability of a FE model allows for higher-end damage characterization and, if a degradation model is available or can be defined based on monitored data, they can further provide salient information regarding the remaining service life of the structure.

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In model-based methods, damage is detected through the updating of the parameters of a numerical and/or probabilistic model, in which the parameters related to resistance and actions can be modelled with probability density functions.

The numerical 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, thresholds. should be consistent with the ultimate or serviceability requirements and, additionally, consider the continuously increasing loss of function due to the damage response characteristics of the structures. In order to maintain a prescribed performance level, it is possible to distinguish between:

- Attention thresholds
- Alarm thresholds

Additional threshold levels characterized by different severity degree may be considered based on specific needs (e.g., road operators and asset owners operational requirements).

When a model is inserted in the analysis loop, it is possible to more straightforwardly achieve higher end SHM tasks, such as localization and quantification, albeit at the cost of higher computational effort.

For further information on model updating techniques, see Chapter 8 of [4].

2.5 Damage detection procedure

The purpose of the damage detection is to determine whether the structure under examination is affected by a damage process, and which are the causes for the occurrence of damage.

Hence, based on the existing literature ([7], [3]), the following damage detection procedure is proposed and envisages a four Level process:

1. **Damage identification and localization**;
2. **Damage quantification**;
3. **Damage modelling and prediction**;
4. **Damage monitoring**.

2.5.1 Damage identification and localization

The first step for the damage detection procedure consists in the **identification** of the type of process(es) responsible for the damage, which can have a chemical, physical, biological, or mechanical origin or even act in a combined way. It also consists in the **damage localization**, which is defined as the process, deterministic or probabilistic, of ascertaining where the damage to structure is located.

2.5.2 Damage quantification

The second step for the damage detection procedure consists in the **damage characterization**, which is defined as the process, deterministic or probabilistic, of determining the time of occurrence, , the size and other features of the damage, such as the origin of the damage and the current progress in the damage phenomenon (e.g., carbonation front depth, crack width, etc.). .

2.5.3 Damage modelling and prediction

The third of the procedure consists of damage modelling and prediction, which aims to control the evolution of damage over time.

2.5.4 Damage monitoring

The last step of the procedure consists of the damage monitoring.

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2.6 Damage and Performance Indicators

Damage indicators (Dis) and performance indicators (Pis) are used for the implementation of management strategies of infrastructure networks and infrastructure objects, which must be kept at a desired performance level.

These indicators can be qualitative or quantitative based, and they can be obtained during principal inspections, through a visual examination, a non-destructive test or a temporary or permanent monitoring system: these values are the basis for the assessment of bridges and tunnels state condition.

As outlined in [4], damage processes, if detected, must be documented as observations by the bridge inspectors. Inspectors are also responsible for observing and assessing bridges/structures to determine those damage processes that are most likely to affect the structure or its components: the inspector must make a meaningful diagnosis with the help of the observations and knowledge of damage processes.

Relevant observations taking place during 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).

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.

For further information about Performance Indicators and their categorization, these concepts are thoroughly described in [4].

Hence, in the context of the IM-SAFE Project, with regard to the data used in a performance assessment, the following distinction has been made:

- **Data coming from documents**
- **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.
- **Performance Indicator**: an observation or a parameter derived from observations that quantitatively describes property of the structure and/or of the aspect of its performance and are used to qualify fitness of the structure for its purpose during service life.
- **Damage indicator**: an observation or a parameter derived from observations that serves for quantitative or qualitative damage detection, damage localization and/or damage characterization.
- **Monitoring parameters**

The categories of data described above enable the characterization of the performance of the structures.

Once it has been assessed that a damage process, whether active or inactive, is occurring, then it could be identified with a damage indicator (DI), which may coincide with a performance indicator and/or an observation or not.

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Observations, Pis and Dis can be obtained directly from the raw data or as a result of a data processing process. In Figure 2.6 the relationship between the different types of data is represented: it is possible to see that observations, PIs, DIs, and data coming from documents are closely related, since both observations and data coming from documents sets overlap part of the PIs and Dis ones. The data processing boundary line separates the area on the chart where raw data can be used from the area where post processing is required.

Parameters to be monitored can be selected by the data set of observation, PIs and DIs.

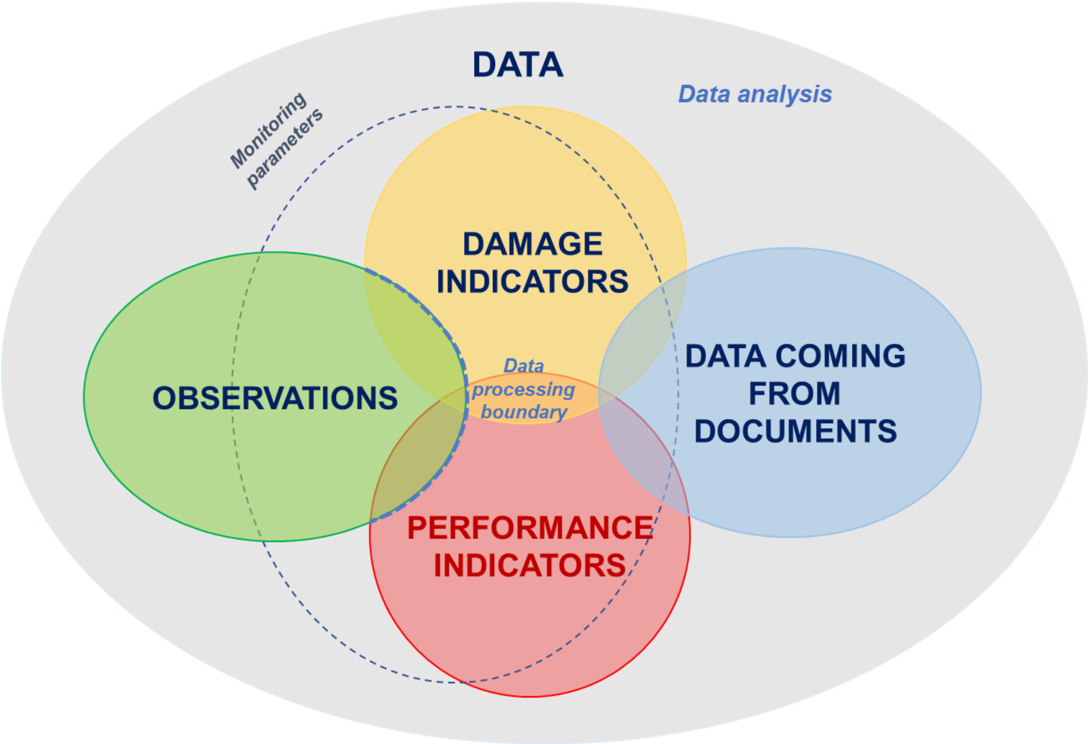

Figure 2.6 – Correlation between observations, DIs, PIs and data coming from documents

It should be noted also that all observations collected through inspection and maintenance activities must be included in reports, which then become also part of the data coming from documents set.

2.7 System limitations

Diagnostics of structures should consider also risk arising from system limitations, since it can increase due to degradation and damage processes. The concept of “system limitations” is not related to loads acting on structures, and neither are to deterioration mechanisms. It is directly related, instead, to the intrinsic properties of the structures and therefore to their design and construction and to the vulnerability, which is defined as the degree to which a system, or part of it, may react adversely during the occurrence of a hazardous event. This concept implies a measure of risk associated with the physical, social and economic aspects and implications resulting from the system's (e.g. bridges or tunnels ability to cope with the resulting event).

Hence, each structure is characterized by vulnerable zones, which can be detected with several methods [4]. A typical example is a Gerber hinge in a girder or frame bridges. They consist in a Gerber deck, suspended span or reduced end beam, supported on nibs of abutments or adjacent beams. Their position influences the magnitude and the distribution of stresses and the sensitivity to actions such as differential settlements or thermal variations.

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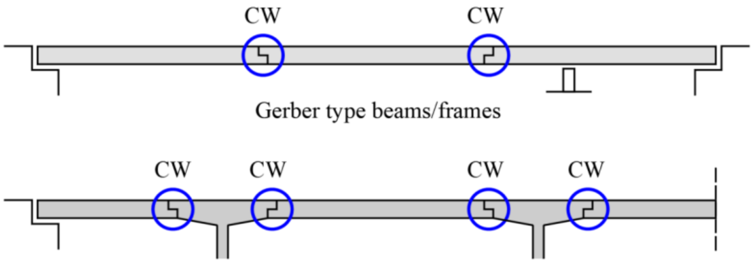

Figure 2.7 – Conceptual weaknesses in a bridge deck (Extract from [4])

Another example of conceptual weakness may be associated to cross-sections, such the precast multicellular one:

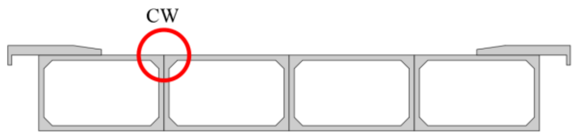

Figure 2.8 – Conceptual weaknesses in a precast multicellular cross section (Extract from [4])

The limitation lies in the fact that these types of elements, generally, can hardly be inspected without a traffic disruption, so, if not properly designed and/or protected during construction and operation, costly interventions are needed.

Locations of vulnerable zones in girder and frame bridges are presented in [4].

2.8 Survey techniques and data analysis methods for damage identification, localization, quantification, and prediction

The assessment of a deteriorating structure requires the use of inspection and/or testing techniques to define the mechanism(s) and severity of any structurally significant degradation, as well as to be able to make a prognosis about future condition. Defects associated with tunnel and bridge structures can be subdivided into classes, types and categories defined in [4].

Data about the structure can be gathered in several ways by the following types of activities:

- Inspections
- Survey
- On-site testing and measurements
- Material sampling
- Laboratory testing
- Monitoring

Each type of information that can be obtained from the activities above represents a specific level of detail: the relationship between the expense, in terms of extent and number of measurements, relative to the insight/information gained is represented in Figure 2.9.

The execution of good quality tests is crucial for a correct decision-making in the perspective of condition control and optimum maintenance.

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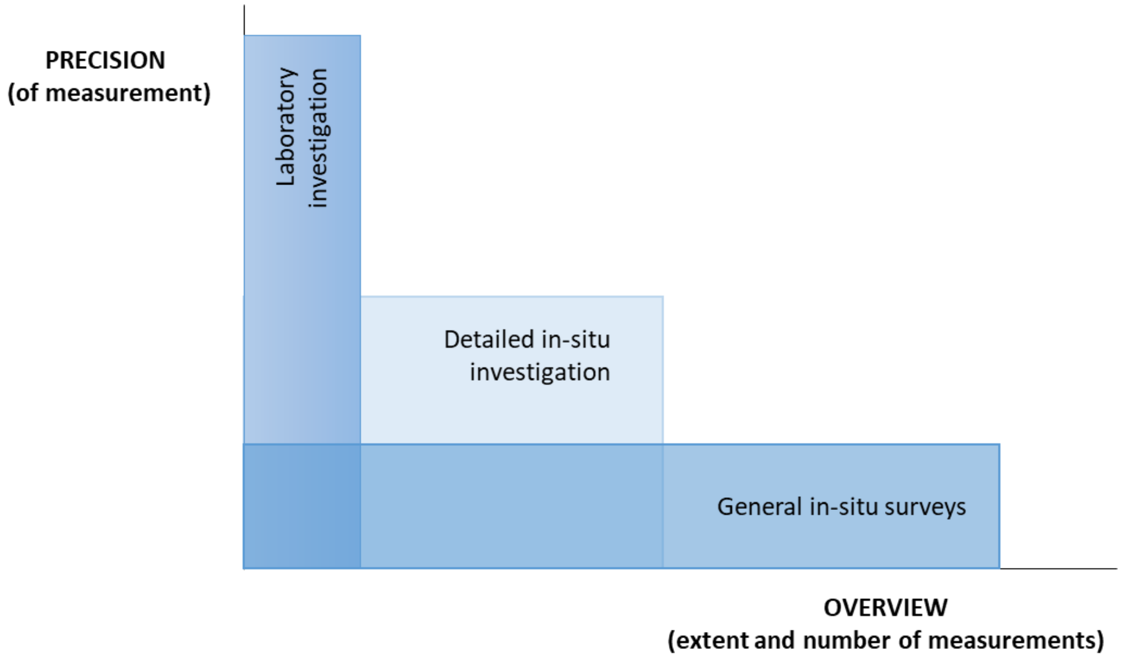

Figure 2.9 – Type of information in dependence of the different levels of investigation (Adjusted from [8])

According to [9], locations of the inspection, survey, testing, investigation, monitoring, or other information gathering or condition monitoring activities should be carefully selected so that the desired information about the current condition and deterioration of materials and/or structural performance can be obtained, considering factors such as:

- The likely mechanism(s) and rate of deterioration
- The environmental conditions
- The conservation strategy and tactics, together with the inspection, testing, investigation and monitoring regimes defined at the time of design or re-design.

Three different inspection levels can be distinguished:

I. **Preliminary assessment (Visual Inspection)**: consists of a preliminary assessment of the structure based on information obtained through inspections, study of the available historical project documentation and a rough check on the structural safety.

II. **Detailed investigation**: consists of detailed inspections, tests and monitoring. The detailed inspection of the structure, which aims to dispel or confirm any doubts as to whether the structure is safe.

III. **Assessment and prediction by advanced analysis**: includes structural monitoring and modelling. This inspection level might be used for problems with substantial consequences, for which an advanced analysis may be needed.

For further information on inspection levels and their use in performance assessment, see Chapter 9 of [10].

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Each inspection level is based on the use of surveying techniques, which can be classified based on several criteria.
Firstly, based on the impact of testing on structures, it is possible to distinguish:

- Destructive techniques
- Non-destructive techniques
- Semi-destructive techniques

Based on the environment in which tests are performed, it is possible to distinguish:

- In-situ surveys
- Laboratory investigations
- Remote analysis

Based on the object under investigation, instead, the following categorization is made:

- Local analysis
- Global analysis

Lastly, concerning the type of information, data can be qualitative or quantitative.

In the context of IM-SAFE project, surveying technologies are categorized as summarized in Figure 2.10.

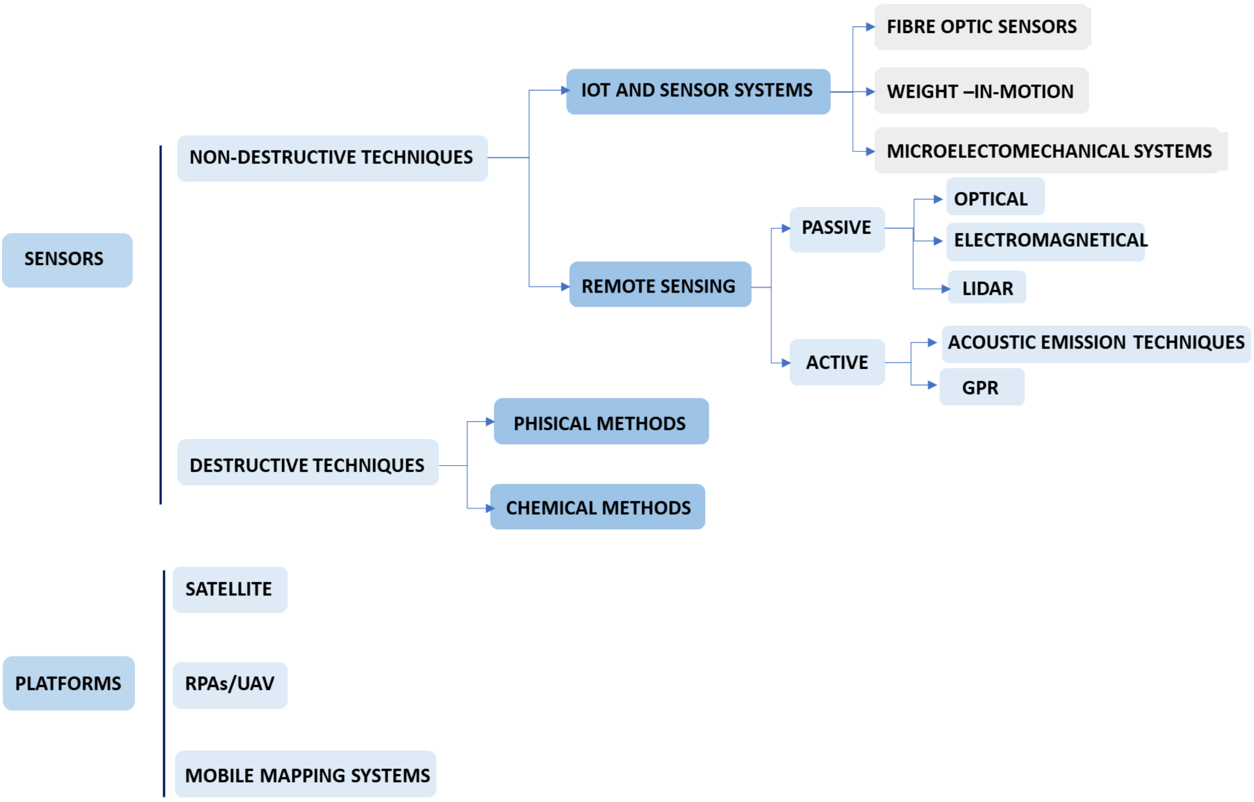

Figure 2.10 – Surveying techniques classification

Hence, surveying techniques are divided in the following main categories based on technology involved.:

- **Sensors** involve destructive and non-destructive techniques, which include either IoT and sensor systems/remote sensing or physical/chemical methods, respectively.
- **Platforms** include satellite, RPAs/UAV and mobile mapping systems.

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The classification of surveying techniques described in [11] based on the principles described above is summarized in Table 2.8.

CLASSIFICATION SYSTEM	TECHNOLOGY		IMPACT			TYPE OF DATA		DATA ACQUISITION		METHOD OF TESTING		TYPE OF METHOD		ENVIROMENT OF TESTING			ANALYSIS	
SURVEYING TECHNOLOGIES	SENSORS	PLATFORMS	DT	NDT	SDT	QUALITATIVE	QUANTITATIVE	IoT AND SENSORS SYSTEMS	REMOTE SENSING	ACTIVE	PASSIVE	PHISICAL METHODS	CHEMICAL METHODS	IN SITU	IN LAB	REMOTE ANALYSIS	GLOBAL	LOCAL
LiDAR	●			●		●	●		●	●		●		●			●	●
SATELLITE		●		●		●	●		●	●	●	●				●	●	
GPR	●			●		●	●		●	●		●		●			●	●
RPAs-UAV		●		●		●	●		●	●		●		●		●	●	●
ACCELEROMETERS	●			●			●	●				●		●		●	●	
CLINOMETERS	●			●			●	●				●		●		●	●	
CRACKMETERS	●			●			●	●				●		●		●		●
MECHANICAL TESTS	●		●			●	●		●			●		●	●			●
ENDOSCOPY	●			●	●	●			●		●	●		●				●
BOROSCOPY	●			●	●	●			●		●	●		●				●
GUIDED WAVES TECHNIQUE	●			●		●	●		●		●	●		●		●	●	●
ACOUSTIC EMISSION	●			●		●	●		●	●		●		●		●	●	●
FIBRE OPTIC SENSORS	●			●		●	●	●		●	●	●				●	●	●
WATER RESISTANCE AND PENETRATION	●		●			●	●		●		●		●		●			●
RADIOLOGICAL METHODS	●		●			●	●		●		●	●			●			●
CHEMICAL METHODS	●		●			●	●		●		●		●	●	●			●
MAGNETIC AND ELECTRICAL METHODS	●			●		●	●		●	●	●	●	●	●	●			●

Table 2.8 – Classification of surveying techniques included in [11]

2.8.1 Determination of Damage and Performance Indicators by survey techniques

The section aims to correlate the following information:

- Performance Indicators (PIs),
- Damage Indicators (Dis)
- Damage processes
- Surveying techniques
- Analysis phase.

For this purpose, the following tables have been built, summarizing information included in Annex A:

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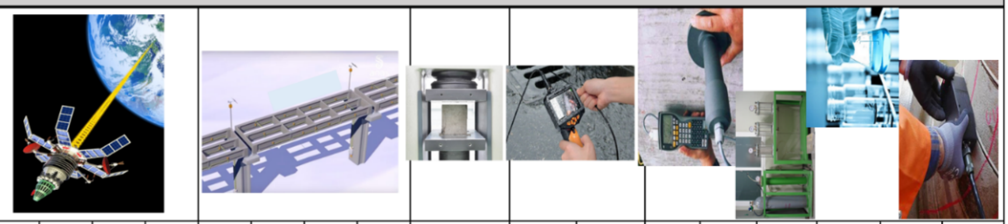
- **PIs vs SURVEYING TECHNOLOGIES**, which also includes a colour scale regarding the analysis phase in which the specific technology is being used.
- **DAMAGE PROCESS vs DIs**
- **DAMAGE PROCESS vs PIs**

The combined use of the tables above allows to define which are the best surveying techniques to be used in order to identify each damage process and which are the related Damage and Performance Indicators.



LEGEND
- 🟡 INSPECTION
- 🟠 DETAILED INSPECTION
- 🟢 MONITORING

PERFORMANCE INDICATORS	SURVEYING TECHNOLOGIES: LIDAR	SATELLITE	GPR	RPAS-UAV	ACCELEROMETERS	CLINOMETERS	CRACKMETERS	FIBRE OPTIC SENSORS	MECHANICAL TESTS	ENDOSCOPY	BOROSCOPY	GUIDED WAVES TECHNIQUE	ACOUSTIC EMISSION	WATER RESISTANCE AND PENETRATION	RADIOLOGICAL METHODS	CHIMICAL METHODS	MAGNETIC AND ELECTRICAL METHODS
RUPTURE	🟢🟡	🟢🟡	🟢🟡	🟢🟡				🟢	🟠🟢			🟢	🟢🟡		🟡		🟠
HOLES	🟢🟡	🟢🟡	🟢🟡	🟢🟡			🟢	🟢	🟠	🟠	🟠	🟡	🟡		🟠		🟠
DEFORMATION	🟢🟡	🟢🟡	🟢🟡	🟢🟡		🟢		🟢	🟠🟢	🟡	🟡	🟡	🟢		🟠		🟠
WIRE BREAK	🟢🟡		🟢🟡	🟢🟡				🟢	🟢	🟡	🟡	🟡	🟢		🟠		🟠
LOSS OF SECTION	🟢🟡		🟢🟡				🟢	🟢	🟡	🟡	🟡	🟢	🟢🟡	🟠	🟡	🟠	🟠
DETERIORATED MORTAR JOINTS				🟡	🟢			🟢					🟡		🟠		
FREQUENCY					🟢🟠							🟢	🟢				
VIBRATIONS/ OSCILLATIONS					🟢							🟢	🟢				
OBSTRUCTION/ IMPENDING	🟡	🟡						🟢		🟡	🟡	🟡	🟡				
DISPLACEMENT	🟢	🟢		🟡		🟢		🟢	🟠	🟡	🟡	🟢	🟢	🟠	🟠	🟠	🟠
CRACKS	🟡	🟢	🟡	🟡			🟢	🟢		🟠🟡	🟠🟡	🟢	🟢	🟠	🟠	🟠	🟠
SCALING	🟡		🟡												🟠		
REINFORCEMENT BAR FAILURE/BENDING	🟡			🟡					🟠			🟠	🟠	🟠	🟠		🟠
STIRRUP RUPTURE	🟡		🟡	🟡					🟠			🟠	🟠	🟠	🟠		🟠
CRUSHING	🟡	🟡							🟢	🟡	🟡						
DEBONDING	🟡	🟡🟢	🟡					🟢	🟠	🟠🟡	🟠🟡	🟢	🟢	🟠	🟠	🟠	🟠
DELAMINATION	🟡		🟡				🟢	🟢	🟠	🟠	🟠	🟠	🟠	🟠	🟠	🟠	🟠
PRESTRESSING CABLE FAILURE					🟢			🟢									
SPALLING	🟡									🟠	🟠	🟡	🟡	🟠		🟠	
TENSIONING FORCE DEFICIENCY					🟢🟠			🟢	🟠			🟢	🟢	🟠	🟠		🟠

Table 2.9 – PIs vs Surveying technologies (for both bridges and tunnels)

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2.8.1.1 Bridges

DAMAGE PROCESSES			PERFORMANCE INDICATORS: 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
DAMAGE PROCESSES	PHYSICAL	ABRASION			●				●				●	●					●	●	●	●
		AGGRADATION (ALLUVIATION)									●	●	●									
		EROSION	●		●		●		●			●	●	●		●	●		●	●	●	●
		CHANGING GEOTECHNICAL PROPERTIES	●	●	●				●			●	●	●	●	●	●	●			●	●
		AGING OF MATERIAL	●							●		●	●					●	●	●	●	●
		FATIGUE	●		●				●				●	●	●	●	●			●	●	●
		IMPACT DUE TO AN ACCIDENT			●						●	●										
		OVERLOADING OF AN ELEMENT	●	●	●							●	●	●	●	●	●	●		●		
		FREEZE THAW	●			●	●	●	●	●			●						●	●		
		HIGH TEMPERATURE				●						●	●					●		●	●	●
	CHEMICAL	ALKALI AGGREGATE REACTION	●			●						●	●			●	●	●			●	●
		SULPHATE REACTION	●			●	●	●	●			●	●			●	●	●			●	●
		CHEMICAL ATTACK				●	●						●	●	●	●	●		●	●		
		CORROSION	●				●							●	●	●	●		●		●	
	BIOLOGICAL	BIOLOGICAL GROWTH	●	●	●				●	●	●	●	●									
	DESIGN AND CONSTRUCTION ISSUES	DESIGN AND CONSTRUCTION ISSUES	●				●							●		●	●		●		●	

Table 2.10 – Damage process vs PIs (bridges)

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DAMAGE PROCESSES																
DAMAGE PROCESSES RELATED TO DESIGN AND CONSTRUCTION ISSUES	BILOGICAL GROWTH	PITTING CORROSION	CHEMICAL ATTACK	SULPHATE REACTION	ALKALI AGGREGATE REACTION	HIGH TEMPERATURE	FREEZE THAW	OVERLOADING OF AN ELEMENT	IMPACT DUE TO AN ACCIDENT	FATIGUE	AGING OF MATERIAL	CHANGING GEOTECHNICAL CONDITIONS	EROSION	AGGRADATION (ALLUVIATION)	ABRASION	DAMAGE INDICATORS
															●	Abrasion resistance
															●	Surface hardness
						●				●					●	Compressive strength
													●	●		Concrete spalling
										●		●				Displacement rate
●	●									●		●				Vertical stresses
											●					Wave propagation velocity
											●					Path of deformation
									●							Induced stresses due to an impact
									●							Induced dispalcements
									●							Vibrations
								●								Average crack density
								●								Longitudinal crack density
								●								Transverse crack density
								●								Crack opening
						●	●									Modulud of elasticity
							●									Fundamental transverse frequency
							●									Water saturation
						●										Tensile strength
					●											Phase composition
					●											Gel presence
					●											Content of alkali
					●											Moisture
					●											Presence of deformed areas
					●											Internal expansion
					●											Mechanical properties of concrete
			●	●												Internal heat distribution
●		●	●	●												Chloride ions content
			●													Waveform amplitude
			●													PH over cross section
●		●	●													Dielectric properties
●		●	●													Dielectric constant changes
●		●														Presence of defects
●	●															Displacement ratios
	●															Bulging
●																Deformation
●																Horizontal stress

Table 2.11 – Damage process vs DIs (bridges)

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2.8.1.2 Tunnels

			PERFORMANCE INDICATORS																		
			Cracks	Crushing	Rupture	Delamination	Scaling	Spalling	Mechanical instability	Debonding	Obstruction/Impending	Displacement	Deformation	Wire break	Prestressing cable	Reinforcement bar failure/bending	Stirrup rupture	Tensioning force deficiency	Loss of section	Changes of concrete pH	Induced stresses
DAMAGE PROCESSES	PHYSICAL	CONTINUOUS VERTICAL ROCK MOVEMENT	●	●	●																
		LOCAL ROCK MOVEMENT (PUNCHING)	●	●	●							●	●								
		HIGHER HORIZONTAL ACTIONS	●	●	●											●			●		
		BENDING STRESS	●	●	●																
		DEBONDING	●							●						●					
		PARTIAL SPALLING OF CONCRETE COVER	●					●		●						●			●		
		OVERLOADING (ROCK MOVEMENT) OF PRESTRESSING	●	●	●			●			●	●									
		DEFORMATION OF THE GROUND			●						●	●									
		WATER IMPACT			●				●											●	●
	DESIGN AND CONSTRUCTION ISSUES	MISSING REINFORCEMENT	●	●	●											●			●		
		DEFORMATION DUE TO SHRINKAGE, TEMPERATURE WITHIN THE SHELL BLOCKS	●									●	●								
		DIFFERENT CASTING TIMES	●									●	●								
		DIFFERENT CONCRETE QUALITIES	●			●						●	●								
		DELAMINATION OF CONCRETE LAYERS	●			●						●	●								
		ANCHOR FAILURE	●	●	●						●	●									

Table 2.12 – Damage process vs PIs (tunnels)

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DAMAGE INDICATORS \ DAMAGE PROCESSES	Anchor failure	Delamination of concrete layers	Different concrete qualities	Different casting times	Deformation due to shrinkage	Missing reinforcement	Water impact	Deformation of the ground	Overloading (rock movement) of prestressing	Partial spalling of concrete cover	Debonding	Bending stress	Higher horizontal actions	Local rock movement (punching)	Continuous vertical rock movement
Induced stresses					●		●	●							●
Vertical fault dislocation														●	
Vertical stress													●	●	
Horizontal stress													●	●	
Deformation rate														●	
Displacement ratio													●		
Stress distribution profiles												●			
Strenght mesurement			●	●							●				
Existence of depressions										●					
Ultrasonic pulse velocity										●					
Elongation at maximum load									●						
Modulus of elasticity									●						
Shear modulus									●						
Induced deformations								●							
Bulging							●								
Mechanical instability							●								
Vhanges of concrete pH							●								
Magnetic flux density						●									
Field fluctuations						●									
Shrinkage rate					●										
Chemical composition			●												
Compression rate			●												
Hardnbess			●												
Delayed finishing		●													
Surface crushing		●													
Superficial crushing		●													
Anchor weaknesses	●														
Chloride ions content	●														
Presence of defects	●														

Table 2.13 – Damage process vs DIs (tunnels)

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3 Inspection

3.1 General

3.1.1 Definition of inspection

On-site examination within the scope of quality control and damage and/or condition assessment, aiming to assess the present condition of a structure.

3.1.2 Inspection's classification

Within the inspection in structures field, three main levels can be distinguished:

The first level corresponds the routine maintenance inspection. It usually consists in a visual inspection to provide qualitative (or semiqualitative) and rather subjective description about the structural elements and/or the structure itself.

The second level is the condition rating inspections, which is based on perform more detailed evaluations of the problematic components and the defects that were estimated in the previous level in order to quantify the effects of the damage on the structural performance

The third level entails extraordinary inspections to provide improved knowledge of the condition, structural capacity, in-service performance, or any characteristic beyond the scope of other types of inspection. In this stage, some tools like instrumentation and/or detailed modelling are usually employed. This level includes different techniques such as structural engineering tests, Asbestos Containing Material (ACM) identification and verification, underwater inspection, fracture critical/redundancy and sub-standard load rating among others.

3.1.3 Role of inspection in the evaluation of a structure health status

Bridges and tunnels are ones of the most critical structures in the transport network. Thus, their collapse usually entails critical economic consequences and human life losses. As is stated in [12] only in the EEUU, the National Bridge Inventory revealed that, currently, there are 691.060 bridges and the detailed information on the number of U.S bridges that have failed or were in a severe condition is not readily available elsewhere. In the same research it is mentioned that during the period between 1989 and 2000, a total of 503 bridge collapses were reported in the United States. In [13] is commented that the statistics on bridge failures from 2009 to 2019 in China shows that most of these failures are related to anthropic factors, and that a lack of real-time monitoring, risk assessment and other managements issues are potential factors that can cause collapses. This study also conclude that the management issues related to construction, design, maintenance and supervision are the key of these collapses being a 69.9% of the collapsed structures. As can be denoted, nowadays, with the great transport network with the current number of critical infrastructures such as tunnels the maintenance is a priority. Within the maintenance operations in structures, the most frequent task are the inspections, that are performed systematically following the requirements of the manual, guidance or standard of the responsible agency. Routine inspection processes can detect anomalies, damage or critical components and require further investigation if it is necessary.

The different inspection manual, guidance or standards were developed in recent year fitting to current needs and leaning from the experience of previous collapses as the case of New York's Schoharie creek bridge in 1987 that provoked a greater attention in underwater inspection and the corresponding agency (FHWA) responded with "Scour at bridges", a new guidance for developing and implementing a scour evaluation program.

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Nowadays with the current experience and the high develop of the technology, the different responsible maintenance agencies elaborated several standards, guidance, or manuals for each type of structure, and the most common potential problems. As it was highlighted in [14], only in the European Union, several different manual or guidance's are available for each agency or each country. In these documents the classifications damage, the forms or the damage indicators are different, thus, currently arise the need of a common procedure to follow the different agencies of each country.

3.1.4 Best practices

Several manuals or guidelines can be found to perform a suitable inspection in function of the country or the type of structure. Among others may be highlighted classified by country:

EEUU:

- Manual for Bridge element inspection, AASHTO, EEUU.
- Metrics for the Oversight of the National Bridge Inspection Program, Federal Highway administration, EEUU.
- Commonly Recognized (CoRe) Structural Elements (2001).
- Manual for Condition Evaluation of Bridges, 2nd ed. (2000).
- Movable Bridge Inspection, Evaluation, and Maintenance Manual (1998).
- Standard Specifications for Structural Supports for Highway Signs, Luminaires, and Traffic Signals, 4th ed. (2001), 272 pp.
- Bridge Inspector's Reference Manual, FHWA NHI 03-001(2002).
- Bridge Scour and Stream Instability Countermeasures Experience, Selection, and Design Guidance, 2nd ed., NHI-01-003 (2001).
- Culvert Inspection Manual, FHWA-IP-86-2 (1986).
- Highway and Rail Transit Tunnel Inspection Manual, FHWA-IF-05-002 (2005).
- Inspection of Fracture Critical Bridge Members, FHWA-IP-86-26 (1986).
- Recording and Coding Guide for the Structural Inventory and Appraisal of the Nation's Bridges, FHWA-PD-96-001 (1995).
- Underwater Inspection of Bridges, FHWA-DP-80-1 (1989).
- USDA Timber Bridges Design, Construction, Inspection, and Maintenance (1992), Forest Service.
- Tunnel Inspection Handbook, massDOT (2018).
- Tunnel Operations, Maintenance, Inspection, and Evaluation (TOMIE) Manual, FHWA-HIF-15-005 (2015).

Australia:

- Structures Inspection Manual, Department of Transport and main roads.

France:

- Road tunnel civil engineering inspection guide. Book 1: from disorder to analysis; from analysis to rating, Centre d'Étrudes des Tunnels (2015).

United Kingdom:

- Guide to surveys and inspections of buildings and associated structures, IStructE, London.

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- Requirements for Inspection and Management of Bridges, BD 62/94 and BD 63/94.
- CD 352-Design of road tunnels, Standards for Highways (2020).

Spain:

- Guía para la realización de inspecciones principales de obras de paso en la red de carreteras del estado, Ministerio de fomento.
- Inspección básica de puentes de ferrocarril, Adif.
- Orden circular 27/2008 sobre metodología de inspección de túneles, Ministerio de Fomento (2008).

Denmark:

- Inspection of Bridges (1994), Danish National Road Directorate.

Finland:

- Guidelines and Policy for Bridge MR&R Operation
- Guidelines for Bridge Inspection
- Bridge Inspection Manual
- Bridge Repair Manual (SILKO–Guidelines)

Germany:

- Highway Structures Testing and Inspection, DIN 1076 (1999), Deutsche Norm.
- Preservation and Maintenance (n.d.), Construction and Housing, German Federal Department of Transportation.
- Guideline for the Structural Design and Equipment of Bridges for Monitoring, Inspection and Maintenance (1997), German Federal Department of Transportation.
- Recording and Assessment of Damages, Guideline RI-EBW-PRÜF, 1998.
- ASB Structure Inventory, (coding manual for SIB–Bauwerke) (1998).

Norway:

- Handbook for Bridge Inspections (2001), Norwegian Public Roads Administration.

Canada:

- BIM Inspection Manual, Version 3 (2005), Alberta Infrastructure and Transportation.
- BIM Inspection Manual—Level 2, Version 1 (2004), Alberta Transportation.
- Structure Inspection Manual (2000), Ontario Ministry of Transportation.

Although each country has different guidelines for defining the inspections, most of them are based on the same fundamentals. Throughout this chapter, the three previously defined inspection levels will be described and related to each of the inspections carried out in some of these countries.

3.2 Routine maintenance inspections

The routine maintenance inspection has variations depending on the country and the used guideline or standard. Nevertheless, all of them share the same main objective, checking the serviceability and functionality of the structure through a visual inspection. Table 3.1 shows some of the inspections in different countries that can be assimilated to the routine maintenance inspections.

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Country	Inspections
Australia	Routine maintenance inspection.
Denmark	Daily inspection, routine inspection and reports from users.
Finland	Annual inspection.
France	Routine visit and annual inspection.
Germany	Superficial inspection and minor test.
Norway	General inspection.
Sweden	Regular inspection, superficial inspection and general inspection.
United Kingdom	Superficial inspection and general inspection.
United States	Routine inspection.

Table 3.1 – Inspection similar to routine maintenance inspections in different countries

3.2.1 Objectives

The purpose of the Level 1 inspection is to verify the serviceability of the structure. Besides, in the routine maintenance inspection the inspector must search any new sign of damage, defects and unusual behaviour and follow up on the known ones.

3.2.2 Description

All the inspections in Table 3.1 try to achieve the objective of a routine maintenance one through visual inspections. Nevertheless, depending on the country, this inspection is carried out by different inspectors, and it is even divided into several inspections. In Australia, Finlandia, Norway and the United States the routine inspection is performed in a single stage that comprehends the observations required for determining the functionality of the structure, the identification of any visible changes in the initial conditions and the follow up of the previously recorded defects. In addition, these inspections may be performed in conjunction with the routine maintenance. On the other hand, in countries such as Denmark, France, Germany, Sweden and the United Kingdom some prior inspections are also performed. These daily or superficial inspections or routine visits are carried out by the road maintenance crews and the road agents (during their patrols) or the contractor to detect new conditions of the structure and ask for routine inspections if it is necessary. Besides, in Denmark, the users have the possibility of generating reports for warning about any impact damage, vandalism, erosion, etc.

Prior to the inspection, the accredited inspector must check that the structural inventory, the safety equipment and the required permits are in place. The structural inventory is a report made during the construction of the structure that shows its original condition. In fact, this inventory may be changed to include new defects found during previous inspections. Hence, the inspector has an updated report to compare the actual condition of the structure with the original or previous inspection structural health.

During the routine maintenance inspection, components of the structure such as the deck, the footways, the signposts, the substructure, the walls, etc. have to be studied to find and follow up visible defects like cracking in the material, displacements, corrosion, delamination, depression of some component, settlements, concentration of vegetation or debris, drainage, functionality of the light and security inspection, etc. In addition, some pictures can be taken to help with their localisation and identification.

It must be noted that, during the inspection, the confined spaces of the structure, inaccessible areas or components below the water level and ground do not need to be evaluated.

3.2.3 Frequency of execution

The frequency of execution depends on the guidelines and the standards of each country so as it can be observed in Table 3.2.

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Country	Inspection	Frequency of execution
Australia	Routine maintenance inspection	One per year
Denmark	Daily inspection	One per day
	Routine inspection	One per year
Finland	Annual inspection	One per year
France	Routine visit	Frequently
	Annual inspection	One per year
Germany	Superficial inspection	Four per year
	Minor test	Three years after major tests
Norway	General inspection	One every year or two
Sweden	Regular inspection	Frequently
	Superficial inspection	One per year
	General inspection	One every three years
United Kingdom	Superficial inspection	Frequently
	General inspection	One every two years
United States	Routine inspection	One every year or two

Table 3.2 – Frequency of execution of the inspections similar to routine maintenance inspections

Furthermore, the countries usually established the requirement of performing a Level 1 inspection after a major natural event like floods or earthquakes to verify the serviceability of the structure. However, these inspections are usually called damage inspections in the guidelines.

3.2.4 Data collection

The information obtained in the routine maintenance inspection shall be recorded in a Level 1 Inspection Report, where the accredited inspector must write all the new defects identified during the inspection and the condition of the damage that were already recorded in the structural inventory. Usually, these reports are a checklist of defects that the inspector must complete. Furthermore, the report has enough space for adding the location and any comment about the identified defects. Finally, the inspector also has to write down its data, the data about the structure (e.g., name, location, material, type, etc.), the date of the inspection and the designation of any other inspection, if necessary. Hence, it is also a useful guide for assisting the inspectors. On the other hand, all the drawn sketches and taken photographs must be included in a Photographs and Sketches Record.

3.2.5 Inspection outcomes

The outcomes of a Level 1 inspection are the reports commented before. These reports collect the position and nature of the visible defects in accessible areas of the structure. In addition, the Level 1 Inspection Report allows inspectors to include comments about not identifiable defects and the reason for their omission. Finally, in the conclusions of the report, the inspectors have to discuss the need to monitor the structure and give recommendations about the necessity of maintenance actions, minor repairs and higher-level inspections.

3.3 Condition rating inspection

In the condition rating inspections, the current structural health of the construction and all its elements are studied. Nevertheless, similar to the Level 1 inspection, each country has different inspections to reach this goal. These inspections can be observed in Table 3.3.

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Country	Inspections
Australia	Condition rating inspection.
Denmark	Principal inspection.
Finland	General inspection and basic inspection.
France	IQOA evaluation and detailed inspection.
Germany	Major test.
Norway	Major inspection.
Sweden	Major inspection.
United Kingdom	Principal inspection.
United States	Hands-on inspection and fracture-critical member inspection.

Table 3.3 – Inspections similar to condition rating inspections in different countries

3.3.1 Objectives

The purpose of the Level 2 inspection is to rate the current load carrying capacity and the condition of a structure and all its components. Besides, in the condition rating inspection, the accredited inspector must look for any defect in the structure, analyse its origin and measure its magnitude. Hence, the condition rating inspection is used for identifying and quantify the defects of a structure, determine its residual life, assess its current load carrying capacity and define the required maintenance actions.

3.3.2 Description

The condition rating inspection is an arm-length through visual examination of the structure, using some destructive and non-destructive testing techniques to obtain any necessary measurement or material sampling. Nevertheless, depending on the country, this inspection may be divided into several types.

In countries like Australia, Denmark, Germany, Norway, Sweden and the United Kingdom the evaluation of all the structure and its components is performed in a singular inspection. However, in countries such as France it is carried out in two different inspections, a detailed arms-length inspection for noting all the defects existing in the structure and an IQOA (Image de la qualité des Ouvrages d'Art) evaluation of these defects for the assessing of the structure. In the United States, in addition to the thorough visual inspection, an inspection of the most unfavourable components of the structure is also carried out. On the other hand, in Finland an additional general inspection (called basic inspection) of 125 reference bridges is performed to extract the information that is the basis for the creation and updating of bridge deterioration models. Finally, it must be noted that in countries like Germany, Norway, Sweden or the United States, the Level 2 inspections include the evaluation of structural elements below the water level.

Prior to the inspection, the accredited inspector shall ensure that all the necessary documentation, inspection equipment, safety equipment, required permits and access equipment is prepared and reviewed.

During the condition rating inspection, all the defects and damage in the structure are mainly identified by an arm-length thorough visual inspection. In addition, the accredited inspector must also determine the origin of these defects, their magnitude, their rate of variation and the effect that they have in the structure and its behaviour. Some examples of typical defects and damage identified during these inspections are scouring, cracking, impact damages, visible settlement or rotation of substructure elements, displaced bearings, corrosion, cracking in weld beads, rot, etc. Subsequently, the inspector has to grade the condition of each structural component based on this information. Furthermore, the inspector shall take several photographs, with digital timestamp, of all the structure and each one of the identified defects to help in their evaluation and localization. On the other hand, the possible modifications in the structural elements due to the previous maintenance works and the exposure of each of the components must be identified, listed and evaluated. All this information has to be used

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for the inspector to evaluate the structural health of the construction. This assessment must be based on the experience and the criteria of the accredited inspector. The inspector can use any non-destructive or destructive testing techniques (e.g., timber drilling, resistograph testing, ultrasonic testing, ground penetrating radar, nuclear densitometry, etc.) and closure the necessary lanes to ease the evaluation process and to acquire all the required data of the inspection. Besides, all this information can be used for developing structural models that allow the performance of several structural analyses. Finally, if the evaluation of some specific behaviour is necessary, the inspector can require a Level 3 special inspection.

3.3.3 Frequency of execution

The frequency of execution depends on the guidelines and the standards of each country so as it can be observed in Table 3.4.

Country	Inspection	Frequency of execution
Australia	Condition rating inspection	Between one and five years
Denmark	Principal inspection	One every six years minimum
Finland	General inspection	Between five and eight years
	Basic inspection	One every five years
France	IQOA evaluation	One every three years
	Detailed inspection	Between one and nine years
Germany	Major test	One every six years
Norway	Major inspection	Between five and ten years
Sweden	Major inspection	One every six years
United Kingdom	Principal inspection	One every six years
United States	Hands-on inspection	One every three years minimum
	Fracture-critical member inspection	One every two years

Table 3.4 – Frequency of execution of the inspections similar to condition rating inspections

It must be noted that the frequency of execution of some inspections in Australia, France and Norway also depend on the typology and the condition of the structure.

3.3.4 Data collection

The information extracted from the condition rating maintenance shall be recorded in several reports. The Level 2 Inspection Report saves the inspection component inventory, the assessment of each of these components, their exposure, the origin and effect of all the identified defects and the overall condition of the construction. Besides, the taken photographs must be also introduced in the Photographs and Sketches Record. On the other hand, the accredited inspector must also generate a Defective Components Report to write down all those components that require monitoring or Level 3 inspections, and the Standard Procedure Exceptions Report, which shows the components that cannot be evaluated and the reason for their omission in the inspection. Finally, special reports have to be created if additional testing surveys were carried out.

3.3.5 Inspection outcomes.

The outcomes of a Level 2 inspection are the reports listed above. Hence, the condition rating inspection main outcome is the condition grade of all the components that belongs to the studied structure. This grade takes into account the structural health of the element, the defects presented on it, as well as their magnitude and origin, and the risks of the atmosphere where the component is located. Besides, the inspection reveals the general condition of the construction, recommending the required maintenance and restoration works, as well as their costs, for keeping its correct behaviour. Finally, in the reports, the accredited inspector also discusses the need to monitor the structure or some of its elements and to request a special inspection.

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3.4 Extraordinary inspections

Level 3 inspections are intended to provide improved knowledge of the condition, load carrying capacity, in-service performance and other characteristics that are beyond the scope of Level 1 and Level 2 inspections. These inspections are not carried out systematically. These arise as consequence of detected damage during inspections at previous levels. In this type of inspections, a visual inspection is carried out together with different test or complementary measurements. This level requires a previous inspection plan in which are detailed the elements object of study and the employed tools or test to be carried out, this inspection plan is usually elaborated for an accredited inspector. The level of accreditation depends on the manual, standard or guidance of the responsible agency. This inspector will decide in function of the structure and the results of the previous level inspections which tests will be performed, specifying the execution procedure if there are no any corresponding standards.

3.4.1 Objectives

The main objectives changes in function of the type of special inspection that will be performed. Briefly the general main objective is to know in greater depth the current state of the structure or of some elements reported in the previous inspection levels. The conditions of the inspection are established in the project-specific brief.

In the case of the detailed structural engineering inspection the particular objectives are identify and quantify the current deterioration process, determine the structural condition in terms of safety, reliability and the behaviour of a structure through physical testing and structural analysis, and finally to develop appropriate management and repair strategies.

In the Asbestos Containing Material (ACM) identification and verification inspection the first purpose is to identify the potential permanent/sacrificial inclusion of asbestos on structures in order to ensure that the department's asbestos register is up to date with the Work Health and Safety counterpart. The Program Manager will make arrangements for a visual inspection. If an ACM is identified the following objective is to confirm the presence of asbestos in suspected ACM where the material may be disturbed through any proposed activity on the structure. The inspection involves hands-on practices to gain access to the areas of concern and may involve the breaking back of limited areas of concrete to facilitate removal of samples for testing by a NATA-accredited laboratory under ISO 17020.

In underwater inspection the objective is to determine the current state of the underwater components and to provide a benchmark for future inspections. In case of the fracture critical/lack of redundancy inspection the objective is to inspect and to identify more deeply the fracture of critical members in the bridge with no load path redundancy. To correctly develop this inspection, diving equipment and specially trained and accredited personnel is required.

Finally, in other extraordinary inspections, the objectives are detailed in the defined inspection plan.

3.4.2 Description

Each extraordinary inspection presents its own procedure that is defined in the corresponding standards, manual or guidance of the agency responsible for the maintenance of the structure.

The detailed structural engineering inspection entails a visual examination of all accessible components of the structure. Where is necessary, it could be complemented by examinations, testing or analysis specified in the project-specific brief such as geotechnical investigations, static or dynamic load testing of the structure or non-destructive measuring technologies. In addition, a load capacity assessment may be included in the brief to determine the repeated live load capacity for the remaining service life of the structure. This inspection comprises different task such as the review of any previous inspection and testing reports, traffic counts,

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studies, environment factors, carry out measurements and testing campaigns to supplement the visual inspection, determine the material properties and structural behaviour. Besides, it usually includes other tasks like identifying components which are limiting the performance of the structure or are likely to deteriorate, probable causes and projected rate of deterioration, durability, residual life of the structure, influential factors in the dynamic load allowance, and hydraulic performance among others.

Focusing on ACM inspection, this is a visual inspection to confirm the presence of potential ACM. If ACM is identified through a visual inspection the asbestos register has to be updated. Once the ACM is identified starts the process of ACM verification. If some of the components present ACM, the register must be updated indicating the elements and the ACM condition. In addition, it must warn Workplace Health and Safety and advise the responsible of the inspection to implement appropriate control measures.

An underwater inspection usually includes some tasks like the mapping of local scour, metal corrosion, reinforced concrete cracking and spalling, prestressed concrete splitting, pile loss and residual section, debris mapping, extensive photographic record or extraction of underwater samples among others. In addition, tidal and splash zone areas should also be inspected, and scour soundings undertaken.

Other extraordinary inspections (e.g., fracture critical member identification, confined space inspection, etc.) are carried out following the inspection plan requirements which are individually elaborated depending on the agency responsible for the maintenance of the structure and its nature and typology.

3.4.3 Frequency of execution

Unlike Level 1 and Level 2 inspections, performed at predetermined frequencies, a Level 3 inspection is carried out on an as-needed basis. A detailed engineering inspection will be carried out in some particular circumstances such as assessing the condition of a structure prior to carrying out programmed works like rehabilitation, strengthening or widening, as the result of recommendations in a Level 2 inspection, to provide a load rating for the structure among others.

In the particular case of the ACM inspection, it will be undertaken once only, at the earliest available opportunity, as part of the Level 1 or Level 2 inspection program. Concerning other extraordinary test, the frequency of execution is in function of the corresponding standard or manual of the responsible of the structure or as a result of the systematically inspections.

3.4.4 Data collection

Data recording requirements will be in accordance with those specified by the project-specific brief. The data recording will be, at least, similar to those required for a Level 2 inspection. Therefore, through visual inspections and measuring technologies, the inspector must extract information about the location, magnitude and effect on the structure of all the defects that must be evaluated during the specific inspection.

In the case of the underwater inspection, the responsible for the maintenance of the structure drafts a specific brief to define the scope of the work. The brief should include the elements that will be inspected and how the inspection will be performed if there are some specified requirements, specified in some guideline, standard or inspection pro forma sketches, the brief must specify the level of the reporting required. The accredited inspector shall review the data collected by the diver and ensure that the required level of detail has been recorded for reporting purposes.

3.4.5 Inspection outcomes.

A draft report is usually elaborated including the compilation of standard inspection forms and supplemented by a written report that is required by the plan inspection brief. All the

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information from the tests carried out shall be included in these reports. Furthermore, the accredited inspector must write down the conclusions about the inspection and the request of any maintenance action, monitoring, etc. if necessary.

In the case of the detailed structural engineering inspection, a written report that includes the specific requirements outlined in the project brief shall be submitted. These requirements should include the diving surveys and materials testing, the rating of all primary defects, the identification of the deterioration mechanism and determination of the overall condition of the structure, the results of any load capacity assessment and, if it is required in the brief, a bridge equivalence rating. If the condition rating of the components or the overall rating condition of the structure differs from the Level 2 report, then a new Level 2 inspection report shall be prepared revising condition ratings of the inspected components.

Concerning the outcomes of the ACM inspection, three main tasks will be carried out: update the register of ACM, advise Workplace Health and Safety that a check has been conducted and potential asbestos containing material has been identified, and place signs on the structure warning of the risk in it. If the presence of ACM is confirmed, the register will be updated indicating the elements and the ACM condition. Besides, Workplace Health and Safety and the responsible of the inspection must be advised of the findings and the need to implement appropriate control measures. In other tests, the outcomes shall be the reports of the actions determined in the results, following the requirements of the certified inspector, the manual guidance or the corresponding standard if it is available.

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4 Monitoring

4.1 General

In recent decades there has been an increasing diffusion of structural monitoring systems in combination with the most widespread and common structural management procedures: structural monitoring is the result of a set of technologies aimed at determining the state of a structural system and its evolution over time, to detect and quantify phenomena of degradation or damage and to allow the evaluation of the integrity of the system itself and its ability to remain in operational condition with adequate levels of safety for a given period of time. [8] identifies the following possible reasons for the implementation of a monitoring system, summarized in Figure 4.1.

MONITORING FOR STRUCTURE MANAGEMENT		
UNDER NORMAL CONDITIONS	**UNDER SPECIAL CIRCUMSTANCES**	**DAMAGED OR DETERIORATING STRUCTURES**
• Structures susceptible to long-term movements • Durability monitoring • Fatigue assessment • Comparison with design	• Modification or demolition of existing structures • Structures affected by external works • Special investigations or assessments	• Structures subject to long term movements • Structures affected by degradation of materials • Control tool for extending service life
RESEARCH MONITORING		
• Evaluation of novel forms of construction/materials • Cost/benefits to be derived from monitoring • Feedback into the design process/standards • Improved understanding of in-service behaviour of structures		

Figure 4.1 – Examples for reasons for the use of monitoring systems (Adjusted from [8])

Hence, condition monitoring activities, combined with inspections and testing, are key tools for the through-life management of the structure: actions to be taken concerning condition control, conservation and maintenance works should be planned based on the information gathered through the activities above, which, according to [15], must be carried out from an early stage in the service life of the structure. Planned activities are included in the Conservation Plan, which states the types of inspection, testing and condition monitoring that have to take place, what components of the structure are to be inspected/monitored, what the frequency of the inspections should be, etc.

In this respect, the IM-SAFE project proposes in [10] a framework for the data-informed safety assessment highly interrelated with the through-life management of the transport infrastructure which 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 collected from inspection and monitoring activities during the lifespan and maintenance process of the structure. Inspections methods are described in chapter 3, while chapter 4 focuses on monitoring.

The use of monitoring systems has the following objectives:

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- a better correlation between loads/actions acting on the structure, the consequent state of deformation and the technical predictions assumed as a basis for the design, thus a more reliable knowledge of the behaviour of the structure;
- the identification of more precise modelling, more efficient dimensioning criteria, and an improved safety assessment;
- an early detection of any anomalies in the structural response and thus the basis for possible reinforcement interventions and/or limitations of use, especially caused by the decay of structural resources due to behaviour under cyclical actions repeated over time or to occasional actions, such as those due to earthquakes or environmental and anthropic agents;
- the definition of strategies aimed at extending the expected life of the structure;
- an improved asset management;
- a collection of statistical data that could have an impact on normative provisions, also with regard to the effects of climate variations.

Hence, structural monitoring provides the opportunity to observe the behaviour of the structure over time and the load history to which it is subjected: the need for data acquisition about its static and dynamic behaviour is increasingly pressing for existing structures, especially for bridges and tunnels, which are critical components of the road infrastructure.

This information is obtained using a network of sensors suitably placed on the structure, data acquisition systems, units for storing and analysing measurements, systems for transmitting data to processing units, including remote ones, and software for the analysis and the interpretation of the data.

However, while structural monitoring techniques are relatively widespread and well established in the aeronautical and aerospace fields, the same cannot be stated for the civil one. There are many reasons for this, for instance, according to [16]:

- Each structure differs in terms of its properties and challenges: this means that the development and implementation of automated monitoring processes is less easy and therefore the intervention of human knowledge is always required even at intermediate process levels. This implies in most cases a specific know-how.
- Lack of knowledge about cost/benefit analysis for the use of a SHM system.
- The technologies involved in instrumental monitoring are profoundly interdisciplinary and therefore their diffusion depends on the sensitivity of the technical-scientific context to knowledge which differs from the cultural matrix of origin, and on the attitude to adopt the innovative contents.
- The service life of civil structures is much longer than the industrial systems, which means that degradation phenomena occur much more quickly, so it takes longer to demonstrate the effectiveness of monitoring systems.

Nevertheless, the need to monitor the behaviour of modern large-scale and technologically very complex structures and the awareness of the limits of traditional approaches to the management of structures, which in several countries were manifested in serious accidents, have motivated the technical and scientific community to research and develop SHM systems suitable for the needs of the civil infrastructure field.

The most typical applications are:

- Structural monitoring
- Seismic monitoring
- Geotechnical monitoring

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- Hydraulic monitoring
- Monitoring of natural and artificial slopes

The principles described in the following paragraphs are mainly based on the provisions given in [16].

4.2 Objectives and requirements

Structural monitoring systems aim to implement an in-service control process, which consists of identifying the value assumed by specific parameters that characterise the behaviour of the structure and determining the changes in the values of these parameters that occur due to deterioration processes. According to [16], the following fundamental functions of any monitoring scheme, simple or complex, are to be performed:

- **A data collection and management system**: information obtained by structural diagnostics (inspections, samples, load tests, etc.) and by a network of sensors directly installed on the structure is processed in the system.
- **A set of data processing procedures**: assessment of the condition of the structure and its developmental trend.
- **Decision-making procedures**: guidance on the choice of subsequent actions based on the indications provided by monitoring.
- **Numerical model of the structure**: this model is validated through an initial calibration, which is developed with a level of detail consistent with the complexity and relevance of the structure.

This process requires a combination of very different but closely interacting activities, both of an experimental nature (structural diagnostics and data collection with a range of sensors) and of an engineering nature (numerical modelling and system identification procedures).

The general objectives of a monitoring system can be summarised as follows:

- **Monitoring the condition of the structure with respect to potential limit states**: the system is designed to control the value of some relevant parameters appropriately chosen in order to identify in a reliable way the approach or the exceedance of a limit state of relevant damage or, in the most critical cases, of possible collapse.
- **Identification of an occurring degradation process**: the system may detect the occurrence of abnormal behaviour of some characteristic responses of the structure to external actions, not in line with predictions or past measurements indicated either by an anomalous trend in measured data, , or by the exceedance of a critical absolute value.

The choice of the monitoring system and its degree of complexity is the result of a case-by-case analysis and aimed at obtaining the best use of the economic resources available and the professional skills involved. The main factors influencing this choice are related to the structure, the environment/loading conditions, and the monitoring system features/potentialities:

- **The relevance of the structure**: to be assessed in terms of economic value and function carried out.
- **Environmental conditions**: structural monitoring should be given higher priority in the case of a very aggressive environment in which the structure is located, in consideration of a higher possibility of damage.
- **The complexity and degree of innovation of the structure**: structures using new materials or innovative construction techniques have a particular interest in the use of monitoring systems.

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- **The degree of reliability of monitoring and cost-benefit analysis**: structural monitoring is a natural development of the control process.

Monitoring, in any case, cannot meet all the performance requirements of an efficient control and must be supported in its conclusions by dedicated investigations: hence the need to consider the actual benefits of monitoring against the costs incurred, which obviously increase with the size and complexity of the system.

Ultimately, the decision on the commissioning of the monitoring system and its characteristics is based on an in-depth cost-benefit analysis, on the basis of which it is possible to identify, in realistic terms, the role to be assigned to the monitoring system and to design the sensor network and, more generally, the whole system in order to be proportionate to the expected benefits

Monitoring data should be properly analysed, reported, and evaluated in a series of periodic reports and in a concluding report containing condition assessments and recommendations for follow-up actions. Source data should also be stored off-line for later use. They may be considered suitable for immediate acquisition, analysis, and evaluation systems, capable of providing early warnings about reaching predefined alarm values.

4.3 Monitoring classification

The classification of monitoring system is provided in the Figure 4.2.

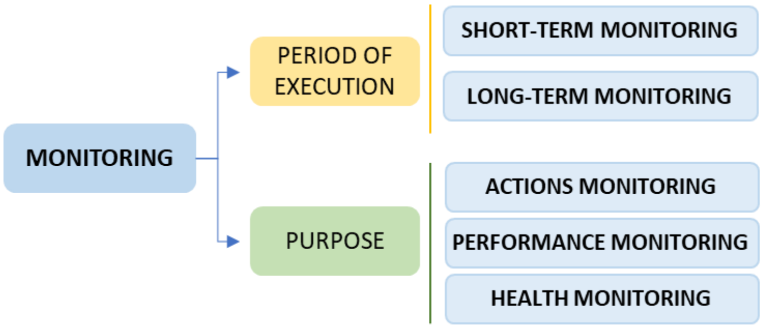

Figure 4.2 – Monitoring classification

Based on the period of execution, the monitoring approaches can be divided into the following categories:

- **Short-term monitoring**: the installation of a monitoring system for a limited period and, if necessary, the repetition of the installation at more or less regular intervals over time. According to [17], this approach is recommended in the following cases:
    - *Extraordinary maintenance or upgrading*- the installation of instrumental systems before, during and after the intervention to evaluate its effectiveness is recommended.
    - *Study of the behaviour of repetitive structural types*.
    - *Boundary transitory situations*, e.g., analysis of slope behaviour for preventive stabilisation interventions.
    - *Analysis of anomalous degradation/damage phenomena*, for which it is required to investigate causes and evolutionary nature.

The installation of systems of this type is particularly suitable in cases where the aim is to study a known phenomenon observed during inspections. This approach includes occasional or periodic dynamic monitoring.

- **Long-term monitoring**: the implementation of a permanent monitoring strategy, in which the hardware/software system is designed to be operational for long periods up

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to the service life of a structure. According to [17], this approach is recommended in the following cases for bridges:

- *Cable-stayed or suspended bridges and large span bridges* (> 200 m).
- *Bridges with spans of more than 50 m in pre-stressed concrete built more than 40 years ago*.
- *Bridges with inspection difficulties (box girders and non-inspectionable piers) in prestressed concrete or steel*.
- *Bridges with innovative structural solutions*.
- *Bridges of historical relevance*.
- *Bridges in critical environments*, characterised by high traffic loads (e.g., frequent transit of exceptional transports), with fatigue problems, in areas with high seismic risk or with critical boundary situations, such as the high risk of flooding and landslides or bridges where accidental phenomena, such as impacts, are likely to be of great importance.

Based on the purpose, the monitoring approaches can be divided into the following categories:

- **Actions monitoring**, which allows the evaluation of the actual loads acting on the structures.
- **Performance monitoring**, which allows an assessment of whether a structural system or component meets the performance requirements under a known or any load:
  - *System performance monitoring*
  - *Component performance monitoring*
- **Health monitoring**: provides real time information for the assessment of the safety and serviceability of a structure or structural components. It is founded on the condition that a sufficient number of measurable health indicators exist and can supply relevant information on the state of the structure.

4.4 Monitoring system architecture

The system architecture represents the topological structure by which the different components of a monitoring system are connected.

A monitoring system is characterised by sensing components (sensors), a system architecture connecting the different sensors, a data collection unit (gateway) and a data repository (cloud).

In details, the elements that compose the acquisition pipeline connecting sensor and storage system typically includes the following components:

- **Sensor**, in which the transduction from a physical quantity of interest (e.g., acceleration, displacement) into an analogue signal, typically electrical or optical, occurs.
- **Analogue communication system**, which connects the sensor to the analogue/digital converter.
- **Analogue/Digital Converter**, which is the device that transforms the analogue signal into a digital signal.
- **Digital communication system**, which connects the converter to the gateway.
- **Gateway**, device to which all digital signals of the monitoring network converge.
- **Cloud**, which represents the repository and the post-processing unit.

Based on the topological distribution of these components, it is possible to distinguish between:

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- **centralised system**, in which the analogue/digital conversion takes place in a single device to which all the sensors are connected.
- **digital sensor network**, in which the analogue/digital conversion takes place at each sensor and typically sensor and converter are integrated in the same device (called digital sensor). Based on the communication mode of digital systems, it is possible to further distinguish between:
  - **cabled** systems.
  - **wireless** systems.
- **mixed system**, in which the conversion takes place at several devices distributed over the structure, each of which carries multiple analogue sensors.

The architecture of the monitoring system must be properly designed: the choice of system architecture and digital communication mode should consider the following requirements.

- Reliability
- System robustness
- Easiness of assembly
- Transmission speed
- Transmission distance
- Scalability and expandability
- Encumbrance and visual impact
- Availability of power supply

4.5 Preliminary monitoring system design

The flowchart in Figure 4.3 represents the sequence of activities that are carried out in the preliminary monitoring system design phase, which has the task of providing the detailed specifications for the development of the final design.

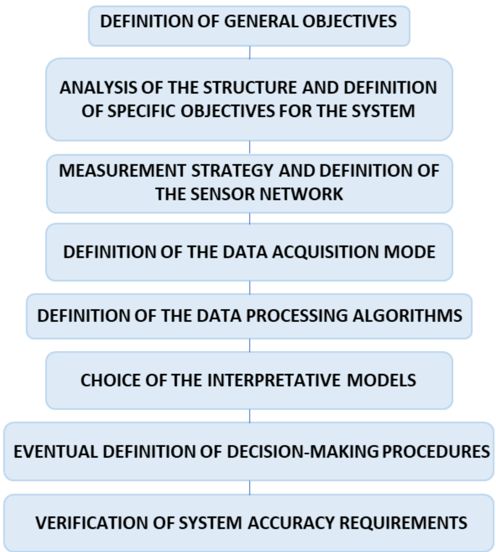

Figure 4.3 – Diagram of the monitoring system design process (Adjusted from [16])

4.5.1 Definition of general objectives

The client and the system designer should jointly analyse the existing technical and economic issues in order to define the desired objectives, which are often the result of a compromise between the desired performance and the financial resources available.

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However, it is common that the request for the implementation of a monitoring system is made without a clear definition of the objectives to be achieved by the client.

They are usually expressed in generic terms such as:

- Assessment of the condition of the structure in order to intervene in case of significant damage.
- Detection of any deterioration processes occurring.

4.5.2 Analysis of the structure and definition of specific objectives for the system

A preliminary analysis should be carried out, the objective of which is to achieve a detailed knowledge of the structure and the evolution of its behaviour. This is hardly ever achievable, except by using systems that are so sophisticated and complex as to be economically unfeasible. The most logical way to proceed is to identify a reduced number of significant quantities that provide the essential information about the structure.

The preliminary analysis allows the identification of quantities related to the type and structural organization, the materials used, the presence of essential components for a satisfactory performance and, therefore, critical components: the specific mechanisms of collapse or damage of the structure are thus identified, depending on the expected loads and environmental conditions, and the quantities which control these mechanisms are identified. The information needed for the preliminary analysis is to be found in the available data, such as those provided by structural diagnostics (all type of inspections, samples, load tests, interventions, etc.)

These quantities can be:

- **State variables**, such as characteristics of the structural response or characteristics of the actions on the structure.
- **System parameters**.
- **Quantities defining structural behaviour**, such as transfer functions or modal parameters.

There are only a few cases where these quantities are directly measurable (state variables). They are generally obtained by re-processing one or more quantities measured by the system.

The quantities thus defined therefore become the specific objectives of the monitoring and provide the input data for the interpretative models which are then used, together with other data that may come from experimental diagnostics or other information, to meet the client's requirements for damage recognition, the estimation or updating of an index of structural safety or an index of the residual life of a structure.

It is important, hence, that the choice of these quantities is made with extreme care, in order to avoid including in the monitoring system the detection of quantities that are irrelevant in the interpretation models or not to include quantities that could be significant.

4.5.3 Measurement strategy and definition of the sensor network

Once the quantities to be monitored and the required accuracy have been identified, the system designer identifies the most appropriate measurement strategy, developing a detailed layout of the sensor network defining the type of quantity measured, the number, and position of each sensor.

4.5.3.1 Types of quantities

The most common quantities of interest in monitoring are:

- **Mechanical** quantities characteristic for:

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  - *Structural behaviour*
    - Cinematic quantities
    - Dynamic quantities
    - Deformations
  - *Actions*
    - Forces
    - Impulses
    - Environmental actions
- **Thermodynamic** quantities:
  - Temperature
  - Irradiation
- **Chemical** quantities:
  - Ph
  - Humidity
  - Agent concentration
- **Electromagnetic** quantities:
  - Electrical and magnetic potentials

It should be preferred to monitor physical quantities that are directly related to the behavioural parameters, in order to limit uncertainties relating to the interpretation model.

4.5.3.2 Number and position of measurement points

In general, the number and the position of the measurement points depend on the structural type, the monitoring objectives, the required accuracy, and the type of instrumentation expected to be used.

In case non-point instrumentation is used, the measurement can be referred to any point in the field of observation. In all other cases (instrumentation applied to the surface of the structure or embedded in it), the number and position of the sensors should be such as to provide all the necessary information for the use of the chosen interpretative model.

In some cases, instrumental monitoring requires the measurement or identification of a field, e.g., the deformation or modal shape of a structure: in this case the choice of the number and position of the measurement points is always a compromise between the need to observe the field exhaustively, limiting the uncertainties due to the incompleteness of the observed field, and the cost of the sensors.

Concerning both damage and structural response, the following methods can be used. [18], in this respect, identifies the selection of an appropriate sensor arrangement as an application-specific task, where the procedure for sensor placement critically depends on the method that is pursued:

- **Local methods** -are used for the evaluation of damage and structural response on a local scale: they are based on the principles of placing sensors where damage is expected, assuming that it directly affects the measured response quantity, so the sensor's proximity to the anticipated damage locations is pivotal.
- **Global methods** are used for the evaluation of structural response on a global scale**:** they assume that damage alters the mass, stiffness, or energy-dissipating properties of the structure, which in turn alter the measured dynamic response of the global system
- If numerical models are available, the sensor placement can be optimized based on mathematical **performance criteria**.

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Concerning actions, sensors layout must be properly chosen based on the type of action to be monitored and on the instrumentation to be used. See chapter 8 for further information.

Due to the nature of the sensor placement problem, the number and position of the sensors can be determined using dedicated optimization algorithms.

4.5.3.3 Measurement system specification

The measurement of a quantity involves the use of a measurement chain, ideally consisting of the actual sensor, the electronics conditioning the signal output from the sensor, the connection cable to the analogue/digital conversion system.

It is recommended that the preliminary design should be limited to clearly defining the metrological specifications required based on accuracy considerations, while leaving the developer of the executive project the task of choosing the most convenient sensor and hardware solution in terms of network logistical configuration, cost, and reliability, while still respecting the metrological specifications required.

The metrological specifications include, where applicable, at least the following instrument properties:

- Precision
- Repeatability
- Stability
- Sensitivity to environmental conditions
- Measuring range
- Linearity
- Signal/noise ratio
- Transfer function
- Sensitivity

The specifications may also include the following features:

- Calibration and adjustment
- Durability or service life
- Maintainability and replaceability
- Level of protection
- Size and weight
- Electrical consumption
- Type of power supply

4.5.4 Data acquisition and processing

Data acquisition and processing are discussed in 4.9 and 4.10, respectively.

4.5.5 Interpretative models and calibration of reference model

The identification of the condition of the structure and the presence of damage is in principle based on the minimization of an error function constructed from real results which photograph the real state of the structure at a certain moment in its life and the results of a numerical model which is built (calibrated) in such a way to be able to describe the real behaviour of the structure with an assigned accuracy.

The same goals can also be reached through a data-driven approach, determining state variables, from which behaviour parameters are extracted, and/or detecting anomalies on the basis of analyses of measurements, which are pre-treated and generally turned into a time series of observations.

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Models are characterised by the following properties:

- period of time needed to establish a stable reference model;
- minimum length of observation time for damage of a given magnitude to produce anomalies in model parameters;
- minimum detectable damage severity;
- reliability of the information provided.

The construction and calibration of the model can be a very complex and costly operation, especially in the case of existing structures, for which the information on the materials, the history of the loads acting on them, the history of the interventions and modifications to which they have been subjected are often not known or only partly known, and only the use of in-depth experimental surveys to reconstruct the structural organization, the materials used and their characteristics and the possible state of deterioration allow a reliable numerical model to be developed.

For further information about model calibration methods, see [4].

4.5.6 Decision-making procedures

The monitoring system may be required to manage thresholds, generally established on the basis of reference limit states, to which the implementation of countermeasures is associated: targeted inspections and surveys in minor cases, execution of provisional measures and decommissioning of the structure in more severe cases.

For further information on thresholds, see [10].

4.5.7 Accuracy requirements

The effectiveness of structural monitoring depends on the reliability of the identification of the condition and behaviour of a structure. Hence, the level of accuracy required to identify system parameters and state variables should be specified at the design stage. The way in which the required accuracy is specified depends on the type of variable (Section 4.5.2) and the form in which the uncertainty is described (deterministic or probabilistic).

4.5.7.1 Definition of required accuracy

The specification of the required accuracy depends on the objective and on the variables to be obtained.

If monitoring leads to a classification of the structural state the case of a classification the uncertainty is the probability of misclassification, defined as the relative frequency, normally expressed as a percentage, with which the system identifies an incorrect state.

The maximum acceptable uncertainty of an individual parameter represented by its numerical value can be described by the maximum error of estimate, or tolerance, of the parameter, defined as the maximum deviation between the estimate of the parameter obtained through monitoring and its exact value. Different tolerance values for positive or negative deviations may be specified where necessary. When the acceptable error depends on the amplitude of the parameter, it may be appropriate to describe the tolerance in terms of a percentage error.

Alternatively, it is possible to describe the tolerated uncertainty probabilistically through the acceptable values of variance or standard deviation of the parameter. When the standard deviation depends on the amplitude of the parameter, it may be appropriate to describe the tolerance in terms of a coefficient of variation.

The acceptable error of a function can be described through the confidence band. Also, in this case different tolerance values for positive or negative deviations can be specified.

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4.5.7.2 Sources of error and evaluation of uncertainty

There are sources of errors of a variety of nature and importance throughout the process leading up to measurement and damage assessment.

In principle the following types of errors can be identified:

- Errors related to the measurement phase and the actual acquisition that determine the **metrological uncertainty** of the data.
- Errors related to data processing and interpretation for damage assessment and numerical updating leading to **model uncertainty**.

The assessment of metrological uncertainty considers all error components that may occur in the measurement chain, including:

- The transducer's instrumental error.
- The noise of the cabling system.
- The noise of the signal conditioning system.
- The analogue/digital conversion error.

Metrological uncertainties related to each source of error can be assessed according to existing regulations or, in lack of them, by statistical analysis of observations and/or a priori estimates based on probabilistic models. It is in anyway necessary to guarantee the correct functioning of the measuring chain through a periodic calibration to be carried out based on the design specifications of the system and/or in the case of exceptional events and extraordinary maintenance.

The following forms of error are considered in the assessment of model uncertainty:

- Estimation errors due to incompleteness of the field.
- Errors due to the uncertainty of the quantities assumed to be deterministic in the model.
- Numerical errors.
- A priori uncertainty in the auxiliary parameters of the model.
- Uncertainties due to model approximation or truncation.
- Other epistemic uncertainties.

The single uncertainties can be expressed in deterministic form through the maximum expected error, which can be expressed in absolute value or in percentage terms. Alternatively, it is possible to describe the tolerated uncertainty probabilistically through the expected values of variance or standard deviation of the parameter.

It should be noted that while the metrological uncertainty is easy to assess a priori once the various components of the system have been identified and the technology has evolved, it can be maintained at the level required for monitoring through the appropriate choice of hardware and the adoption of suitable measures in the installation of the system. The assessment of model uncertainty, on the other hand, still represents a very difficult problem to solve.

4.6 Final design

Once the monitoring strategy has been developed through the preliminary system design, the designer provides a final design of the monitoring system, specifying the system architecture (Section 4.4), hardware (sensors, acquisition units and communication devices), and software (application that allows the acquisition of measurements according to project specifications and their storage in digital format) components.

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The executive design is always accompanied by an installation manual and the specification of the management protocol:

- The installation manual should be complete with explanatory drawings and diagrams, giving instructions for both hardware and software components. The manual also contains methods for on-site quality control of the installation and functioning of the system.
- The management protocol must be appropriately described in an operating and maintenance manual for the monitoring system.

In conclusion, the design of a monitoring system is represented by a set of documents and graphic tables:

- A report containing the objectives of the monitoring, also including a description of the monitored structure with all the information of interest for the monitoring.
- A document containing the formal description of the monitoring system.
- Graphic reports showing the position of the system components, any interaction of these components with existing plants and the position of any signs of structural deterioration.
- Detailed graphic reports providing details for the execution of the system installation.
- A document containing the specifications of the system's software components.
- The installation manual.
- The user and maintenance manual.

4.7 Installation and management

4.7.1 Installation

The installation of the system, for all its components, should be carried out and supervised by competent personnel. The installation of the sensors, to be carried out based on proven valid standards and/or detailed instructions provided by the executive designer and/or producer, is supported by pre- and post-installation acceptance checks, for which appropriate documentation should be provided by the technical supervisor. The sensors must be adequately protected against mechanical damage and the aggression of environmental phenomena. The accessibility of the sensors installed on the structure must be ensured as far as possible.

In the case of sensors pre-installed in materials or components of the structure, their correct functioning should be verified and documented before connection to interrogation lines is made.

The sensors subject to calibration are fine-tuned at the end of the installation according to specifications and the results of the procedure are recorded accordingly. In particular, when possible, an on-site calibration of the entire measurement chain is performed.

The installer should provide the technical supervisor and/or system operator with all documentation relating to the installed sensors, including manufacturer's certificates, verification during installation and certification of calibrations and adjustments performed.

The cabling and other data transmission systems should be properly fixed to the structure and not be subject to vibration, shock, or damage of any kind. The continuity of the lines and the quality of the transmitted signals should be verified and documented for each individual section as well as for all transmission lines. Particular care must be taken with accidental discharge protection subsystems.

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4.7.2 Testing and commissioning

After the installation has been completed, functional tests are carried out on all system components. The technical supervisor should provide the operator with appropriate documentation of all operations carried out in the presence of the parties involved.

4.7.3 Management

The system operator is responsible for ensuring that the system works properly by supervising the data acquisition, storage, and analysis operations if they are carried out automatically, or for supervising the execution of measurements if they are carried out manually or with assistance.

The operator shall also promptly inform the client of the occurrence of any early warning and alarm signals of malfunctioning of the system or parts of it. The management of the system generally includes the provision of periodic reports on the measurements and functioning of the system. The preparation of data analysis and interpretation reports and the final assessment of the condition of the structure may be a task assigned to the operator, or may be assigned to other specialists, typically the designer or structural consultant, whether or not they are part of the client's organisation.

4.8 Monitoring system maintenance

Routine maintenance of the system includes periodic calibration and adjustment operations, to be programmed according to sensor specifications, replacement of damaged sensors, maintenance of power supply subsystems, cleaning of electrical and optical contacts, implementation of software updates. In case of sensor replacement, zero readings must be updated and, if necessary, also the reference numerical models.

To simplify **routine maintenance** operations, the sensors should be placed with some redundancy, at least at the most critical measuring points, and the durability guaranteed by the supplier of the most critical sensors in terms of significance of readings and difficulty of replacement should be adequate.

Extraordinary maintenance is required when, due to unexpected events or exceeding of the service life, significant parts of the system have to be replaced. Such situations generally concern electronic components of the system which have become obsolete or damaged for various reasons and can no longer be repaired.

4.9 Monitoring data acquisition and processing

4.9.1 Data acquisition

The design of the acquisition system should be developed based on the number and location of the measurement points as well as the size of the structure to be monitored, the organization of the system operator and economic considerations regarding the costs of installation and management.

The preliminary design sets out the system performance requirements and provides the system architecture. System performance requirements include:

- Number of channels (or inputs) and types of sensors that can be connected.
- Maximum sampling frequency.
- Possibility of continuous, programmed, manually controlled, automatic acquisition on trigger or pre-trigger.
- Presence of signal pre-analysis electronics.
- Database dimensions.

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- Size.
- Power supply system.
- Energy consumption.

For each type of sensor, acquisition methods should be defined based on the nature of the quantity measured and its significance in the content of subsequent processing:

- Sampling frequency.
- Periodicity of acquisition.
- Signal synchronization requirements.

4.9.1.1 Sampling frequency

The minimum sampling frequency is chosen according to the type of physical phenomenon observed and its variability over time, which can be represented by its maximum frequency. In particular, the Shannon-Nyquist theorem specifies that the sampling frequency cannot be less than half the maximum frequency of the phenomenon:

The down-sampling of the analogue signal determines, due to the aliasing phenomenon, an alteration of the frequency content. When the maximum frequency is much higher than the frequency of interest, the sampling frequency can be reduced, provided that low-pass filters are introduced before the conversion from analogue to digital.

Although in general the quality of the digital signal increases with the sampling frequency, the maximum sampling frequency is nevertheless limited in order to consider the limitations of the data transmission and storage system and in particular:

- Memory capacity of the converter buffer.
- Available transmission bandwidth in the local and remote network.
- System storage capacity.
- Data processing time.
- Speed of access to the database.
- Reporting time to data display.

4.9.1.2 Periodicity of acquisition

In the case of periodic acquisition, the acquisition rate, and the length of each record (or the number of samples) should be specified. It is also necessary to specify whether these parameters can be modified by the user during system management. The length of the record is chosen considering the required accuracy of the post-processed data too. For instance, if the acquired signal is in the frequency domain, the frequency resolution of the transformed signal depends on the total length of the record:

In the case of event-driven acquisition (triggering), the algorithm enabling event recognition should also be specified. The simplest trigger condition is the exceeding of a threshold, which may be related, for example, to:

- The maximum, average or RMS (Root Mean Square) amplitude of the signal.
- The variation of the measurement from the previously acquired sample.
- The amplitude (maximum, average or RMS) of one of the spectral properties of the acquired signal.

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It should also be specified whether the trigger condition applies to the individual sensor or to a combination of sensors. More refined trigger conditions can be implemented to recognize or classify particular events (e.g., earthquake).

Particular attention should be paid to the robustness of the algorithm, in order to avoid false positives, i.e., the acquisition of unwanted signals, and false negatives, i.e. the non-acquisition of significant signals.

4.9.2 Data processing

The raw data coming out of the acquisition system are:

- Single numerical values in the case of static quantities (quantities which vary very slowly over time).
- Temporal sequences of data acquired at a rate appropriately chosen for the needs of the monitoring system in the case of dynamic quantities.

Raw data are hardly significant in itself and, therefore, are subjected to a process that transforms them into valid information to be used for the interpretative models subsequently employed to recognize the behaviour of the structure and to update the interpretative model.

In principle, three successive processing phases can be identified:

- Data pre-processing.
- Evaluation of the significant quantities to be monitored.
- Updating of the interpretative models.

4.9.2.1 Data pre-processing

Data pre-processing aims to improve the quality of the measured data without, however, transforming the quantity under consideration. The main steps consist of:

- Elimination of noise effects, elimination of spikes (anomalous readings), zero drifts, etc.
- Control of the correctness of the acquisition and a verification that a full scale has been chosen that does not lead to instrument saturation or to values that are too low, close to the resolution of the analogue/digital converter.
- Transformation into engineering values.
- Data validation using structure symmetries or with correlations in case of redundant values.

4.9.2.2 Evaluation of the significant quantities to be monitored

The evaluation uses the data recorded and validated after pre-processing to calculate the quantities identified as system-specific objectives (behaviour parameters or state variables). In this case, the operations, of various degrees of complexity, generally involve a real transformation of the measured quantity into other related quantities. Typical processes are, for example:

- Integration of accelerometric values to obtain the displacement.
- The determination of transfer functions.
- The separation of effects into quantities which depend on several causes.
- The determination of modal parameters.

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4.9.2.3 Updating of the interpretative models

The updating includes all the operations required by the damage identification processes carried out through the use of interpretative models that lead to updating the parameters of the numerical model of the structure.

4.10 Data treatment

The data acquired by the instrumentation, once stored and referred to homogeneous periods of acquisition, are processed to obtain time series that can be analysed with the interpretative models and with the recognition processes of the state of the structural system.

In particular, the algorithms used in this phase of the process aim to eliminate noise affecting signal quality, eliminate drifts and non-significant frequencies, recognize and eliminate spurious readings, integrate missing data and finally separate the effects of non-significant environmental components in order to perform the necessary compensations.

A large number of procedures, generally statistical, and numerical filters are available for this purpose and can be conveniently applied.

Other important algorithms for data processing are those aimed at the compression (data reduction) of time series. In the case of long-term monitoring, in fact, the amount of data quickly becomes very large, requiring special informatic solutions for storage and preservation (e.g., clouds), and specific algorithms to optimize the occupation of space and retrieve data efficiently and quickly.

For further information, see D4.1.

4.11 Bridges monitoring

To analyse the behaviour over time of a bridge during the operational life and in case of occurring events that can affect the stability of the bridge itself, it is advisable to use a monitoring system which consists of several instrumentation types. The aim of this section is to provide general guidance on the instrumentation that can be used in a monitoring system. The following elements are analysed, providing for each one an example of a possible monitoring system able to describe its static/dynamic behaviour and, eventually, to detect the appearance and/or evolution of degradation/damage processes:

- Deck
- Piers/abutments
- Bearings
- Joints
- Prestressing cables
- Stay-cables/hangers

It should be noted, though, that the information contained herein represent only an example of the type of instrumentation that could be installed, given that other technologies could be used, either new or existing.

4.11.1 Deck

The information regarding the instrumentation to be used for a bridge deck monitoring system, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.1.

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ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
DECK	ACCELEROMETER	MODAL PARAMETERS	HIGH
		VIBRATIONAL LEVELS	HIGH
	CLINOMETER	DEFORMATION (DISPLACEMENTS/ROTATION)	MEDIUM
	CRACKMETER	CRACK WIDTH	MEDIUM/LOW
	THERMOCOUPLE	TEMPERATURE	LOW
	HUMIDITY SENSOR	HUMIDITY	LOW
	CORROSION SENSOR	RESISTIVITY INDICATOR, POTENTIAL DIFFERENCE, PH	LOW

Table 4.1 – Deck instrumentation.

- **Accelerometers** are devices that measure the vibration or acceleration of the element on which they are fixed. They measure linear acceleration and allow the analysis of vibrations and structures dynamic behaviour. Aiming to obtain a correct dynamic representation, the essential measuring points are quarter and middle span. Additionally, in case of girders, it is recommended to monitor at least the lateral beams, which are the ones more likely to be vulnerable to damage. Besides, in order to correctly derive modal shapes, global vibrational levels and to identify asymmetries and torsional components in vibration modes, it is advisable to install at least two accelerometers for monitored cross section, so that the deck transversal behaviour may be comprehensively described. However, accelerometers positioning must be properly chosen, based on the specific structure needs.
- **Clinometers** analyse structures static behaviour. They allow the detection of structures elements deformations, highlighting the appearance of eventual damage processes. The number of measuring points must be sufficient to allow a correct reconstruction of the deformed shape of the structure. In case of girders, it is recommended to monitor at least the lateral beams, which are the ones more likely to be vulnerable to damage. A minimum number of five clinometers per beam is generally recommended.
- **Crackmeters** are devices that allow to monitor the evolution of cracks width. They are placed on existing cracks to assess their evolution over time or where it is likely that a crack may be appearing.
- **Thermocouples** measure the temperature, thus assessing the environmental conditions in which the structure is located. It is recommended to have at least one probe at the intrados and one at the extrados in order to be able to detect both uniform and variable temperature variations along the height of the section. Temperature sensors location should also be chosen according to the exposure of the structure (sun/shade) to fully characterise the structure behaviour. It is advisable to have at least one thermocouple per monitored structure, to assess the environmental conditions and to eventually correlate changes in monitoring parameters values to thermic phenomena that affect the structure static and/or dynamic behaviour.
- **Humidity sensors** measure humidity and, thus, assess the environmental conditions. Typically, one sensor per monitored structure is sufficient, assuming the punctual measurement can be generalized to the entire structure.
- **Corrosion sensor** can be of various types depending on the desired measurement. Since a local measurement is given, sensors should be placed at points where corrosion has already occurred. This kind of instrumentation can also be implemented for a general measure of the environmental conditions (e.g. pH measure) to assess whether the environment where the structure is located is unfavourable or favourable for potential corrosion phenomena.

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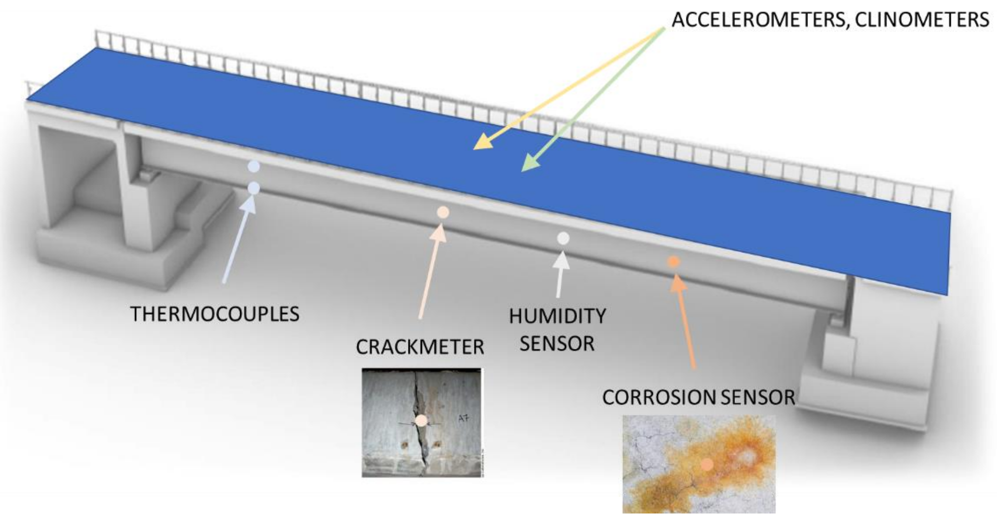

Figure 4.4 – Deck instrumentation (Adjusted from [19]).

4.11.2 Piers/abutments

The information regarding the instrumentation to be used in a pier/abutment monitoring system, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.2.

ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
PIERS/ABUTMENT	ACCELEROMETER	MODAL PARAMETERS	HIGH
		VIBRATIONAL LEVELS	HIGH
	CLINOMETER	DEFORMATION (DISPLACEMENTS/ROTATION)	MEDIUM
	CRACKMETER	CRACK WIDTH	MEDIUM/LOW
	CORROSION SENSOR	RESISTIVITY INDICATOR, POTENTIAL DIFFERENCE, PH	LOW

Table 4.2 – Piers/abutments instrumentation.

- **Accelerometers** are devices that measure the vibration or acceleration of the element on which they are fixed. They measure linear acceleration and allow the analysis of vibrations and structures dynamic behaviour. Aiming to obtain a correct dynamic identification and to measure accelerations, the essential measuring points are the top and the bottom of the pier. In this way, modal shapes that involve or are governed by piers can be derived, and the accelerations transferred from to bottom of the structure to the deck in case of extreme events, such as earthquakes, can be evaluated. Additionally, in case of frame piers, it is recommended to monitor all piers or at least the outer alignments. However, accelerometers positioning must be properly chosen, based on the specific structure needs.
- **Clinometers** analyse structures static behaviour. They allow the detection of structures elements deformations, highlighting the appearance of eventual damage process. It is recommended to monitor all the structure piers and, in case of frame piers, to install clinometers at least on the outer ones (on the two opposite alignments) in order to detect eventual differential settlements. It is advisable to install at least two clinometers per pier (top and bottom).
- **Crackmeters** are devices that allow to monitor the evolution of cracks width. They are placed on existing cracks to assess their evolution over time or where it is likely that a crack may be appearing.

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- **Corrosion sensor** can be of various types depending on the desired measurement Since a local measurement is given, sensors should be placed at points where corrosion has already occurred.

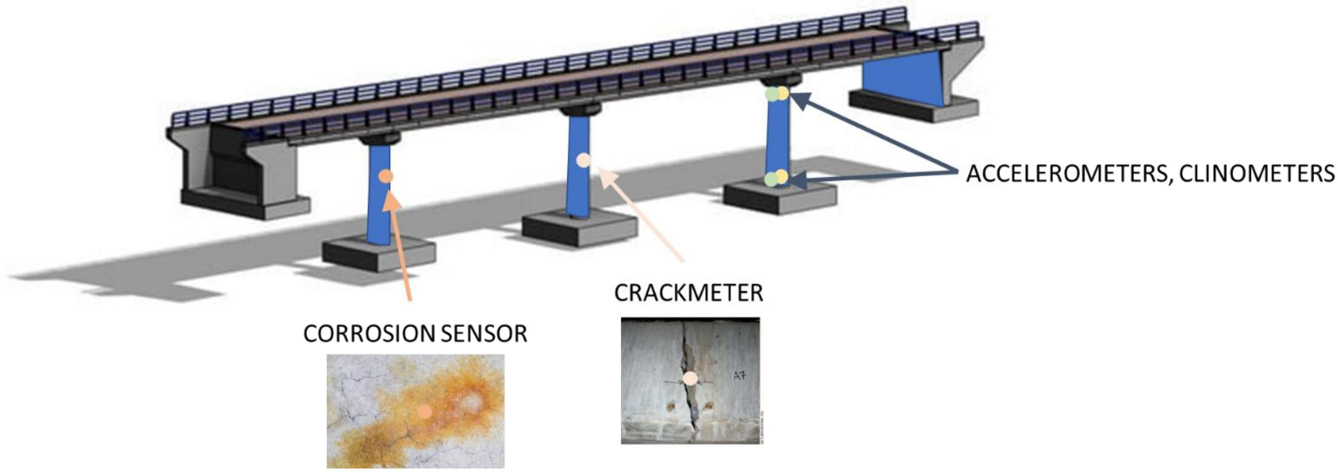

Figure 4.5 – Piers/abutment instrumentation (adjusted from [20]).

4.11.3 Bearings

The information regarding the instrumentation to be used for a bearing monitoring system, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.3.

ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
BEARINGS	DISPLACEMENT TRANSDUCER	DISPLACEMENT	MEDIUM*
	CLINOMETER/ ROTATION TRANSDUCER	DEFORMATION (DISPLACEMENT/ROTATION)	MEDIUM
	LOAD CELL	BEARING REACTION	LOW

* HIGH in case of sudden events, e.g., earthquakes..

Table 4.3 – Bearings instrumentation

The objective of bearing monitoring is to measure their displacement and to detect any anomalies in their behaviour, as well as to assess, in the case of isolated systems, the relative displacement between deck and piers. Each bearing should have its own monitoring system. In order to measure bearing displacements/rotations in space in the three dimensions, it is advisable to install displacement transducers, clinometers or rotation transducers. In addition, if the support reaction wants to be derived and monitored, a load cell can also be installed.

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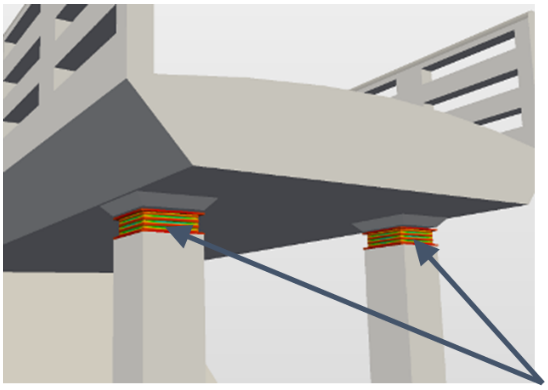

Figure 4.6 – Bearings instrumentation (Adjusted from [21]).

4.11.4 Joints

The information regarding the instrumentation to be used for a joint monitoring system, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.4.

ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
JOINTS	ACCELEROMETER	VIBRATIONAL LEVELS	HIGH
	DISPLACEMENT TRANSDUCER	DISPLACEMENT	MEDIUM/LOW

Table 4.4 – Joints instrumentation.

Joints are the connecting elements between two spans, and, as such, are particularly susceptible to deterioration and require continuous maintenance. In order to monitor any maintenance issues, accelerometers should be installed to identify joints vibrational levels so as to associate any abnormal impulses with localised damage. In addition, it is advisable to install displacement transducers to assess the relative displacement between spans across the joint. If possible, the displacement transducer should be installed at both endpoints of the investigated span.

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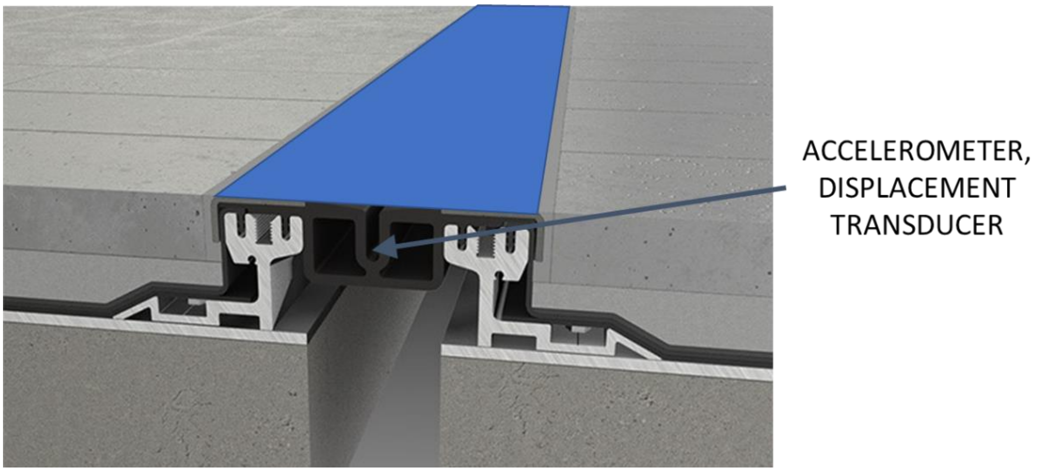

Figure 4.7– Joints instrumentation (adjusted from [22]).

4.11.5 Prestressing cables

The information regarding the instrumentation to be used for a prestressing cable monitoring, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.5.

ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
PRESTRESSING CABLES	ACCELEROMETER	VIBRATIONAL LEVELS	HIGH
		MODAL PARAMETERS	HIGH

Table 4.5 – Prestressing cables instrumentation.

Accelerometers are used to assess any abnormal vibrations and, above all, to identify any changes in modal parameters that could be attributed to a variation of stress in the cable (loss of prestressing force and/or reduction of the cross-section area, e.g. due to corrosion). It is recommended to install at least one accelerometer per segment between two constraint points (header/damper), and to add measuring points in the weak zones and as many as deemed appropriate to collect information along the entire longitudinal development of the cable.

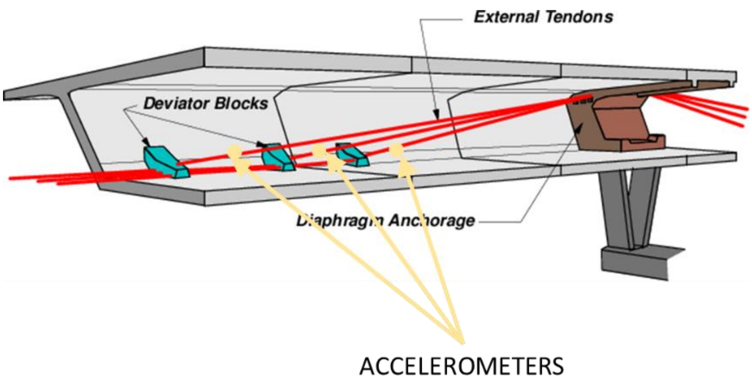

Figure 4.8– Prestressing cables instrumentation (adjusted from [23]).

4.11.6 Stay-cables/hangers

The information regarding the instrumentation to be used to monitor stay-cables and hangers, the corresponding monitoring parameters and the suggested sensors frequency of acquisition is summarized in Table 4.6.

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ELEMENT	INSTRUMENTATION	MONITORING PARAMETER	FREQUENCY OF ACQUISITION
STAY-CABLES/HANGERS	ACCELEROMETER	MODAL PARAMETERS	HIGH
		VIBRATIONAL LEVELS	HIGH
	STRAIN GAUGE	DEFORMATION/STRESS	LOW*

*HIGH in case fatigue behaviour is investigated

Table 4.6 – Stay-cable/hangers instrumentation.

Accelerometers are used to assess any abnormal vibrations and, above all, to identify any changes in modal parameters that could be attributed to a variation of stress in the cable. Moreover, strain gauges can be installed in areas where damage has occurred to measure and monitor the phenomenon evolution.

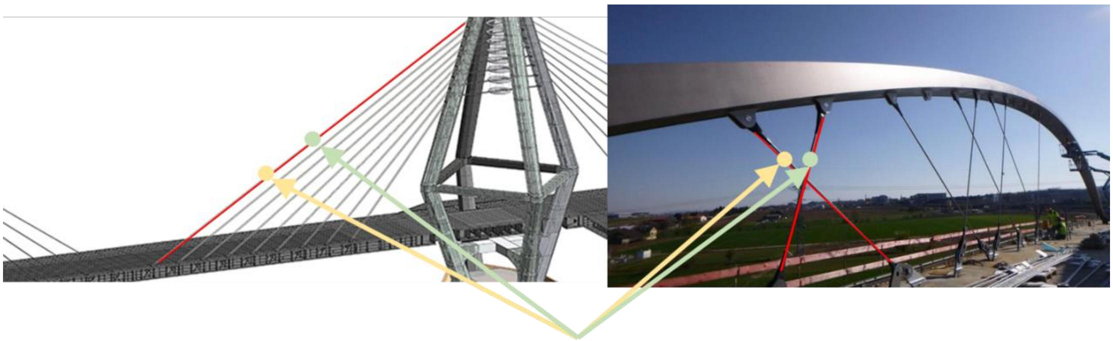

Figure 4.9 – Stay-cable/hangers instrumentation (adjusted from [24] and [25]).

4.12 Tunnels monitoring

Tunnel monitoring is essential for short- and long-term stability monitoring of the tunnel itself, as well as the surrounding soil and adjacent buildings.

Monitoring can be carried out both during the excavation of a tunnel and for an existing gallery.

All the stresses that are caused by the excavation of a tunnel are due to multiple factors, among them the most important relate to the geological-stratigraphic characteristics in which the structure is located, the geotechnical and mechanical parameters of the materials, the excavation methodologies and, lastly, the geometric and elastic characteristics of the linings.

The analysis of the information above leads to a correct decision-making for the design of monitoring systems.

To analyse the behaviour over time of a tunnel during the operational life and in case of occurring events that can affect the stability of the tunnel itself, it is necessary to use a monitoring system which consists of several instrumentation types.

4.12.1 Monitoring system design

The flowchart presented in Section 4.5 can be applied also in the case of tunnels. The fundamental information to be defined is summarized as follows:

- The quantities to be measured (displacements, stresses, interstitial pressures, free surface position, etc.) as a function of the monitoring objectives;

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- The location of the measurement points (on the surface, in the cable, fixed stations, measurement stations, etc.);
- The instrumentation used to perform the measurements;
- The temporal frequency of the measurements;
- The criteria for recording and processing the measurements and the transmission of data to the stakeholders;
- The threshold values of the measures and related provisions to be adopted. The criterion adopted to define these values must be based on the results of the design calculations and on the experience acquired in similar structures.

4.12.2 Displacement measurements

Displacement monitoring requires ad hoc instrumentation, which depends on the type of displacement to be measured and should be placed in an adequate number of cross-sections deemed significant.

The main displacement measurements are described in the following chapters.

4.12.2.1 Convergence measures

“Convergence” is defined as the decrease in the distance between two points of the tunnel. It is calculated by measuring the distance between target points fixed to the excavation walls or to the linings. Through the convergence measures it is possible to check the response of the support system, comparing their values with thresholds and identifying any warning signs of instability (Figure 4.10).

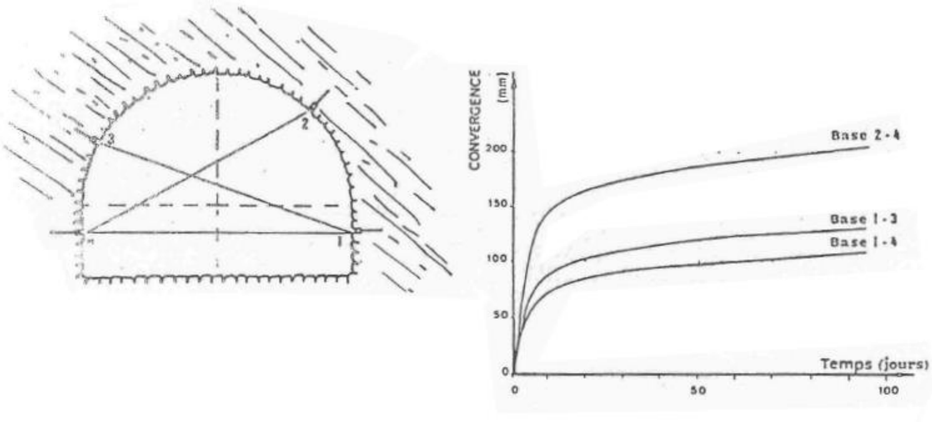

Figure 4.10 – Example of a measure of convergence as a function of time

4.12.2.2 Measurements of mass displacements

It is also possible to detect rock mass displacements.

Their measurement allows to evaluate the deformations, mainly radial, around the cavity and consequently to evaluate the extent of the mass area affected by the excavation.

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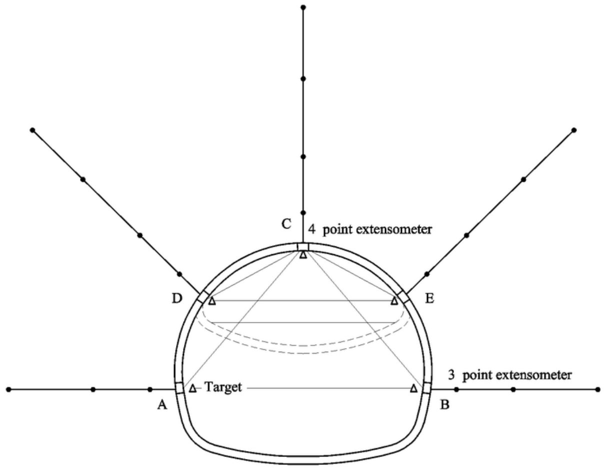

Figure 4.11 – Typical arrangement of strain gauges for measurements of mass displacements [26]

4.12.2.3 Measurements of surface displacements

In the case of "superficial" tunnels, subsidence, and horizontal displacements of both the surface and the rock mass can be measured from the surface.
The objective of surface monitoring is to control the movements of the rock mass, especially in the presence of potentially unstable slopes or escarpments.
The tools used to measure these surface displacements usually include:

- hydraulic levelling chains for the continuous measurement of height variations;
- Topographic levelling for the survey of the level of the spreading of strong points placed on the surface;
- Single-base and multi-base strain gauges to measure the vertical displacements along the tunnel axis and along the vertical ones close to the cable;
- Inclinometers to measure horizontal displacements on the sides of the tunnel;
- Wall clinometers, often installed on pre-existing structures to measure its rotations.

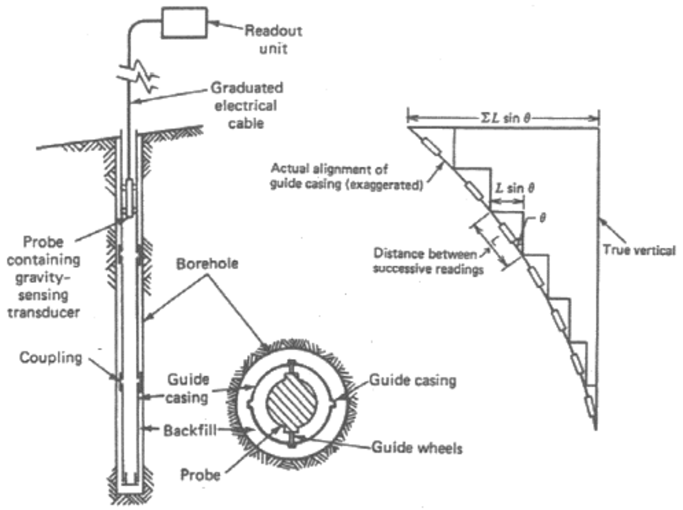

Figure 4.12 – Ground movement measurement by an inclinometer [27]

4.12.2.4 Stress measurements

Tunnels monitoring systems also implies the evaluation of the stresses in the linings and their interaction efforts between the ground/rock mass and the structures. Through the stress measurements it is possible to monitor the stress condition to which the structural elements are subjected.

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The instrumentation used for the measurements usually consists of:

- Pressure cells for the measurement of radial stresses at the mass-lining interface and measurements of radial and circumferential stresses within final concrete linings;
- Load cells often installed at the foot of the ribs and at the piers-shell connection to measure the compression stress in the ribs;
- gauge bars for local deformation measurements. From these measurements, normal voltage values parallel to the axis of the bar are obtained. They can be applied on the ribs or inserted inside the final covering.

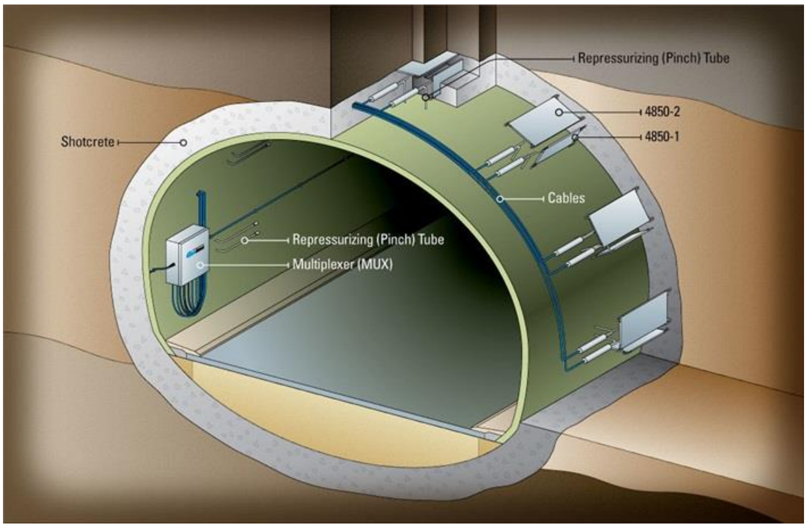

Figure 4.13 – Example of application of pressure cells (Extract from [28])

4.12.3 Standard tunnel monitoring system section

An example for a standard tunnel monitoring system section is provided in Figure 4.14, which includes the number and the positions for the devices related to convergence and loads measurements and the assessment of the state of deterioration. Convergence measurements may be performed through the use of optical sights or laser scanners. It is also possible to evaluate the load to which the tunnel is subjected:

- Stress control: may be achieved with pressure cells or flat jacks;
- Deformation measurements: deformation may be measured by means of MEMS inclinometers and accelerometers or by means of optical fibre technology.

Lastly, it is possible to check the state of deterioration of the tunnel using Georadar.

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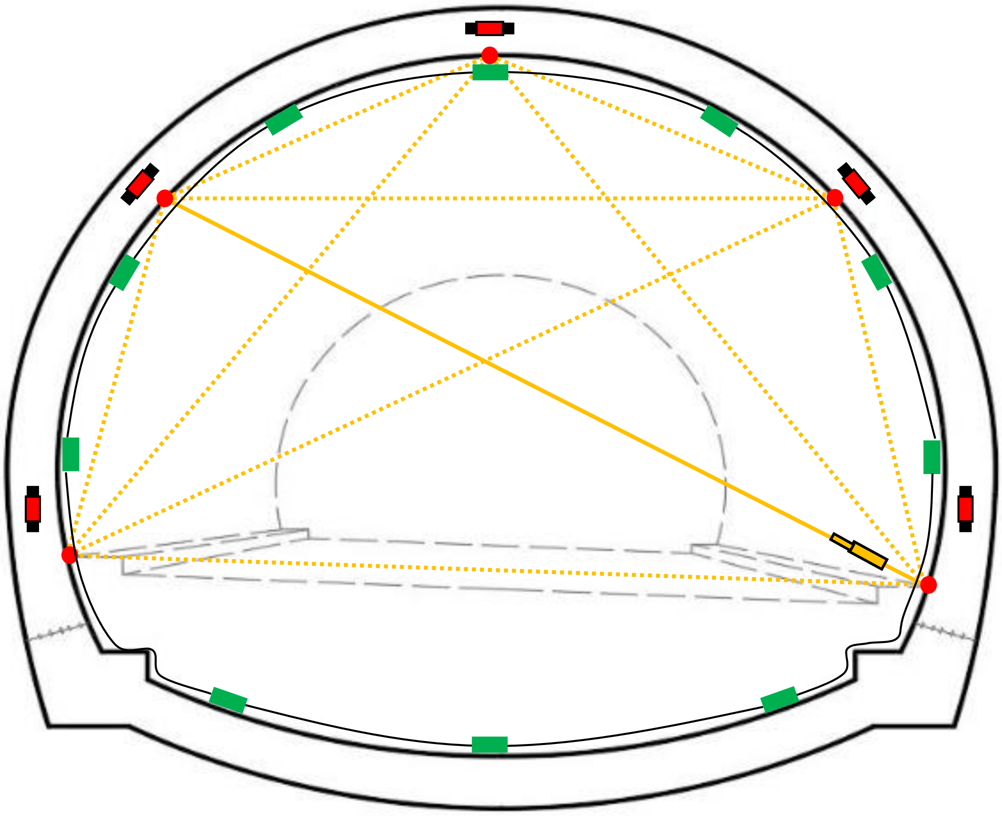

	FIELD	INSTRUMENTATION
CONVERGENCE	Convergence check	Optical sights/ Laser scanner
LOAD	Tension control	Pressure cells/ Flat jacks
	Deformation measures	MEMS inclinometers and accelerometers/ Optic fiber
STATE OF ART	State of deterioration	Georadar

Figure 4.14 – Tunnels monitoring system instrumentation

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5 Damage and degradation processes

Engineering structures are a key component of the infrastructure system and their long-term performance in service plays a critical role on the structural health management. Civil infra-structures are exposed to various external loads such as earthquakes, gusts, environmental actions, traffic, and wave loads during their lifetime, which can lead to damages and degradation processes which need to be taken into account to avoid structural problems and to increase the structural safety and serviceability [29].

One of the most important aspects during the service life of structures is ensuring their safety in time and the early detection of eventual damages and degradations. The principal problem is that structures may get deteriorated and degraded with time in an unexpected way (some damage processes are time-variant dependent, most of them are unavoidable and occur in many cases, even may not be recognizable by visual inspection). This leads to structural failures causing costly repair and heavy loss of human lives. Identifying the principal sources involved in damage and degradation processes is fundamental for understanding them [29]. This chapter presents an overview of the information available in **Error! Reference source n ot found.** regarding the existing damage and degradation processes for bridges and tunnels.

5.1 Damage and degradation processes identification

Damage and degradation processes are classified in two main groups regarding their source. The first group of damages is caused by environmental or external factors (physical, chemical, and biological processes). The second group contains information of damages caused by design and construction issues. in addition, two structural materials are considered to characterize damage and degradation processes: concrete and steel. **Error! Reference s ource not found.** presents in a general way definitions for each damage and degradation process within the scope of the project. All descriptions are focused on explaining how the damage processes are developed in the structure, describing their significant causes and consequences.

5.1.1 Environmental exposure or external factors

Engineering structures are exposed to different environmental conditions (e.g., temperature, wind, precipitation, freeze, traffic), which in combination with other factors lead to damage and degradation processes. These processes can significantly affect material properties. Structure exposure to environmental conditions generally governs the material durability requirements, materials selection, material design, and construction of the structure. Therefore, structures exposure has a high influence on their materials durability, maintenance works, and their costs.

A distinction between physical, chemical, and biological processes is made to understand the effects of environmental exposure and external actions.

5.1.1.1 Physical processes

Damages that lead to a physical disruption in the structure or its components are considered physical damage processes. These processes are usually guided by external actions and their influences, affecting the current or future functionality of the structure. In, **Error! Reference s ource not found.** the descriptions of all physical processes considered can be found, while **Error! Reference source not found.** and **Error! Reference source not found.** show the pr oposed DIs and available technologies for each.

5.1.1.2 Chemical processes

The chemical processes considered involve the dissolution of substances or chemical reactions between components of the materials. Reaction products might cause problems due to dissolution or expansion, leading to a chemical issue. These processes are usually

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prolonged in time, and highly dependent on the material micro-structure such as organic, carbonate, pH value, and sulphate and sulphide ion contents.

5.1.1.3 Biological processes

Biological aggressors such as microorganisms, algae, fungi, and various bacterias can generate biological processes in structures, affecting their service life performance. While **Error! Reference source not found.** describes principal pathologies due to biological p rocesses, focusing on biological growth in substructures, **Error! Reference source not found.** links these pathologies with the proposed damage indicators. Finally, **Error! Reference source not found.** shows the available technology to identify and monitor these biological processes.

5.1.2 Design and construction issues

All structures are prone to be damaged by the construction or rehabilitation of ancillary equipment and services, such as installing additional safety fences and repairing utility apparatus, respectively; these types of processes are covered as design and construction issues. Processes that are developed due to inadequate practices, inadequate design, or lousy construction methods are directly related to poor construction practices, which usually lead to durability issues. **Error! Reference source not found.** defines all design and c onstruction issues in structures.

5.2 Damage indicators

Error! Reference source not found. links damage and degradation process with damage i ndicators (DIs). Damage indicators are carefully assigned to each damage process to understand how the structure is affected and how the combination of them can worsen the structure condition. An overview of damages and degradation mechanisms can be found in Chapter 2.

5.3 Surveying technologies

Error! Reference source not found. contains surveying technologies addressed in the p roject in the Document [11] and related to the detection of specific damage processes mentioned in the Document D2.2, Chapter 4.1. The surveying technologies uniquely considered are a part of broad spectrum category available on the market – however in the document there is mentioned selected group, based on the future potential perspectives and best practices in EU countries.

Tables presented in the Document D2.2 **Error! Reference source not found.** attribute to s pecific damage mechanism a set of surveying technologies that can be proposed for detection and/or quantification of the damage related to bridges and/or tunnels.

Selection of the parameters has been taken into account when comparing surveying technologies – detection effectiveness, accuracy, availability and versatility. Rating scale has been prepared subjectively – should be treated as an overview and preselection suggestions, since the damage mechanisms can be very complex and the choice has to be adjusted to the specific case or more than one surveying methods selected.

Having the description of each surveying methods in the Document [11], and technology explained it is possible for one to consider different factors important in decision on suitability and reliability of the method as a whole and resultant of variables . Apart from the technology itself, it is also quite important to acknowledge whether the method which is the most suitable for the case is also available in the area/country of interest. Depending on the priorities it is obvious to assess, which method will meet the requirements and also will be economically reasonable.

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The section is organised as follows – for each damage process there is a table of techniques dedicated for its detection or symptoms of it according to damage indicators associated with the process. Each surveying technology is rated In the colour scale starting from red colour representing the low level of adjustment, through low-medium/moderate in orange, to moderate-high and high level of adjustment. The rating has been prepared based on the literature review performed in [11] as well as by studying the real cases documentation and the experience of the consortium members. Essential information for each case is to guide the recipient into practical aspects of the decision making process for the large group of surveying methods presented in [11]. with respect to damage process, performance and damage indicators.

The tables demonstrate essential surveying technologies for the damage process and the distinction between them, as it is can be seen – for some of the damage processes the selection is narrowed, which on one hand make the choice easier, but at the same time – it can be also a disadvantage. In other cases there can be quite a variety of techniques, but with similar characteristics – then the choice depends mostly on individual preferences and the budget involved.

5.4 Data analysis methods

Error! Reference source not found. lists all relevant surveying technology and for each t echnology it indicates the applicable and recommended data analysis methods, describing the best practices, state of the art, recent approaches, and further references.

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6 Damage characterization procedure

6.1 General

Throughout this deliverable, along with [11], the different phases experienced in the identification of a damage in an infrastructure have been explained. It comprehended from its detection to the assessment via inspections (Chapter 3) and surveying technologies ([11]), to the monitoring of the structure (Chapter 4). The purpose of this chapter is to summarize and organize this knowledge following a generalized workflow divided in the different phases of the damage characterization procedure. As such, Chapter 6.2 covers damage detection and localization, while Chapter 6.3 deals with its parametrization and assessment. The next two phases, modelling and prediction and monitoring, are explained in Chapters 6.4 and 6.5 respectively. These phases are tightly linked together, as the data generated from monitoring can be used to update models, and the predictions can be checked through monitoring. Nevertheless, modelling and prediction are possible the moment you first identify the damage, it is placed before monitoring. These four phases are represented by the flowchart seen in Figure 6.1.

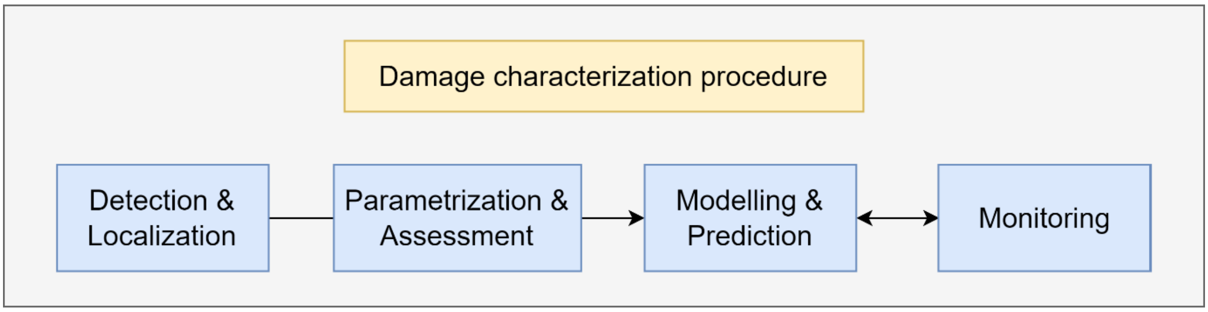

Figure 6.1 – Damage characterization procedure phases

6.2 Phase 1: Damage detection and localization

In order to identify, monitor and model a damage, it is necessary to assert the likelihood that there is a damage in the structure. It must be detected and located with the required level of refinement. This phase is undertaken by the different inspections that can be performed in a structure. In some cases, the damage might be first detected as a result of continuous monitoring, which are explained in Chapter 4 and Chapter 6.5. However, the structure will undergo an inspection regardless, in order to assert the monitoring detection and to determine whether the damage exists in the structure or not.

Since Chapter Inspection provided a detailed view on the type of inspections, along with their motivation and outputs, this chapter will summarize it to better illustrate the damage detection procedure.

There are three levels of inspections: (i) routine maintenance inspections; (ii) conditional maintenance inspections; (iii) extraordinary inspections. Apart from the routine maintenance inspections, the other levels use the reports of lower-level inspections as starting data to better understand the state of the structure. Therefore, the inspection level that first detects the damage is usually the routine maintenance inspection.

This type of inspection aims to verify the serviceability of the structure and is approximately done once per year. It is performed by visual inspection of the structural components in order to detect the possible damages. In order to plan the inspection, the inspector has to check the available documentation and resources at his disposal. In the first place, he should review the structural records, which contain: characteristics of the structure, hazards, the condition of the structure at the time of the last inspection, any worsening of defects over time, significant

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maintenance or modifications since the last inspection, analysis and decision regarding hidden or enclosed structural features that should be accessed. Further steps include risk assessment and methods selection before inspection [30]. Besides the structural records, historical data about fire events, evidence of vandalism, burnt vehicles, other incidents, storage of combustible materials in vicinity of the structure should also be studied. Finally, the location itself must be reviewed. Before data collection, the area around the bridge and nearby access terrain to the designated spaces must be checked, including the clearing of vegetation, and studying the lightening conditions. Also, the relevant areas of study and what information should be collected about the bridge or tunnel are to be set in this stage. It should also include the basic geographical information of the target and surroundings (for instance if satellite imaging will be used), traffic frequency, nearby roads, trees, buildings shall be considered prior to inspection [29]. Once the inspection takes place, its results are reflected in a report that can include both pictures and the inspector comments, to further detail the state of the damage or characterize the possible omissions of the process.

6.3 Phase 2: Damage parametrization and assessment

Once the damage has been located, it is necessary to determine the value of its parameters, including uncertainty, and impact on the overall structure. To do so, the adequate surveying technology or test must be selected, which in turns requires to estimate the physical parameter or event that is to be detected, such as the width of a crack or the erosion of the concrete. The location of the damage must also be addressed, as it might not be superficial, removing certain options.

One of the main challenges when estimating the extent of a damage, is the relationship between the data acquired and the damage. This means how the data taken can serve as a way to estimate its location, extension and impact on structural performance. This challenge is tightly related to the techniques or technologies used to gather data. It also depends not only on the technology itself, but on the damage that is being estimated. The same technique might not be related in the same way to two different kinds of damages.

The different surveying technologies, along with the parameters that they measure, setup, environmental dependency, and their advantages and disadvantages, have been discussed in [11]. Therefore, similarly as in Chapter 6.2, this chapter will represent a support for the information contained in the mentioned deliverable. To do so, Table 6.1 has been created to serve as an index of the different surveying technologies reports contained in [11], together with some others that are out of the scope of IM-SAFE. The purpose of this table is to ease the search for a fitting surveying technology by highlighting the different events/properties that they measure, along with their application, in a single table.

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Surveying Technologies	Measured event / property	Application
LiDAR	Geometric information + Intensity	Asset monitoring
Satellite	Changes over time	Asset monitoring
GPR	**Masonry arch bridges** • Unknown geometries remaining in the interior of the bridge • Evidence of restorations and/or reconstructions • Existence of cavities and fractures/cracking in masonry • Moisture in masonry • Inventory of bridge foundations • Filling distribution in masonry • Thickness of ashlars (pavement, ring arch, spandrel walls, etc.) **Concrete bridges** • Estimation of concrete cover depth. • Mapping reinforcing bars (deck and beams). • Location of cable ducts and other utilities such as deck joints. • Damage detection on concrete (corrosion, cracking, etc.). • Moisture detection and water content estimation. **Tunnels** • Thickness of concrete segment/lining. • Thickness of the backfill grouting layer. • Damages in concrete lining and grouting layer. • Damages (e.g., cracks/fissures and voids) behind tunnel linings. • Moisture/water content. • Depth and location of reinforcement (rebar). • Inspection of reinforced concrete structures (e.g., steel arch). • Location of immersion joints. • Identification of depth and presence of insulation material.	Masonry and concrete bridges. Tunnels
Magnetic and electrical methods	Detection of corrosion in post-tensioned concrete elements. Qualitative analysis, such as localization of the rebars in structure, determination of their dimensions and diameter of cover	Post-tensioned concrete elements
Water resistance test	Characterization of durability of concrete elements and durability of the surface protections. Determine the space of pores and absorption rate in the concrete. Determination of the effectiveness of hydrophobic surface agents used in securing building materials from the influence of water	Concrete
Acoustic Emission Techniques (AE)	• Detection of dynamic processes in materials, • Detection of leaks, • Detection of flaws, • Tracking of degradation processes in concrete, • Detection of damage mechanisms related to corrosion. • Level of intensity of cracking processes, • Integrity testing of metallic structures, • Integrity testing of composite materials, • Integrity testing of concrete structures.	Evaluate possible catastrophic failures or the level of damage. Link the degree of damage in the structure with the operating conditions of the facility.

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Boroscopy and Endoscopy	Detect and photograph abnormal sections on the structural elements of the bridge or tunnel with cracks or deformations or affected by corrosion or chemical attack.	Diagnostics of civil engineer structures
Fibre Optic Sensors (FOS)	• Strain • Deformation • Temperature • Vibration • Pressure • Acceleration • Inclination	Long-term monitoring and remote control of the condition of facilities
Guided Waves Propagation (GW) techniques	• Discontinuity from the wave signal diffracted by the crack. • Detection of delamination and debonding. • Surface cracks depth • Homogeneity of concrete • Quality variation of concrete • Detection of voids, imperfections • Determination of the age of concrete.	Detection of the damage in structural health monitoring of the reinforced concrete
Mechanical tests on cored samples.	Characterization of concrete properties	Reliable assessment of the safety of bridges or tunnels
Qualitative Chemical Methods	Carbonation front depth	Assess the risk of degradation of the structure
Quantitative chemical methods.	•Corrosion risk of rebars resulting from the influence of chlorides ions. •Capacity of concrete to resist chloride ions penetration. •Detection of corrosion parameters of reinforcement •Expansion, cracking, strength loss and disintegration due to sulphate ions. •Quantification of harmful ions.	Quantitatively assessment of the rate of degradation
Radiological and Nuclear Methods	•Cracks dimensions •Early signs of corrosion •Microcracking progress •Quantification of water movement	Assessment of reinforcement characteristics and distribution in reinforced concrete structures.
Surface measurements	Compressive strength and hardness of concrete elements	Monitoring concrete structural elements of bridges
Water penetration test/ Permeability test	Resistance or durability of concrete under hydrostatic pressure	Bridges or tunnels diagnostics
Weight in Motion systems (WIM-Systems)	• Weight of the vehicles (estimation) • Axle group loads, axle loads, wheel loads of the passing vehicles • Tire impact forces • Strain forces • Velocity of the vehicles	Prevention of the overload of the structures
Micro Electro-Mechanical Systems (MEMS) - Accelerometers.	Linear acceleration	Asset monitoring. Analysis of vibrations and structures dynamic behaviour.

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Micro Electro-Mechanical Systems (MEMS) - Clinometers.	Inclination with respect to the horizontal axis	Asset monitoring. Analyse structures static behaviour.
Crackmeters	Crack width	Monitoring of existing/new cracks on bridges or tunnels
Slope clinometers	Slope displacements in the subsurface	Evaluation of eventual relative ground movements near natural slopes, embankments and retaining walls. Monitoring of landslides evolution.
Piezometers	Static water level or hydrostatic pressure in the subsurface	Foundations of tunnels or other structures
Flat Jacks	Stress	In situ stress measurements on tunnel linings and bridge piers.
Displacement Transducers	Displacements	Bridge performance evaluation (e.g., bearing displacements).
Linear Polarization Resistance and AC impedance measurements	Corrosion rate	Laboratory method of determining steel loss during the corrosion process
Radiographic On-site Testing	Location of reinforcement, occurrence of honeycombing	In situ detection
Neutron Activation Analysis	Residual radioactivity	Laboratory method for concrete samples
Ponding test	One-dimensional chloride ingress profile	Laboratory method for concrete samples
Electrical Impedance Spectroscopy	Carbonation depth progress	Laboratory method for studies of corrosion kinetics, morphology of the corrosion
Alternating Current Field Measurements	Crack length, depth sizing of surface cracks	In situ Inspection of fillet welds in highway bridges
Infrastructure Corrosion Assessment Magnetic Method (iCAMM™)	Reinforcement condition monitoring	Condition of bridge culverts
Scanning Electron Microscope (SEM)	Composition of concrete, changes in relationship between constituents as a result of aging or damage process	Laboratory method for microscopic investigation of hardened concrete
Mercury Intrusion porosimetry	Distribution of pore sizes in cement-based materials	Laboratory method for concrete samples

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Active Thermal Imaging/infrared thermography (IRT)	Detecting subsurface deteriorations/ Area and depth of subsurface delamination	Structural Health Monitoring
Hyper Spectral Imaging/ UV/VIS NIR	Water-to-cement-ratio, different curing times	Laboratory method for concrete curing assessment
Laser-Induced Breakdown Spectroscopy	Chloride detection	On-site and laboratory method for concrete structures

Table 6.1 – Surveying technologies summary table

6.4 Phase 3: Damage modelling and prediction

As stated at the beginning of Chapter 6.1, damage modelling and prediction is tightly related to its monitoring, which is described in Chapter 6.5. The reason is that data monitoring constantly generates vast amounts of information in heterogeneous formats as a result of SHM systems, inspections and surveying technologies. This data is stored and managed accordingly to their type and purpose, creating a historical record as the structure advances through its service life. However, the first iteration of the damage model, as well as some early predictions, can be constructed with the first identification of the damage.

With this information, the damage geometry and characteristics can be modelled using appropriate techniques, such as finite elements, and are to be expressed using the appropriate format, such as the Industry Foundation Classes (IFC) or LASer (LAS). Also, the historical data can be transformed into predictive models that can indicate the expected behaviour of the damage. This, in return, facilitates maintenance planning as it can be based on grounded predictions. The purpose of this chapter is to describe the different ways to model a damage, both geometrically and physically, as well as to present the techniques used to predict the evolution of the damage.

6.4.1 Methods for damage modelling

The accurate and reliable structural models must consider the damages that have been estimated and measured in previous phases, along with their uncertainties. Therefore, in this chapter, two different methodologies of damage modelling will be explained.

Geometrical Damage Modelling: In this methodology, the damage is physically represented in the structural model. This modelling can be tackled according to different dimensions depending on the type of information desired. For instance, 3D models can be used to represent the geometry of a solid damage, as well as its position in the structure. These damage models are to be expressed in an appropriate format and are often modelled using the IFC data schema in a BIM model, since these models allow the introduction of semantic information that can further characterize the damage [31]. For instance, [32] modelled a beam crack in IFC 4 using different available software. However, 3D models are computationally expensive, and are used only in special cases where a high level of detail is required. As such, 2D and 1D model simplifications are employed to reduce the computational burden. In 2D models, which use surface bodies, the damages can be expressed as a reduction of the thickness of the body (e.g., if there is some loss of material) or by modifying the stiffness of the contact between bodies (e.g., a damaged connection that turns from a fixed to a pinned connection). For example, in [33], the authors randomly modified the thickness of the elements of a mesh of a surface body to study the variation of its ultimate strength. Concerning 1D models, as they are limited by the deeper level of simplification, usually express the damage either as a variation of the stiffnesses in contacts, or by introducing some releases (e.g. plastic hinges in cracked beams). In [34] the authors update a structural model of a bridge considering

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the nature of its connections by modelling them in different ways (pinned connection, rigid connection and semi-rigid connection using rotational springs).

Physical Damage Modelling: In this approach, the appearance of a damage is bound to change the physical properties of the material. Hence, the damage is not considered by a physical modelling but as an event that modify some of the physical properties in the material. For instance, [35] analysed samples of sea corroded steel and extracted the stress-strain curve of the material to study its variation in presence of corrosion. [36] induced shear deformations in a steel beam to observe the microstructural changes of the material and how they affect the material mechanical properties. [37] & [38] tested and analysed a damaged beam in order to tune their analytical model so that the digital behaviour matches the real word behaviour.

It should be noted that a structural model can use both methods for modelling the damage of the body it represents. Therefore, it is possible to develop a 3D structural model of a tunnel that has the cracks represented geometrically, as well as variations in its material due to the changes in the microstructure that the damage induces in it.

6.4.2 Prediction of damage evolution

The creation of models that estimates the effect of damages on structures is a very useful tool for the prediction of their evolution. In fact, there are multiple methods that have been developed to study and predict them These comprehend from simple regression models to more sophisticated methods such as machine learning or deep learning techniques.

Once the damage of the structure has been modelled, datasets can be created for the training of the algorithms dedicated to the prediction of the evolution of the damage. These datasets must be created by sampling the vector space of the model that represents the damage of the structure, which was modelled using all the information collected in previous sections over the years. Once enough data is obtained, the datasets have to be used to train and validate the commented algorithms, which can be carried out for starting the prediction of the evolution of the studied damage. It should be noted that each of the algorithms commented below are based in particular methodologies or structures:

Linear and non-linear regression: This type of regression describes the relationship between dependent variables and one or more independent variables in which the dependent variables are considered the response or prediction variable. Depending on the nature of our dataset and the behaviour of our variables different methodologies can be used such as simple linear regression, stepwise, multiple, regularization or even mixed regressions based on linear and non-linear effects, more information about the different methodologies can be found in [39] and [40].

Support Vector Machines: This methodology is considered a nonparametric technique that attempts to learn a functional relationship between some pairs of inputs and outputs given an experimental design or dataset. It is considered an extension of the non-linear regression based on the principle of structural risk minimization as is described in [41] and [42].

Polynomial chaos Expansions (PCE): This is a powerful surrogate modelling technique that aims as providing a functional approximation of a computational model through its spectral representation on a suitable built basis of polynomial functions [43], different applications and a more detailed explanation can be found in [44] and [45].

Gaussian process regression (GPR): Also called kriging, is a stochastic algorithm that perform a statistical interpolation method that capitalizes on a gaussian process to interpolate a wide range of complex functions [46]. With the introduction of the GP-regression [47] it is able to support noisy data and has become a widespread use in machine learning. An applied case in reliability analysis will be [48].

Other Machine learning or deep learning methodologies: This field is one of the fields that has experienced in recent years in which different types of architectures and methodologies

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have been implemented for the prediction of all types of applications. Depending on the damage type, its nature, how it develops and changes, the prediction of its evolution can be carried out by different architectures. The different architectures can be classified in two groups, supervised and unsupervised learning. The following table performed in [49] compares some of the most used methodologies y deep learning. More methodologies and information can be obtained in [50] and [51].

Furthermore, if the damage evolution is properly estimated, it is possible to calculate the threshold parameter values that will ensure the safety of the structure. For achieving this purpose, a calibrated numerical model of the structure needs to be generated. The calibration can be achieved by using several model updating techniques to minimize an error function that compares the numerical and experimental structural responses. Starting from the calibrated numerical model, its parameters can be constantly modified to represent the evolution of the damage and to predict the consequences it will have on the structural health response. Therefore, when the failure of the structure is reached, the value of the corresponding parameter can be identified and defined as a threshold value. These values are of real interest for planning the damage monitoring of a structure.

6.5 Phase 4: Damage monitoring

As explained in Chapter Monitoring, the monitoring of a structure aims to determine the state of a structural system and its evolution over time. This also includes the identification and parametrization of phenomena such as degradation and damages, along with their uncertainties. The monitoring intention is that of evaluating the integrity of the system and its ability to perform its function in adequate conditions for a given time. There are different types of monitoring approaches, depending on the intention of the monitoring itself. However, health monitoring is used to obtain real time information that can inform the estimation of the safety and serviceability of a structure.

Structural Health Monitoring (SHM) is the process of implementing a damage identification strategy. This process involves the observation of the structure over time by obtaining periodically spaced measurements. Then, these measurements should be transformed into damage-sensitive features that are to be analysed to estimate the current state of the structure. For long term monitoring, the output of a SHM system reflects the capability of a structure to continue to perform its intended function after being exposed to aging and the accumulation of damage so far. In the case of extreme events, the system can be used as a rapid screening mechanism that indicates the severity of damage and the condition of the structure. The shortcoming of SHM systems is that they are based around the notion that the damage will significantly alter the properties of the structure. However, damages are typically local phenomena's that might not affect the global response of the structure [52]. As a result, the role of inspections or targeted surveys that provide quality and accurate information about a damage and its effect on its local environment is highlighted.

Chapter 6.2, along with Chapter Inspection, noted that routine inspections are usually the tool to detect a new damage, while the conditional and extraordinary inspections usually build up from the knowledge provided by them. These means that higher level inspections are usually linked to the monitoring of a certain damage, since they result in the accumulation of information about the damage. In the same manner, surveying technologies that can be utilized continuously through the lifecycle of the asset provide means to generate quality information that can be used for the estimation of the current state of the damage. Furthermore, the data obtained with surveying technologies that are performed at periodical intervals can be compared to one another in order to assess not only the current state of the damage, but its evolution.

As different SHM systems, inspections and surveying technologies can be used to monitor a certain damage and its influence in the structure, the issue of data interoperability arises. Both SHM systems and surveying technologies usually result in digitalized data, but each one of

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them can provide the data in one or more formats that cannot be directly linked. On the other hand, inspections are usually recorded in a report, which can take the form of a normal document or table, sometimes in physical format. Therefore, digitalization and data interoperability are important enabling technologies and lack of solutions can hinder efficient/real time modelling of damage and prediction of condition development. These issues have been tackled by WP3 and WP4 of this project, so the next chapter will focus solely on the modelling and predictive aspects.

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7 Damage classification

7.1 Damage classification procedures

The development of a damage classification may be a crucial instrument for the assessment of new and existing bridges, as well as for the evaluation of maintenance strategies. Clustering and homogenization of the input data provided by inspection, testing and monitoring is indeed a great deal of effort for road operators and infrastructure managers, which have to manage a huge amount of information in order to keep assets at a desired performance level.

Hence, procedures for damage classification are needed, accounting for type, size and location of defects or other relevant issues depending on the type of structure, the actions on structure, and the risks that may potentially affect the structure in the future, such as the one following from changes in traffic loads or service life demand, and from resilience issues related to climate change and increased use. In case of bridges and tunnels, specific performance indicators (PIs) and damage indicators (Dis) can be included in the database, in order to describe the health status of the assets and accounting for damage in performance assessment and maintenance strategies. These indicators can be qualitative or quantitative based, and they can be obtained during principal inspections, through a visual examination, a non-destructive test or a temporary or permanent monitoring system.

Accordingly, a damage classification procedure consists of the following steps:

1. **Damage detection**: damages affecting the structure under investigation are detected through inspection, testing and monitoring.
2. **Damage characterization**: Once damage has been detected, the following information is needed and must be inserted into the database.
    a. **Level**: network / system / component level at which the damage is detected.
    b. **Location**: identification of the elements of the structure on which damage is located.
    c. **Type**: identification of the type of damage occurring on structures
    d. **Causes**: damage may be due to the overloading of the structure, to the aging of materials and to several damage processes. Damage causes are outlined in [2.3.1].
    e. **Quantification**: identification of the qualitative/quantitative parameters related to the detected damage.
    f. **Extent**: characterization of the extent of damage, which is the basis for intervention and maintenance prioritization and planning.
3. **Information updating**: database information must be updated over time with additional information collected through inspection plan, maintenance interventions and monitoring systems.

7.2 Procedures for including Damage Indicators in evaluating KPIs

Chapter 5 of [4] described Performance Indicators (PI) at the component, system and network levels. Furthermore, in this deliverable (D2.2), Chapter 2 provided an overview of damages and degradation mechanisms, including their basis for evaluation and their possible causes and effects. More specifically, Chapter 2.6 described Damage Indicators (DI) and PI and briefly introduced their relationship. This chapter aims to further characterize this relationship, considering Key Performance Indicators (KPIs) as well.

As mentioned in Chapter 2.6, a Damage Indicator is an observation, or a parameter derived from observations, that serves for quantitative or qualitative damage detection, damage

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localization, damage parametrization or damage assessment. On the other hand, a Performance Indicator is defined as an observation, or a parameter derived from observations, that quantitatively describes property of the structure and/or of the aspect of its performance and are used to qualify fitness of the structure for its purpose during service life. While similar, their domains are vastly different. A DI simply addresses the severity of a single damage, while a PI tackles the asset in generalized manner. However, this also means that they are related. The appearance of a certain damage in a structure, while it might be considered non relevant in structural calculations in the case of minor damages, it is still part of the structure, and will therefore affect the PIs.

The purpose of this chapter is to further characterize the relationship between DIs and PIs. This will be tackled in a generalized manner, as the PIs and DIs of different projects and assets are bound to be different. Nevertheless, this abstraction serves to illustrate the relationship between both concepts, and how they affect one another. Figure 7.1 presents a diagram that expresses the relation between the different concepts.

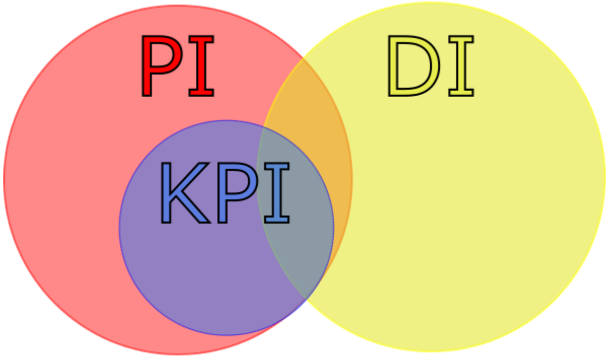

Figure 7.1 – Relationship diagram for KPI - PI – DI

It is better to express their relationship in a top-down approach, starting from the Key Performance Indicators (KPIs). A KPI is a subset of the PIs that are critical for to determine the state of the structure. They carry the most significant information about its current performance and are used to quantify its fitness for purpose. Due to them being a subset of the PIs, the relationship between the three concepts, KPIs, PIs and DIs, is reduced to PIs and Dis..

As such, the first step is to define a hierarchical structure that represents the influence relation between different PIs and the DIs. Generally, a PI might be related to one or more DIs that are performance relevant in its domain. However, as seen in the diagram of Figure 7.1, a certain parameter can be a KPI, a PI and a DI at the same time. As an example, a PI that reflects the behaviour of a bridge would be its frequency. In this case, this PI is influenced by damages (material discontinuities, corrosion…) and ambient conditions (wind, temperature…). However, since it can be directly measured in the structure, no direct relationship to a DI is needed or, if its determined that the frequency measurement is detecting a damage, it can be set as both PI and DI. Furthermore, if the frequency carries sufficiently significant information to determine the fitness for purpose of the structure, it might be declared a KPI as well, falling in the central region of the diagram of Figure 7.1.

In the case of a PI that targets the structural integrity of the truss of a bridge, it should be directly related to the DIs that describe the presence and severity of cracks, or the plastic deformation and displacement of the elements.

In the technical domain, the selected performance measures are to follow certain properties [53]:

- **Appropriateness:** It should adequately reflect one or more goal or objective.
- **Comprehensible and defensible:** It should be clear, simple, and concise.

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- **Comprehensive:** It should cover all the possible consequences.
- **Dimensionality:** It should be comparable across time and geographic regions, and it should have the required level of dimension associated with the decision-making problem.
- **Measurability:** It should be measured objectively.
- **Predictable:** It should be possible to use it as grounds for prediction of future levels.
- **Realistic and operational:** It should be practical, meaning that it should be measurable without excess effort or time.
- **Unambiguous:** It should clearly define its metric and its relationship with the consequences.

While the performance indicators are to be studied for each case, common structural performance indicators are shown below [54]:

- **Structural reliability:** Quantification of the probability of failure. It considers load models and resistance, effects that might occur over time, and the possibility of extreme hazards.
- **Cumulative probability of failure:** Quantification of the probability that the time of failure is less than a generic time interval. It is calculated from the probability density function of the time of failure.
- **Survivor function:** Complement of the cumulative probability of failure. Therefore, it reflects the probability of not failing before a generic time interval.
- **Hazard rate function:** Measure of the instantaneous failure rate. It defines the conditional probability that it will fail in a future time interval, given that it survived until the present time.
- **Structural redundancy:** Estimation of the ability of the structural system to continue carrying load after the failure of one of its components.
- **Structural robustness:** Quantification of the ability of the structural system to suffer damage because of an extreme action.
- **Structural vulnerability:** Reflects the susceptibility to some external natural or man-made action.
- **Structural risk:** Quantification of the combined effects of actions, probability of failures, and related consequences in a given context.
- **Structural resilience:** Estimates the ability of recovery against the occurrence of a hazardous event.

Once the structure that defines which DIs influences which PIs/KPIs is set, the final step is to quantify said influence. Both the structure relations and the quantification are to be evaluated in detail for each case, as the context and resources of each project are different from one another. For instance, an analytic hierarchy process can be used to quantify the importance of a certain set of DIs in a given PI [55]. The starting point of this methodology is the hierarchical/dependence structure between DIs and PIs, as mentioned previously. By knowing which DIs influence a certain PI, it is possible to compare them pair-wise to determine a rank of influence. This is done using a pair-wise comparison matrix, as is shown in Table 7.1 and Table 7.2, which determines the relative importance of different DI pairs to the PI of study. The steps to follow are:

1. To set the importance (**i**) of the DI in the rows with respect of the DIs in the columns. This means that the diagonal of the matrix is always filled with ones. It also means that if DI_1 is given a i_{12} over DI_2, DI_2 has a $1/i_{12}$ over DI_1.

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2. To sum the weights column-wise (∑).
3. To divide the cell values between the sum of its own column (**n**).
4. Average the cells row-wise to obtain the criteria weights (**ω**). These weights determine the rank of importance of the DIs that influence the PI under study.

DIs related to PI_n	DI_1	DI_2	DI_3
DI_1	1	i_{12}	i_{13}
DI_2	$1/i_{12}$	1	i_{23}
DI_3	$1/i_{13}$	$1/i_{23}$	1
Sum	$\sum_1$	$\sum_2$	$\sum_3$

Table 7.1 – Example of Pair-wise comparison matrix for a given PI

DIs related to Pi_n	DI_1	DI_2	DI_3	**Criteria Weights**
DI_1	n_{11}	n_{12}	n_{13}	ω_1
DI_2	n_{21}	n_{22}	n_{23}	ω_2
DI_3	n_{31}	n_{32}	n_{33}	ω_3

Table 7.2 – Example of Criteria weights of DIs for a given PI

7.3 Proposal for the development of a Damage Classification Database

Throughout this deliverable, different types of data acquisition systems or methods have been mentioned, such as inspections or surveying technologies. In Chapter Monitoring and Chapter 6.5, the concept of monitoring has been described and characterized in the context of structural damage. This forms part of the Structural Health Monitoring of an infrastructure and highlights the importance of recording data through time. Damage records are an important part of the structural health of a structure, as it represents the evolution of an asset through its lifecycle. If a specific zone of the structure presents continuous damages over time, despite being maintained, it is necessary to evaluate the possibility of taking actions to ensure a permanent solution. This only represents the value of historical records, but other factors as the time needed for measuring the damage, as well as the equipment used, are important elements in decision making and cost calculations. Most importantly, these records can also be used to assess the implications of the damage for reliability and risk, including the safety of the users.

Therefore, this chapter aims to present a series of fields or attributes that should accompany a damage measurement in a database. Bear in mind that the implementation of these fields is not discussed in this chapter, as databases are part of a vast field of knowledge in computer science [56]. These fields are as follows:

- **Id:** Unique id of the item.
- **Damage parameter:** Damage feature being measured (e.g., crack width).
- **Damage Indicator:** Damage indicator related to the item.
- **Date:** Date of measurement.
- **Location:** Location of the measurement.
- **Duration:** Length of the entire measurement process.
- **Surveying Technology:** Surveying technology used to measure the damage feature.
- **Measurement:** Value of the measure.

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- **Units:** Units of the measurement.
- **Equipment:** Equipment/Sensor used in the survey.
- **Accuracy:** Measurement accuracy. It is related to the equipment.
- **Environmental conditions:** Indication of the environment conditions at the time of measure (e.g., heavy rain or fog). It could be optional for technologies that are not environment-dependant.
- **Responsible:** Indication of the responsible personnel or inspector.

Nevertheless, these items present a simplified set of required parameters needed to infer knowledge from continuous monitoring of a damage. The SHM of an infrastructure often comprehends the use of a vast number of sensors and performing regular inspections. These heterogeneous sources of data are to be studied and joined together to further enrich the knowledge of the asset. Therefore, interoperability and digitalization solutions are to be taken into account, such as the use of Internet Of Things [57] and standardized data formats based around BIM [58], [59].

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8 Actions evaluation

8.1 General

The damages that might appear in a given structure are directly linked to the actions that structure is bearing. Therefore, properly measuring or estimating these actions is a key factor throughout the lifecycle of the asset. In some cases, these actions cannot be directly measured in a practical way, such as the weight of the structure. However, they can be estimated through other means. For instance, the weight of the structure might be estimated from its volume and the density of a series of samples of the materials. The purpose of this chapter is to describe load models provided by standards for actions on both bridges and tunnels, including permanent actions (e.g., weight), variable actions (e.g., wind) and accidental actions (e.g., landslides) and to provide information on how to use in-situ data. In particular, Chapter 8.2 will provide an overview of Eurocode load models and describe how to consider in structural assessment the load effects that these actions generate upon the structure, describing both deterministic and probabilistic load models for each category. Chapter 8.3 will describe the methods for the quantification of actions, direct or indirect, such as using specific sensors. Finally, Chapter 8.4 will describe the use of the obtained data in previous chapters to create representative and accurate load models.

8.2 Actions on structures

Actions are a particular source of potential harm and present hazards to structures. Different actions can act on a structure, directly (direct actions), indirectly (indirect actions) or as environmental actions.

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

- permanent actions, which are present and constant during the entire duration of the reference period and fixed in space (e.g. self-weight, permanent equipment, floor or road finishing, etc.);
- variable actions, that are normally not present during the entire reference period and variable in time and space, 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.

In Table 8.1, examples of potentially relevant actions for bridges and tunnels based on their source and variation in time are given.

Actions	Category
Self-weight	direct & permanent actions
Weight of installations or road finishing etc.	direct & permanent actions
Imposed deformations due to shrinkage	indirect & permanent actions
Imposed deformations due to differential settlements	indirect & permanent actions
Other imposed deformations	indirect & permanent actions
Imposed accelerations	indirect & seismic actions
Traffic loads	direct & variable actions
Wind load	environmental & variable actions

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Snow load	environmental & variable actions
Impact	direct & accidental actions
Explosion	direct & accidental actions
Fire	environmental & accidental actions

Table 8.1 – Examples of potentially relevant actions for bridges and tunnels (Extract from [4])

Concerning actions on bridges and tunnels, two important phenomena should be mentioned: climate change and mobility change.

Climate changes may 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 characteristic values of climatic loads used in the assessment model can significantly vary due to the climate change influence. These characteristic values, according to [60], 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.

For a further description of the way to evaluate Factors of Changes (FC) relying on climate projections, see [4][3.3.1.3].

Moreover, road traffic in Europe has increased significantly over the last decades, both in terms of traffic volume and intensity, with consequent effect on load actions.

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.

8.2.1 Bridges

Actions on bridges can be categorized as follows:

- Permanent actions
    - Structural
    - Non-structural
- Variable actions
    - Snow
    - Wind
    - Traffic
    - Thermal
- Accidental actions

The description of the actions listed above is given in the following paragraphs.

8.2.1.1 Permanent actions

This category includes:

- Structural actions
- Non-structural actions

Self-weight of structural elements belongs to this category, which, according to [61], should be classified as a permanent fixed action and the sum of self-weight of structural and non-structural elements should be taken into account as a single action.

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Appendix A provides the average values of weights per unit volume. The determination of the characteristic values of the self-weight, dimensions and weights per unit volume shall be carried out in accordance with [62], clause [4.1.2].

Materials	Density γ [kN/m³]
pavement of road bridges	
gussasphalt and asphaltic concrete	24,0 to 25,0
mastic asphalt	18,0 to 22,0
hot rolled asphalt	23,0
infills for bridges	
sand (dry)	15,0 to 16,0[1)]
ballast, gravel (loose)	15,0 to 16,0[1)]
hardcore	18,5 to 19,5
crushed slag	13,5 to 14,5[1)]
packed stone rubble	20,5 to 21,5
puddle clay	18,5 to 19,5
pavement of rail bridges	
concrete protective layer	25,0
normal ballast (e.g. granite, gneiss, etc.)	20,0
basaltic ballast	26

Table 8.2 – Weight density of construction materials (Extract from [61])

Permanent actions normally have a small uncertainty related to the magnitude in comparison with other kind of loads. Uncertainties can be generally related to:

- Variability within a structural part
- Variability between different structural parts of the same structure
- Variability between various structures

In particular, according to [61], the characteristic values of the weights per unit volume of structural parts should be updated based on the available data.

8.2.1.1.1 Permanent structural actions

Permanent structural actions are determined by the evaluation of self-weight of the structural elements, which are beams, slab and supporting elements such as cable anchors. The self-weight G is determined by the following equation relation:

$$G = \int_{Vol} \gamma dV \qquad [\ 8\text{-}1\ ]$$

Where V is the volume described by the boundary of the structural part, γ is the weight density of the material. When the material can be assumed homogeneous, the previous equation can be written as:

$$G = \gamma_{av} V \qquad [\ 8\text{-}2\ ]$$

Where γ_{av} is the average value of weight density.

Permanent actions usually have a very small and slow variation in time around their mean or reach monotonically a limiting value.

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From a probabilistic point of view, it is possible to take into account the variation of the self-weight by assuming upper and lower characteristic values (see [4.1.2] of [62]). The evaluation is based on the hypothesis that weight density and dimensions of a structural part are assumed to have Gaussian distributions [63].

Examples for mean values μ_γ and coefficients of variation V_γ are provided in the Table 8.3.

Material	Mean value [kN/m^3]	Coefficient of variation
Steel	77	< 0.01
Concrete		
Ordinary concrete [2)]	24	0.04
High strength concrete	24-26 [4)]	0.03
Lightweight aggregate concrete	[4)]	0.04-0.08
Cellular concrete	[4)]	0.05-0.10
Heavy concrete for special purposes	[4)]	0.01-0.02
Masonry	-	≈ 0.05
Timber [3)]		
Spruce, fir (Picea)	4.4	0.10
Pine (Pinus)	5.1	0.10
Larch (Larix)	6.6	0.10
Beech (Fagus)	6.8	0.10
Oak (Quercus)	6.5	0.10

Table 8.3 – Mean value and coefficient of variation for weight density (Extract from [63])

The values here reported refer to the population of data taken from various sources, but they may be updated based on the available site-specific data (Section 8.4).
Concerning volume, it is usually possible to assume that mean values of dimensions are equal to the nominal values given on drawings, but its mean value V of the structural parts might be calculated directly from the mean values of the dimensions, as well as standard deviation from the values for the standard deviation for the dimensions.

Mean values and standard deviations for deviations of cross-section dimensions from their nominal values are given in Table 8.4.

Structure or structural member	Mean value	Standard deviation
Rolled steel		
steel profiles, area A	0.01 A_{nom}	0.04 A_{nom}
steel plates, thickness t	0.01 t_{nom}	0.02 t_{nom}
Concrete members [2)]		
$a_{nom} \leq 1000$ mm	0.003 a_{nom}	4 + 0.006 a_{nom}
$a_{nom} \geq 1000$ mm	3 mm	10 mm
Masonry members		
unplastered	0.02 a_{nom}	0.04 a_{nom}
plastered	0.02 a_{nom}	0.02 a_{nom}
Structural timber		
sawn beam or strut	0.05 a_{nom}	2 mm
laminated beam, planed	≈ 0	1 mm

Table 8.4 – Mean values and standard deviations for deviation of cross-section dimensions from their nominal values

8.2.1.1.2 Permanent non-structural actions

Permanent non-structural actions are determined by the evaluation of self-weight of the non-structural elements include road pavement, sidewalks, acoustic safety barriers, road safety

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barriers, parapets, finishes, water disposal system, road equipment, side walls and similar. Their value depends on the specific weights of the materials used.

The upper and lower characteristic values of the weights per unit volume of non-structural parts, such as ballast on railway bridges, or backfill on buried structures such as manholes, should be updated if the material is expected to consolidate, become saturated or otherwise change its properties during use.
Hence, the nominal thickness of the ballast on railway bridges should be specified and the upper and lower characteristic values of ballast thickness on railway bridges should be determined considering a deviation from the nominal thickness of ±30%.
In particular, in order to determine the upper and lower characteristic values of the self-weight of waterproofing, decking and other cladding on bridges, where the variability of their thickness may be high, a deviation of the total thickness from the nominal value or other specified value should be taken into account.
According to [61], unless otherwise specified, this deviation should be considered as follows: ±20% if the nominal value includes a coating executed after construction and between +40% and -20% if this coating is not included.
For the self-weight of other non-structural elements, such as:

- handrails, safety barriers, parapets, kerbs, and other finishes of bridges;
- joints/connections;
- lightening elements.

The characteristic values should be taken as equal to the nominal values unless otherwise specified.

8.2.1.2 Variable actions

8.2.1.2.1 Snow actions

In [64] guidance on how to determine the values of loads due to snow is provided. Snow loads are usually classified as variable actions and are assumed to act vertically and to be referred as horizontal projections of the area.

Snow can be deposited on a roof in many different patterns, depending on:

- the shape of the roof;
- its thermal properties;
- the roughness of its surface;
- the amount of heat generated under the roof;
- the proximity of nearby buildings;
- the surrounding terrain;
- the local meteorological climate, in particular its windiness, temperature variations, and likelihood of precipitation (either as rain or as snow).

According to [64], snow load can be determined by the following relation:

$$S = \mu_i C_e C_t s_k \qquad [\ 8\text{-}3\ ]$$

Where:

- μ_i is the snow load shape coefficient;
- s_k is the characteristic value of snow load on the ground;
- C_e is the exposure coefficient;
- C_t is the thermal coefficient.

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Concerning the characteristic value of snow load on the ground s_k, it should be determined in accordance with [62]([4.1.2]): in case of a small variability, one single value G_k may be used, otherwise an upper value $G_{k,sup}$ and a lower $G_{k,inf}$ value shall be used. In this respect, the National Annex specifies the characteristic values to be used, though different characteristic values might be allowed to cover unusual local conditions. For the European ground snow load map, see Annex C of [64].

Alternatively, according to [63], the following relation might be used:

$$S_r = S_g \, r \, k^{h/h_r} \qquad [\ 8\text{-}4\ ]$$

Where:

- S_g is the snow load on ground at the weather station and it is time dependent;
- r is a conversion factor of snow load on ground to snow load on roofs;
- h is the altitute of the building site;
- h_r is a reference altitude (= 300 m);
- k is 1.25 for coastal regions and 1.5 for inland mountainous regions.

Hence, [63] suggests determining the characteristics of the ground snow load S_g based on observations from weather stations.

[63] also includes the description of a probabilistic model for S_g, which is presented by:

- A probability distribution function for the total duration T of the load;
- A probability distribution function for the maximum load S_{gmax} within one year.

The distribution functions $F_{S_{gmax}}$ in case of maritime and continental climates are gamma distributions whose parameters should be based on local observations.

Approaches provided by the references above are summarized here.

8.2.1.2.1.1 Eurocode approach

[64] recommends increasing snow loads values in regions with possible rainfalls on the snow and consecutive melting and freezing, especially in cases where snow and ice can block the drainage system of the roof.

The characteristic value of snow load on the ground s_k may be refined using an appropriate statistical analysis of long records taken in a well sheltered area near the site, given that record periods of less than 20 years are not generally suitable for the variability of maximum winter values. [64] also recommends considering the future development around a specific site of C_e. Provisions about the adjustment of the ground snow load according to return period are given in Annex D and summarized here.

s_k definition is based on annual probability of exceedance of 0,02. Nevertheless, If the available data show that the annual maximum snow load can be assumed to follow a Gumbel probability distribution, then the relationship between the characteristic value of the snow load on the ground and the snow load on the ground for a mean recurrence interval of n years is given by the formula:

$$s_n = s_k \left\{ \frac{1 - V\frac{\sqrt{6}}{\pi}\left[ln\left(-ln(1-P_n)\right)+0{,}57722\right]}{(1+2{,}5923\, V)} \right\} \qquad [\ 8\text{-}5\ ]$$

Where:

- s_k is the characteristic snow load on the ground (with a return period of 50 years, in accordance with [62]
- s_n is the ground snow load with a return period of n years;

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- P_n is the annual probability of exceedance [equivalent to approximately $1/\,n$, where n is the corresponding recurrence interval (years)];
- V is the coefficient of variation of annual maximum snow load.

This expression is represented graphically in Figure 8.1, where X is the return period in years and $Y = s_n/s_k$.

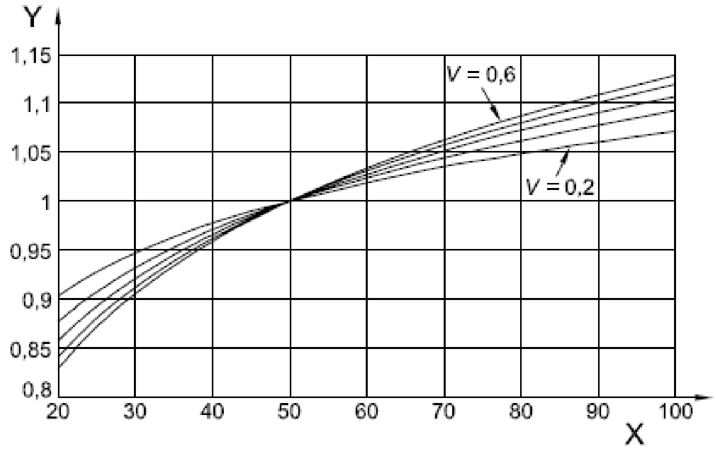

Figure 8.1 – Adjustment of the snow load on the ground in relation to changes of the return period (Extract from [64])

8.2.1.2.1.2 JCSS approach

The characteristics of the ground snow load S_g should be determined based on observations from weather stations, which can provide data in terms of water-equivalents of snow or depths of snow. In the first case the values can be used directly to determine the ground snow load, while in the second case the data on snow depth must be converted to snow load by the following relation:

$$S_g = d\,\gamma(d) \qquad [\ 8\text{-}6\ ]$$

Where:

- d is the snow depth;
- $\gamma(d)$ is the average weight density of the snow, that derives from

$$\gamma(d) = \frac{\lambda\gamma(\infty)}{d} ln\left\{1 + \frac{\gamma(0)}{\gamma(\infty)}\left[exp\left(\frac{d}{\lambda}\right) - 1\right]\right\} \qquad [\ 8\text{-}7\ ]$$

For further information, see [63].

8.2.1.2.2 Wind actions

In [65] and [63] guidance on the determination of natural wind actions is provided.

Wind effects on buildings and structures depend on the exposure of buildings, structures and their elements to the natural wind, but also on the dynamic properties, the shape and dimensions of the building/structure. Wind actions are classified as variable fixed actions and fluctuate with time and may act directly as pressures on the external surfaces, indirectly on the internal surfaces and directly on the internal surface of open structures.

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The evaluation of wind actions is based on wind velocity or velocity pressure, which are composed of a mean and a fluctuating component.

Wind mean velocity v_m can be determined from the basic wind velocity v_b, defined as:

$$v_b = c_{dir}\, c_{season}\, v_{b,0} \qquad [\ 8\text{-}8\ ]$$

Where

- c_{dir} is the directional factor
- c_{season} is the season factor
- $v_{b,0}$ is the fundamental value of the basic wind velocity, which is defined as the characteristic 10 minutes mean wind velocity, irrespective of wind direction and time of year, at 10 m above ground level in open country terrain with low vegetation such as grass and isolated obstacles with separations of at least 20 obstacle heights.

Wind actions on structures and structural elements should be determined taking into account both external and internal wind pressures.

The fluctuating component of the wind is represented by the turbulence intensity. For further information on the coefficients for the evaluation of wind actions, see [65].

Parameter	Subject Reference
peak velocity pressure q_p	
basic wind velocity v_b	4.2 (2)P
reference height z_e	Section 7
terrain category	Table 4.1
characteristic peak velocity pressure q_p	4.5 (1)
turbulence intensity I_v	4.4
mean wind velocity v_m	4.3.1
orography coefficient $c_o(z)$	4.3.3
roughness coefficient $c_r(z)$	4.3.2
Wind pressures, e.g. for cladding, fixings and structural parts	
external pressure coefficient c_{pe}	Section 7
internal pressure coefficient c_{pi}	Section 7
net pressure coefficient $c_{p,net}$	Section 7
external wind pressure: $w_e=q_p\, c_{pe}$	5.2 (1)
internal wind pressure: $w_i=q_p\, c_{pi}$	5.2 (2)
Wind forces on structures, e.g. for overall wind effects	
structural factor: $c_s c_d$	6
wind force F_W calculated from force coefficients	5.3 (2)
wind force F_W calculated from pressure coefficients	5.3 (3)

Table 8.5 – Calculation procedures for the determination of wind actions (Extract from [65])

Wind actions on bridges, in particular, produce forces in the x, y and z directions, as shown in Figure 8.2.

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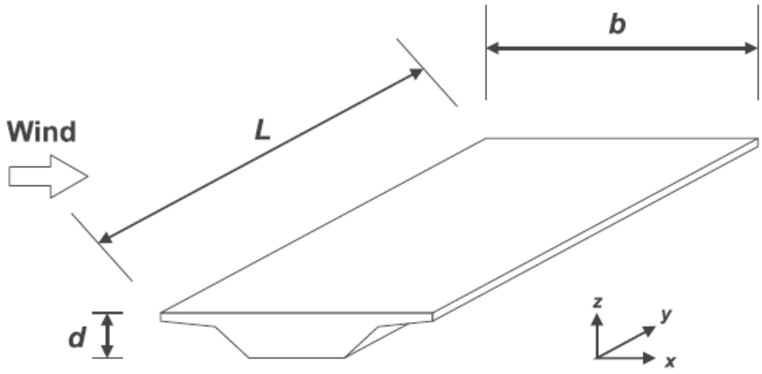

Figure 8.2 – Directions of wind actions on bridges (Extract from [65])

In particular, the wind force in x-direction may be obtained with the following equation:

$$F_W = \frac{1}{2}\,\rho\, {v_b}^2\, C\, A_{ref,x} \qquad [\ 8\text{-}9\ ]$$

Where

- v_b is the basic wind speed
- C is the wind load factor
- $A_{ref,x}$ is the reference area
- ρ is the density of the air

The wind force in z-direction may have significant effects only if it is of the same order as the dead load, whilst the one in y-direction is not usually taken into account.

For further information on wind actions on bridges and on combination of wind and traffic actions in case of simultaneity for road and railway bridges, see chapter 8 of [65].

The influence of dynamic effects related to wind loads, typical in the case of high-rise buildings, should be considered, since they cause oscillations in the along wind, across wind, and torsional directions. These effects are described in annex E of [65].

In [63] the correspondent probabilistic model is given, in which it is assumed that mean wind velocities, for any terrain category, height above the ground and averaging time interval have a Weibull distribution.

$$F_{\bar{U}}(x) = 1 - exp\left[-\frac{1}{2}\left(\frac{x}{\sigma}\right)^k\right] \qquad [\ 8\text{-}10\ ]$$

With k close to 2.

8.2.1.2.3 Traffic actions

8.2.1.2.3.1 ULS models

In [66] guidance on models and representative values for traffic loads on roadway bridges, footbridges and railway bridges is provided.

Loads due to road traffic consisting of cars, trucks and special vehicles, give rise to vertical and horizontal forces, static and dynamic. Vehicle traffic can differ between bridges by:

- composition (e.g. percentage of trucks and trucks types);

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- density (e.g. average number of vehicles per year);
- maximum weight of vehicle weights;
- axle load;
- presence of traffic signs limiting the weight of vehicles.

Loads are applied to conventional lanes, whose location and numbering should be determined in accordance with the recommendation given here.

The width of the carriageway, w, must be measured between the curbs of pavement or between interior boundaries of vehicle containment systems and must not include the distance between the vehicle containment systems or the curbs of the traffic dividers, nor the widths of such vehicle containment systems.

The number of lanes is represented in the following table:

Carriageway width w	*Number of notional lanes n_l*	*Width of a notional lane*	*Width of the remaining area*
$w<5.4$ m	1	3 m	w-3 m
5.4 m $\leq w<6$ m	2	0.5 w	0
6 m $\leq w$	Int(w/3)	3 m	$w-3\times n_l$

Table 8.6 – Subdivision of the carriageway (Extract from [66])

When the carriageway of a bridge deck is physically divided into two parts, separated by a central reservation, then:

- if the two parts are separated by a road containment system permanent, each part, including the emergency lanes or the platforms, are divided into conventional lanes;
- if the two parts are separated by a road containment system temporary, all carriageway, including the central reservation, are divided into conventional lanes.

The conventional lanes are arranged and numbered in order to induce the most unfavourable design conditions: the lane that gives the most unfavourable effect is numbered Lane Number 1, the one that gives the second most unfavourable effect is numbered Lane Number 2, etc (see Figure 8.3).

Legend:

W= Width of the carriageway

- w_l= Lane width
  - 1= Lane Number 1
  - 2= Lane Number 2
  - 3= Lane Number 3
  - 4= Remaining surface

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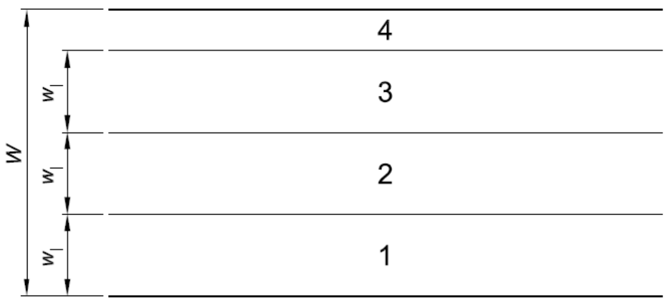

Figure 8.3 – Example of the numbering of the Lanes in the most general case (Extract from (EN-1991-2, 2005))

For each individual verification, the load models, on each notional lane, should be applied on such a length and so longitudinally located that the most adverse effect is obtained, as far as this is compatible with the conditions of application defined below for each model.

Load Models for vertical loads represent the following traffic effects:

- **Load Model 1** (LM1): concentrated and uniformly distributed loads, which they cover most of the effects of truck and motor vehicle traffic.
- **Load Model 2** (LM2): single axle load applied to a specification tire footprint, which covers the dynamic effect of normal traffic on structural elements of small span.
- **Load Model 3** (LM3): a set of vehicle axle loads special (for example for industrial transport) that can travel on roads enabled for exceptional loads. It is to be used for global and local verifications.
- **Load Model 4** (LM4): a crowd load, to be used only for checks global.

8.2.1.2.3.1.1 Load Model 1

Load Model 1 consists of two parts:

- Double-axle concentrated loads (tandem system: TS), each axle having the following weight:

$$\alpha_Q Q_k \qquad [\ 8\text{-}11\ ]$$

where α_Q are correction coefficients.

- A uniformly distributed load (UDL system), with the following weight per square meter of lane:

$$\alpha_q q_k \qquad [\ 8\text{-}12\ ]$$

where α_q are correction coefficients.

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Location	Tandem system Axle loads Q_{ik} (kN)	UDL system q_{ik} (kN/m^2)
Lane Number 1	300	9
Lane Number 2	200	2.5
Lane Number 3	100	2.5
Other Lanes	0	2.5
Remaining Area	0	2.5

Table 8.7 – Load model 1 - characteristic values [66]

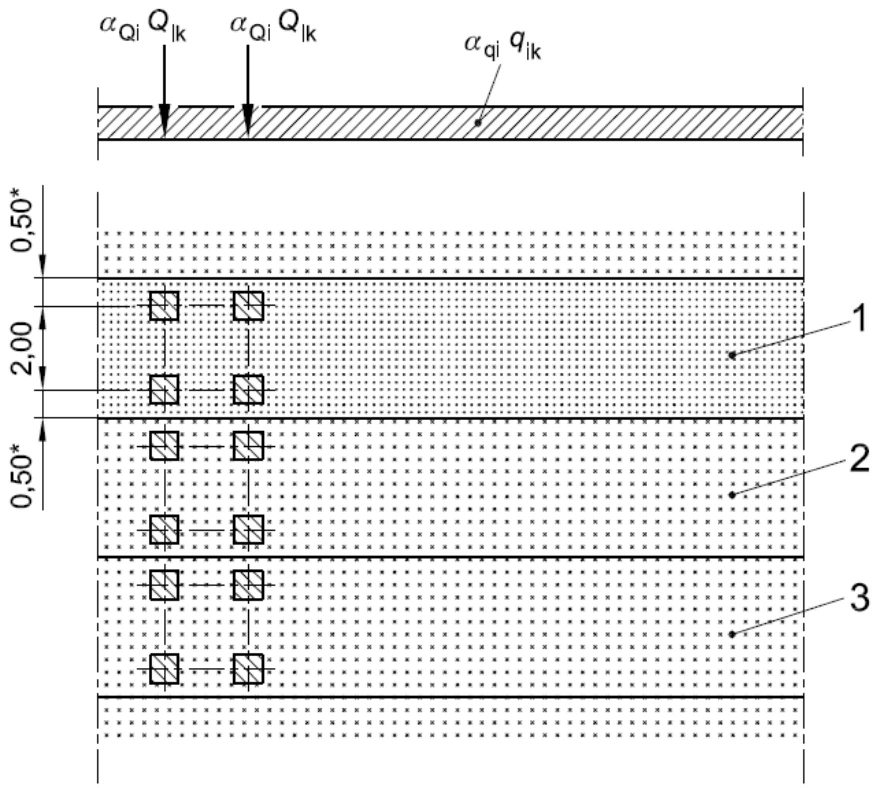

Figure 8.4 – Application of load model 1 (Extract from [66])

For local verifications, a tandem system is applied in the most unfavourable position. If two tandem systems are considered on adjacent lanes, these can be close together at a distance between the axles of the wheels less than 0.5 m (see Figure 8.5).

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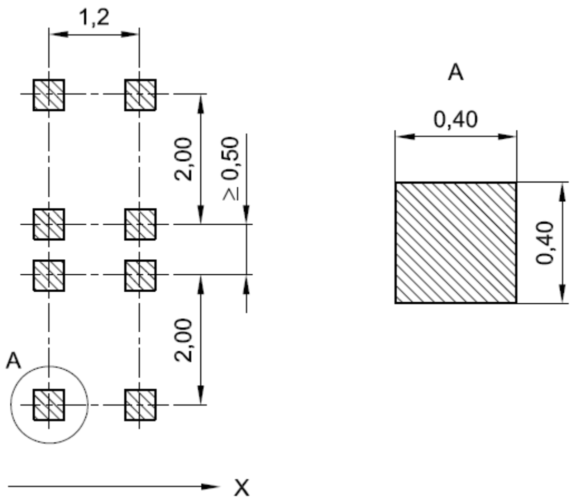

Figure 8.5 – Application of tandem systems for local verifications (Extract from [66])

8.2.1.2.3.1.2 Load Model 2

Load Model 2 consists of a single axle load $\beta_Q Q_{ak}$ with Q_{ak} equal to 400 kN, including dynamic amplification. The model 2 is considered traveling in the direction of the longitudinal axis of the bridge and should be apply in any location on the roadway. If necessary, only one wheel load of $\beta_Q \cdot 200$ kN should be considered. The contact surfaces of the wheel, if not otherwise specified, is a rectangle of sides 35 × 60 cm.

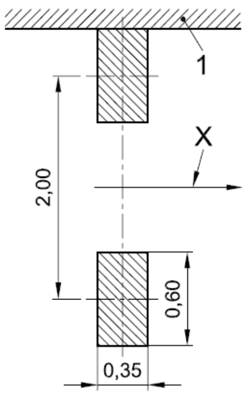

Figure 8.6 – Application of load model 2 (Extract from [66])

8.2.1.2.3.1.3 Load Model 3 (special vehicles)

Special vehicle models must be defined and taken into account when significant.

8.2.1.2.3.1.4 Load Models 4 (crowd load)

The crowd load, if significant, must be represented by a model of load consisting of a uniformly distributed load (which includes amplification dynamic) equal to 5 kN/m^2.

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8.2.1.2.3.1.5 Rail traffic actions

General rules are given for the calculation of the associated dynamic effects, centrifugal forces, nosing force, traction and braking forces and aerodynamic actions due to passing rail traffic.

Rail traffic actions are defined by means of load models, which are the following:

- Load Model 71 (and Load Model SW / 0 for continuous bridges) to represent normal rail traffic on the main railway lines;
- SW / 2 Load Model to represent heavy loads;
- HSLM Load Model to represent the loads of trains used for transport passengers for speeds greater than 200 km / h;
- "Unloaded train" loading model to represent the effects of an unloaded train.

For further information on traffic load on road and rail bridges, see [66].

8.2.1.2.3.1.6 Probabilistic traffic load models

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. 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.

For further information, see 8.4.

8.2.1.2.3.2 Fatigue models

Traffic running on bridges produces a stress spectrum which may cause fatigue. The stress spectrum depends on the geometry of the vehicles, the axle loads, the vehicle spacing, the composition of the traffic and its dynamic effects.

Concerning calculation of fatigue lives, the separate models for road and railway bridges are described in paragraph [4.6] and [6.9] of [66] respectively.

In particular, there are 5 fatigue load models: 1, 2 and 3 aim to determine the maximum and minimum stresses resulting from the possible load arrangements on the bridge of any of these models, whilst 4 and 5 are intended to be used to determine stress range spectra resulting from the passage of lorries on the bridge. Fatigue Load Model 5 is the most general model, since it uses actual traffic data.

In general, a traffic category on a bridge should be defined, for fatigue verifications, at least, by:

- The number of slow lanes
- The number of heavy vehicles observed or estimated, per year and per slow lane.

8.2.1.2.3.2.1 Load Model 1

Fatigue Load Model 1 has the configuration of the characteristic Load model 1 described in 8.2.1.2.3.1.1, with the values of the axle loads equal to $0{,}7\ Q_{ik}$ and the values of the uniformly distributed loads equal to $0{,}3\ q_{ik}$ and (unless otherwise specified) $0{,}3\ q_{rk}$.

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The maximum and minimum stresses ($\sigma_{FLM,max}$ and $\sigma_{FLM,min}$) should be determined from all possible load arrangements of the model on the bridge. The design value of the resulting stress range $\Delta\sigma_{FLM1} = \sigma_{FLM1,max} - \sigma_{FLM1,min}$ should be equal to or smaller than the design value of the constant amplitude fatigue limit of the applicable S-N curve:

$$\Delta\sigma_{FLM1}\,\gamma_{F,fat} \leq \frac{\Delta\sigma_D}{\gamma_{M,fat}} \qquad [\ 8\text{-}13\ ]$$

8.2.1.2.3.2.2 Load Model 2

Fatigue Load Model 2 consists of a set of idealised lorries, called "frequent" lorries: the maximum and minimum stresses should be determined from the most severe effects of different lorries, separately considered, travelling alone along the appropriate lane.

According to [66], each "frequent lorry" is defined by:

- the number of axles and the axle spacing,
- the frequent load of each axle,
- the wheel contact areas and the transverse distance between wheels.

This information are summarized in Table 2.1 – Damage processes for reinforced concrete and steel structures

.

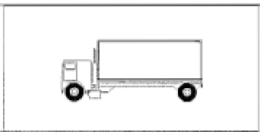
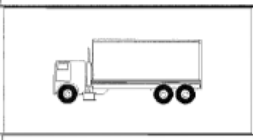
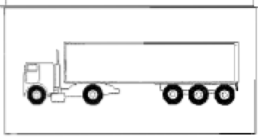
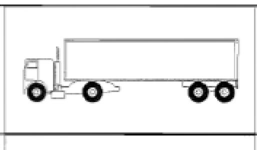
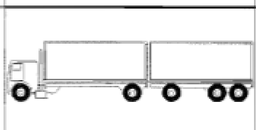
1 LORRY SILHOUETTE	2 Axle spacing (m)	3 Frequent axle loads (kN)	4 Wheel type (see Table 4.8)
	4,5	90 190	A B
	4,20 1,30	80 140 140	A B B
	3,20 5,20 1,30 1,30	90 180 120 120 120	A B C C C
	3,40 6,00 1,80	90 190 140 140	A B B B
	4,80 3,60 4,40 1,30	90 180 120 110 110	A B C C C

Table 8.8 – Set of "frequent" lorries [66]

The maximum and minimum stresses ($\sigma_{FLM2,max}$ and $\sigma_{FLM2,min}$) should be determined from all possible load arrangements of the model on the bridge.

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The design value of the resulting stress range $\Delta\sigma_{FLM2} = \sigma_{FLM2,max} - \sigma_{FLM2,min}$ should be equal to or smaller than the design value of the constant amplitude fatigue limit of the applicable S-N curve:

$$\Delta\sigma_{FLM2}\,\gamma_{F,fat} \leq \frac{\Delta\sigma_D}{\gamma_{M,fat}} \qquad [\ 8\text{-}14\ ]$$

8.2.1.2.3.2.3 Load Model 3

This model consists of four axles, each of them having two identical wheels. The geometry is shown in Figure 8.7. The weight of each axle is equal to 120 kN, and the contact surface of each wheel is a square of side OAO m.

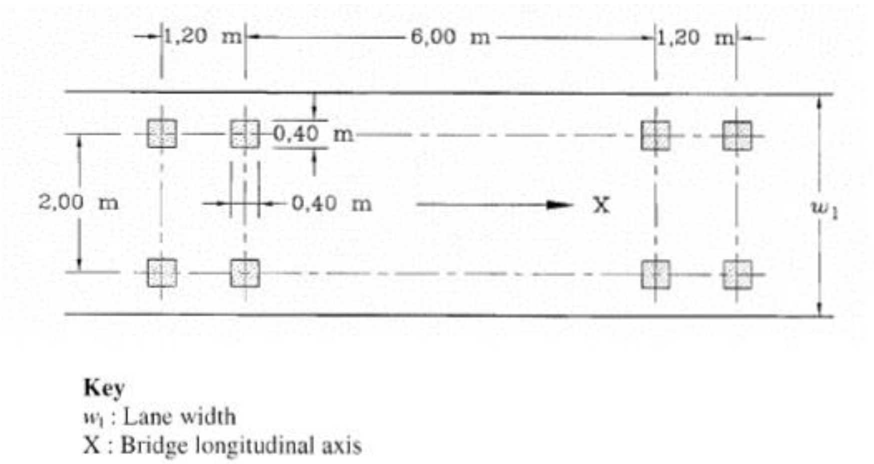

Figure 8.7 – Fatigue Load Model 3 [66]

8.2.1.2.3.2.4 Load Model 4

Fatigue Load Model 4 consists of sets of standard lorries which produce effects equivalent to those of typical traffic on European roads. A set of lorries appropriate to the traffic mixes predicted for the route as defined in Table 8.9 and Table 8.10 should be taken into account. This model, based on five standard lorries, simulates traffic which is deemed to produce fatigue damage equivalent to that due to actual traffic of the corresponding category.

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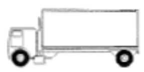
VEHICLE TYPE			TRAFFIC TYPE			
1	2	3	4	5	6	7
			Long distance	Medium distance	Local traffic	
LORRY	Axle spacing (m)	Equivalent axle loads (kN)	Lorry percentage	Lorry percentage	Lorry percentage	Wheel type
	4,5	70 130	20,0	40,0	80,0	A B
	4,20 1,30	70 120 120	5,0	10,0	5,0	A B B
	3,20 5,20 1,30 1,30	70 150 90 90 90	50,0	30,0	5,0	A B C C C
	3,40 6,00 1,80	70 140 90 90	15,0	15,0	5,0	A B B B
	4,80 3,60 4,40 1,30	70 130 90 80 80	10,0	5,0	5,0	A B C C C

Table 8.9 – Set of equivalent lorries [66]

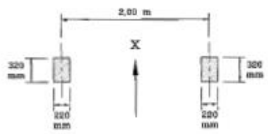
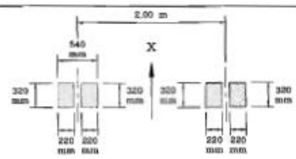
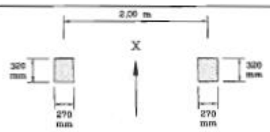
WHEEL/ AXLE TYPE	GEOMETRICAL DEFINITION
A	
B	
C	

Table 8.10 – Definition of wheels and axles [66]

According to [66], each standard lorry is defined by:

- the number of axles and the axle spacing,
- the equivalent load of each axle
- the wheel contact areas and the transverse distances between wheels.

The stress range histogram and the corresponding number of cycles from each fluctuation in stress during the passage of individual lorries on the bridge should be the Rainflow or the Reservoir counting method. This histogram contains all stress cycles $\Delta\sigma_i$. The fatigue damage, D, should be determined using the following equation:

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$$D_d = \frac{1}{5*10^6} \sum_{i=1}^{n} min\left[\left(\frac{\gamma_{M,fat}*\gamma_{F,fat}*\Delta\sigma_i}{\Delta\sigma_D}\right)^3, \left(\frac{\gamma_{M,fat}*\gamma_{F,fat}*\Delta\sigma_i}{\Delta\sigma_D}\right)^5\right] \quad [\ 8\text{-}15\ ]$$

The design damage should be equal to or smaller than 1.

8.2.1.2.3.2.5 Load Model 5

Fatigue Load Model 5 consists of the direct application of recorded traffic data, supplemented, if relevant, by appropriate statistical and projected extrapolations.

A stress history can be obtained by analysis using recorded representative real traffic data, multiplied by a dynamic amplification factor, which should take into account the dynamic behaviour of the bridge and depends on the expected roughness of the road surface and on any dynamic amplification already included in the records.

8.2.1.2.3.2.6 Probabilistic fatigue load models

According to [67], the current fatigue load models applied in Europe are based on traffic load measurements in 1986 and the most frequently used fatigue load model is unable to represent the fatigue action effects of today's European traffic. Hence, [67] proposes a new fatigue load model whose parameters can be calibrated on WIM data, in order to have a significant improvement in accuracy compared to the existing models. This approach in summarized in 8.4.

8.2.1.2.4 Thermal actions

In [68] and [63] guidance on the determination of thermal actions is provided.

Thermal actions can be classified as variable and indirect actions. The temperature distribution within an individual structural element may be split into the following four essential constituent components, as illustrated in Figure 8.8:

a) A uniform temperature component, ΔT_u ;
b) A linearly varying temperature difference component about the z-z axis, ΔT_{MY} ;
c) A linearly varying temperature difference component about the y-y axis, ΔT_{MZ} ;
d) A non-linear temperature difference component, ΔT_E. This results in a system of self-equilibrated stresses which produce no net load effect on the element.

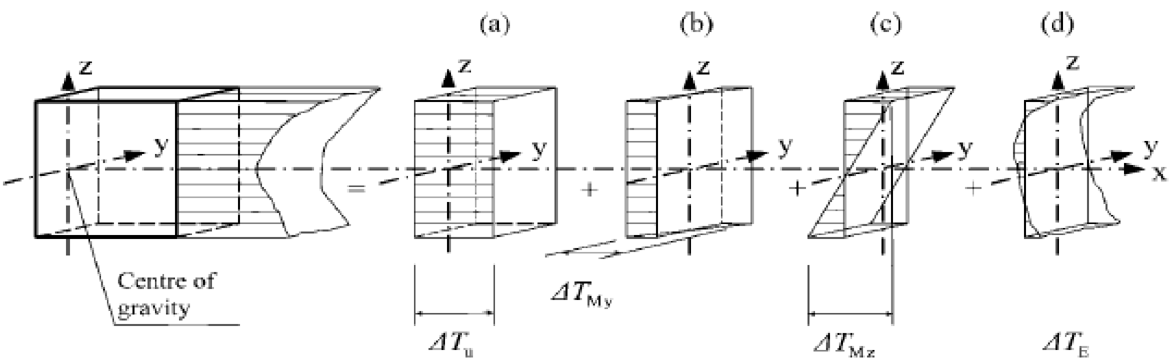

Figure 8.8 – Diagrammatic representation of constituent components of a temperature profile (Extract from [68])

The strains and therefore any resulting stresses, are dependent on the geometry and boundary conditions of the element being considered and on the physical properties of the

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material used. When materials with different coefficients of linear expansion are used compositely the thermal effect should be taken into account.

Representative values of thermal actions should be assessed by the uniform temperature component and the temperature difference components. In particular, the vertical temperature difference component should generally include the non-linear component.

8.2.1.2.4.1 Uniform temperature component

The uniform temperature component depends on the minimum and maximum temperature which a bridge achieves. This results in a range of uniform temperature changes which, in an unrestrained structure would result in a change in element length, though the effects related to restraint of associated expansion or contraction due to the type of construction, to friction at roller or sliding bearings, to non-linear geometric effects ad to the interaction between the track and in the bridge (in case of railway bridges) due to the variation of the temperature of the deck and of the rails, which may induce supplementary horizontal forces on the bearings, should be taken into account if relevant.

8.2.1.2.4.2 Range of uniform bridge temperature component

The values of minimum and maximum uniform bridge temperature components for restraining forces shall be derived from the minimum (T_{min}) and maximum (T_{max}) shade air temperatures.

The correlation between minimum/maximum shade air temperature (T_{min}/T_{max}) and minimum/maximum uniform bridge temperature component ($T_{e,min}$/$T_{e,max}$) is given in figure 6.1 of [68].

The characteristic value of the interval of maximum contraction of the uniform temperature component of the bridge is:

$$\Delta T_{N,con} = T_0 - T_{e.min} \qquad [\ 8\text{-}16\ ]$$

while that of maximum expansion is:

$$\Delta T_{N,exp} = T_{e.max} - T_0 \qquad [\ 8\text{-}17\ ]$$

where T_0 is the initial bridge temperature at the time that the structure is restrained.

8.2.1.2.4.3 Vertical component

According to the [68], the effect of vertical temperature differences should be considered by using an equivalent linear temperature difference component with $\Delta T_{M,heat}$and $\Delta T_{M,cool}$, whose values should be applied between the top and the bottom of the bridge deck.

Specific values for each country should be used. Some examples are given in Table 8.11.

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Type of Deck	Top warmer than bottom	Bottom warmer than top
	$\Delta T_{M,heat}$ (°C)	$\Delta T_{M,cool}$ (°C)
Type 1: Steel deck	18	13
Type 2: Composite deck	15	18
Type 3: Concrete deck: - concrete box girder - concrete beam - concrete slab	 10 15 15	 5 8 8
NOTE 1: The values given in the table represent upper bound values of the linearly varying temperature difference component for representative sample of bridge geometries. NOTE 2: The values given in the table are based on a depth of surfacing of 50 mm for road and railway bridges. For other depths of surfacing these values should be multiplied by the factor k_{sur}. Recommended values for the factor k_{sur} is given in Table 6.2.		

Table 8.11 – Recommended values of linear temperature difference component for different types of bridge decks for road, foot and railway bridges (Extract from [68])

8.2.1.2.4.4 Horizontal component

The temperature difference component generally needs only to be considered in the vertical direction. In particular cases, however (for example when the orientation or configuration of the bridge results in one side being more highly exposed to sunlight than the other side), a horizontal temperature difference component should be considered.

8.2.1.2.4.5 Probabilistic model

Probabilistic models for thermal actions are described in section 2.14 of [69],in which the basic variables that may significantly influence effects of thermal actions on structures are listed:

- Climatic agents
    - Shade air temperature (daily and seasonal changes)
    - Solar radiation (direct and diffused)
    - Wind speed (influenced by regional wind climate and orography)
- Operating process temperatures
    - Inner environment of the structure
- Characteristics of construction works
    - Space orientation of the structure
    - Shape of the structure, dimensions and cross-sectional geometry
    - Joints of the structure, types of materials and used colours
    - Structural system
    - Thermal properties of materials
    - Initial temperature at which the structure is restrained, properties of atmosphere and terrain
- Geographical location of site
- Properties of atmosphere and terrain.
    - Emissivity of the atmosphere and terrain, location near some water source

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According to [69], development of temperatures in a structure may be obtained either theoretically by means of the heat conduction method or experimentally by measuring the temperatures in many points of relevant structure with adequate frequency. A reliable theoretical prediction of the temperature fields in a structure is a quite difficult problem because it needs the integration of the Fourier heat conduction equation on a generally complex shaped domain under appropriate initial conditions, and with non-linear and time-dependent boundary conditions.

Given that the thermal exchange between the structure and its environment is influenced by three basic factors which may appear simultaneously (solar radiation and irradiation, thermal convection, and conduction), for bridges the evaluation of the vertical difference temperature component should take into account the measurements of shade air temperatures and effects of solar radiation.

The statistical characteristics of the model of temperature difference component may be assessed on the basis of the extreme value probabilistic distribution. For further information, see [69].

8.2.1.3 Accidental actions

In [70] and [63] guidance on the determination of accidental actions is provided. This category includes actions due to impact and explosions, though, in case of bridges, explosions might be disregarded.

In practice, the occurrence and consequences of accidental actions can be associated with a certain risk level, which may lead to additional measures, when necessary. It should be noted, however, that zero risk level is impracticable and that in most cases it is necessary to accept a certain level of risk. For bridge structures such emergency measures may involve the closure of the road or rail service within a specific limited period.

Hence, the risk analysis of the accidental actions to be considered depend upon the measures to be taken for preventing or reducing the severity of an accidental action, the probability of occurrence and the consequence of failure of the identified action, the public perception, and the level of acceptable risk.

8.2.1.3.1 Impact

[70] defines accidental actions due to the following events:

- impact from road vehicles (excluding collisions on lightweight structures);
- impact from forklift trucks;
- impact from trains (excluding collisions on lightweight structures);
- impact from ships;
- the hard landing of helicopters on roofs.

According to [63], Impact is an interaction phenomenon between the object and the structure. In the case of bridges, the actions due to impact and the mitigating measures provided should consider, amongst other things, the type of traffic on and under the bridge and the consequences of the impact, therefore actions due to impact from road vehicles, trains and ships must be taken into account.

In general, actions due to impact may be determined either by a dynamic analysis or by an equivalent static force, where the forces at the interface of the impacting object and the structure depend on their interaction.

The parameters to be considered are:

- impact velocity of the impacting object;

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- mass distribution;
- deformation behaviour;
- damping characteristics (of both the impacting object and the structure);
- angle of impact;
- construction and movement of the impacting object after collision.

The basic model for impact loading consists of:

- Potentially colliding objects;
- The occurrence of a human or mechanical failure that may lead to a deviation of the intended course;
- The course of the object after the initial failure;
- The mechanical impact between object and structure.

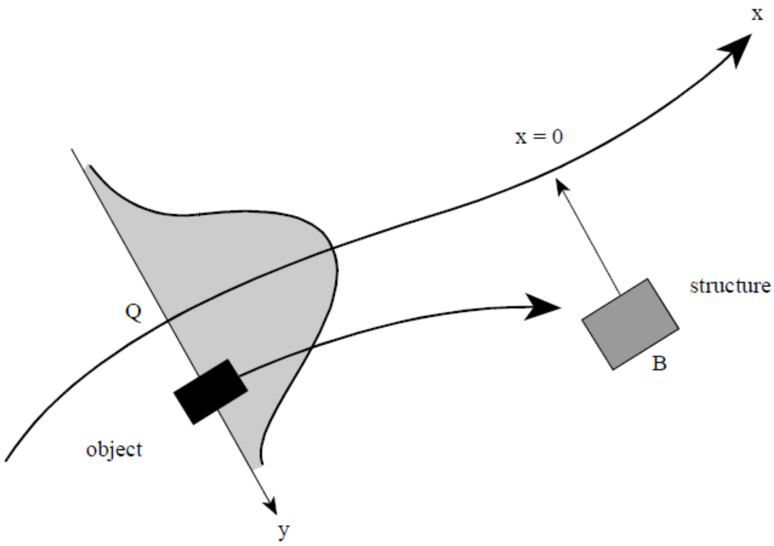

Figure 8.9 – Probabilistic collision model (Extract from [63])

Using the assumption of rigid structure and colliding object modelled as an elastic single degree of freedom system, with equivalent stiffness k and mass m, the maximum possible resulting interaction can be calculated as:

$$F_c = v_c\sqrt{km} \qquad [\ 8\text{-}18\ ]$$

Where v_c is the object velocity at impact and the duration of this load is:

$$\Delta t = m v_c / F_c \qquad [\ 8\text{-}19\ ]$$

8.2.1.3.1.1 Impact from road vehicles

Accidental actions caused by road vehicles include the ones due to impact on supporting substructures and to impact on superstructures.

[70] provides examples for the value of equivalent static force to use.

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Category of traffic	Force F_{dx} a) [kN]	Force F_{dy} a) [kN]
Motorways and country national and main roads	1 000	500
Country roads in rural area	750	375
Roads in urban area	500	250
Courtyards and parking garages with access to: - Cars - Lorries b)	 50 150	 25 75
a) x = direction of normal travel, y = perpendicular to the direction of normal travel. b) The term "lorry" refers to vehicles with maximum gross weight greater than 3,5 tonnes.		

Table 8.12 – Indicative equivalent static design forces due to vehicular impact on members supporting structures over or adjacent to roadways (Extract from [70])

Category of traffic	Equivalent static design force F_{dx} a) [kN]
Motorways and country national and main roads	500
Country roads in rural area	375
Roads in urban area	250
Courtyards and parking garages	75
a) x = direction of normal travel.	

Table 8.13 – Indicative equivalent static design forces due to impact on superstructures (Extract from [70])

For further information on impact loads on kerbs and parapets and impact for traffic on bridges, see [66], while accidental actions caused by road vehicles on bridges also carrying rail traffic are described in UIC leaflet 777.1R.

Concerning the condition of impact from road vehicles, [70] suggests using the following recommendations:

- for impact from lorries the collision force F may be applied at any height h between 0,5 m to 1,5 m above the level of the carriageway or higher where certain types of protective barriers are provided. The recommended application area is a = 0,5 m (height) by 1,50 m (width) or the member width, whichever is the smaller.
- for impact from cars the collision force F may be applied at h = 0,50 m above the level of the carriageway. The recommended application area is a = 0,25 m (height) by 1,50 m (width) or the member width, whichever is the smaller.

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Key

a Is the height of the recommended force application area. Ranges from 0,25 m (c
0,50 m (lorries)

h Is the location of the resulting collision force *F*, i.e. the height above the level
carriageway. Ranges from 0,50 m (cars) to 1,50 m (lorries)

x Is the centre of the lane

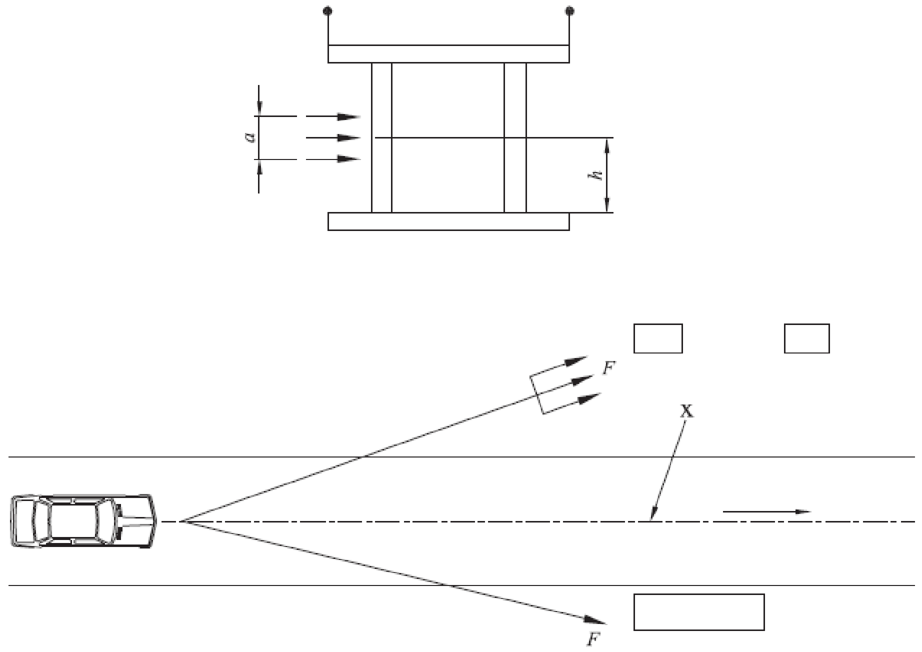

Figure 8.10 – Collision force on supporting substructures near traffic lanes for bridges and supporting structures for buildings (Extract from [70])

For $h_0 \leq h \leq h_1$, these values may be multiplied by a reduction factor r_F, where

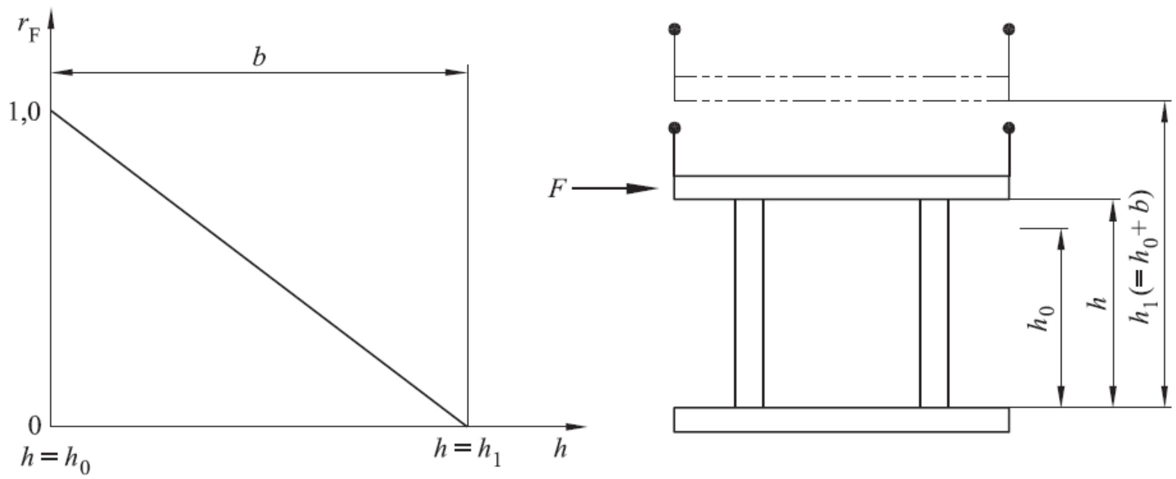

Figure 8.11 – Recommended value of the factor r_F for vehicular collision forces on horizontal structural members above roadways, depending on the clearance height h (Extract from [70])

- h Is the physical clearance between the road surface and the underside of the bridge deck;
- h_0 Is the minimum height of clearance between the road surface and the underside of the bridge deck below which an impact on the superstructure need to be taken into account. The recommended value of h_0 is 5,0 m;
- h_1 Is the value of the clearance between the road surface and the underside of the bridge deck. For values of h_1 and above, the impact force F need not be

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considered. The recommended value of h1 is 6,0 m (+ allowances for future re-surfacing, vertical sag curve and deflection of bridge);

- b is the difference in height between h_1 and h_0, i.e. $b = h_1 - h_0$. The recommended value for b is 1,0 m. A reduction factor for F is allowed for values of b between 0 and 1 m, i.e. between h_0 and h_1.

On the underside surfaces of bridge decks the same impact loads as above with an upward inclination may have to be taken into account: the conditions of impact may be given in the National Annex. The recommended value of upward inclination is 10°, see Figure 8.12.

Key
X Direction of traffic
h Height of the bridge from the road surface measured to either the soffit or the structural members

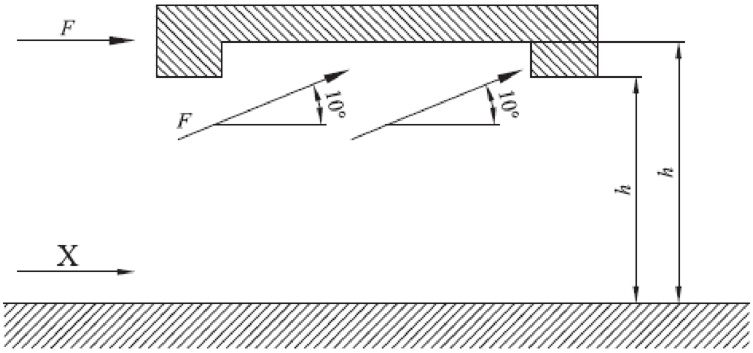

Figure 8.12 – Impact loads on members of the superstructure (Extract from [70])

[70] also considers accidental actions caused by derailed rail traffic under or adjacent to structures, using the following classification of structures.

Class A	Structures that span across or near to the operational railway that are either permanently occupied or serve as a temporary gathering place for people or consist of more than one storey.
Class B	Massive structures that span across or near the operational railway such as bridges carrying vehicular traffic or single storey buildings that are not permanently occupied or do not serve as a temporary gathering place for people.

Table 8.14 – Classes of structures subject to impact from derailed railway traffic (Extract from [70])

For further information, see [70].

8.2.1.3.1.2 Impact from ships

Accidental actions due to collisions from ships should consider:

- the type of waterway,
- the flood conditions,
- the type and draught of vessels and their impact behaviour, and
- the type of the structures and their energy dissipation characteristics.

According to [70], the action due to impact should be represented by two mutually exclusive forces:

- a frontal force (in the direction of the normal travel, usually perpendicular to the longitudinal axis of the superstructure (deck))
- a lateral force with a component F_R parallel to F_{dx} .

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The following equation may be used for the evaluation of the impact force due to friction F_R acting simultaneously with the lateral impact force:

$$F_R = \mu F_{dy} \qquad [\ 8\text{-}20\ ]$$

Where μ is the friction coefficient.
The position and area over which the impact force is applied depend upon the geometry of the structure and the size and geometry (e.g. with or without bulb) of the vessel, the vessel draught and trim, and tidal variations.

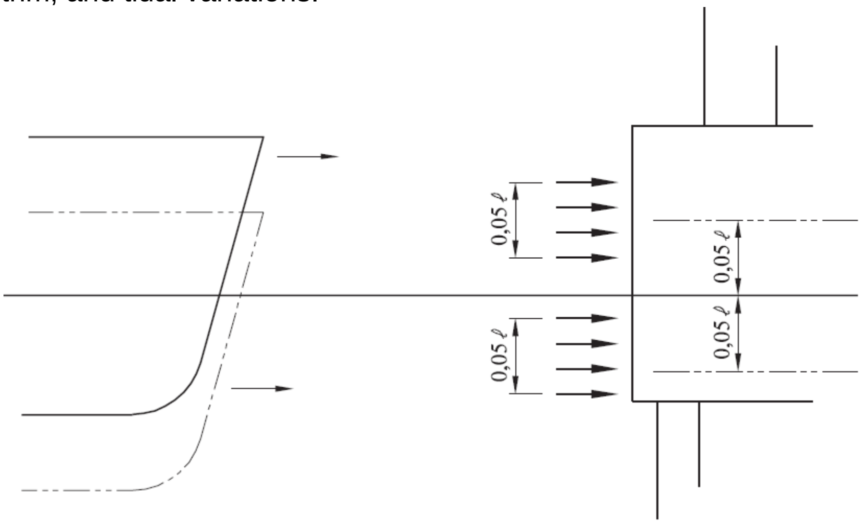

Figure 8.13 – Indicative impact area for ship impact. (Extract from [70])

Moreover, the forces on a superstructure should be determined by taking account of the height of the structure and the type of ship to be expected. In general, the force on the superstructure of the bridge will be limited by the yield strength of the ships' superstructure.

8.2.1.3.2 Explosions

An explosion is defined as a rapid chemical reaction of dust, gas, or vapour in air, which results in high temperatures and high overpressures. Explosion pressures propagate as pressure waves. The pressure generated by an internal explosion depends primarily on the type of dust, gas or vapour, the percentage of dust, gas or vapour in the air and the uniformity of the dust, gas or vapour air mixture, the ignition source, the presence of obstacles in the enclosure, the size, the shape, and the strength of the enclosure in which the explosion occurs, and the amount of venting or pressure release that may be available.

Explosion pressures on structural members should be determined considering reactions transmitted to the structural members by non-structural members. The explosive pressure should be assumed to act effectively simultaneously on all the bounding surfaces of the enclosure in which the explosion occurs.

For further information, see [70].

8.2.1.4 Seismic actions

In [71] guidance on design and construction of buildings and civil engineering works in seismic regions is provided. In cases of low seismicity, reduced or simplified seismic design procedures for certain types or categories of structures may be used.

Seismic actions evaluation depends on the "basic seismic hazard" of the construction site, since they are defined in terms of the maximum horizontal acceleration expected in free field

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conditions on a rigid reference site with a horizontal topographic surface, as well as in terms of the ordinates of the elastic response spectrum in acceleration corresponding to it, with reference to predetermined probabilities of exceedance in the period of reference.

Hence, the first step of the evaluation consists of the identification of the category of soil to which the site belongs.

Ground type	Description of stratigraphic profile	Parameters		
		$v_{s,30}$ (m/s)	N_{SPT} (blows/30cm)	c_u (kPa)
A	Rock or other rock-like geological formation, including at most 5 m of weaker material at the surface.	> 800	_	_
B	Deposits of very dense sand, gravel, or very stiff clay, at least several tens of metres in thickness, characterised by a gradual increase of mechanical properties with depth.	360 – 800	> 50	> 250
C	Deep deposits of dense or medium-dense sand, gravel or stiff clay with thickness from several tens to many hundreds of metres.	180 – 360	15 - 50	70 - 250
D	Deposits of loose-to-medium cohesionless soil (with or without some soft cohesive layers), or of predominantly soft-to-firm cohesive soil.	< 180	< 15	< 70
E	A soil profile consisting of a surface alluvium layer with v_s values of type C or D and thickness varying between about 5 m and 20 m, underlain by stiffer material with v_s > 800 m/s.			
S_1	Deposits consisting, or containing a layer at least 10 m thick, of soft clays/silts with a high plasticity index (PI > 40) and high water content	< 100 (indicative)	_	10 - 20
S_2	Deposits of liquefiable soils, of sensitive clays, or any other soil profile not included in types A – E or S_1			

Figure 8.14 – Ground types (Extract from [71])

The earthquake motion at a given point on the surface is represented by an elastic ground acceleration response spectrum, henceforth called an "elastic response spectrum".
The elastic response spectrum in acceleration is expressed by a spectral form referred to a conventional damping of 5%, multiplied by the value of the maximum horizontal acceleration on a rigid horizontal reference site.
The defined spectrum can be used for structures with fundamental period less than or equal to 4.0 s. For structures with higher fundamental periods, it must be defined by specific analyses, otherwise seismic actions should be described by accelerograms.
The elastic response spectrum of the horizontal component is defined by the following expressions:

$$0 \leq T < T_B \qquad S_e(T) = a_g \cdot S \cdot \left(1 + \frac{T}{T_B} \cdot (\eta \cdot 2.5 - 1)\right)$$

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$T_B \leq T < T_C$ $\quad$ $S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5$

$T_C \leq T < T_D$ $\quad$ $S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5\left(\frac{T_C}{T}\right)$

$T_D \leq T$ $\quad$ $S_e(T) = a_g \cdot S \cdot \eta \cdot 2.5\left(\frac{T_C T_D}{T^2}\right)$

[8-21]

where

- $S_e(T)$ is the elastic response spectrum;
- T is the vibration period of a linear single-degree-of-freedom system;
- a_g is the design ground acceleration on type A ground;
- T_B is the lower limit of the period of the constant spectral acceleration branch;
- T_C is the upper linlit of the period of the constant spectral acceleration branch;
- T_D is the value defining the beginning of the constant displacement response range of the spectruln;
- S is the soil factor;
- η is the damping correction factor with a reference value of $\eta = 1$ for 5 % viscous damping.

8.2.2 Tunnels

There are four main types of actions acting on tunnels:

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

According to [72], the most common loads distribution model is shown in the following figure.

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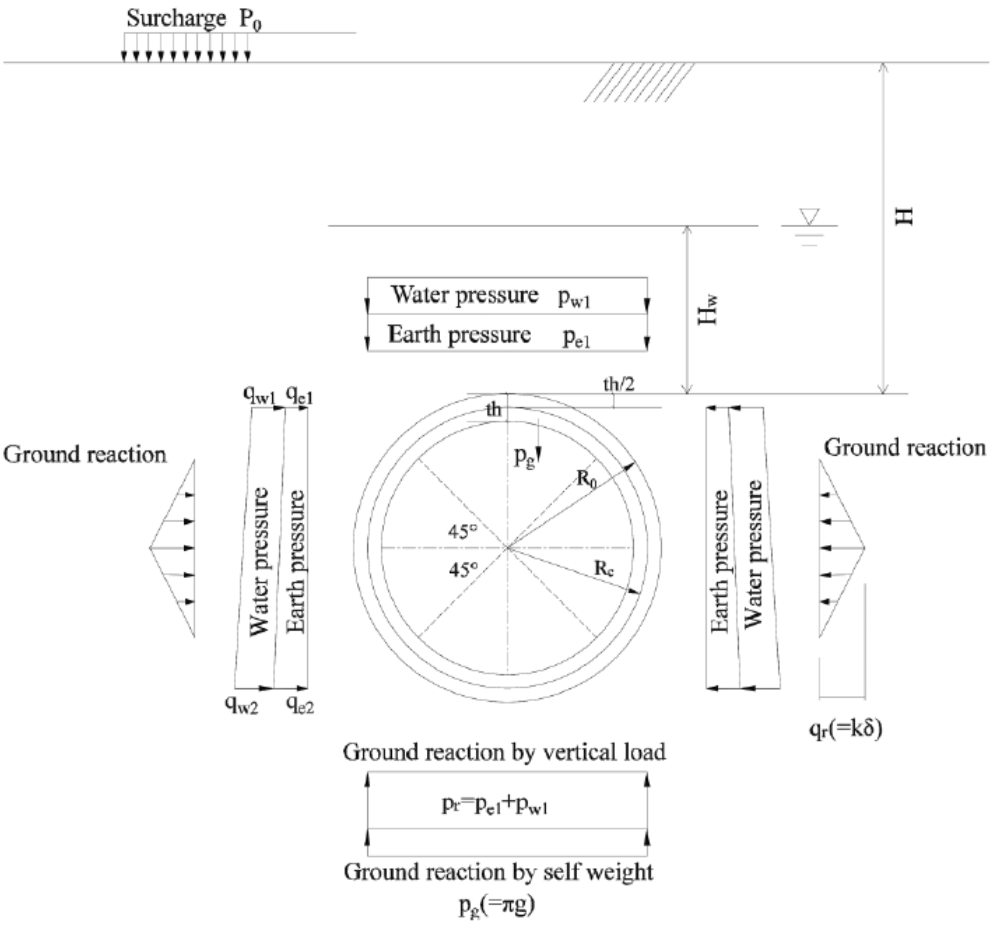

Figure 8.15 – Loads on the segmental ring (Extract from [72])

Where:

- p_{w1} is the vertical water pressure;
- p_{e1} is the vertical earth pressure;
- p_r is the ground reaction due to vertical load;
- p_g is the self-weight;
- q_{w1} is the lateral water pressure at tunnel top;
- q_{w2} is the lateral water pressure at tunnel bottom;
- q_{e1} is the lateral earth pressure at tunnel top;
- q_{e2} is the lateral earth pressure at tunnel bottom;
- q_r is the ground reaction due to lateral deformation;
- H_w is the depth of groundwater level;
- H is the buried depth of tunnel;
- k is the coefficient of ground reaction;
- δ is the lateral deformation;
- P_0 is the surcharge;
- t is the thickness of segment;
- R_o is the outer radius of segmental ring;
- R_c is the calculative radius of segmental ring, i.e. radius of segmental ring used in calculations)

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The description of the actions listed above is given in the following paragraphs.

8.2.2.1 Permanent actions

According to [73], dead load is vertical load acting along the centroid of the cross-section of tunnel and is calculated in accordance with the following equation:

$$p_g = \frac{W}{2\,\pi\,R_c} \qquad [\ 8\text{-}22\ ]$$

Where:

- W is the weight of lining per meter in longitudinal direction;
- R_c is radius of centroid of the linings.

8.2.2.2 Soil actions

The load distribution model consists of applying uniform vertical ground pressures, linearly varying lateral earth pressures, and a triangularly distributed horizontal ground reaction. Their effect can be analysed using elastic equations, beam-spring models, finite element methods (FEM) and discrete element methods (DEM). The elastic equation method is a simple method for calculating lining internal forces for circular tunnels.

The magnitude and distribution of the upward ground reaction due to vertical load P_r are assumed to be the same as the downward earth in the upper part. The lateral earth pressures are assumed, instead, to increase uniformly with depth. The values of lateral earth pressure can be evaluated multiplying the vertical loads by a coefficient of lateral earth pressure. [73] suggested that the value of the coefficient of lateral earth pressure (λ) to be used in the design calculation should be between the value of the coefficient of lateral earth pressure at rest (K_0) and the value of the coefficient of lateral active earth pressure (K_a). It was proposed by JSCE (2006) that the value of K_0 should be regarded as λ when the horizontal ground reaction is difficult to be obtained, and that the value of K_a or a reduction of K_0 should be used as λ when the horizontal ground reaction is available. Hence, the value of λ should be taken as half of the sum of K_0 and K_a:

$$\lambda = \frac{1}{2} \times (K_0 + K_a) \qquad [\ 8\text{-}23\ ]$$

K_a is the coefficient of lateral active earth pressure. K_a can be calculated using equation proposed by Rankine:

$$K_a = tan^2\left(\frac{\pi}{4} - \frac{\emptyset}{2}\right) \qquad [\ 8\text{-}24\ ]$$

To estimate vertical loads, it can be referred to the concept of Terzaghi's solid [74], formulations and [75], used respectively for the soil and rock conditions. In soft ground condition, the vertical ground pressure should be equal to the overburden pressure, if the designed tunnel is a shallow tunnel (H < 2D). If it is a deep tunnel (H ≥ 3D), the reduced earth pressure can be adopted in accordance with Terzaghi's formula considering the wet unit weight for soil above groundwater table and the submerged unit weight for one below groundwater table.

$$\sigma_v = \frac{B_1(\gamma - c/B_1)}{k_0\,tan\,\phi}\left(1 - e^{-K_0 tan\emptyset H/B_1}\right) + p_0 e^{-K_0 tan\emptyset H/B_1} \qquad [\ 8\text{-}25\ ]$$

On the other hand, in rock ground condition, the rock load (P) based on Unal concept is calculated as [75]:

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$$P = \frac{100 - RMR}{100} \gamma D \qquad [\ 8\text{-}26\ ]$$

where RMR is Bieniawski's Rock Mass Rating, and D is the diameter of the tunnel.

The ground reaction q_r correspond to the lateral tunnel deformation multiplied by a coefficient of ground reaction k. The coefficient of ground reaction k is an empirical coefficient and its value ranges widely in different kinds of soil. Its value will be suggested in the geotechnical or design report for a specific project and can be estimated with Galerkin Method:

$$K_n = \frac{E}{R_t(1+\nu)} \quad ; \quad K_t = \frac{1}{3} K_n \qquad [\ 8\text{-}27\ ]$$

where E is rock mass deformation modulus, v is the ground Poisson coefficient, and R_t is the radius of tunnel.

8.2.2.3 Hydraulic actions

The water pressure acting on lining is represented by a hydrostatic pressure. The resultant of water pressure acting on tunnel linings is the buoyancy. If the resultant of the vertical earth pressure at crown and the dead load is greater than the buoyancy the difference between them acts as the vertical earth pressure at bottom (subgrade reaction). If the buoyancy is greater than the resultant of the vertical earth pressure at crown and the dead load, the tunnel would float. According to [73], water pressure load can be estimated as:

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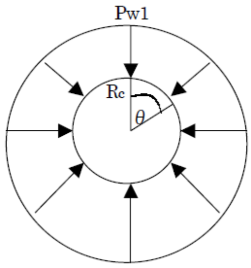

Figure 8.16 – Hydrostatic pressure (Extract from [73])

$$P_{w1} = \gamma_w H_w \text{ [At tunnel crown]} \quad [\ 8\text{-}28\ ]$$

$$P_{w1} = P_{w1} + \gamma_w R_c(1 - cos\theta) \text{ [At the bottom]} \quad [\ 8\text{-}29\ ]$$

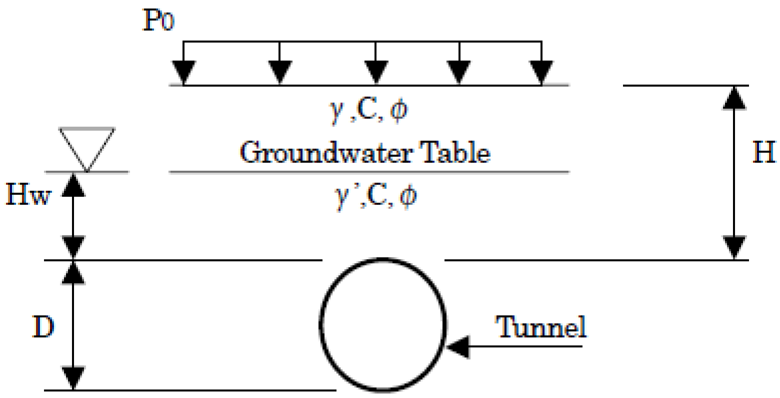

Figure 8.17 – Load scheme (Extract from [73])

8.2.2.4 Landslide actions

In the planning of underground infrastructures, geological, geomorphological, as landslides, and hydrogeological conditions of the area interested by the works should be carefully considered to evaluate, in term of deformation response of the rock mass, the interaction between the excavation and the existing structures at the surface. Landslide location and morphology, active or quiescent in nature, must be evaluated in detail to define the tunnels' alignment. Landslides are particularly sensitive to changes of the stress-strain in the rock mass induced by underground excavation (both in conventional and mechanized system) and to the tunnel size and shape, slope inclination and shape of the sliding surface; the excavation of a tunnel can lead to a rapid evolution of landslides. For this reason, tunnel design must include, especially in a landslide area, an extensive investigations and geotechnical monitoring, as well as the development of numerical models (FEM or FDM analyses) to forecast the evolution of landslides during tunnel excavation in the short and long term (e.g., an investigation campaign with boreholes fairly deepened down the future tunnel invert needs to be planned, in addition to the studies of the area and to the bibliographic research).

The delimitation of landslides bodies in the depth of the slopes allows to define a conceptual "kinematic model" to be used to evaluate slope stability before, during and after tunnel construction. The excavation, if driven inside a landslide or close to a major slipping surface, may increase the natural movement of the landslide or even reactivate quiescent bodies. The presence of a landslide, even inactive, on a slope where a tunnel must be driven, must be carefully considered also for the potential damages to the tunnel itself that may be caused by the induced ground movements. At the ground level these movements usually start when the

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face of the excavation is at a distance between one and two times the tunnel overburden and stop when the face is passed through at the same distance. Different tunnel locations – in plan and elevation - should be analysed, to find the geometric solution compatible with the slope and landslide stability. In this stage, empirical approaches and numerical evaluation in “closed form” are generally used, in order to obtain “qualitative” responses, to be compared in term of cost and time and environmental sustainability.

8.3 Actions evaluation

As per Chapter 8.2 structures are subjected to different kinds of actions. These actions are sometimes specific to their type, such as wind loads on bridges. However, they can also be consistent among several, such as self-weight. This chapter aims to explain the different methodologies that can quantify the action that affect a given structure. This evaluation is directly involved with the design of the asset, as it must consider the possible actions that will influence it in a certain location or across seasons. To better summarize this information, the chapter presents a table that collects the different methods or sensors used to evaluate a given action. These items will then be briefly described following the separation between bridges and tunnels.

Type	Actions	Evaluation
Permanent	Structural	Load cells Weight estimation via density.
Permanent	Non-structural	
Variable	Thermal	Infrared thermography Thermocouples Thermistors
Variable	Snow	Rods Snow pillows/scales Passive gamma radiation
Variable	Wind	Anemometer
Variable	Traffic	Weight in motion Cameras
Accidental	Accidental	Simulation
Seismic	Seismic	Accelerometers

Table 8.15 – Most common sensors for bridge action evaluation

Type	Actions	Evaluation
Permanent	Structural	Load cells Weight estimation via density.
Permanent	Non-structural	
Variable	Soil actions	Strain gauges Pressure cells Weight estimation via soil stratification
Variable	Hydraulic actions	Strain gauges Pressure cells Piezometers
Accidental	Landslide	Slope clinometers

Table 8.16 – Most common sensors for tunnel action evaluation

8.3.1 Bridges

Permanent loads: For the measurement of structural and non-structural loads the most common sensor in structures are the load cells. A load cell is a device that converts a force

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into a measurable output (e.g., voltage). They can be used for directly measuring the permanent load G (i.e., self-weight) of the desired component such as the deck, the pavement, the non-structural elements on the superstructure of the bridge, etc.

There are different types of load cells according to their measurement technique. Most common load cells are the strain-gauge load cells. These sensors calculate the weight of the bodies on them by measuring the intensity of the electrical resistance variation in the strain gauges, which is proportional to the intensity of the applied force on them. Due to their measurement technique, these load cells can be glued to any beam, but it is recommended to placed them onto the ones that are critical in the structure e.g., arches, stringers, etc.

Other load cell types are the hydraulic load cells that measure the weight as a change in pressure in the internal filling fluid; the pneumatic load cells that operate on the force-balance principle or the inductive load cells whose output is proportional to the displacement of a ferromagnetic core. All these load cells must be placed between the zone where the desired permanent load is being applied and a base. In order to have the most possible accuracy, these two surfaces must be parallel, and the load has to be applied vertically on the loading surface.

Besides, there are a relationship between the permanent loads G and mass of the structural elements $m_{element}$ using the gravity constant g.

$$G = m_{element} \cdot g \qquad [\ 8\text{-}30\ ]$$

Therefore, if a measuring of the whole structure is required, weight estimations using the volume of all the elements and the density of their materials must be done. The volume can be calculated by using the dimensions of the structure by looking at historical documents such as the drawings of the structure, or by measured them with instruments like terrestrial laser scanners or callipers. On the other hand, density of the constituent material can be measured directly using some instruments such as solid density meters or by testing an extracted sample from the bridge (measuring both, its volume and mass).

Thermal loads: Thermal loads, as it is introduced in Chapter 8.2.1.2.4, are provoked by temperature differentials in the structure. These differentials can be measured through electric devices such as thermocouples, thermistors or RTD (resistance temperature detectors). Thermocouples are devices based on the Seebeck effect [76], which relates the open circuit voltage ΔV between semiconductors and the temperature differential ΔT between the hot and the cold point of the circuit, using the Seebeck coefficient α_S.

$$\Delta V = \alpha_S \cdot \Delta T \qquad [\ 8\text{-}31\ ]$$

Thermal actions can derive from uniform or non-uniform temperature differentials along the cross-section height. Therefore, for non-uniform temperature differentials, ends of the thermocouple must be placed both on deck intrados and extrados, so that non-uniform variations can be measured.

On the other hand, thermistors and RTD are temperature-sensitive resistors, so the temperature differential can be estimated by the difference in the resistance value.

Nowadays, it was verified that thermographic methodologies are also suitable for assessing the temperature effects in different structures [77]. Furthermore, it is possible to estimate the temperature variations in a structure by observing the effects that this differential provokes in some structural responses such as the dynamic behaviour. Thus, accelerometers can be used for measuring the temperature differentials in the bridge.

Snow loads: Recently, the European Cooperation in Science & Technology and some partners [78] conducts a survey about the different measurement techniques related with snow

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in Europe. In the document, four different measurements were asked about: snow depth, snow presence, depth of snowfall and water equivalent of snow cover. The snow depth and the depth of snowfall quantify the height of the snow in the ground in any time or after a specific period of time (usually 24 hours) respectively, so they can be measured by using rods and rulers. On the other hand, the snow presence identifies the existence of snow in a defined area, which can be checked by cameras or satellite images. Although relevant, these measurements do not provide information related to snow loads. Nevertheless, the water equivalent of snow cover can be expressed as a pressure that can be easily extrapolated to a snow load. In order to measure this data, manual and automatic approaches can be undertaken. In manual measurements, a worker must conduct a snow course. The snow course consists in take from 5 to 10 SWE sample locations spaced 30 meters apart. A SWE sample consists in filling a metallic tube with known dimensions to calculate the density of the snow in it. After weighting the sample, the density of the snow is calculated using the following equation.

$$\rho_{snow} = \frac{m_{sample}}{V_{tube}} \qquad [\ 8\text{-}32\ ]$$

Subsequently, the water equivalent of snow cover is calculated using the equation below.

$$SWE = \sum_{samples} L \cdot \rho_{snow} \qquad [\ 8\text{-}33\ ]$$

Where ρ_{snow}, m_{sample} and V_{tube} are referred to the density of the snow, the mass of the SWE sample and the volume of the metallic tube. And L is the height of the snow of the snow sample.

Concerning the automatic approaches, the water equivalent of snow cover can be directly measured using weighing mechanisms such as snow pillows or scales or passive gamma radiation instruments. The snow pillows are synthetic rubber or stainless-steel bladders filled with an antifreeze fluid that measure the pressure of the snow on them by measuring the hydro-static pressure in the bladder. On the other hand, the snow scales performed the weight measurement using an electronic load cell. Both, snow pillows and scales, must be placed on a ground surface where the snow is going to be accumulated. Finally, the passive gamma radiation instruments are able to calculate the water equivalent of snow cover by measuring the attenuation of the gamma radiation by the water in the snowpack and the soil. However, this last technique is only adequate for measuring low SWE values. These instruments consist of a gamma radiation source separated from a gamma radiation detector. The source must be positioned on the ground surface, while the detector must be positioned above the snow surface pointing towards the gamma radiation source.

Wind loads: In Chapter 8.2.1.2.2 is commented that the wind loads are calculated by considering the direction and the speed of wind. Anemometers are the most common sensors for measuring these variables. There are a lot of different anemometer types, but the windmill and the ultrasonic anemometers are the most used ones in bridges. The first one calculates the wind speed by measuring its angular velocity when its propellers are pushed by the air. On the other hand, ultrasonic anemometers have an ultrasonic emitter and receiver separated one from each other. The emitter sends ultrasonic pulses to the receiver, but these may be disrupted by the air if this is moving quickly. Hence the velocity of the wind is calculated by measuring this disruption. Besides, there are some measuring techniques for measuring the wind pressure that some authors are studying recently. Concerning the ones that could be implemented in civil engineering structures like bridges, in [79] researchers develop a capacitive pressure sensor based on a membrane of dielectric elastomer that correctly identifies the wind pressure when it is placed on a surface.

Traffic loads: In order to quantify the traffic loads a lot of bridges have installed Weight in Motion (WIM) systems. There are several types of WIM systems such as the pavement WIM systems, the Bridge WIM (BWIM) systems and the Dynamic On-Board WIM (OBW) systems.

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All of them are thoroughly described in WP2. Pavement WIM systems are embedded in the roadway or under/on the bridge deck, they can be installed before or on the bridge. On the other hand, the BWIM systems comprehend different sensors attached to the soffit the bridge to measure its stresses and strains so results can be obtained by inverse modelling. Finally, the OBW systems have its sensors installed on the vehicles instead of the infrastructure. All these WIM systems measure the frequency and the weight of the vehicles that cross it. Besides, they are also able to measure the tire impact forces, the strain forces and the velocity of the vehicles. All this information is studied for establishing the load models that correctly describe the traffic loads of a bridge.

Nevertheless, some authors like [80] said that most Weigh in Motion technologies do not work correctly in congested traffic. Hence, they have proposed a methodology to estimate the traffic loads using digital cameras and image analysis. This methodology consists of taking several pictures of a zone of the bridge using the digital camera in order to capture data of the vehicles crossing the structure e.g., dimensions and type of vehicles, speed and volume of traffic, etc. Moreover, the authors have defined a statistical correlation between these data (dimensions and type of vehicles mostly) and weight of vehicles, allowing the calculation of traffic loads.

Accidental loads: An accidental load has a complicated nature. The load will depend on the mass, velocity and direction of the vehicle but also on the mechanical properties of the crashed body and the structural element where it collapses. For instance, the magnitude of the load will be different if the collision is plastic or elastic. Besides, accidental loads have a low frequency of occurrence by definition. Therefore, it is complicated to install some sensors in the structure for quantifying them. In fact, the current standards provide several tables that shows the most appropriate load to consider in each specific collision, as it is showed in the Chapter 8.2.1.3. Hence, a recommendable approach for assessing the damage that an accidental load provokes is by modelling the structure and simulate its response for every accidental action showed in the tables of the standards.

Seismic loads: According to the standards [81], the seismic loads have to be represented by an elastic ground acceleration response spectrum, but they can also be modelled using accelerograms and spatial models. Therefore, the main surveying instrument for quantifying them in bridges are the accelerometers, which have been thoroughly described in WP2. Some authors [82] claim that it is also interesting to capture the dynamic behaviour of the structure while measuring the seismic loads. In addition, they also comment that these measurements can be improved by including some sensors such as strain and displacement gauges, weather stations, etc. for recording all the data related to the structural vibrations under strong motions and the data that can interferes in the measuring of the accelerometers. This information is useful for the structural health monitoring and the study of the combined effects of the soil-structure interactions.

8.3.2 Tunnels

Permanent loads: Similar to bridges, the most common sensor used for quantifying the permanent loads of the structural and non-structural elements in the tunnels are the load cells, particularly the strain-gauge load cells. Besides, if a quantification of the permanent load of the whole structure is required, it can also be estimated by measuring the dimensions of all the structural elements and check the density of their materials by looking at the standards or by measuring it using devices such as solid density meters. The permanent loads can be calculated from the measured or calculated weights by multiplying them by the standard earth gravity.

Soil actions: The soil actions act on the walls of the tunnels pressing them. Therefore, it is possible to quantifying them by measuring the deformations that they provoke in these walls using strain-gauge pressure plates. As it was commented before, the strain gauges calculate the applied forces into them using the proportionality between them and the intensity of the

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electrical resistance variation in the gauges. Some authors [83] have implemented a system consisting in pressure plates and fiber-optic to quantify these loads.

Besides, the soil actions can be also estimated by knowing the density of the soil above the tunnel. However, soils have stratums made of different materials. Therefore, it is necessary to study the soil stratification in order to identify them and, consequently, their depths and densities. These stratums can be identified by looking at the bibliographical data (e.g., historical documents), by studying the soil stratification in the laboratory or by performing in-situ surveys. Concerning the surveys, the cone penetration test is the most used technique to identify and classify the stratums of a ground. It consists of vertically penetrate a cone into the ground at a constant low velocity to measure the resistance of the soil to this penetration. It should be noted that it is necessary to carry out a preliminary study to identify the survey positions. These are defined according to the depth of the ground and the continuity of its stratums. On the other hand, some non-destructive measuring systems can be also used for measuring the soil stratification. The most commonly used is the Ground Penetrating Radar (GPR), which is thoroughly described in WP2.

Hydraulic actions: Similar to the soil actions, these can be measured by looking at the strains that they provoke in the walls of the studied tunnel. Hence, pressure plates explained before can be also implemented for measuring hydraulic actions.

Nevertheless, these actions can also be quantified by measuring the pore water pressures. These pressures can be obtained by piezometers. A piezometer is a sensor capable of measuring these water pore pressures and the groundwater level. It can also be used to measure the pressure of the water in aquifers. Piezometers have to be driven into the ground, so a borehole has to be drilled beforehand. The position of these boreholes must be previously defined in a preliminary study. There are different types of piezometers depending on their measuring principle such as the hydraulic piezometers, pneumatic piezometers or the open standpipe piezometers.

Landslide actions: In order to monitoring the land slope and predict its possible collapses, a measurement of its displacements and accelerations is required. Besides, these displacements can also be useful to determine the mass of displaced debris if a landslide occurs. Therefore, the most common sensors for measuring them are the slope clinometers. The clinometers are sensors that measure the slope gradient and the shear displacements of the ground over time. They are also used for measuring this gradient during activities such as tunnelling, excavation, etc. Inclinometers can be classified attending to the nature of their accelerometers. Most used inclinometers are the servo clinometers and the MEMS clinometers. Last ones have been thoroughly explained in WP2. There are several papers [84] that describe different monitoring of landslides systems that are based on these types of sensors.

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8.4 Use of data for actions modelling [SAC]

Load models in 8.2 describe the temporal, spatial and directional properties of the actions across the structure. This section aims to provide information on how to include available data in the models described above, identifying parameters and variables which may be updated in order to have a reasonably accurate representation of the current loads. In this respect, each section will include the following scheme:

- Surveying technology
- Site-specific data provided by each technology
- Parameters for actions evaluation which can be updated based on the site-specific data.

8.4.1 Bridges

8.4.1.1 Permanent actions

Concerning the evaluation of permanent actions, paragraph 8.3.1 lists the following technologies:

- Load cells
- Terrestrial laser scanner
- Calliper
- Solid density meters
- In-situ/lab tests on samples.

In this section the measured quantity and the associated parameter to be updated (element weight, volume, weight density) related to each technology is given. Load cells allow to evaluate the weight of desired element, whilst terrestrial laser scanners and callipers provide volume measurements. Weight density can be updated based on data coming from solid density meters or in situ/lab tests on samples.

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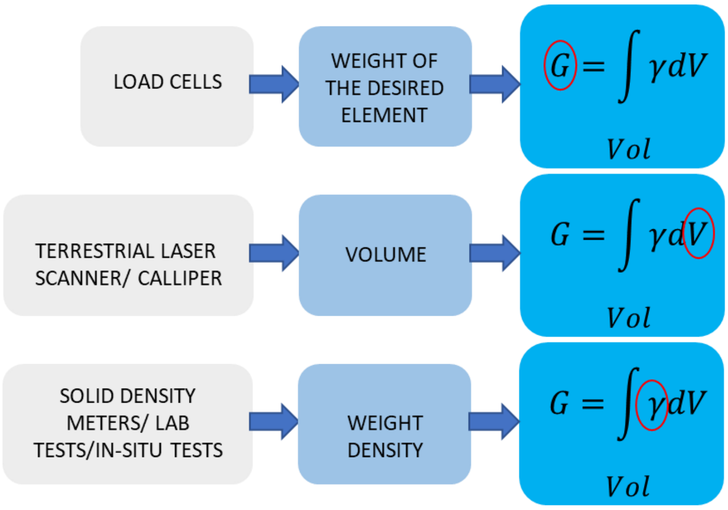

Figure 8.18 – Use of data for permanent actions

8.4.1.2 Variable actions

8.4.1.2.1 Snow actions

Concerning the evaluation of snow actions, paragraph 8.3.1 lists the following technologies:

- Rods
- Rulers
- Cameras
- Satellite images
- Snow pillows/scales
- Passive gamma radiations

In this section the measured quantity and the associated parameter to be updated is given. Cameras and satellite images allow to detect the presence of snow, but cannot provide a direct input for the evaluation of snow loads, whilst rods and rulers can be used for the quantification of snow depth and therefore of the value of the snow load on ground S_g.

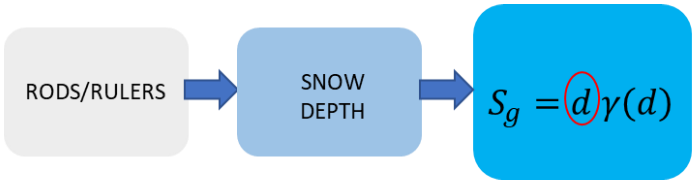

Figure 8.19 – Use of data for snow actions-rods/rulers

Snow pillows/scales and passive gamma radiations are used for the evaluation of the water-equivalent of snow cover, which is directly related to S_g.

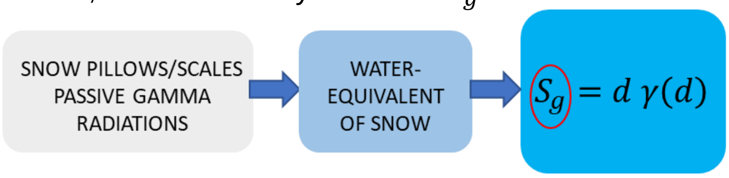

Figure 8.20 – Use of data for snow actions-snow pillows/scales, passive gamma radiations

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As outlined in 8.2, indeed, the characteristics of the ground snow load S_g can be determined based on observations from weather stations, either water-equivalents of snow or depths of snow, given that in the first case the values can be used directly to determine the ground snow load, while in the second case the data on snow depth must be converted to snow load by the relation:

$$S_g = d\,\gamma(d) \qquad [\ 8\text{-}34\ ]$$

8.4.1.2.2 Wind actions

Wind actions evaluation depends on the location and on the availability and quality of meteorological data, the type of terrain, etc. Mean wind velocities may vary over the year: if data are available, it is possible to consider updated values of $v_{b,0}$ in the calculation.
Anemometers allow the measurement of the direction and the speed of wind, based on which wind actions can be evaluated.

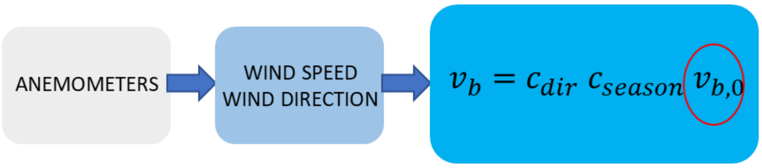

Figure 8.21 – Use of data for wind actions

8.4.1.2.3 Traffic loads

8.4.1.2.3.1 Ultimate loads

Load models included in [66] do not describe actual loads: they have been selected and calibrated so that their effects (with dynamic amplification included where indicated) represent the effects of the actual traffic in the year 2000 in European countries.
Hence, structure-specific traffic load models may be developed in order to have consistent data with real traffic loads, which 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. The general scheme is provided in figure 8.21 and includes:

- **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 is performed to generate artificial WIM data for the time period of interest.
- **TRAFFIC LOAD EFFECT SIMULATION**: load effects/values are calculated.

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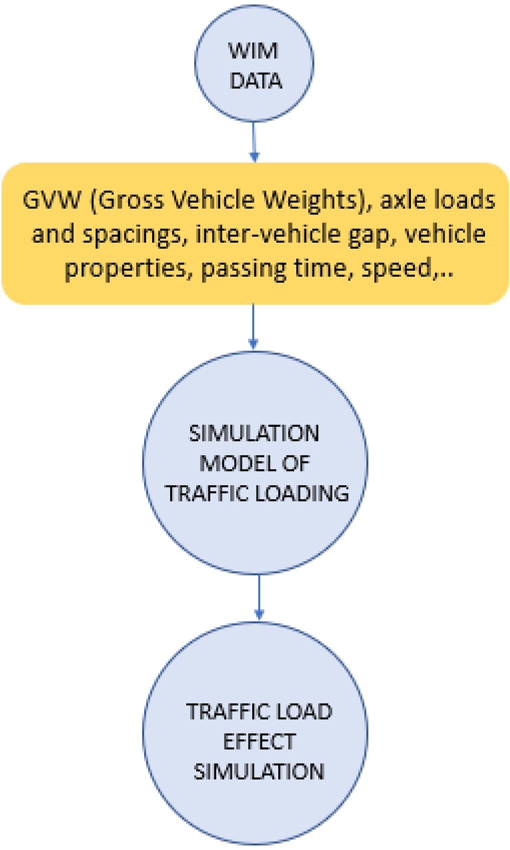

Figure 8.22 – General scheme (Extract from [4])

For further information on how to build site-specific traffic load models, see [4].

8.4.1.2.3.2 Fatigue loads

The fatigue load models (FLMs) described in 8.2 were calibrated using traffic load measurements, in terms of traffic composition, heavy vehicle construction, intervehicle distances, number of traffic jams, legislation and number of heavy vehicles. But, as these measurements have changed a lot since 1986, FLMs should be updated based on actual traffic data. In this respect, traffic data measured with a WIM station may provide a good basis for the calibration of fatigue load models for road bridges. An example of how WIM data can be used for the creation of a WIM database to be directly used for fatigue verifications instead of using random simulations with vehicle weight distributions-based measurements is provided in [67] and summarized here.

- The first step of the procedure consists of the creation a **WIM database**, obtained from a WIM measurement station. WIM databases should allow an accurate representation of the fatigue loads.
- Because of the differences in traffic composition between different countries, **site-specific load models** can be derived, in which the stress ranges $\Delta\sigma$ defined by Eurocodes are used to derive one equivalent range:

$$\Delta\sigma_{FLM}{}^{*} = \begin{cases} \Delta\sigma_l \quad if\ n_l = 1 \\ \left[\Delta\sigma_l{}^5 + \left(1 + a\,\frac{L}{m}\right)\sum_{l=1}^{n_l}\Delta\sigma_l{}^5\right]^{1/5} \quad if\ n_l > 1 \end{cases} \qquad [\ 8\text{-}35\ ]$$

Where:

- n_l is the number of cycles encountered by crossing the vehicle over the influence line;

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- $\Delta\sigma_l$ is the largest stress range encountered by crossing the vehicle over the influence line;
- L is the span (in case of a single span bridge, otherwise it is the average of the adjacent spans)
- a is a factor depending on the density of traffic, whose value must be calibrated with WIM data.

For further information, see [67].

8.4.1.2.4 Thermal actions

The magnitude of the thermal effects depends on local climatic conditions, together with the orientation of the structure, its overall mass, finishes (e.g. cladding in buildings).

Moreover, an additional contribution assessed on the basis of weather station data or data from temperature sensors can be considered, which consists of a linear variation between extrados and intrados of the element under investigation.

Thermal differentials can be measured through electric devices such as thermocouples, thermistors or RTD (resistance temperature detectors). Based on the available data, the minimum and maximum uniform bridge temperature components $T_{e.min}$ and $T_{e.max}$ might be determined.

As outlined in 8.3.1, thermal actions can derive from uniform or non-uniform temperature differentials along the cross-section height. Therefore, for non-uniform temperature differentials, ends of the thermocouple must be placed both on deck intrados and extrados, so that non-uniform variations can be measured.

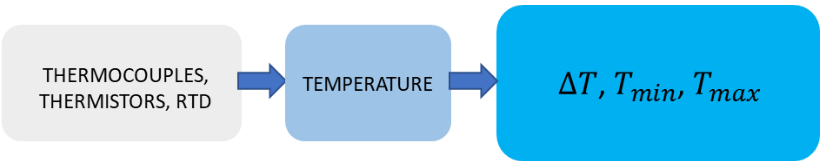

Figure 8.23 – Use of data for thermal actions

8.4.1.3 Accidental actions

The strategies recommended by [70] for accidental design situations are illustrated in Figure 8.24.

In the perspective of prevention, accidental actions should be evaluated considering the most unfavourable scenario within the analysis carried out for the specific structure. This implies that the value of the action cannot in general be modified on the basis of the available data, unless exceptional situations which are not foreseen by regulations are taken into account.

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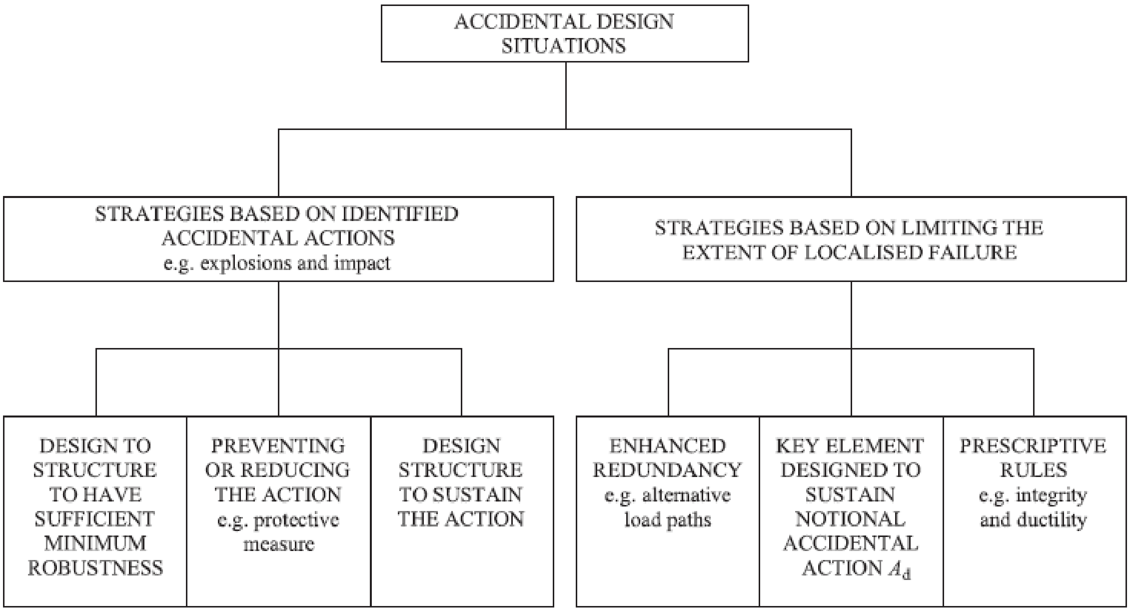

Figure 8.24 – Strategies for accidental design situations (Extract from [70])

As stated in 8.3.1, a recommendable approach for assessing the damage that an accidental load provokes is by modelling the structure and simulate its response for every accidental action showed in the tables of the standards.

8.4.1.4 Seismic actions

National territories are subdivided into seismic zones, depending on the local hazard. Hence, by definition, the hazard within each zone is assumed to be constant. Nevertheless, seismic maps might be updated either based on the occurrence of unusual seismic events or depending on the acceleration histories recorded by seismic stations in the surrounding area of the site under investigation.

Hence, the main surveying instrument for quantifying them in bridges may be the accelerometers, since they allow the evaluation of the dynamic behaviour of the structure while measuring the accelerations provoked by seismic events.

8.4.2 Tunnels

8.4.2.1 Permanent actions

The considerations made in paragraph 8.4.1.1 can also be applied to tunnels.

8.4.2.2 Soil actions

As outlined in 8.3.2, soil actions act on the linings of the tunnels pressing them. Hence, they may be quantified by measuring the deformations that they provoke using strain-gauge pressure plates.

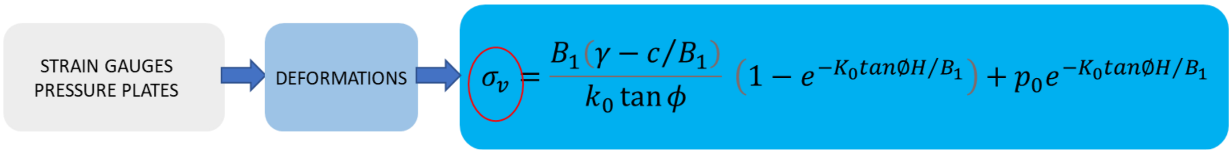

Figure 8.25 – Use of data for soil actions-pressure plates

Moreover, they depend on the stratigraphic characteristics of the soil/rock crossed by the tunnel, whose geotechnical and mechanical parameters can be updated on the basis of the

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results of investigation campaigns. Therefore, it is necessary to study the soil stratification in order to identify them and, consequently, their depths and densities.

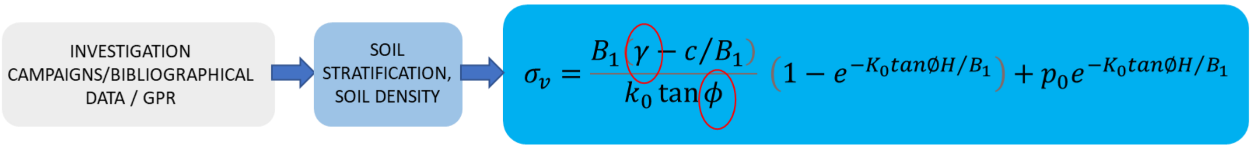

Figure 8.26 – Use of data for soil actions-GPR, investigation campaigns bibliographical data

8.4.2.3 Hydraulic actions

Hydraulic actions can be measured by looking at the strains that they provoke in the linings of the tunnel under investigation.

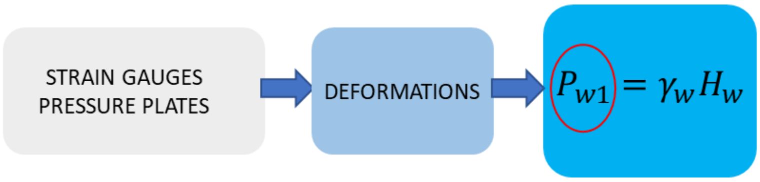

Figure 8.27 – Use of data for hydraulic actions-pressure plates

Nevertheless, water pressure load can be estimated based on H_w, which is the depth of groundwater level. The theoretical value of H_w is provided by the tunnel design documents, but it may be updated based on a piezometric survey campaign in the proximity of the section to be analysed.

Figure 8.28 – Use of data for hydraulic actions-piezometers

8.4.2.4 Landslide actions

To consider the effect derived from the activation of an accidental load, as a landslide, it's necessary to develop a numerical FEM or FDM model of the slope and the tunnel which consider the results of a preliminary and extensive structural and geotechnical investigation. Geotechnical investigation may include campaign with piezometer to measure the heigh of groundwater table at specific point, with slope inclinometers to detect zones of movement and establishing whether a movement is constant, accelerating, or responding to remedial measure.

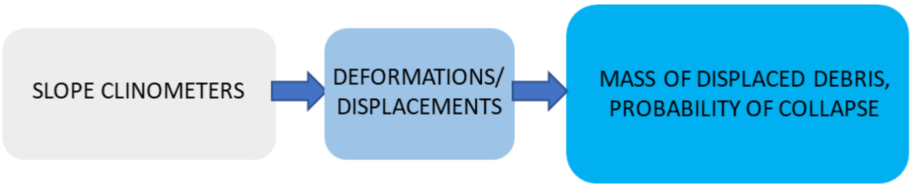

Figure 8.29 – Use of data landslide actions

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9 Model updating based on diagnostic load testing

9.1 General

Structures and structural components are designed in such a way to ensure that the performance level remains above the one required for the structural safety and serviceability during their entire life cycle. To allow effective and efficient life cycle management, though, the following factors should be taken into account:

- deterioration mechanism(s), acting or suspected;
- aging of materials;
- environmental conditions;
- increase/decrease in loads over time;
- change in standards performance requirements.

Hence, the factors which may influence the deterioration and progress of deterioration are related to changes in loads acting on structures and to the properties of the structures themselves. Figure 9.1 shows the probability density function of resistance and load respectively: assuming that variables have a Gaussian distribution resistance, it should be noted that resistance is characterized by a reduced mean value and by an increased standard deviation, because of the higher level of uncertainty, whilst loads increase over time without substantial changes in standard deviation values.

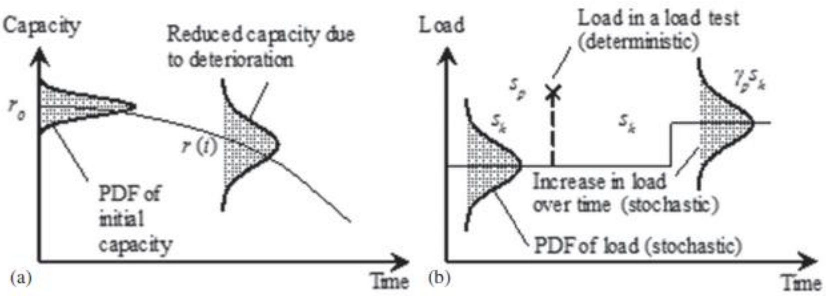

Figure 9.1 – Changes over time of resistance and loads (Extract from [85])

Accordingly, it is necessary to determine the deterioration mechanisms (see chapter 2), present deterioration levels and deterioration rates of materials and/or structural performance using appropriate models based on information obtained during from inspection, testing and monitoring activities (chapter 3 and 4), from the design and construction records, information upon previous interventions and the environmental conditions. Model updating allows to evaluate the condition of structures and, therefore, to make a prognosis of current performance.

In particular, loads acting on structures may vary over time due to several reasons:

- Standards update (to account for the increase of actions);
- change of the intended use;
- change of traffic volume;
- change of permanent structural actions.

Resistance variation, instead, is related to changes in the properties of materials, sections, structural components and structural systems (e.g., aging, delamination, spalling, etc).

The variation in the resistance of structures can be determined from the analysis of design documentation, drawings, results of visual inspections and testing on elements and materials,

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and from information obtained through a load test, either a proof load test or a diagnostic load test. Design documentation and drawings can be used for a first estimate of the resistance, whilst inspections and testing are the basis for the updating over time of parameters related to the structural behaviour. Load test, instead, 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.

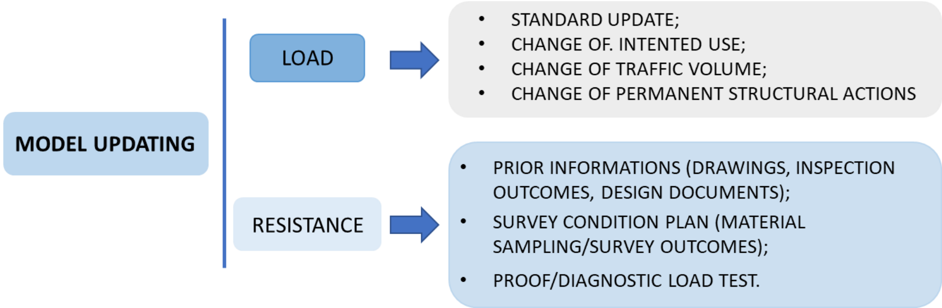

Figure 9.2 – Model updating methods

The difference between proof loading and diagnostic load testing is that proof loading focuses on improving the analytical assessment of an existing structure revealing the potential hidden safety reserve, giving 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, instead, 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.

For further information on proof loading and diagnostic load test, see [4].

In the following paragraphs reference is made only to diagnostic load testing.

Load tests may be performed in order to update the failure probability of a structure and, therefore, the reliability index β, in order to implement a maintenance and interventions prioritization plan.

In particular, the actual β should be compared to the target reliability levels proposed for the assessment of existing structures, based on which actions to be taken may vary:

- 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.

A detailed framework for the data-informed safety assessment is thoroughly described in [10].

9.2 Model updating

Model updating techniques are widely used in the SHM field and 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 is used for design verification and validation, to obtain improved predictions of structural response quantities, or simply to identify unknown system characteristics [86]. It is assumed 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 [87].

Both data acquired from dynamic analysis and static diagnostics load tests are used for updating purposes, as they provide detailed information regarding the global and local

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behaviour of the structure of interest and can be measured in an operational state of the structure.

Starting from an initial numerical model of the structure, diagnostic load tests are conducted in order 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. In the static diagnostic load test, the load is applied following application systems and is placed in each intermediate and final phase long enough for the measurements to stabilize. The structural behaviour is controlled by selected measurement parameters (flexures, deformations, etc.), which must be constantly monitored and evaluated. Test should be considered successful when the structure carries the applied load without signs of distress and therefore fulfils the code requirements.

The dynamic test, instead, is performed to evaluate the response of the structure to the real dynamic load. In this test the dynamic amplification factor is evaluated, that is the coefficient that amplifies the structural response when the load acting from the static to the dynamic mode is changed. For further information on static and dynamic diagnostic load tests, see [4]. 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.

In this section, a framework for the model updating related to diagnostic load testing is described, starting from the identification of the state of the structures up to the experimental validation of the model.

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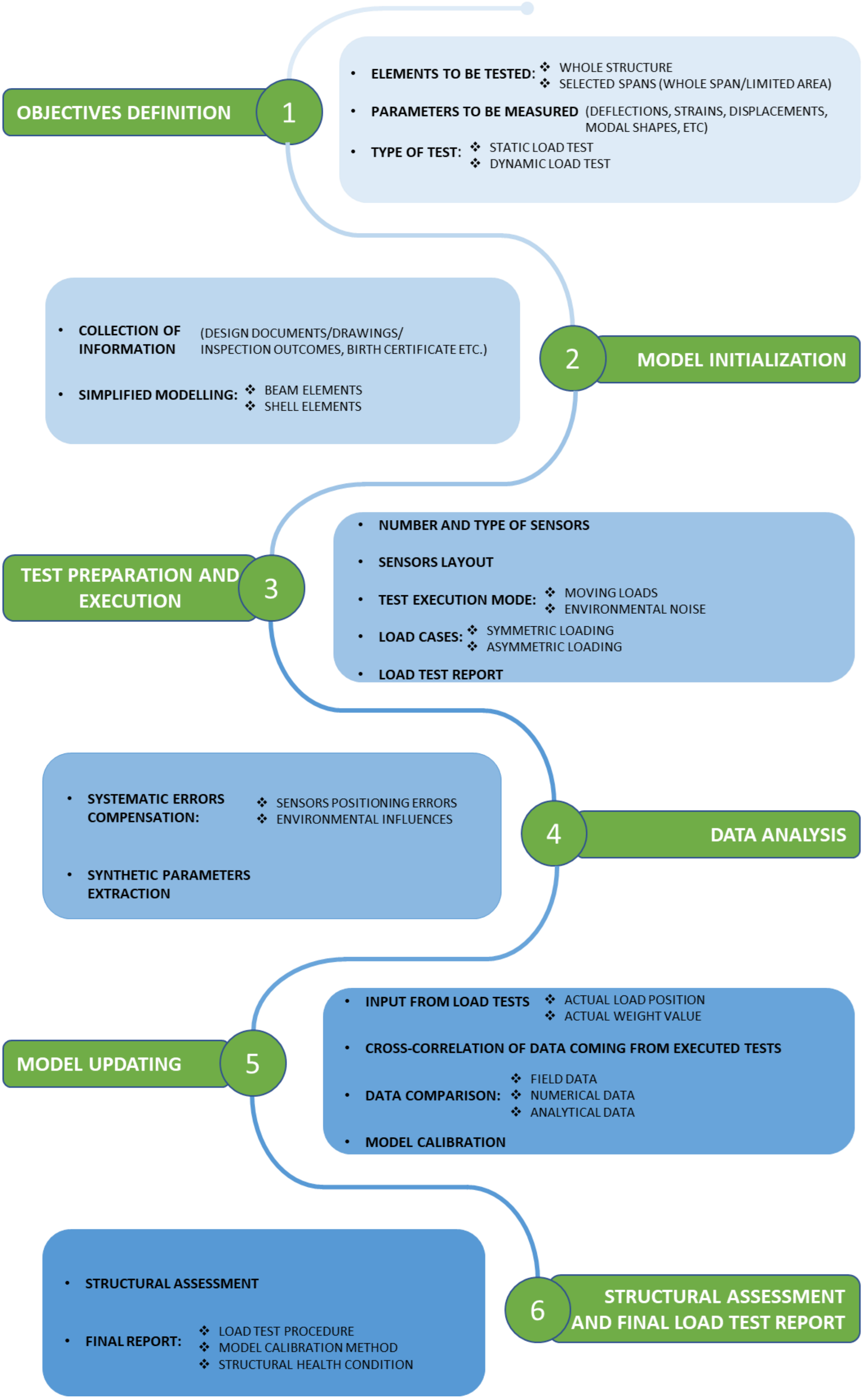

Figure 9.3 – Model updating framework

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The framework consists of the following steps:

1. **OBJECTIVES DEFINITION**: includes the definition of the elements of the structure to be tested (whole structure/ selected spans), the parameters to be measured and the type of test (static/dynamic) to be performed.
2. **MODEL INITIALIZATION**: information, e.g., from design documents, drawings, inspection and intervention reports, is collected and used for a simplified modelling, which can be based either on beam elements or on shell elements.
3. **TEST PREPARATION AND EXECUTION**: based on the objectives identified in the first step, the instrumentation plan is developed. In particular, the following information is required:
   a. *Number and type of sensors*
   b. *Sensors layout*
   c. *Test execution mode* - In the case of static load tests, load weight, load positions have to be defined. In the case of dynamic load tests in which moving loads are applied, the test duration, vehicle speed, load path must be specified too. Otherwise, the environmental noise must be specified.
   d. *Load cases* to be considered (symmetric/ asymmetric).
   e. *Load test report*- summary of the information regarding the load test procedure and results.
4. **DATA ANALYSIS**: systematic errors compensation, due to sensors positioning error and/or to environmental influences, and extraction of synthetic parameters.
5. **MODEL UPDATING**: model is updated with the actual positions and values of the loads. At the model level, a cross-correlation should also be made between data from the tests. During this phase an experienced engineer must compare field data to numerical and analytical data but also identify which parameters should be adjusted to yield an accurate field-validated model (best-fitting), e.g., flexural and torsional stiffness (both in terms of sections and materials).
6. **STRUCTURAL ASSESSMENT AND FINAL LOAD TEST REPORT**: structural assessment, in order to evaluate the reliability index and eventually to assess potential deterioration processes which may be occurring, and summary of the results in a final report, providing information on the load test procedure, the model calibration method and the health status condition.

9.3 Methods of calibration of models: Deterministic and probabilistic models

There are several methods for model calibration. According to [16], it is possible to distinguish between:

- **Deterministic models**: the basic idea is to calibrate a model trying to minimize an objective function (objective function) capable of expressing the residuals (algebraic differences) between numerical data and experimental data. For the process of updating the FE model, modal data are typically used, in particular natural frequencies or eigenvalues. Given a set of measured modal data of the real system, the update problem is that the model realistically approximates the mass and stiffness matrices that will produce modal data as close to real systems as possible.
  Deterministic models can be divided into ***direct*** (one step) or ***parametric*** (multi-step).

  a) **Direct deterministic methods**: non-iterative methods in which the solution is reached by applying the variation of the parameters directly on the matrices that describe the system. Direct methods are the fastest and easiest ones, but sometimes the solution found has no physical meaning. To obtain valid and

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physically compatible results, it is necessary to start from a very detailed finite element model, as close as possible to reality and minimized errors.

b) **Parametric deterministic methods**: iterative methods which modify the values of the parameters until convergence is reached.
Parametric methods were introduced to correct the weaknesses of direct methods. There are three main problems when comparing measured data and the corresponding numerical estimates: the first one is that the natural frequencies and the experimental and theoretical modal forms are physically relevant in the same way; for this purpose it is useful to use MAC (Media Access Control) .
The second problem concerns the scaling of modal shapes. Once calculated eigenvectors are usually normalized to mass. The modal forms do not all intervene with the same scale, therefore each one is independently scaled; it is, then, possible to use a modal scaling factor (MSF):

$$MSF = \frac{\Phi_i^T \Phi_{mi}}{\Phi_{mi}^T \Phi_{mi}} \quad [\ 9\text{-}1\ ]$$

Moreover, in the uncommon case damping is taken into account in the finite element model, the eigen solutions of experimental origin can be obtained in complex form.

- **Probabilistic models**: The parameters to be updated and are not treated neither with direct formulation nor with multi-step techniques. During the model calibration procedure, errors are taken into account. The most widely applied approach to merging models with monitoring data is using a Bayesian approach, which leverages Bayesian statistics to treat parameters as random variables, thus assigning probability distributions to them. The Bayesian update leads to updated probability distributions of the parameters for use in the updated numerical model. When the analysis is complete, a model is obtained in which each of the parameters has a probability density function and an error relative to its evaluation.

9.3.1 Direct deterministic methods

Method of Lagrange multipliers:

The Lagrange multiplier method is a simple tool to minimize an objective function in the presence of well-defined constraints and boundary conditions on the independent variables.
In this method, one set of parameters is considered fixed and immutable, while other sets of parameters are updated separately until an objective function is minimized.
An example is that of Berman and Nagy [88], who minimized the following objective function:

$$J_M = \left\| M_A^{-1/2}(M - M_A)M_A^{-1/2} \right\| + \sum_{i=1}^{m} \sum_{j=1}^{m} \lambda_{ij}(\phi^T M \phi - 1) \quad [\ 9\text{-}2\ ]$$

Where:
- M are the updated masses;
- M_A are the masses extracted from the mass matrix of the model;

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- λ_{ij} are the Lagrange multipliers used to force the orthogonality of the vectors with respect to the updated masses.

The results of the minimization procedure led to the following expression for the updated masses:

$$M = M_A + M_A \phi m_a^{-1}(1 - \phi^T M_A \phi)(\phi^T M_A \phi)\phi^T M_A \qquad [\ 9\text{-}3\ ]$$

$\Phi \in R^{P\times M}$ is an incomplete modal matrix because M<P and P is the order of the updated numerical model. After the calculation of M, the matrix K can also be calibrated by minimizing an additional objective function:

$$J_k = \left\| M_A^{1/2}(K - K_A)M^{1/2} \right\| + \sum_{i=1}^{m}\sum_{j=1}^{m} \lambda_{K_{ij}}(K\phi - M\phi\Lambda)_{ij} + \sum_{i=1}^{m}\sum_{j=1}^{m} \lambda_{O_{ij}}(\phi^T K\phi - \Lambda)_{ij} + \sum_{i=1}^{m}\sum_{j=1}^{m}(K - K^T)_{ij} \qquad [\ 9\text{-}4\ ]$$

Where Λ represents the spectral matrix.
The stiffness updating equation can be expressed as:

$$K = K_A + (\Delta - \Delta^T) \qquad [\ 9\text{-}5\ ]$$

Where Δ is obtained with the following equation:

$$\Delta = \frac{1}{2} M\Phi(\Phi^T K_A \Phi + \Lambda)\Phi^T M - K_A \Phi\Phi^T M \qquad [\ 9\text{-}6\ ]$$

In this case the Lagrange multipliers are used to force the respect of the equations of motion, in conditions of orthogonality, and the symmetry of the stiffness matrix.

"Matrix Mixing" approach:

This method assumes that modal forms are orthonormal and can be used only if modal forms for each degree of freedom are given.

$$\Phi^T M \Phi = 1;$$
$$M = \Phi^{T^{-1}} \Phi^{-1};$$
$$M^{-1} = \Phi\Phi^T = \sum_{i=1}^{m} \Phi_i {\Phi_i}^T \qquad [\ 9\text{-}7\ ]$$

The same approach can be applied to the stiffness matrix:

$$\Phi^T K \Phi = \Lambda;$$
$$\Phi^{T^{-1}}(\Phi K \Phi)\Phi^{-1} = \Phi^{T^{-1}} \Lambda\, \Phi^{-1};$$
$$K = \Phi^{T^{-1}} \Lambda\, \Phi^{-1}; K^{-1} = \Phi \Lambda^{-1} \Phi^T = \sum_{i=1}^{m} \frac{1}{\omega_i^2} \Phi_i {\Phi_i}^T \qquad [\ 9\text{-}8\ ]$$

The matrix-mixing approach uses modes for which some experimental data are missing.

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$$M^{-1} = \sum_{i=1}^{m} \phi_{T_i}{\phi_{T_i}}^T + \sum_{i=m+1}^{p} \phi_{A_i}{\phi_{A_i}}^T;$$
$$K^{-1} = \sum_{i=1}^{m} \frac{\left(\phi_{T_i}{\phi_{T_i}}^T\right)}{\omega_{T\,i}^2} + \sum_{i=m+1}^{p} \frac{\left(\phi_{A_i}{\phi_{A_i}}^T\right)}{\omega_{A\,i}^2}$$

[9-9]

Where subscripts A and T indicate numerical data and experimental data respectively.

If m is the number of measured modal shapes and p is the number of degrees of freedom, the issue with assembling the mass and stiffness matrices from the experimental data is that the number m of components of the eigenvectors of experimental origin is usually way smaller than the order p needed by the model. The eigenvectors extracted from the numerical and experimental analyses ϕ_{A_i} and ϕ_{T_i}, must have dimension p.
Structural matrices assembled with criteria $m < p$ self-solutions are incomplete.
The matrix-mixing method generally provides matrices of complete mass and stiffness, but which have little consideration for physical connectivity.

Eigenstructure Assignment approach

The assumption of this method is that equations of motion are expressed as first or second order ordinary differential equations.
Stiffness and damping matrices are updated, while the mass matrix of the model does not vary.
The equation of motion in terms of displacement is:

$$M\ddot{x} + C\dot{x} + Kx = B_0 u \qquad [\ 9\text{-}10\ ]$$

Where:

- M, C, K are positive definite matrices;
- u is the input of the vector of control forces;
- B_0 is the matrix that correctly distributes the exciting forces on the displacement degrees of freedom.

It is hardly ever possible to measure all displacement variables. The measured vector containing displacement and velocity information is called y:

$$y = D_0 x + D_1 \dot{x} \qquad [\ 9\text{-}11\ ]$$

D_0 and D_1 are the matrices governing displacements and velocities in the state space.
They depend on the excitation positions and on the positions and type of measurements.
In the updating procedure the matrices B_0, D_1, D_0 are initially imposed. One sets $u = Gy$ where G is the gain matrix, on which the calibration depends. The equation of motion is:

$$M\ddot{x} + C\dot{x} + Kx = B_0(D_0 Gx + D_1 G\dot{x});\ M\ddot{x} + (C - B_0 G D_1)\dot{x} + (K - B_0 G D_0)x = 0 \qquad [\ 9\text{-}12\ ]$$

The gain matrix G induces perturbations on the damping and stiffness matrices by updating them in such a way that, once the real components of G are identified, the model is able to

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reproduce the eigen solutions of experimental origin. The updated stiffness and damping matrices are:

$$K = K_A + B_0 G D_0$$

$$C = C_A + B_0 G D_1 \qquad [\ 9\text{-}13\ ]$$

The matrix B_0 can be chosen arbitrarily, C_0 and C_1 are chosen so that the sum $C_1 \Phi \Lambda + C_0 \Phi$ is not singular and is therefore invertible. The matrices Φ and Λ contain incomplete experimental eigenvectors and eigenvalues. Only $m < p$ eigenvectors of experimental origin are needed.

9.3.2 Parametric deterministic methods

Methods based on penalty functions

In general, penalization functions are defined by considering an additional term to the objective function, which penalizes the violation of the constraints. Penalization functions are generally non-linear functions of parameters that need to take into account boundary conditions and constraints. Methods based on penalty functions are based on the use of an iterative procedure that, due to the nature of the cost function, requires the evaluation of the congruence, compatibility and significance of the modal model at each iteration. If the variation of the parameters in successive iterations is small the convergence of the solution has been reached.

Approach based on Sensitivity Analysis

The relationship between the calculated measurable quantities (natural frequencies, modal shapes or displacements) and the parameters of the model for which the correction is sought, is generally non-linear. The method is based on the linearization of this relationship and uses a Taylor series expansion truncated after the linear term:

$$\varepsilon_x = x_m - x_A(\theta) \approx r_i - G_i(\theta - \theta_i);$$

$$r_i = x_m - x_{Ai} \qquad [\ 9\text{-}14\ ]$$

$$G_i = \left[\frac{\partial x_j}{\partial \theta_k}\right]_{\theta=\theta_i}$$

Where:

- x_m is the vector of the experimental results (frequencies and modal forms, etc.);
- x_{Ai} is the vector of the results estimated numerically at iteration i;
- r_i represents the residual between x_m and x_{Ai}.
- G_i is the sensitivity matrix;
- θ_i is the vector of the parameters of the model at iteration i^{th}.

Natural frequencies and modal shapes can be used in the updating procedure. Modal shapes often contain large measurement errors, and individual vector components can be up to 20% uncorrect. Conversely, natural frequencies can be accurately measured close to 1% and can be confidently used. In many practical cases, the number of unknown parameters exceeds the number of measured data. The system is under-determined and there are infinite groups of parameters able of satisfying the equations, in accordance with the Rouché-Capelli

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Theorem. In this case, the choice must take into account the physical significance of the parameters found.

Minimum variance methods

These methods can be considered as based on penalty functions in which the weighting matrices change in a particular way from one iteration to another. This approach uses Bayesian statistics to manage a large amount of data. Both the measured data and the estimates of the initial parameters are affected by errors that can be expressed in terms of variance matrices.

Methods that use frequency domain data

In these methods the Frequency Response Function (FRF) is optimized through a penalty function that directly involves the FRF itself. FRF data can be used directly without extracting natural frequencies and modal shapes. In these methods the damping, in particular the proportional damping, is modeled in the finite element model.

9.3.3 Probabilistic methods

Probabilistic estimators

Through the Maximurn Likelihood Estimation (MLE) it is possible to determine the best estimate of the parameters θ of the model $f(\theta)$ starting from the observed measurements y and the likelihood function $L(y|\theta)$, which provides the probability of observation of the measurements given the parameters. The best parameter estimate provided by the MLE satisfies the following equation:

$$\theta_{mle}(y) = \underset{\theta}{arg\,max} L(y|\theta) \qquad [\ 9\text{-}15\ ]$$

The value of θ_{mle} defined in [9-15] can be determined with the least squares method by minimizing the error function

$$S = [y - f(\theta)] \times [y - f(\theta)]^T \qquad [\ 9\text{-}16\ ]$$

In the determination of parameters θ it is possible to introduce a priori information in the data analysis with the use of an a priori distribution $g(\theta)$. The posterior distribution $p(\theta)$ is therefore

$$p(\theta) \propto /(y|\theta) \times g(\theta) \qquad [\ 9\text{-}17\ ]$$

The type of a priori distribution and its uncertainty are generally based on information provided by documented cases or the literature of the sector [Kass and Wasserman 1994] In the Bayesian inference of the model parameters, the best estimate of the parameters θ is the set of the same θ_{ML} to which the maximum posterior probability (MAP) corresponds:

$$\theta_{ML}(y) = \underset{\theta}{arg\,max} \big(L(y|\theta) \times g(\theta)\big) \qquad [\ 9\text{-}18\ ]$$

For the estimation of parameters in a continuous domain, it is necessary to assess the posterior distribution of the parameters and the related uncertainties The uncertainty can be determined through direct formulas for the propagation of the error or it can be obtained

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through the application of numerical methods such as Markov chain Monte Carlo methods (MCMC).

If the following assumptions are satisfied:

- the state parameters are defined in a continuous domain;
- the value of the parameters θ_k at a certain instant k is a linear function of the value of the parameters θ_{k-1} at the instant k-1, and of the realization of the model uncertainties w_k, that is:

$$\theta_k = F_k \theta_{k-1} + w_k \quad [\ 9\text{-}19\ ]$$

  Where F_k is a matrix representing the temporal relationship of the parameters.
- the measurements y_k observed at instant k can be considered a linear function of the state θk at instant k, and of the realization of the instrumental error v_k, that is:

$$y_k = H_k \theta_k + v_k \quad [\ 9\text{-}20\ ]$$

  where H_k is a matrix representing the relationship between parameters and measures.
- the probabilistic distributions of the realizations w_k and v_k are multinormal with zero averages;

then the estimation of the parameters at the instant k and the corresponding covariance matrix can occur through an application of the Kalman filter.

Monte Carlo methods

Through the application of Monte Carlo methods, numerical results can be obtained on the basis of repeated random sampling. This method is used when it is difficult or impossible to analyze variables in closed form or it is not practical to apply a deterministic algorithm. In the Monte Carlo method the following particular scheme is followed:

- define a domain of possible inputs;
- randomly generating inputs by means of a probability distribution on the domain;
- perform the deterministic calculation on the basis of the inputs;
- aggregate the results using a probabilistic function.

Deterministic evaluation is the direct solution of a system of equations based on each given input. In this way, once input and output data are given, it is possible to understand which are the variables on whose variability the output depends.

For further information on Monte Carlo methods, see [89].

Multi-Model Approach

This methodology is based on the generation of a large number of candidate models: by comparing many direct solutions to well-determined problems, the solution of an inverse problem is found. The most significant advantage is that each of the generated models can certainly respect the physical-mechanical compatibility since its creation, which in classical methods are verified a posteriori.

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Each model is associated with the value of an objective function and is identified by its value. The “right” model is the one associated with the minimum value of the objective function.

The first thing to do is to create the model and then the optimization procedure starts. A global stochastic search and optimization algorithm is used to select a population of candidate models. A global stochastic research algorithm called PSGL (Probabilistic Global Search Lausanne) is used to minimize the cost function. Initially, it is assumed that the probability density function is uniform for each parameter. There is no model generation during this phase. The second step is the sampling cycle. In this cycle, several models are generated and the cost function (objective function) is calculated for each model. At the end of this cycle, only the model associated with the minimum of the cost function, which is called Best Sample (BS), is stored. To edit the PDF for each variable, a probability update cycle is used, which works as follows:

- identification of the range in which the BS is located;
- multiplying the interval by a factor greater than one;
- normalization of PDF.

This probability updating cycle is applied multiple times to find the range containing the Current Best Point. The current best point at the end of this focus cycle is called CBEST. Since both measurements and models contain errors, they are taken into account all models whose objective function is below an adequate threshold. The threshold value therefore depends on the rough estimate of modeling and measurement errors. Applicable models are grouped into classes using known data mining techniques. Each class has a representative "flag" model. The final comparison takes place between the "flag" models.

The objective function to be minimized is:

$$f(p) = \sum_{i=1}^{n} \left[\left[1 - MAC \left(\varphi_{id,i}\ \varphi_{fe,i}(p) + \left\| \frac{\omega_{id,i} - \omega_{fe,i}(p)}{\omega_{id,i}} \right\| \right) \right] \right] \qquad [\ 9\text{-}21\ ]$$

Where:

- ϕ represents the modal forms;
- ω represents the modal angular frequencies;
- the subscripts “id” refers to the quantities identified experimentally;
- the subscripts “fe” refers to the quantities extracted from the finite element model.

9.4 Procedures for identifying KPIs from diagnostic testing

Once model updating has been performed through the procedures described in the previous paragraphs, it is possible to identify the PIs and the related KPIs among the parameters which have been initially selected.
In general, there is no clear distinction between PI and KPI. The KPIs are determined by a number of performance indicators collected at the operational level.
According to [2], Key Performance indicators are:

- Reliability
- Availability
- Safety
- Economy
- Environment

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IM-SAFE proposal, though, is based on the **RAMSSHE€P approach** [90], which includes:

- Safety, Reliability and Security (S, R, S) - a combined KPI
- Availability and Maintainability (A, M) - a combined KPI
- Economy € (i.e. Costs)
- Environment (E)
- Health and Politics (H, P) - a combined KPI

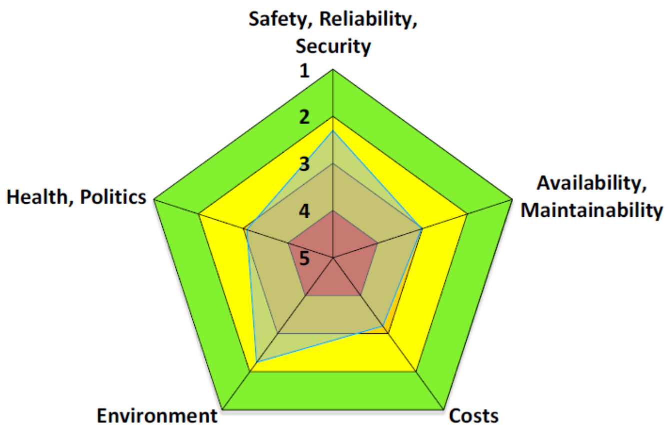

Figure 9.4 – RAMSSHE€P approach (Extract from [90])

Since some of the KPIs above are difficult to be assessed, it should be noted that within the IM-SAFE project only KPIs relating to structural safety are taken into consideration.

Hence, starting from a diagnostic load test, it is possible to obtain information only in relation to the Safety, Reliability and Security combined KPIs: measurements deriving from the performance of diagnostic load testing includes displacements, deformations, frequency, vibrations/oscillations and eventually, the presence of cracks.
The parameters obtained from the measurements carried out during the diagnostic load test are compared with the field parameters. The latter are not necessarily parameters directly measured by the sensors or may be derived quantities. Given that, the parameters can be direct or indirect, depending on whether they already have the required information inside them or whether they need to be reprocessed to obtain the desired measurement. An example of an indirect parameter can be the frequency derived from the acceleration measured by the accelerometers. Measurements listed above can be identified as Performance Indicators: among them it is possible to collect which PIs may be considered also as KPIs.

Among parameters in Figure 9.5 – Performance Indicators (PIs) and related Key Performance Indicators (KPIs) from diagnostic load testing., which lists a series of PIs, it is possible to select KPIs that can be derived from a diagnostic load test. Figure 9.5 shows PIs and the related KPIs.

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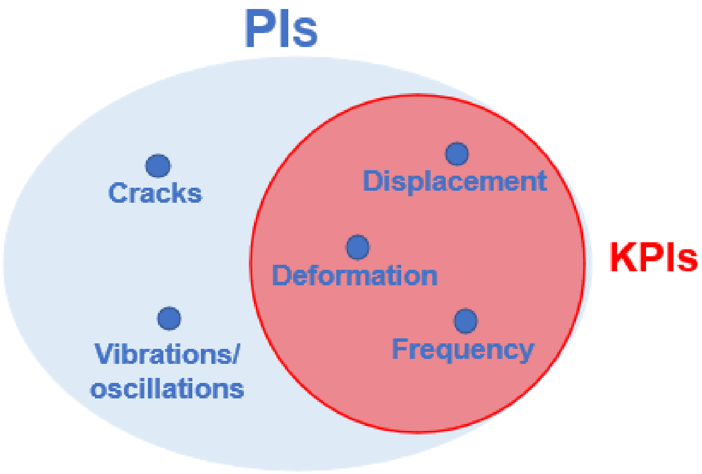

Figure 9.5 – Performance Indicators (PIs) and related Key Performance Indicators (KPIs) from diagnostic load testing.

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10 Conclusions

This report aims to provide technical background for the formulation of the proposal for the mandate to CEN for a further amendment to the existing EU standards on diagnostics of structures based on survey data and for a new standard for monitoring strategies. For this purpose, this document contains an overview of the damage and degradation mechanisms that can affect bridges and tunnels, focusing only on concrete structures (non-reinforced/reinforced/prestressed). For each damage process, Damage Indicators (Dis) and Performance Indicators (PIs) have been identified, as well as the surveying technologies which may be used for its detection.

Based on the information above, a four-step damage detection procedure has been developed: it consists of damage identification and localization, damage quantification, damage modelling and prediction and damage monitoring. This procedure accounts for type, size and location of defects or other relevant issues depending on the type of structure, the actions on structure, and the risks that may potentially affect the structure in the future.

Since damage identification and localization is based, among other things, on inspection outcomes, a classification of inspection methodologies is given, describing each type of inspection with respect to its objectives, frequency of execution, data collection methods and outcomes.

Principles of structural monitoring are also provided, introducing the objectives and requirements of monitoring systems and their classification based on the period of execution and on the purpose. Monitoring systems architecture and design processes are also described, as well as a guidance for monitoring systems installation and management and for data acquisition, processing and treatment.

This document also includes a procedure for damage classification and for the development of a damage classification database.

Since damages that might appear in a given structure are directly linked to the actions that structure is bearing, a description of load models provided by standards on both bridges and tunnels is given. Furthermore, this document provides procedures for determining actions on structures based on condition survey data, which can be used for the development of representative and accurate load models.

Lastly, procedures for assessing KPIs from diagnostic testing are given, including the description of a decision-making process for model updating purposes and a review of the existing model calibration methods.

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12 Appendixes

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REVIEW OF METHODOLOGIES AND INSTRUMENTS FOR DIAGNOSTICS OF TRANSPORT INFRASTRUCTURE APPENDIXES

Annex A - Damage and degradation processes

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Appendix A1 Damage and degradation processes identification

A1.1 Bridges

A1.1.1 Environmental exposure or external factors

A1.1.1.1 Physical processes

A1.1.1.1.1 Abrasion

Abrasion is defined as the process by which relative motion between a surface and hard particles or protuberances on an opposing surface produces abrasive wear of the surface [91]. This process involves the removal of material from a solid object when loaded against hard particles which have equal or greater hardness; these particles can originate externally or from fractures of asperities. Abrasive wear of materials changes the surfaces and dimensions of the components of a structure. All construction materials have abrasion resistance which mainly depends on their constitutive properties.

Properties which mainly define the abrasion resistance of concrete structures are the surface finishing and the curing conditions [92]. Nevertheless, the abrasion resistance of concrete can be affected by environmental conditions (e.g. by the action of solid particles carried by the water or frictional forces due to ice formation), dosage of aggregates, or the use of supplementary cementitious materials. Abrasion can be generated by a number of sources; the most common is the action of airborne or waterborne particles although the collision of vehicles with the soffit and/or superstructure of bridges is also an important source of abrasion.

A1.1.1.1.2 Aggradation (alluviation)

Aggradation results when the bed-material load supplied to a reach of river from upstream exceeds the rivers capacity to transport it. Aggradation at bridges is critical because the deposited material (which can lead to biological growth in different areas) can significantly decrease the waterway opening beneath a bridge, thereby increasing contraction scour, upstream flooding, and bridge overtopping [93]. Long-term aggradation may become a major problem at some bridges, especially in multiple-span bridges, because in this typology aggradation at one or more spans can lead to the potential for increased contraction scour at the remaining spans.

A1.1.1.1.3 Erosion

Erosion is a progressive loss of original material (concrete) from a solid surface due to mechanical interaction between that surface and a fluid, a multicomponent fluid, or impinging liquid or solid particles [91], although wind impact can lead also in erosion.

In bridges, erosion is generated by water inflowing under bridge foundations (scour). Scour occurs in three main forms, namely, general scour, contraction scour and local scour. Firstly, general scour occurs naturally in river channels, may occur because of changes in the hydraulic parameters governing the channel form such as changes in the flow rate or changes in the quantity of sediment in the channel. Secondly, contraction scour occurs because of the reduction in the channel's cross-sectional area that arises due to the construction of structures such as bridge piers and abutments. Finally, local scour occurs around individual bridge piers and abutments. Downward flow is induced at the upstream end of bridge piers, leading to very localized erosion in the direct vicinity of the structure. Erosion in bridges is quantified in terms of rate, that means loss of material (erosion) with exposure duration.

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A1.1.1.1.4 Changing geotechnical properties

Geotechnical properties (e.g cohesion, angle of internal friction, capillarity, permeability) are defined as all specifications of the soil material that are considered to properly characterize the soil under the structure. Geotechnical properties are highly important to investigate the impact of soil-structure interaction and to evaluate the structure stability.

Geotechnical properties are prone to change due many reasons, most common are due seepage, ground water, encountering materials different in classification from those predicted, and replacement of supplementary inappropriate material. However, failures identifying subgrade conditions, aggradation, abrasion, and inaccurate estimation of erosion materials are also important causes of changing geotechnical properties [94].

A1.1.1.1.5 Aging of material

Degradation contributes to the formation of irreversible or much less reversible changes in the structure and physical properties of construction materials. Under natural conditions, most of construction materials undergo aging. Aging of the material is the result of the interaction of different types of degradation with different intensities [95].

There are many types of degradations due aging. Between more relevant are thermal degradation, which occurs under the influence of the increased operating temperature or its rapid changes. Chemical degradation, caused by the effect of chemical compounds. Radiation degradation, which occurs under the impact of high-energy radiation. Biodegradation, caused by the activity of microorganisms and enzymes on the material. Finally, mechano-degradation occurring under the influence of the breakage of macromolecules as a result of exceeding the cohesion forces.

Aging of material starts in the surface of materials, causing fogging or microcracks, their presence has a destructive effect on the material since it allows the penetration of volatile substances in their deeper layers, which facilitates the intensification strength (tensile or bending strength) affecting the structure stability.

A1.1.1.1.6 Fatigue

Fatigue is understood as the process of initiation of cracks through a structural element due to the action of fluctuating stress [96]. Loads that produce fatigue could be a set of actions or parameters based on typical loading events. Fatigue damage often occurs at joints, starting with micro-cracks that finally lead to holes. In the case of steel bridges, fatigue often occurs in the welded type of joints. Micro-cracks may grow larger when bridges are exploited and subjected to iterative loading and unloading. As a result, the cross-sectional area of members may decrease in joints location [97]. In steel bridges, corrosion can significantly reduce the fatigue life of steel wires. This phenomenon leads because the corrosion substantially affects the fatigue crack initiation life of the wire during the service period, thus reducing the steel wire's service life and residual strength [98].

A1.1.1.1.7 Impact due to an accident

Accident occurrences has significant influences on the safety of bridge structures, impact accident includes all damages arising from accidental human activity including impact, collisions from wheeled vehicles, ships, derailed trains, gas explosions.

All impacts may affect bridge in different magnitudes some collision accidents resulted in slight damage to the piers, such as concrete cracking at the impact location. While others caused severe damage to the bridge structures, such as pier fracture and bridge collapse [99]. Effect

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of impact velocity and impact mass is very important, because a larger impact velocity (and greater impact mass) can result in higher impact energy, causing a larger peak impact force.

Piers are the principal components affected under impact due to an accident. Potential failure modes due this damage process are the spalling of concrete cover, the plastic hinge formation, the breakage of the piers and the rebars fracturing. Mentioned failure modes lead in a big increment on maintenance costs. Impact is also influenced by the bridge material. In case of concrete, the peak impact force and the impact duration are independent of concrete strength, because concrete is brittle material, and it is difficult for the brittle material to perform the high shear capacity in the short impact duration [99]. In case of steel higher steel strength can resist the larger impact energy, resulting in a larger impact force.

A1.1.1.1.8 Overloading of an element

When a load (in the load-carrying unit) that exceeds the rated load is applied to an element, it is overloading [100]. Same time, stresses are induced throughout the element, unless both the components geometry and the distribution of the applied loading are extremely uniform, the distribution of stresses throughout the component will be nonuniform [101].This could produce damages to elements that are no prepared to carry the increased loads.

Overloading can be the result of static loading, in the short or long term, including creep effects or as a result of dynamic loading, resulting from impacts or seismic loading. In both instances, once the load is cancelled, the material retains some memory of the previous overload and cannot go back to its initial state [102]. Internal defects could arise creating a weak point for further aggressions.

A1.1.1.1.9 Freeze-thaw

Freeze-thaw affects concrete structures inducing an internal phase change and flow of pore water in concrete. When the porous space of concrete is highly saturated, these processes can generate internal stress high enough to rupture the material during freezing. It can result either in surface scaling and spalling, or in material volume expansion which usually induces a network of cracks [102]. Moreover, with the presence of salts the deterioration process can be greatly reduced [103]. Accordingly, the action intensity of freeze-thaw environment is graded by the basic factors contributing to the concrete damage by freezing, that is, the climate frost intensity, concrete saturation degree and presence of salts.

Concrete resists freeze–thaw damage when free water can move through capillary poresuntil reaching ‘bubbles’ where the ice is able to freely expand, without creating excessive internal stresses. The freeze/thaw resistance of a material is understood as the ability to withstand repeated water exposure and subsequent freeze cycling for 100 freeze/thaw cycles. This ability will be quantified in terms of the % retention of the mechanical property of interest as compared to samples exposed to water for a comparable time without subsequent freeze/thaw cycling.

A1.1.1.1.10 High temperature

A bridge structure is continuously exposed to fluctuating environmental temperature with distinct annual and diurnal trends. Changes of temperature have several effects in structures. The modulus of the material can change with the temperature thus affecting its rigidity [104]. Modal properties change with temperature gradients induced in the structural elements, usually the top of the bridge exposed to direct sun could induce temperature gradients and associated thermal stresses which lead in changes in geometric stiffness.

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The internal thermal stresses could also be introduced by a change in boundary constraints such as frozen bearings or change in end fixity constraints caused by freezing and thawing of the foundations of the piers rigidly connected to the super structure [104].

Overall, hight temperature can induce changes in material stiffness, changes in modal parameters, increased stresses, changes in bearing behaviour, problems on expansion joints, between others.

A1.1.1.2 Chemical processes

A1.1.1.2.1 Alkali aggregate reaction (alkali-silica reaction)

Alkali aggregate reaction (AAR) is an expansive chemical deterioration process that is caused by the chemical reaction between reactive aggregate and the inherently alkaline cement-based composites. Symptoms of AAR include volume increase, the mapping of crack pattern formation, gel seeping from cracks, and pop-outs of aggregates [105]. Apart from the unsightly appearance that results from the presence of ASR, it can also cause significant deterioration in the mechanical properties of especially stiffness (i.e. Youngs modulus) and strength. Crack formation typically presents access to deleterious substances, which subsequently ingress into the matrix, and which may lead to various other deterioration processes, such as to the chloride-induced corrosion of embedded steel reinforcement.

Certain types of siliceous rock and mineral aggregate constituents can react with alkali hydroxides in the concrete pore solution to form a gel product which can expand to cause cracking and other damage to the hardened concrete [103]. These AAR aggregates can therefore be undesirable, but the possibility of reaction occurring and the magnitude of any resultant damage is dependent upon a combination of critical factors.

A1.1.1.2.2 Sulphate reaction

Sulphate attack affect concrete causing either softening, decay of the concrete matrix and expansive cracking. These reactions largely involve the cement paste of concrete, however, in some unusual circumstances, the sulphate action may derive from a constituent of the aggregate, or the aggregate itself might be vulnerable to attack [103]. Sulphate attack can have endogenous origin (developing without any contribution from the environment) or exogenous origins (such as sulphates contained in the soils or in liquids). In both cases, the consequence is some volume expansion owing to the delayed formation of ettringite, which is an expansive component [102].

A1.1.1.2.3 Chemical attack

A chemical attack involves all dissolution of substances or chemical reactions between substances and components that could cause problems of the concrete, due to dissolution or expansion.

The ability of concrete to resist chemical attack is primarily dependent upon the properties of the hydraulic binder, including the cement type, the presence and type of any mineral additions, the water/cement ratio and the degree of compaction. Aggregate properties only rarely directly influence the overall chemical resistance of concrete [103].

A1.1.1.2.4 Corrosion

Corrosion is an electro-chemical process by which the cross-section of steel reinforcement is reduced either reasonably uniformly or locally (that is, through pitting). Corrosion is generated by an interaction between a metal and its environment that results in changes in the properties

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of the metal, and which may lead to significant impairment of the function of the metal [106]. By other hand, corrosion results from the fact that steel tends towards finding their natural form, which is oxidized. The corrosion rate of passivated steel can be less than 1 μm per year.

Corrosion of ordinary structural steel reinforcements commences quite soon following exposure to humid air, and can then proceed rapidly in honeycombed or extensively cracked concrete. However the process is relatively slow in the absence of aggressive species, such as chloride and sulphate ions. Initially there are no visible traces of corrosion on the concrete surface, but with time staining followed by cracking, spalling and delamination become evident [106]. The development of active corrosion in reinforced concrete results from two mechanisms whose common feature is the diffusion of external agents through the pores in the concrete. These mechanisms are the carbonation process and the chloride diffusion process. In reinforcement concrete structures, the steel is normally protected by the alkalinity of the cement pore solution (pH around 13). At lower pH levels, steel attains a high corrosion potential that leads to passivity, with the formation of a thin surface film, about two and three nanometres thick, of iron hydroxides, which provides corrosion resistance [102].

Corrosion of prestressing reinforcement will also commence in a humid environment (as may be present at inadequately filled joints between precast segments) due to carbonation of the concrete, and in the presence of aggressive species - such as chloride ions. Corrosion due to chloride ions can occur in partially grouted or open ducts when contaminated water comes in contact with the tendon. Prestressing steel may also fail through stress corrosion and/or hydrogen embrittlement [106].

Corrosion can also affect metallic components, this type of corrosion can be initiated and promoted in several ways, the main ones are environment corrosion, stray electric currents, stress corrosion cracking, galvanic corrosion, crevice corrosion and bacteriological corrosion. Finally, Aluminium structures can suffer from pitting corrosion, but its effects are rarely serious. However aluminium culverts and underpasses are particularly susceptible to pitting corrosion and so are not permitted in some States [106].

A1.1.1.3 Biological processes

A1.1.1.3.1 Biological growth

Biological growth could be defined as an excessive organism growth that may affect structures inducing damages caused by itself. Biological growth usually starts with primary colonisation by micro-organisms, algae, fungi and various types of bacteria. This is followed by visible growths of algae and lichens. Dirt then collects on the surface and, together with the decaying remains of dead organisms, provides an environment suitable for more advanced biological activity [107]

In advanced cases of biological growth problems associated to concrete structures, dead lichen can provide a footing for mosses and larger plants, which can affect drainage and lead to more serious damage. The roots of plants can grow into cracks and weak spots in the concrete, resulting in bursting stresses that can increase the size of cracks and may lead to spalling [107].

A1.1.2 Damage processes related to design and construction issues

Deterioration related with design and construction errors can lead in many problems of durability in structures.

One of the commonest problems met with concrete structures is inadequate cover to the reinforcement. The evidence of this may not be immediately evident but it will show itself with time through the corrosion of the reinforcement. Problems related to stresses that arise during

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construction (due construction in multiple stages) are also common. Other defects at the construction stage may include:

- Inadequate load capacity due to a design error
- Reinforcement incorrectly placed
- Poor reinforcement detailing and concrete specification
- Inadequate cover to the reinforcement
- Honeycombing due to aggregate grading or poor compaction
- Poor quality control during construction
- Use of reinforcing/structural steel that did not meet design requirements metallic
- Use of metallic fasteners that promotes corrosion of the reinforcement
- The properties of materials are untested or not well understood at the time of construction

A1.2 Tunnels

A1.2.1 Environmental exposure or external factors

A1.2.1.1 Physical processes

A1.2.1.1.1 Continuous vertical rock movement

In general, settlement is defined as a downward vertical movement of a point. Usually, vertical rock movements are mainly due to the following two factors: stress changes and pressure variations. When the tunnel is shallow, movements caused by total stress changes are larger than movements owing to pore pressure variations [18].

By other hand, the dissipation of pore pressure within the rock matrix, caused by drainage through the tunnel walls, and resulting consolidation processes are also responsible of continuous vertical rock movements in tunnels. Significant surface movements can also occur through consolidation processes in crystalline rock masses in which pore pressure is lowered. Indeed, in this case, neither porosity nor compressibility, are directly relevant for this problem [18]. To avoid several damages in tunnels due vertical rock movements, impermeabilization on the fractured sections is commonly used, that helps to reduce the movements on surface strongly.

A1.2.1.1.2 Local rock movement (punching)

Tunnels are usually subjected to punching and settlement loads during their service life due to local rock movements. These load effects need to be carefully studied to avoid tunnels instability problems.

Punching effect due to local rock movements in tunnels derive from a close relationship between the penetration rate and brittleness index of the rock (ratio of uniaxial compressive strength and tensile strength of rock) [19]. There is an inverse correlation between these two parameters, as brittleness index decreases, the penetration rate increases. In harder rocks with lower rock brittleness, the expansion of fragmentation area decreases, and the number and length of main cracks are also reduced outside the fracture.

A1.2.1.1.3 Higher horizontal actions (underestimation of lateral action)

Horizontal actions in tunnels can be generated by the settlement of the foundation; excessive earth pressure; failures of earthworks adjacent to a structure; water pressures produced by

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inadequate or blocked drains; and by changes in the strength or degree of consolidation of the subsoil or backfill [20].

The underestimation of lateral action can lead in transverse horizontal movements, such as tilting, bulging, and sliding of the tunnel ring. This underestimation may also produce bearings and rotational movements which usually results from unsymmetrical settlement or lateral movements [20].

A1.2.1.1.4 Bending stress

Excessive circumferential bending stress can lead in excessive distortions and cracking in tunnel concrete linings [21]. In low resistance bending zones (poor rock zones or low shear zones) there is an important effect in the structural stability of tunnels due to bending stresses; in those cases bending reinforcement is usually required even though the remainder of the tunnel may have received no reinforcement or only shrinkage reinforcement. Failure due to bending stress involves formation of a single lobe parallel to the axis of the tunnel.

Bending stress is especially relevant in tunnels joints; the effect of bending on joints is generally considered by applying a reduction factor to the bending stiffness of the tunnel lining [22]. Based on the bending sign, the detectable damage will be different: the failure due to negative bending moment is just terminated by the crushing of intrados concrete, while failure due to positive bending moment is usually initiated by the yield of tunnel bolts.

A1.2.1.1.5 Debonding

Debonding could be defined as a separation at the interface between the substrate and the near-surface mounted or externally bonded materials [23]. Debonding phenomena are strongly depending on the load transfer mechanisms at the concrete/matrix interface. Debonding failures can take place at the ends of the strengthening surfaces in presence of high stresses at the interface between the strengthening system and the concrete (end debonding) or away from the ends of the bonded strengthening surfaces when they are induced by flexural or flexural-shear cracks (intermediate crack induced debonding).

A1.2.1.1.6 Partial spalling of concrete cover

Spalling is defined as the breaking off fragments or solid particles from a surface, in consequence, spalling phenomena can leads to a reduction of the concrete section in tunnels.

Cracks seen on tunnel lining surfaces have a range in widths, from small to large, and it is assumed that closed cracks with greater widths have a greater risk of spalling. The structural effect of the spalling are two folds: a reduction of the concrete section leads to a decrease of the bearing capacity of the lining but also a stiffness decreases leading to a reduction of the actions due to thermal expansion [24].

In tunnels with plain concrete linings, for which there is no adhesion of the concrete to reinforcing bars, any closed cracks that run across the width of the lining can cause spalling. In practice, the risk of closed cracks causing concrete to spall off is reduced by the roughness of the mating faces of the cracks causing frictional resistance [25].

A1.2.1.1.7 Overloading (rock movement) of prestressing

Excessive deflections can be generated by higher than anticipated live loads and through a reduction in the load carrying capacity of the superstructure or substructures. Excessive deflections of prestressed superstructures can result from the use of prestressing reinforcement with a higher relaxation than assumed in design. In case of tunnels, anchor

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systems are prone to be overloaded due to local movements or global mountains deformations, leading in excessive strengths and anchor elongations [26].

A1.2.1.1.8 Deformation of the ground

Some structures have a highest potential of ground deformations (e.g. due to groundwater changes, mining or liquefaction); their effects on the structures and on the performance requirements for the road link shall be taken into consideration in the development and design of appropriate mitigation measures [27].

There are many factors which influence the deformation of the ground in structures, some of them are horizontal, vertical forces and displacements induced on or within the structure, settlements, seismic actions, between others. Main causes of ground rupture, instability and soil deformation can result from earthquake shaking, earthquake induced liquefaction or cyclic softening, lateral spreading with or without associated liquefaction or cyclic softening, fault rupture associated with earthquakes and subsidence from other causes, such as groundwater changes, mining, etc [27].

A1.2.1.1.9 Water impact

Water inflow into the tunnels is highly dependent on the hydraulic conductivity of the rock mass in case of grouted and un-grouted tunnel zones. Furthermore, factors like joint roughness, percentage of contact areas and infillings can also control the seepage inflow into the tunnels.

Water inflow and water pressure controls have a big impact in the structural behaviour of tunnels. These factors need to be carefully designed. Uncontrolled water behaviour causes mechanical instability, hostile to construction progress and causes adverse environmental impacts in and around the surrounding area [28]. Environmental impact is controlled by factors including the depth of groundwater, groundwater mineralized degree, suspended moisture content and salt content etc [29].

A1.2.2 Damage processes related to design and construction issues

A1.2.2.1 Missing reinforcement

During the tunnel design, as long as the lining is seen as a structure loaded by the rock mass, moments and tensile stresses are projected to the lining and must be covered by reinforcement. Missing design reinforcement can lead in weak tunnels with low capacity resisting shear force and bending moment [30]. When an overloading as a strong earthquake happens, a large shear force and bending moment will be induced in the lining due to dynamic squeezing of the surrounding ground. Thus, the lining will be easily damaged, and can even collapse.

A1.2.2.2 Deformation due to shrinkage, temperature within the shell-blocks

Deformations due to shrinkage and temperature induce early-age cracking in concrete tunnels. Early age cracking occurs when thermal and shrinkage induced stresses in concrete exceed the tensile strength of concrete. The induced stress caused by restraint to early-age deformation in response to temperature reduction, moisture loss, and chemical reaction correspond to thermal, drying, and autogenous shrinkages. Although plastic and carbonatation shrinkage also can lead in additional stresses around the time [31].

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Thermal and autogenous shrinkages dominate early age cracking, while drying shrinkage may occur when the concrete surface is exposed to environmental conditions (it usually occurs within 3–5 days even before the removal of formwork) [31]. Thus, more attention should be given to avoid excessive heat of hydration, reducing or compensating shrinkage and decreasing restraint to control early age cracking.

A1.2.2.3 Different casting times

The cyclic behaviour and effects of fatigue on interfaces between concrete cast at different times, subjected to shear stress is particularly relevant in structures subjected to important cyclic loads. In general, concrete-to-concrete interfaces are classified into three categories: (1) monolithically cast uncracked interface, (2) monolithically cast pre-cracked interface, and (3) interface between concretes cast at different times (that is, cold joint). The transfer of shear force across cold joints is a key factor to consider in concrete structures with concrete which was casting at different times [32].

A1.2.2.4 Different concrete qualities

The construction quality of concrete is possibly degraded by the distinct layer casting in real structures due to the time variable rheology of cement paste [33]. Many factors have influence in concrete quality of structures, so that is almost impossible to have an entire structure with same quality of concrete. More influenced factors in concrete quality are rheological properties of aggregates, cement type, additive type, water quality, binder ratio (w/b), between others.

When different layers of concrete are casted, a weak interface is induced (the interface bond strength decreases with the longer delay for casting the second layer). Main problems with weaknesses due different concrete qualities are related to shear and adherence strengths.

A1.2.2.5 Delamination of concrete layers (e.g. spreaded concrete)

Delamination is defined as a zone of weakness or separation along a plane parallel to the concrete surface caused by a either material, processing and/or environmental factors. The result of air voids or bleed water trapped under a dense polished concrete surface is the creation of zones of weakness (defects can be caused by several factors and mechanisms including premature finishing, excess air, delayed finishing, surface crusting, unusual bleeding and long-term factors) [20].

A1.2.2.6 Anchor failure

Most common damage mechanisms of the anchor systems are failures of the steel tendon caused by corrosion or failures of anchor body caused by weak condition of performed injected element [34]. Corrosion can occur when significant variations exist in the ground along the ground anchor length, particularly with variations in pH and resistivity. The potential for excessive loss of metal by corrosion in soil is high in the several environments, more relevant environment is soils near the groundwater table and soil exhibiting low pH, although in soils with high concentrations of aggressive ions such as chlorides and sites where stray currents are present the potential for excessive loss of metal by corrosion in soil is also important [35].

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Appendix A2 Damage indicators

A2.1 Bridges

A2.1.1 Environmental exposure or external factors

A2.1.1.1 Physical processes

A2.1.1.1.1 Abrasion

Abrasion resistance, Surface hardness, Compressive Strength.

A2.1.1.1.2 Aggradation (alluviation)

Concrete spalling.

A2.1.1.1.3 Erosion

Concrete spalling.

A2.1.1.1.4 Changing geotechnical properties

Displacement rate, Vertical Stresses.

A2.1.1.1.5 Aging of material

Wave propagation velocity, Path of deformation.

A2.1.1.1.6 Fatigue

Compressive strength, Displacement rate, Vertical Stresses.

A2.1.1.1.7 Impact due to an accident

Induced stresses due an impact, Induced displacements, Vibrations.

A2.1.1.1.8 Overloading of an element

Average crack density, Longitudinal crack density, Transverse crack density, Crack opening.

A2.1.1.1.9 Freeze-thaw

Relative dynamic modulus of elasticity, Fundamental transverse frequency, Water Saturation (with/without de-icing agent).

A2.1.1.1.10 High temperature

Modulus of elasticity of concrete, Compressive resistance, Tensile strength.

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A2.1.1.2 Chemical processes

A2.1.1.2.1 Alkali aggregate reaction (alkali-silica reaction)

Phase composition, Gel presence, Content of alkali, Moisture, Presence of deformed areas, Internal expansion, Mechanical properties of concrete.

A2.1.1.2.2 Sulphate reaction

Internal heat distribution, Chloride ions content.

A2.1.1.2.3 Chemical attack

Waveform amplitude, Internal heat distribution, pH over cross section, Chloride ions content, Dielectric properties, Dielectric constant changes.

A2.1.1.2.4 Corrosion

Chloride ions content, Dielectric properties, Dielectric constant changes, Presence of defects.

A2.1.1.3 Biological processes

A2.1.1.3.1 Biological growth

Displacement ratios, Vertical stresses, Bulging.

A2.1.2 Damage processes related to design and construction issues

Displacement ratios, Deformation, Vertical stress, Horizontal stress, Chloride ions content, Dielectric properties, Dielectric constant changes, Presence of defects.

A2.2 **Tunnels**

A2.2.1 Environmental exposure or external factors

A2.2.1.1 Physical processes

A2.2.1.1.1 Continuous vertical rock movement

Induced principal stress.

A2.2.1.1.2 Local rock movement (punching)

Vertical fault dislocation. Vertical stress. Horizontal stress. Deformation rate.

A2.2.1.1.3 Higher horizontal actions (underestimation of lateral action)

Horizontal stress, Vertical stress, Displacement ratios.

A2.2.1.1.4 Bending stress

Stress distribution profiles.

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A2.2.1.1.5 Debonding

Strength measurement.

A2.2.1.1.6 Partial spalling of concrete cover

Existence of round or oval depressions along surfaces, Ultrasonic pulse velocity.

A2.2.1.1.7 Overloading (rock movement) of prestressing

Elongation at maximum load, Modulus of elasticity, Shear modulus.

A2.2.1.1.8 Deformation of the ground

Induced deformations, Induced stresses.

A2.2.1.1.9 Water impact

Bulging, Mechanical instability, Induced stresses, Changes of concrete pH.

A2.2.2 Damage processes related to design and construction issues

A2.2.2.1 Missing reinforcement

Displacement ratios, Strength measurement.

A2.2.2.2 Deformation due to shrinkage, temperature within the shell-blocks

Induced stresses, Shrinkage rate.

A2.2.2.3 Different casting times

Strength measurement.

A2.2.2.4 Different concrete qualities

Chemical composition, Strength, Compression rate, Hardness.

A2.2.2.5 Delamination of concrete layers (e.g. spreaded concrete)

Delayed finishing, Surface crushing, Superficial crushing.

A2.2.2.6 Anchor failure

Anchor weaknesses, Chloride ions content, Presence of defects.

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Appendix A3 Surveying technologies

A3.1 Bridges

A3.1.1 Environmental exposure or external factors

A3.1.1.1 Physical processes

A3.1.1.1.1 Abrasion

GPR, LiDAR, Satellite, Schmidt Hammer test, Windsor Probe test, Abrasion resistance test, Boroscopy, Endoscopy, Guided Waves Propagation, Acoustic Emission, Fibre Optic Sensors. Compressive Strength Test.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Satellite				
Schmidt hammer test				
Windsor Probe test				
Abrasion resistance test				
Guided Waves Propagation				
Acoustic Emission				
Fibre Optic Sensors				
Boroscopy, Endoscopy				
Compressive Strength Test				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

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A3.1.1.1.2 Aggradation (alluviation)

GPR, LiDAR, Satellite, Clinometers, Endoscopy, Boroscopy, Acoustic Emission, Fibre Optic Sensors.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Satellite				
Clinometers				
Endoscopy, Boroscopy				
Acoustic Emission				
Fibre Optic Sensors				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.3 Erosion

GPR, LiDAR, Satellite, Rpas-UAV.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Satellite				
Rpas-UAV				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

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A3.1.1.1.4 Changing geotechnical properties

GPR, LiDAR, Satellite, Rpas-UAV, Acoustic Emission, Boroscopy, Endoscopy, Guided Waves Propagation, Fibre Optic sensors, Clinometers, Accelerometers.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
FOS				
Guided Waves Propagation				
Acoustic Emission				
Clinometers				
Accelerometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.5 Aging of material

GPR, LiDAR, Satellite, Rpas-UAV, Boroscopy, Endoscopy, Fibre Optic Sensors, Computed Tomography, Infrared Thermography, Infrared Spectroscopy, Half-Cell potential method, Neutron Radiography, Clinometers, Accelerometers, Mechanical tests on cored samples.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
Fibre Optic Sensors				
Computed Tomography				
Infrared Thermography				
Infrared Spectroscopy				

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Half-cell Potential method				
Neutron Radiography				
Clinometers				
Accelerometers				
Mechanical tests on cored samples				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.6 Fatigue

GPR, LiDAR, Satellite, Rpas-UAV, Boroscopy, Endoscopy, Fibre Optic Sensors, Pull-out test, Fatigue test, Computed Tomography, Infrared Thermography, Mechanical tests on cored samples, Clinometers, Accelerometers.

Surveying Techniques	**Detection effectiveness**	**Accuracy**	**Availability**	**Versatility**
GPR				
LiDAR				
Rpas - UAV				
Satellite				
Boroscopy, Endoscopy				
FOS				
Pull-out test				
Fatigue test				
Computed Tomography				
Infrared Thermography				
Mechanical tests on cored samples				
Clinometers				
Accelerometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.7 Impact due an accident

WIM Systems, LiDAR.

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Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
WIM Systems				
LiDAR				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.8 Overloading of an element

LiDAR, Satellite, Weight-in-Motion, Fibre Optic Sensors, Accelerometers, Clinometers.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
Weight-in-Motion				
LiDAR				
Satellite				
FOS				
Accelerometers				
Clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.1.9 Freeze-thaw

GPR, LiDAR, Satellite, Spectral Analysis of Surface Waves, Water penetration test, Boroscopy, Endoscopy.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Spectral Analysis of Surface Waves				
Satellite				
Water penetration test				
Boroscopy, Endoscopy				
Ion Chromatography				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

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A3.1.1.1.10 Hight temperature

GPR, LiDAR, Rpas-UAV, Satellite, Infrared Thermography, Spectral Analysis of Surface Waves, Mechanical tests on cored samples.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Spectral Surface Waves Analysis				
Mechanical tests on cored samples				
Infrared Thermography				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.2 Chemical processes

A3.1.1.2.1 Alkali aggregate reaction

GPR, LiDAR, Potentiometric Titration, Water Penetration test, Mechanical tests on cored samples, Argentometric Titration, Ion Chromatography, pH indicators, Schmidt Hammer test, Windsor Probe test.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Potentiometric Titration				
Satellite				
Boroscopy, Endoscopy				
Chloride Diffusion test/ Ion migration test				
Mechanical tests on cored samples				
Ion Chromatography				
pH Indicators				
Schmidt hammer test				
Windsor Probe test				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

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A3.1.1.2.2 Sulphate reaction

GPR, LiDAR, Satellite, Half-cell Potential method, Ion Chromatography, pH Indicators, Boroscopy, Endoscopy, Infrared Spectroscopy, NMR.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Satellite				
Boroscopy, Endoscopy				
Argentometric Titration				
Potentiometric Titration				
Ion Chromatography				
pH Indicators				
Half-cell potential				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.2.3 Chemical attack

GPR, LiDAR, Satellite, Rpas-UAV, Infrared Spectroscopy, NMR, Boroscopy, Endoscopy, Argentometric titration, Potentiometric titration, Ion Chromatography, pH Indicators, infrared Spectroscopy.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
Argentometric Titration				
Potentiometric Titration				
Ion Chromatography				
pH Indicators				
Infrared Spectroscopy				
NMR				

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A3.1.1.2.4 Corrosion

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

GPR, LiDAR, Satellite, Rpas-UAV, Infrared Thermography, Accelerometers, Acoustic Emission, Chloride Diffusion Test, Magnetic Flux Leakage, Pulsed Eddy Current, Boroscopy, Endoscopy, Galvanostatic Pulse Technique.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
Galvanostatic Pulse Technique				
Pulsed Eddy Current				
Acoustic Emission				
Chloride Diffusion Test				
Magnetic Flux Leakage				
Infrared Thermography				
Accelerometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.1.3 Biological processes

A3.1.1.3.1 Biological growth

GPR, LiDAR, Satellite, Rpas-UAV, Satellite, Boroscopy, Endoscopy, pH Indicators.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
pH Indicators				

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Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.1.2 Damage processes related to design and construction issues

GPR, LiDAR, Satellite, Rpas-UAV, Satellite, Boroscopy, Endoscopy, Accelerometers, Clinometers.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
RPAS UAV				
Satellite				
Boroscopy, Endoscopy				
Accelerometers				
Clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2 Tunnels

A3.2.1 Environmental exposure or external factors

A3.2.1.1 Physical processes

A3.2.1.1.1 Continuous vertical rock movement

GPR, LiDAR.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.2 Local rock movement

GPR, LiDAR.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Clinometers				

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Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.3 Higher horizontal actions

LiDAR.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
LiDAR				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.4 Bending stress

LiDAR, Fibre Optic Sensors, Clinometers, Flat jacks.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
LiDAR				
FOS				
Clinometers				
Flat jacks				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.5 Debonding

GPR, LiDAR, Guided Waves Propagation, Acoustic Emission, Impact–echo, Fibre Optic Sensors, Infrared Thermography, Magnetic Memory Method, Magnetic Flux Leakage, Magnetic Pulse Induction, Neutron Radiography.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Guided Waves Propagation				
Acoustic Emission				
Impact-echo				
FOS				
Infrared Thermography				
Magnetic Memory Method				
Magnetic Flux Leakage				

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Magnetic Pulse Induction				
Neutron Radiography				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.6 Partial spalling of concrete cover

GPR, LiDAR.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.7 Overloading (rock movement) of prestressing

LiDAR, Clinometers.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
LiDAR				
Clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.8 Deformation of the ground

LiDAR, FOS.

Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
FOS				
LiDAR				
Clinometers				
Slope clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.1.1.9 Water impact

Acoustic Emission, LiDAR, Water resistance test, Water penetration test, Boroscopy, Endoscopy, Schmidt Hammer test, Ultrasonic wave propagation.

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Surveying technology	Detection effectiveness	Accuracy	Availability	Versatility
Acoustic Emission				
LiDAR				
Water resistance test				
Water Penetration test				
Boroscopy, Endoscopy				
Schmidt Hammer				
Ultrasonic pulse wave propagation				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2 Damage processes related to design and construction issues

A3.2.2.1.1 Missing reinforcement

Clinometers, LiDAR, GPR, Magnetic Memory Method, Magnetic Flux Leakage, Magnetic Pulse Induction.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
Magnetic Memory Method				
Magnetic Flux Leakage				
Magnetic Pulse Induction				
LiDAR				
GPR				
Clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2.1.2 Deformation due to shrinkage, temperature within the shell-blocks

Clinometers, LiDAR, GPR, Concrete block potential shrinkage test.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Clinometers				

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Concrete block potential shrinkage test				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2.1.3 Different casting times

LiDAR, GPR, Schmidt Hammer test, Windsor Probe test, Compressive strength test., Crackmeters.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Schmidt Hammer test				
Windsor Probe test				
Compressive strength test				
Crackmeters				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2.1.4 Different concrete qualities

LiDAR, GPR, Schmidt Hammer test, Windsor Probe test, Compressive strength test, Acoustic Emission, Neutron Radiography.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Schmidt Hammer test				
Windsor Probe test				
Compressive Strength test				
Acoustic Emission				
Neutron Radiography				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2.1.5 Delamination of concrete layers

GPR, LiDAR, Acoustic emission, Guided Waves propagation, Magnetic Memory Method, Magnetic Flux Leakage, Magnetic Pulse Induction, Boroscopy, Endoscopy, Infrared Thermography, Impact-echo, Schmidt Hammer test, Neutron Radiography.

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Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Acoustic Emission				
Guided Waves Propagation				
Magnetic Memory Method				
Magnetic Flux Leakage				
Magnetic Pulse Induction				
Boroscopy, Endoscopy				
Infrared Thermography				
Impact-echo				
Schmidt hammer test				
Neutron Radiography				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

A3.2.2.1.6 Anchor failure

GPR, LiDAR, Clinometers.

Surveying Techniques	Detection effectiveness	Accuracy	Availability	Versatility
GPR				
LiDAR				
Clinometers				

Rating Scale	Low	Low Medium	Moderate	Moderate-High	High

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Appendix A4 Data analysis methods for specific surveying technologies

Surveying technology	**Data analysis methods for the specific surveying technology** - Best practice or state of the art - Examples of novel/recent approaches - Further references
Visual inspection	**Task**: Defect detection and segmentation on high resolution images of concrete structures **Best practice/state-of-the-art**: EN 13018:2016 **Novel/Recent approaches**: Literature is mostly focused on detection of cracks, also because public benchmark datasets exist mostly for this defect type. State-of-the art results are obtained using deep learning architectures. Lately a novel Crack Transformer network (CrackFormer) for fine-grained crack detection was proposed [1]. It is composed of novel attention modules in a SegNet-like encoder-decoder architecture and new scaling-attention modules to combine outputs from the corresponding encoder and decoder blocks to suppress non-semantic features and sharpen semantic ones. **Further references**: Deep hierarchical CNN (DeepCrack), to predict pixel-wise crack segmentation in an end-to-end method [2].
Boroscopy	**Task**: **Best practice/state-of-the-art**: EN 13018:2016 **Novel/Recent approaches**: A deep learning framework [3] based on state-of-the-art Fully Convolutional Networks (FCN) was proposed to identify and locate damages from borescope images. The framework successfully detected two major types of damages (cracks and burns) in borescope images and extracted their regions with high prediction accuracy. The framework was further optimized to significantly reduce the amount of training data by applying fine-tuning methods.
Water penetration	**Task**: Determine maximum range of water penetration into concrete element from the surface **Best practice/state-of-the-art**: EN 12390-8:2019
Weight-in-motion method	**Task**: Determine the weight and wheel loads of the vehicles Focus: Assess the quality of data acquired by the WIM system in terms of accuracy, outlier detection and error analysis. **Best practice/state-of-the-art**: State of the art in the measurement of the accuracy referred to weights and static loads [4] is based on the application of statistical analysis with calculation of mean, standard deviation, confidence level, confidence interval and tolerance. WIM data for the accuracy test are acquired using instrumented calibration lorries. **Novel/Recent approaches**: This paper [5] describes a method based on determining the Steering Axle Load Spectra (SALS) to investigate the accuracy of 77 operative weigh-in-motion stations. In order to include several factors—such as the type of axle load sensor, pavement temperature and vehicle speed—various cases of SALS were derived using a series of filters. Means and standard deviations were then calculated on normally distributed SALS (Lilliefors test at 99.9% level). By analysing the impact of temperature and vehicle speed on the cumulative distributions of mean values of SALS, the authors concluded that systematic error in WIM measurement can be the result of temperature change.
Fibre optic sensors	**Task**: Measure the deformation of concrete surface **Best practice/state-of-the-art**: EN IEC 61757-1-1:2020-12

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Surveying technology	Data analysis methods for the specific surveying technology - Best practice or state of the art - Examples of novel/recent approaches - Further references
	Novel/Recent approaches: Recently, Cross Correlation Analysis (CCA) and Robust Regression Analysis (RRA) were applied to data generated on a laboratory model of a four-span highway bridge under different global and local damage scenarios [6]. A system of Fibre Bragg Grating (FBG) strain sensors was used to capture SHM data under different temporal damage scenarios. In CCA, cross-correlation coefficient matrixes of strain data amongst all possible individual pairs of sensors were created for the baseline (un-damaged) and the damaged condition. Statistically significant changes of sensor pairs cross-correlation coefficients with respect to the baseline during monitoring helped detect and locate the damage. In RRA, only sensor pairs that have high correlation were chosen and then rearranged into a new matrix. Only correlations of these pairs were used during the monitoring phase to detect any abnormal behaviour within a robust regression model. Both methods detected the damages. RRA outperformed CCA in terms of time-to-detection but also resulted more computationally expensive. **Further references**: Comparison of acquired data vs model, calibration of the model, determination of the density function of continuous measured data with genetic algorithms [7].
Endoscopy	**Task**: Detection and segmentation of damaged area **Best practice/state-of-the-art**: EN 13018:2016
Magnetic and Electrical Methods	**Task**: Detect debonding, loss of diameter in rebars **Best practice/state-of-the-art**: BS 1881, DIN 1045 **Novel/Recent approaches**: The Magnetic Force Induced Vibration Evaluation [8] (M5) is a unique mechanical and electromagnetic method that allows examining the bond between rebars and concrete with higher sensitivity. The M5 method can be considered as a type of modal analysis using magnetic coupling and electromagnetic excitation. Using a strong magnetic coupling (excitation element with alternating magnetic field, strong magnet attached to the detection element), vibrations are induced only in the rebar and not the whole structure. Data analysis consist in the visual inspection of the frequency spectrum of the measured signal. Signal changes between a healthy rebar and a de-bonded one are observed.
Radioactive and Nuclear Methods	**Task**: Structural integrity of concrete samples (Loss of section, delamination) **Best practice/state-of-the-art**: ASTM D2950-91, NDIS 1401-1992
Guided waves Techniques	**Task**: Crack detection in the damaged area **Best practice/state-of-the-art**: BS 9690-1:2011, ASTM E2775 –16, ISO 18211:2016
Surface measurements	**Task**: Concrete strength, Hardness of the surface **Best practice/state-of-the-art**: EN 12 504-2, ASTM C805-97 STM, ISO/DIS 8046
Acoustic Emission	**Task**: Location of active destructive processes from emission of acoustic signals. **Best practice/state-of-the-art**: EN 13554:2011, ASTM E3100 – 17 **Novel/Recent approaches**: The waveform-based analysis approach is believed to be better than traditional parameter-based approach in source discrimination. Acoustic emission methods are usually based on a first denoising or signal conditioning step, in which the recorded waveforms are processed using Fourier (frequency domain), short-time Fourier (time-frequency domain),

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Surveying technology	Data analysis methods for the specific surveying technology - Best practice or state of the art - Examples of novel/recent approaches - Further references
	wavelet or fast-wavelet transforms. Latest literature agrees with the superiority of the wavelet over Fourier Transform based filter techniques [9]. This signal feature extraction step is the eventually followed by location analysis, cluster analysis of locations in physical space, measurement of variance and energy of location clusters, cluster analysis of locations in feature space, visualisation for final fracture identification [10]. Since similar source mechanisms emit similar signals, search for similarity is used for source discrimination. Cross-correlation coefficients in time domain and magnitude squared coherence (MSC) in frequency domain [11] are two proposed algorithms for source discrimination. **Further references**: Method based on image correlation [10] Use of wavelet algorithms and Coherence Functions [9]
Water resistance	**Task:** Water absorption of concrete as a measure of resistance against carbonation and chloride migration **Best practice/state-of-the-art**: PN-EN 772-11:2011
Qualitative chemical methods	**Task**: Distribution of the pH changes on the sample, carbonation **Best practice/state-of-the-art**: EN 14630:2006, EN 1239-12:2020, ASTM C876
Quantitative chemical methods	**Task**: determine the presence of cracks or the loss of section in rebars from chloride ion concentration in steel reinforced concrete (wet solution, x-ray fluorescence spectroscopy (XRF)) **Best practice/state-of-the-art**: AS 1012.20-1992, EN 14629:2008, PN-EN 1767:2008, ASTM C 1202 **Novel/Recent approaches**: **Additional references:** Big data in quantitative chemistry [12] (PCA, Factor Analysis, Wavelet transform, Machine learning) Recent approaches of Factor Analysis in chemistry [13] Machine learning in electrochemistry [14]
Mechanical tests on cored samples	**Task**: **Best practice/state-of-the-art**: EN 12390-3:2001, EN 12504-1:2002, ASTM C900-94 STM, ISO 5725-1 **Novel/Recent approaches**: Verification of statements concerning the random variations of mechanical behaviour of concrete cores, supported with statistical approaches, is important. Statistically significant data sets under such conditions are very rare. In this study [15], the issue is addressed by exploring the mechanical behaviour of cores using probabilistic concepts. Two-parameter Weibull, normal and log-normal distributions are used to fit the test data (strength, elastic modulus and strain at peak stress) of concrete specimens. The Weibull distributions most accurately describe the experimentally measured data. A basic theory of damage mechanics is introduced to deal with stress-strain behaviour of cores. In this theory, a statistical method is used to describe mechanical properties on a mesoscopic scale in order to generate realistic behaviour at a macroscopic scale. It is shown that there is a relatively good coincidence between the theoretical results and the measured data.
Mems accelerometer	**Task**: **Best practice/state-of-the-art**:

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<table>
<tr><th>Surveying technology</th><th>Data analysis methods for the specific surveying technology
- Best practice or state of the art
- Examples of novel/recent approaches
- Further references</th></tr>
<tr><td></td><td><u>Vibrational parameters</u>:
1. Statistical analyses can be performed at sensor level (evaluation of the sensor vibrations), component level (analysis of data collected by all the sensors installed on the same element), and system level (analysis of data collected by all the sensors installed on the same structure).
2. As a result, the following quantities can be estimated:
- Sensor, element or structure average vibrational levels
- Anomalous vibrations induced by localized damages (e.g., road pavements defect)
- Presence of exceptional loads or accidental actions
- Earthquake detection
3. <u>Modal parameters </u>[16,17]:
4. Structure modal parameters estimation (natural frequencies, damping ratio)
5. Structure modal shapes estimation
6. Damage detection in operational conditions, by evaluating changes in time of frequencies values/damping ratios/modal shapes
7. Structure health status evaluation after an exceptional event, as an earthquake.
Novel/Recent approaches:
Machine learning for the detection of vibrational levels</td></tr>
<tr><td>Mems clinometer</td><td>Task:
Best practice/state-of-the-art:
8. Data analysis methods on clinometers data mainly consist in statistical methods, aimed to evaluate structure residual deformations due to degradation/damage mechanisms (plastic deformations, settlements, bearings deformations, structural joints defects, etc.) and evolving deformations. It used also to estimate the structure deformed shape under load.
Novel/Recent approaches:</td></tr>
<tr><td>Satellite</td><td>Task: Change detection
Best practice/state-of-the-art:
Due to the limited resolution of optical satellite images, structural failures can only be detected as “changes” over infrastructure images acquired overt time. Heuristic methods are based on simple thresholds (decided by experts), usually applied to determine if there is a change in the infrastructure. This paper, for example, extracts joint planar-vertical features to delineate the structure of interest and applies a multi-temporal change detection model to simultaneously capture the structure change over time [18]. The detected changes are usually referred as rupture, deformation and displacement interchangeably.
Novel/Recent approaches:
Machine learning can be used to determine changes in the image over time. This paper [19] describes an optimized architecture (EffCDNet) which adopts a siamese-based pre-trained encoder with an Attention-based UNet decoder for semantic segmentation. The network is built with pre-trained EfficientNet architecture with shared weights to extract robust features and to overcome the limitations caused by insufficient training data. The attention-based UNet decoder uses the attention-gate layer mechanism right before concatenation to obtain more discriminative relevant features and improve segmentation</td></tr>
</table>

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Surveying technology	Data analysis methods for the specific surveying technology - Best practice or state of the art - Examples of novel/recent approaches - Further references
	performance. This allows the user to reconstruct fine-grained feature maps with significant context information. To obtain enhanced information difference map, the Undecimated Discrete Wavelet Transform (UDWT) fusion was used as a post-processing technique for spatial and temporal analysis of multi-resolution images. **Further references**: Alternative machine learning approaches [20,21].
LiDAR	**Task**: Detect damages in concrete (cracks, spalling, etc) **Best practice/state-of-the-art**: Damages are detected using heuristic methods. Point clouds from reconstruction are compared to previously acquired point clouds of the same infrastructure or to the model of the structure [22]. **Novel/Recent approaches**: This paper [23] presents a framework for automated defect inspection of concrete structures, that comprises LiDAR data collection, defect detection, scene reconstruction, defect assessment and data integration stages. Deep learning algorithms are implemented to efficiently detect defects from the collected LiDAR images, and a simultaneous localization and mapping algorithm is adopted for site reconstruction. Based on the images of detected defects, assessment is conducted to evaluate the defect conditions, complemented with the defect dimensions estimated from the aligned image and LiDAR data. The defect position could also be mapped to the respective structural elements and the inspection results integrated into existing Building Information Modelling files. The proposed workflow was validated using a case study for determining concrete cracks and spalls in a real-world facility. **Further references**: Heuristic [24] Alternative machine learning approaches [25,26]
GPR	**Task:** **Best practice/state-of-the-art**: Heuristic methods are based on comparing the measured GPR signal to synthetic GPR data [27]. GPR data are often integrated with other complementary NDT data. Some additional imaging techniques like textural analysis or Hilbert transforms are used to highlight some signal attributes (amplitude, frequency, attenuation). The most typical machine learning techniques, based on detecting hyperbolic reflections or pattern recognition, are Support Vector Machine (SVM), Genetic algorithms, Fuzzy logic and Hidden Markov model (HMM). Histogram of oriented gradients (HOG) may be used for feature extraction. **Novel/Recent approaches**: Machine learning approaches are based on object and damage detection through CNNs and RNNs. In this example (28), a Mask Region-based Convolutional Neural Network (R-CNN) is improved by incorporating a new loss function for detection and segmentation. This new loss function is called *distance guided intersection over union* (*DGIoU*) and considers the center distance between two bounding boxes, to overcome the weakness of intersection over union (*IoU*) in training and evaluation. In addition, a new method is proposed to extract data points from the segmented mask patches containing both object signatures and background noises. The extracted data points can be further processed for object localization and characterization. Experiments conducted using GPR scans

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Surveying technology	**Data analysis methods for the specific surveying technology** - Best practice or state of the art - Examples of novel/recent approaches - Further references
	collected from a concrete bridge deck demonstrate that the hyperbolic signatures of rebars can be accurately detected and segmented using the proposed method.

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Appendix A5 References

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IM-SAFE.EU

REVIEW OF METHODOLOGIES AND INSTRUMENTS FOR DIAGNOSTICS OF TRANSPORT INFRASTRUCTURE APPENDIXES

Annex B - Damage characterization procedure

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Appendix B1 Surveying technologies – Phases

Technology \ Phase	Phase 1	Phase 2	Phase 3
Accelerometers			○
Abrasion resistance test		○	
Acoustic Emission			○
Argentometric Titration		○	
Boroscopy	○		
Chloride Diffusion test/Ion migration test		○	
Compressive strength test		○	
Computed Tomography		○	
Covermeters		○	
Crackmeters			○
Electrical resistivity tomography		○	
Endoscopy	○		
Fatigue test		○	
Fiber Optic Sensors			○
Galvanostatic Pulse Technique		○	
GPR	○		
Gravimetric method		○	
Half-cell Potential method		○	
Impact-Echo method		○	
Infrared Spectroscopy		○	
Ion Chromatography		○	
LiDAR	○		○
Magnetic Flux Leakage Method		○	
Magnetic Memory Method		○	
Neutron Radiography		○	
NMR Spectroscopy		○	
Phenolphthalein test		○	
Potentiometric Titration		○	
Pull-out test		○	
Pulsed Eddy Current Method		○	
Rainbow test		○	
Satellite	○		○

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Technology \ Phase	Phase 1	Phase 2	Phase 3
Schmidt hammer		○	
Spectral Analysis of Surface Waves		○	
Tensile test		○	
Thymolphthalein test		○	
Torque test		○	
UAV	○		○
Ultrasonic Pulse Velocity		○	
Uranyl-Acetate treatment		○	
Water penetration test		○	
Water resistance test		○	
Weight-In-Motion Systems			○
Windsor Probe test		○	

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