



TU1406
COST ACTION

Quality specifications for roadway bridges,
standardization at a European level

WG2

Technical Report

Performance Goals for Roadway Bridges
OF COST ACTION TU 1406

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1 INTRODUCTION

1.1 Scope

Europe is more dependent than ever on its transportation system to consistently support a thriving economy and to afford its citizens a satisfactory quality of life. Transport enables economic growth and job creation: it must be sustainable in the light of the new challenges we face (EC, 2011). However, a large part of the road network, including bridges, which represent significant assets, are reaching the end of their service life, whilst traffic volumes are steadily growing, environmental legislation is becoming more stringent, and the occurrence of extreme weather events (such as large floods) is increasing. At the same time, in recent years many national road agencies in Europe have faced budgetary restriction with the result that funding for maintaining, renovating and reconstructing road infrastructure has decreased significantly. It is therefore extremely important that we improve the way European infrastructure is inspected, assessed and maintained, and that better methods are developed to allocate limited resources, in order to fulfil the ever growing requirements. Together with these developments the evolution and upgrading of existing bridge management practice is essential.

In recent years Infrastructure Asset Management (IAM) has been applied as a strategic governance approach with the aim of achieving more value from assets whilst reducing the use of resources. By combining engineering and economic principles with sound business practice, asset management strives for cost-effective investment decisions throughout the life-cycle of infrastructure assets (Tao et al., 2000). In order to support IAM, agencies must collect, store, manage, and analyse large amounts of data in an effective and efficient manner. Although agencies have strongly emphasized the importance of data collection and integration, the process of linking the data to the decision-making processes has potential for significant further improvement (Flintsch and Bryant 2009). The available data should be translated into useful information about the performance of the asset, in the form of performance indicators (PI). By focusing on the use of the PIs and translating them into Key PIs (KPIs) and the needs of the decision and processes to be supported, road agencies could define which assets and which KPIs about these assets are more important for decision-making and tailor their data collection accordingly.

1.2 COST Action TU1406

The objective of the COST TU 1406 Action is to investigate the way bridge PIs and KPIs are collected and quantified, how performance goals are specified across Europe, and finally to produce guideline documents linking collection and quantification of PIs, KPIs, performance goals, standards, and practices to decision making processes. Performance goals are usually defined at different levels, from high-level strategic decisions to low-level, object-specific objectives. This report gives the overview of different performance aspects, how they could be quantified and used for bridge performance assessment as a result of activities in the Working Group 2 of the COST TU 1406 Action.

The main ambition of the Action is to develop a guideline for the establishment of Quality Control plans in roadway bridges, by integrating the most recent knowledge on performance assessment procedures with the adoption of specific goals (Matos, 2016, Matos et al., 2016). This guideline will focus on bridge maintenance and life-cycle performance at two levels: (i) performance indicators, (ii) performance goals. The possibility to incorporate new indicators related to sustainable performance will also be considered. By developing new approaches to quantify and assess bridge performance, as well as quality specifications to assure expected performance levels, bridge management strategies will be significantly improved, enhancing asset management of ageing structures in Europe.

In order to reach this main general aim, the following more specific objectives/deliverables have been considered (Matos, 2016, Matos et al., 2016): (i) to systematize knowledge on QC plans for bridges, which will help to achieve a state-of-art report that includes performance indicators and respective goals; (ii) to collect and contribute to up-to-date knowledge on performance indicators, including not only technical indicators but also environmental, economic and social ones; (iii) to establish a wide set of quality specifications through the definition of performance goals, aiming to assure an expected performance level; (iv) to develop detailed examples for practicing engineers on the assessment of performance indicators as well as in the establishment of performance goals, to be integrated in the developed guideline; (v) to create a data basis from COST countries with performance indicator values and respective goals, that can be useful for future purposes; (vi) to support the development of technical/scientific committees.

To achieve the objectives, it was decided to structure the work in several Working Groups (WG), as presented in by Matos et al. 2016 and Casas 2016:

WG1: Performance indicators. The goal is to explore those performance indicators of bridge structures, in the course of international research cooperation, which capture the mechanical and technical properties and its degradation behavior, already partly covered by code specifications. Considerations also include: natural aging, quality of the material; service life design methods; sustainable indicators; environmental, economic and social based indicators, performance profiles. The final result is the implementation of a performance indicator data base for Europe with flexibility to accommodate country-specific requirements. Further information on this WG can be found in Strauss (2016). *Objectives of Working Group 1* therefore are among others the characterization of bridge performance indicators, which can address: (a) the safety: the load factor, the reliability index to ULS; (b) the serviceability: the condition index, the reliability index to SLS; (c) the availability, robustness; (d) the costs: the total LCC, values related to durability aspects; and (e) aspects of environmental efficiency: CO2 foot-print. **Leader:** Alfred Strauss, **Vice-Leader:** Ana Mandić Ivanković.

WG2: Performance goals. The objective is to provide an overview of existing performance goals for the indicators previously identified in WG1 and to develop technical recommendations which will specify the performance goals. These goals will vary according to technical, environmental, economic and social factors. Further information on this WG can be found in Stipanovic and Klanker (2016). *Objectives of Working Group 2* therefore are among others to identify existing performance goals (where the term goal pertains to quantifiable requirement and/or threshold value) for the indicators previously indicated in WG1. The performance goals will vary according to technical, environmental, economic and social factors. **Leader:** Irina Stipanovic **Vice-Leader:** Lojze Bevč.

WG3: Quality Control plans. Based on the results of WG 1 and WG 2 as well as on survey of existing approaches in practice, the objective of this WG is to provide a methodology with detailed step-by-step explanations for establishment of QC plans for different types of bridges. The QC plan has to relate performance goals, which are user / society related, e.g.: Traveling time; Traffic allowance; Safety level; Comfort / Serviceability; Further information on this WG can be found in Hajdin (2016). *Objectives of Working Group 3* therefore are among others based on results from WG1 and WG2, as well as on a survey of existing approaches in practice, the objective of Working Group 3 is to provide a report with detailed step-by-step explanations for the establishment of QC plans for different types of bridges. The QC plans will address the dynamics and uncertainty of the processes that may significantly compromise bridge performance. **Leader:** Rade Hajdin, **Vice-Leader:** Matej Kušar.

WG4: Implementation in a case study. A series of benchmarks will be developed during Working Group 4. To this end, some of the performance indicators identified in WG1 will be computed for a set of roadway bridges over EU. These indicators will be then compared with specific goals, as identified in WG2. At the end of the process, a QC plan will be applied to those bridges utilising recommendations from WG3. A data basis will be then established for benchmarking. **Leader:** Amir Kedar, **Vice-Leader:** Sander Sein.

WG5: Drafting of guideline/recommendations. Working Group 5 focuses on the development of guidelines, drawing support from all the other WGs. These guidelines for a systematic maintenance and management of highway bridge assets will acknowledge the variation of philosophical, technical and implementation methodologies throughout the EU, with the expectation that the delivered framework will be scalable and portable for standardised implementation in existing or new infrastructure networks. **Leader:** Vikram Pakrashi, **Vice-Leader:** Helmut Wenzel.

WG6: Dissemination. The aim of this WG is to disseminate all results which were obtained in all the other WGs. Dissemination consists in establishing liaisons with existing national and international associations, conferences, working groups and journals. Also, this group will be responsible to continuously update the website as well as all the other dissemination frameworks. **Leader:** Guðmundur Guðmundsson, **Vice-Leader:** Stavroula Pantazopoulou

The target groups and end users who will exploit the outcome of this Action are (Matos, 2016, Matos et al., 2016): (i) public/private owners, as their assets will be maintained in an upscale level; (ii) operators, as standardized procedures for reducing maintenance costs, guaranteeing the same quality-level, will be introduced; (iii) design and consultant engineers, as the assessment of roadway bridges performance will be established in a uniform way, according to the developed guideline; (iv) equipment and

software companies, as a new perspective will be given, regarding the most suitable equipment and software for the assessment of roadway bridges; (iv) academics and researchers engineers, as they will take an advantage of their involvement in the guideline preparation; (v) students, as they will benefit from COST tools (e.g. training schools) and from the contact with different stakeholders involved in this Action; (vi) relevant European, international and national associations, with which the main outcomes of this Action will be shared; (vii) standardization bodies and code writers, which will benefit from the developed guideline.

The functionality of an infrastructure network can be assessed through a performance framework which has to be set in advance by decision makers. Decision makers can be either asset owners (public entity, often a national or state level ministry), or in the case of concessions the owner is public, the concessionaire is the asset manager. In practice they should share decision making responsibility between them. But depending on the level, the decision maker can also be manager or designer of a certain project/structure. In this framework performance goals are established at a component level (e.g. a bridge beam), a system level (a bridge) and at a network level (a collection of bridges that form a network) and objects are evaluated according to this based upon measurable performance indicators. Performance goals can be very different in varying countries or agencies.

The decision making process can be performed at a number of different levels; varying from strategic/network (top-down approach) level to tactical, operational/object (bottom-up approach) level. The purpose is establishment of optimal investment funding levels and performance goals for the whole network, together with identification of appropriate combinations of measures and timing for each individual part/object of the network.

1.3 WG2 Report

In this report, the path from measuring and assessing performance indicators to linking them with certain performance goals are presented. Often this cannot be established directly especially when multi-objective performance based decision making is needed and preferred.

Reliability performance assessment is analysed for three actions: structural performance, seismic loading and scour.

The economic, financial and social implications that arise during the lifetime of an object are explained. For infrastructure objects a traditional view of Life Cycle Cost (LCC) would be the costs incurred by government bodies for realisation, maintenance and disposal of the object in question. In recent years user costs and society costs are taken into account. The user costs are those that are borne directly by the user of the infrastructure, such as vehicle operating costs and traffic accident costs. The society costs are costs borne by society at large and usually take into account the environmental costs.

Modern decision making processes dealing with bridge management have to go far beyond choosing an optimal solution based just on single indicators (i.e. lowest long-term cost), and multi-performance criteria have to be implemented. Since the analysis of complex problems involving many parameters cannot be solved without optimization algorithms, the final chapter addresses this issue.

2 PERFORMANCE GOALS FOR ROADWAY BRIDGES

2.1 Introduction

In most European countries maintenance is the core task of the road agencies. In the past, maintenance was mostly approached from a technical point of view rather than a user point of view. This situation has changed significantly during the past decade. Societal aspects are now an integral part of the decision making process and for example, user benefits should now be considered for all management activities, including maintenance. In recent years, infrastructure asset management has been applied as a strategic governance approach to achieve more value from assets and minimize the use of scarce resources. By combining engineering and economic principles with sound business practice, asset management strives for cost-effective investment decisions throughout the life-cycle of infrastructure assets (Tao et al., 2000). The principle of asset management has been described in the Specification PAS 55 (BSI 2008) and ISO 55000 standard (ISO 2014). Within asset management, physical assets are to be considered in relation with all other activities of the organization to deliver the required performance. For physical assets such as bridges this means bridge management is to be part of the

management of the network, and the bridge performance levels have to fulfil the network performance requirements. (Velde et al. 2013)

There are two approaches to asset management in practice: top-down and bottom-up. The top-down approach focuses on planning and strategy management at the network-level of the bridge network. The bottom-up approach focuses on project-level analysis which dictates network strategy.

Performance assessment is commonly encountered in a number of activities and processes related to engineering, economics, environment, health, etc. Its definition in this context is straightforward, in that performance essentially refers to how successfully a task, system or operation functions. From this perspective, performance measurement is a task required for assessing and improving characteristics and operations of a system, process, or infrastructure (Karlaftis and Kepaptsoglou, 2012). A comprehensive definition of performance measurement is offered by the US Federal highway Administration (Shaw, 2003):

“Performance measurement is a process of assessing progress toward achieving predetermined goals, including information on the efficiency with which resources are transformed into goods and services (outputs), the quality of those outputs (how well they are delivered to clients and the extent to which clients are satisfied) and outcomes (the results of a program activity compared to its intended purpose), and the effectiveness of government operations in terms of their specific contributions to program objectives.”

In the road sector, performance can be measured from a number of different perspectives and for a number of reasons, e.g. at a network level, system level and a component level.

The required network performance is related to functional requirements and to policy goals applied to the network as a whole. These goals are rather abstract and have to be specified from a network level to requirements on a system level (Klatter et al., 2012) The required structural performance is usually related to the structural safety for example as a target reliability value. Existing international codes and standards are almost exclusively focused on structural safety and serviceability. In the next paragraphs we will explain the difference between performance goals and requirements on the network and on the system level, supported with the examples of existing codes and guidelines.

2.2 Literature overview

2.2.1 From PIs to KPIs

Strategic decision-making for networks is done by the asset owner, in the case of road networks which is most often a public entity. The asset owner deals with the “why” question, and has the responsibility to come up with the overall network policy, set targets for performance on a network level and for an acceptable risk level. In Figure 1 this is presented at the top left of the triangle.

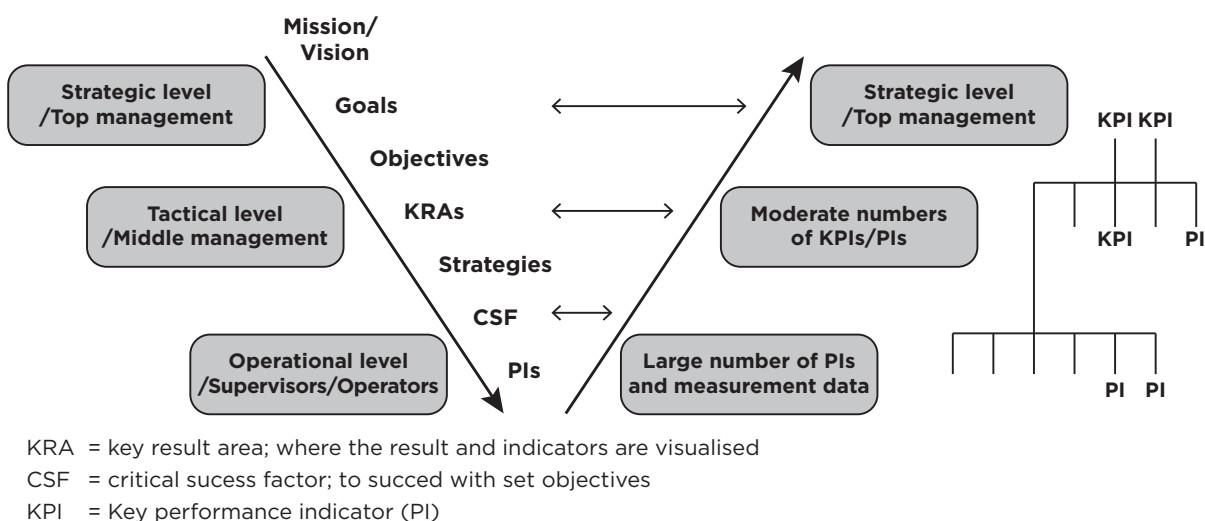


Figure 1. Breakdown of goals and objectives and aggregation of data. C. Stenström, A. Parida and D. Galar (2012) Performance Indicators of Railway Infrastructure, International Journal of Railway Technology Volume 1, Issue 3, Pages 1-18, 2012. doi:10.4203/ijrt.1.3.1, ©Saxe-Coburg Publications, 2012,

The asset manager is responsible for meeting the performance goals whilst meeting the constraints set by the asset owner. The asset manager will translate the targets and strategies of the asset owner into what, where and when to take actions at the tactical and operational level. The asset manager is responsible for investment strategies, maintenance concepts, technological standards, risk management and network management. In Figure 1 it can be seen that in order to assess performance objectives Key Performance Indicator, KPIs that need to be defined, at strategic and tactical level. Those KPIs are determined from a number of Performance Indicators, PIs collected at an operational level. For the purpose of this report the PIs are determined at the bridge element level. An overview of existing Performance Indicators, was collated as a result of an extensive survey in WG 1 of this Action, is reported in the Technical Report (WG1, 2016).

The role of service provider, at the operational level, is often outsourced to private companies or can be fulfilled within the asset management organisation itself. The service provider is responsible for project delivery, maintenance, execution and services, including the delivery of asset information and project management. In bridge assessment procedures they are regularly responsible for the collection of PIs through visual inspections, on-site tests, structural health monitoring etc. The collection of PIs should be in line with the KPIs and pre-defined procedures for bridge assessment.

2.2.2 Performance goals at different levels

The concept of efficient transport network management has been introduced in the past two decades as the “process of maintaining and improving the existing road network to enable its continued use by traffic efficiently and safely, normally in a manner that is effective and environmentally sensitive; a process that is attempting to optimize the overall performance of the road network over time” (Karlaftis and Kepaptsoglou, 2012). Maintaining the national road network plays an important part in achieving the strategic goals. However, the relationship between goals and network performance is not exclusive. For example, network performance may be dependent on traffic patterns, weather conditions, economic growth and oil prices, to name a few variables. It is therefore often not possible to derive performance indicators directly from network level goals.

In the Netherlands, the required network performance is defined on a 4-yearly basis in a Service Level Agreement between Rijkswaterstaat and the Ministry of Infrastructure and the Environment. Performance aspects are categorized in: availability, traffic safety and sustainability. For each category, a number of indicators have been specified as shown in Table 1. Where the goals are stated as a desired outcome, required network performance is specified in terms of output.

Table 1. Overview of the network performance aspects and belonging Performance Indicators
(Ministry of Infrastructure and the Environment, 2011)

	Availability
1	Availability of the road
2	Traffic delays caused by road works <ul style="list-style-type: none"> Caused by construction projects and planned maintenance Caused by unplanned maintenance
3	Availability of traffic information <ul style="list-style-type: none"> Availability Accuracy
A	Availability of the road during rush hour
B	Traffic jams: total
C	Traffic kilometers travelled on the main road network
	Traffic safety
4	Traffic safety: <ul style="list-style-type: none"> Pavements meet demands for skid resistance and rutting Preventive spreading of de-icing salts within appointed timeframe
D	Number of deaths caused by traffic accidents
E	Number of hospitalized by traffic accidents
	Sustainability
F	The number of encroachments of the norm

An example of performance goals on all three levels strategic, tactical and operational level, based on the literature study and related possible performance indicators is given in Table 2.

Table 2. Example of performance goals, tasks and indicators on different levels, based on the literature from Australia, Netherlands and UK (Austroads, 2009; RWS, 2011; DfT, 2013)

Level	Performance goals (examples)	Tasks	Performance indicators (examples)
STRATEGIC LEVEL (network or part of network)	<ul style="list-style-type: none"> - To provide safe, responsive and sustainable network; - To provide an efficiently and effectively operated strategic road network; - To support and facilitate economic growth; - To minimize its negative impacts on users, local communities and the environment; - To balance the needs of individuals and businesses that use and rely on it. 	<p>State performance requirements at network level, for both short and longer term.</p> <p>Resources boundaries at network level, across regions, assets or types of activities for long term (period is arbitrary)</p>	<ul style="list-style-type: none"> - Average availability of the road (% of time) - Long-term trend of traffic jams - Long-term trend of number of traffic accidents - Environmental impacts
TACTICAL LEVEL (part of network or object)	Meet technical, socio-economic and sustainability requirements in line with performance goals	<p>Risk assessment</p> <p>Development of maintenance strategies and technological standards</p> <p>Resources allocation across regions, assets or types of activities for medium term, usually for the period of up to 5 or 10 years</p>	<ul style="list-style-type: none"> - Availability of the road (% of time) - Traffic delays caused by road works - Availability of the road during rush hour - Traffic jams - Traffic kilometres travelled on the main road network - Number of deaths caused by traffic accidents - Number of hospitalized by traffic accidents - Environmental impacts - Unplanned maintenance / operational costs
OPERATIONAL LEVEL (object or element)	Meet technical, socio-economic and sustainability requirements in line with performance goals	<p>Resources allocation across assets (usually within one type of asset or a part of the network) within 1 to 5 years</p> <p>Decision on alternative choice on the project / bridge level, e.g.</p> <ul style="list-style-type: none"> - do nothing and monitor - repair - rehabilitate - replace 	<p>Technical indicators:</p> <ul style="list-style-type: none"> - condition level - reliability index - risk level <p>Socio-economic:</p> <ul style="list-style-type: none"> - maintenance costs - operational costs - user delay costs - environmental costs <p>Environmental indicators:</p> <ul style="list-style-type: none"> - air pollution - noise - soil and water pollution

Structures like bridges are necessary for a functioning transport infrastructure network. Bridge perfor-

mance goals can be set in order to ensure bridge performance is in line with network level performance goals. When defining bridge performance indicators, some difficulties may present themselves. First, the timescale for which network performance goals are set is typically much shorter than the estimated service life of a bridge. In the Dutch example, the Service Level Agreement is set for a period of four years. In contrast the estimated service life of a bridge is up to 100 years. Therefore bridge performance goals should not only enable meeting the short term performance goals, but also facilitate life cycle optimization.

Furthermore, where bridge management is traditionally focused, on evaluating the condition of the bridge, the desired condition now needs to be expressed or translated into goals reflecting network performance.

Based on the Service Level Agreement, a set of Performance Indicators for structures are defined. These indicators are based on a set of performance criteria (in our approach we would call them performance aspects), the RAMS SHEEP criteria - which are the acronym for Reliability, Availability, Maintainability, Safety, Security, Health, Environment, Economic and Politics, respectively. Each criterion is defined as follows:

- Reliability: the probability that the required function of the system can be carried out under the given conditions for a given time interval.
- Availability: the probability that the required function of the system can be carried out under the given circumstances during a given arbitrary time.
- Maintainability: the probability that the maintenance activities are possible within the specified time and under circumstances that the required function continues to run.
- Safety: related to the freedom from unacceptable risks in terms of injury to people.
- Security: related to the safety of a system regarding to vandalism and unreasonable human behavior.
- Health: being related to physically, mentally and socially defined aspects.
- Environment: concerns the physical environment requirements.
- Economics: regarding the relationship between cost and value.
- Politics: concerning political-administrative and social requirements.

In Table 3 details of all the RAMSHEEP performance criteria are given, as they are currently used by Rijkswaterstaat. A clear definition of the required performance forms the start of the process of risk based inspections. The demands on a network level will be translated to demands for parts of the network and subsequently to demands for objects, in this case bridges. COST action TU 1406 the terminology criteria would be related to the performance aspect and a sub-criteria would performance goal.

**Table 3. Performance criteria following RAMSSHEEP aspects.
(Ministry of Infrastructure and the Environment 2012)**

Criteria / Performance Aspect		Sub-criteria / Performance Goal
Reliability	1.1.R	Satisfy reliability requirements for moving parts and equipment
	1.2.R	Meet structural requirements in relation to damages
	1.3.R	Meet structural requirements in relation to revised standards
	1.4.R	Meet structural requirements in relation to different use
	1.5.R	Meet structural requirements in relation to defects in design, execution or management
Availability	2.1.A	Meet object specific requirements with regard to the fulfilment of the object functions
	2.2.A	Prevention of calamities
Maintainability	3.1.M	Meet requirements relating to the maintainability of elements
Safety	4.1.Sa	Meet object specific requirements with regard to the safe performance of the object functions
	4.2.Sa	Prevention of calamities
Security	5.1.Se	Meet the requirements with regard to the prevention of vandalism
	5.2.Se	Meet the requirements relating to the protection of the object
Health	6.1.H	Meet health and safety decisions
Environment	7.1.E	Meet design requirements
	7.2.E	Meet environmental requirements
	7.3.E	Comply with requirements relating to use/ comfort
Economics	8.1.Ec	Water management in order
	8.2.Ec	Prevent widespread or irreparable damage
Politics	9.1.P	Meet requirements for image

These criteria can be applied during inspection and maintenance processes (Bakker et.al. 2012, Klankner et.al. 2014). , but the link from performance indicators obtained and the exact criteria addressed, doesn't necessarily have to be direct.

2.3 COST TU 1406 Approach

In order to develop maintenance strategies it is necessary to establish a link between the performance aspects at the bridge component level and those at the network level. The framework which links performance indicators (PIs) and performance goals (PGs) at different levels is presented, based on the WG 1, WG 2 and WG 3 objectives within COST TU 1406 project (www.tu1406.eu) is shown in Figure 2. Depending on the asset management approach chosen, bottom-up or top-down, it can be observed and applied in both directions.

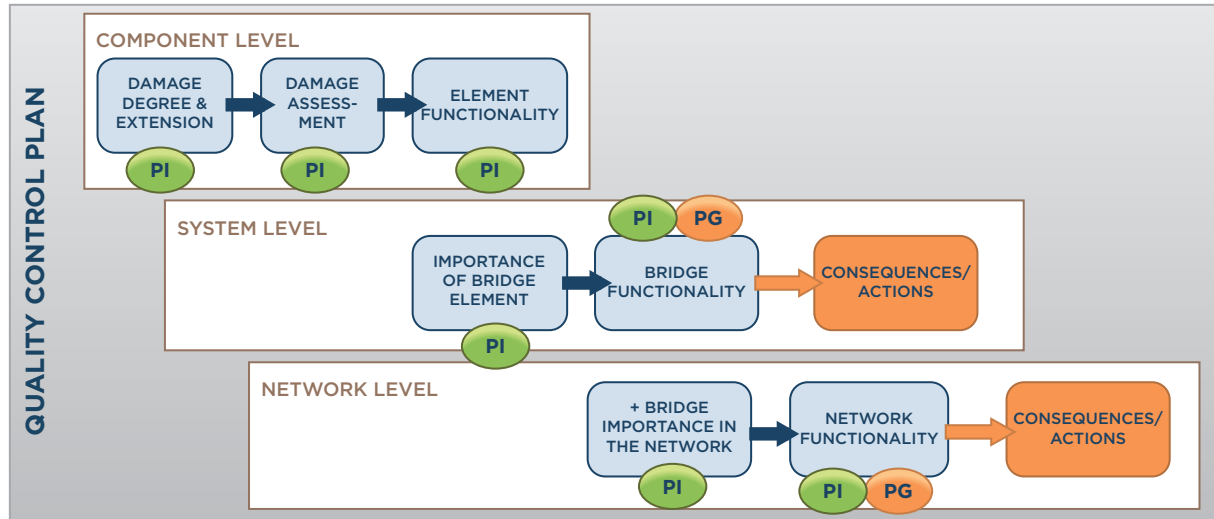


Figure 2. Assessment procedure from component to the system and network level based on the PIs and PGs.

(COST TU 1406, www.tu1406.eu)

2.3.1 Reliability

Bridge inspections are generally carried out on the bridge component level, as shown in Figure 2, and often are divided subsystem groups, i.e. substructure, superstructure, roadway. In WG 1 report (2016) the examples of bridge components related to subsystems are given, and can be different from country to country. Through bridge inspections a number of PIs are collected and analyzed (processed), in order to determine the aggregated KPI at the system level. Those PIs are usually related to the technical aspects of the bridge performance, mostly defined as structural performance or reliability. When the PIs are transformed from component (bridge) to the system (network) level, the importance of the bridge element for bridge functionality has to be taken into account. For regular (smaller) roadway bridges the assessment is usually done based on the condition indices, finally delivering an aggregated Bridge Condition Index (BCI). The index is calculated based on the condition of the bridge's structural elements and the service provided by the bridge. Historical records of BCI should help track the general system condition over time, evaluate the benefits of an agency's bridge maintenance and rehabilitation programs, and serve as a basis for allocating resources to bridges within a network.

The increased availability of element-level inspection information has influenced the redevelopment of BCIs used around the globe. Currently, most BMSs rely on element-level information for calculating BCIs. Based on the computational approach used, current methods for developing condition indices that can be grouped into the following four approaches (Chase et al. 2016):

- Ratio-based methods assign a BCI or bridge condition number (BCN) based on the ratio of the current condition to the condition of the structure when it was new. The objective for this method is to calculate the remaining value of the bridge.
- The weighted averaging approach is suitable for planning bridge maintenance and rehabilitation activities. The approach estimates the condition of the whole structure by combining condition ratings of all individual bridge elements weighted by their significance or contribution to the structural integrity of the bridge. This approach is common in systems that rely on element-level inspection data. BCIs used in Australia (BCN), the United Kingdom (BCI), South Africa (BCI), and Austria (BCI) are the examples of weighted combination approaches discussed in the report of Chase et al. (2016).

- The worst-conditioned component approach is common in systems that carry out inspections on key bridge components. This method is used to extract the critical defects in bridge components. In this approach, the BCI is approximated to the rating of the component in the worst condition. Some States also use the worst (lowest) National Bridge Inventory (NBI) rating to report bridge conditions at performance dashboards. The Michigan Department of Transportation uses the lowest NBI rating in its Bridge Condition Forecasting System (BCFS). BCFS helps Michigan with bridge project selection decisions. The German and Japanese BCIs are the examples of this approach and are discussed in the report of Chase et al. (2016).

- Qualitative methods do not report the condition of the bridge on a numerical scale. They describe a structure as either “Poor,” “Fair,” or “Good,” based on the condition state and importance of the elements under investigation. In the United States, Washington, Florida, and other States use NBI condition ratings to classify bridges as “Good,” “Fair,” or “Poor.” The Bridge Health Indicator used by Roads and Maritime Services (merger of Roads and Traffic Authority and New South Wales Maritime) in Sydney, Australia, is discussed in this report as an example to highlight the use of qualitative methods in the assessment of overall bridge health.

The aggregated BCI is then used as KPI for reliability performance aspect assessment. The threshold value or minimum required value for BCI is defined by the bridge owner.

When larger bridges or bridges with higher importance on the network are assessed, i.e. landmark bridge, the probabilistic models are applied to determine bridge reliability index, β . Actually, the existing bridge evaluation should be carried out, not only as the result of periodic inspection on the base of subjective evaluation of actual bridge condition, but from the viewpoint of the bridge reliability, i.e. from the viewpoint how the actual bridge condition affects the bridge reliability for remaining bridge lifetime.

The reliability level for the newly designed bridges for the total design lifetime T_d , which is represented by probability of failure P_{fd} or by reliability index b_d , is given in Eurocode (2009). However, the reliability level for evaluation of existing bridges for remaining lifetime t_r is not defined in the Eurocodes. Since 2016 fib has established large international Committee in order to develop a new Model Code 2020 for Concrete Structures combining both new and existing structures (see <http://www.fib-international.org/>).

There is significant research effort focussed on the development of reliability assessment frameworks, such as in EU project Destination Rail, see www.destinationrail.eu. The process for reliability assessment starts with the determination of the critical limits states. This should be performed by a competent engineer using advanced analysis techniques. Probabilistic modelling need not be performed at this stage. The next step is to define the required reliability index. The overview of some standardized values is given in chapter 3. Stochastic modelling of the load and resistance variables is then performed, which can be based on reported probabilistic models in the literature, as a first pass. Next, the reliability index is calculated for the system in question, for each limit state assessed. This should ideally be done with a combination of simulation, FORM and/or SORM analysis, as described in Report Deliverable 2.1; 2017 (<http://www.destinationrail.eu/documents>). If the reliability index and/or risk be found to be within acceptable bounds, a sensitivity analysis should always be performed to identify critical variables and ensure that the analysis is not overly sensitive to any variable, as this can indicate an overly simplistic analysis. If the reliability is found to be unacceptable, the analysis should be performed again, incorporating additional information. This is where the SHM data should be used for the improvement of the models. Consideration of monitoring such as strain measurements, vibration, water content or site testing allows a much more accurate calculation of the reliability, and will often show the infrastructure to have significantly greater capacity than previously calculated. The information can be considered stochastically in the analysis using maximum likelihood estimation of distribution parameters or Bayesian statistical updating of the previously calculated statistical models. If the risk is still found to be unacceptable, repair and remediation strategies need to be considered in the reliability domain to optimise their performance.

At present, Infrastructure Managers make safety critical investment decisions based on poor data and an overreliance on subjective visual assessment. Therefore, their estimates of risk are highly question-

able. The probabilistic methodology developed for hazard assessment allows an accurate calculation of the failure probability which, when applied alongside the detailed whole life cycle costing provides a much more reliable calculation of risk.

European bridges are more frequently exposed to natural hazards, such as scour caused by flooding, earthquakes or the combined effect of both. Although hazard resilience is a very important aspect for management and maintenance planning of the bridge network, consensus metrics do not currently exist for it. Research programs are actively working on defining metrics and acceptable thresholds to address bridge hazard resilience aspects. Based on recent research efforts (SMART RAIL, DESTination RAIL) resilience assessment can be performed based on the fragility curves which are representing probability of failure as a function of the intensity of the hazard. The advantage of the approach is that different limit states can be considered simultaneously.

2.3.2 Other performance goals

When the reliability level is in a deteriorated state, maintenance options need to be considered for each bridge below the threshold. Usually three main options are considered, from do nothing, to minor repair and finally major repair or reconstruction. The chosen maintenance option will have direct and indirect impacts, such as direct costs related to the maintenance activity and indirect costs caused by maintenance activities and borne by the society, such as user delay and environmental impacts. The direct impacts are regularly calculated as owners cost and will represent economy performance aspect of the bridge. Other impacts are in our approach finally categorized as availability and environmental aspects.

Traffic safety is also one of the other performance aspects, which can be quantified at two periods, one during the regular operation and the other during maintenance activities. The overview of suggested KPIs for each performance aspect is given in Figures 3 and 4. The methodology how each of proposed KPIs can be quantified is explained in following chapters.

Multiple bridge performance goals should be set as multi-objective system, taking into account different aspects of bridge and network performance. In our approach we have finally five performance aspects:

- Reliability
- Availability
- Economy
- Environment
- Traffic Safety

Multi-criteria decision-making (MCDM) provides a systematic approach to combine these inputs with benefit/ cost information and decision-maker or stakeholder views to rank the alternatives. Different methodologies are presented in Chapter 5. Hierarchy structure for linking multi-objective bridge performance goals, covering most of the previously mentioned aspects with performance indicators is shown in Figure 3.

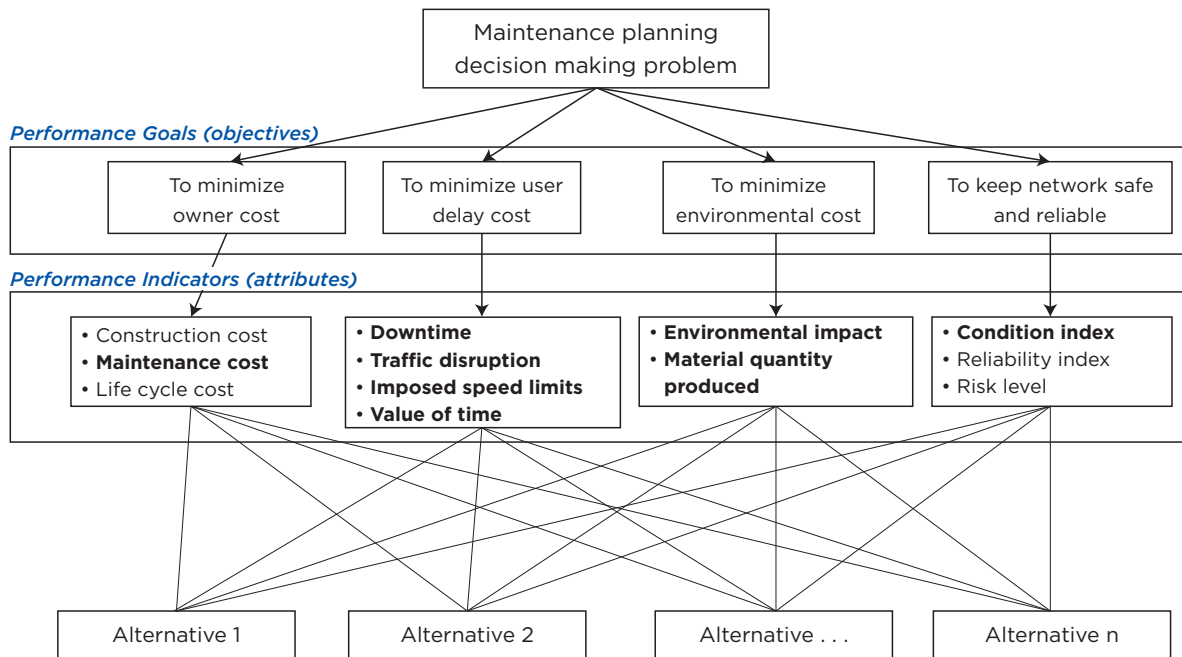


Figure 3 Linking multiple performance goals (called multiple objective) to performance indicators (adapted from Rashidi (2011))

2.3.3 Framework for KPIs assessment

In Figure 4 the process of the multiple performance goals assessment is presented, also showing the transition from a system to the network level.

3 RELIABILITY

3.1 Structural Performance assessment

Bridges form an integral part of critical infrastructure, facilitating mobility, social and economic development by enabling the free movement of people and goods (Imhof, 2004; fib, 2013). The life span of a bridge comprises a function of diverse factors such as materials, environment, level of use, and level of maintenance. In most European countries, agencies classify bridges using data gathered through bridge inspections based on their condition, which then provide basis for prioritization of the bridge maintenance planning.

3.1.1 Performance Goals

Safety aspects for existing structures are provided in national and international standards and recommendations (Diamantidis and Bazzurro, 2007), including the guidelines of the American Concrete Institute (ACI, 2003), the recommendations by the Joint Committee on Structural Safety (JCSS, 2001), as well as the Swiss note SIA (1994). Required structural performance is usually related to the goals of structural safety and serviceability, or expressed as a target reliability, evaluated on the component or the system level. Indicators relating to structural performance in the context of safety, serviceability and durability often come with explicit definitions in relevant standards and codes of practice (Dette & Sigrist, 2011). However, a large disparity is noted within Europe regarding the way performance indicators are quantified and with respect to the specification of goals.

3.1.2 Assessment at the Ultimate Limit State

As elaborated upon in (Bouassida et al., 2010), ultimate limit states are defined in EN 1990:2002 as limit states that concern the safety of people, and/or the safety of the structure. The Eurocode foresees six ultimate limit states, with three of these (EQU, UPL and HYD) pertaining to stability, and three (STR, GEO and FAT) linked to resistance. The level of safety is usually determined at the ultimate limit state. It is typically verified by two alternative types of performance measures: by the relation of the design resistance to the corresponding sectional design force, or alternatively by the reliability index associated to the corresponding limit state function.

As part of the workings of WG1 (COST TU 1406, 2016; Strauss et al. 2016), an exhaustive database has been compiled, where a number of PIs are defined that are linked to the goal of safety on the component, system and network level. As an example, crack pattern, length and formation stage form PIs that are rendered via inspection at the component level, and relate to safety with respect to defects. On the other hand, PIs that correspond to the system level include the safety and reliability indices, while risk and the importance of the bridge in the overall network comprise indicators at the network level. Below follows a description of the two major fronts for assessment with the goal of safety, namely the deterministic and probabilistic front.

Deterministic Assessment

For the first class, in its simplest form a safety factor may be defined as (Heidkamp and Papaioannou, 2011):

$\frac{R}{\gamma} \geq S$ where R denotes resistance, S designates the action and γ is an empirically determined global safety factor, or alternatively a more refined version may be adopted as

$\frac{R_k}{\gamma_R} \geq S_k \cdot \gamma_S$, where the partial safety factors γ_S, γ_R are introduced. These are calculated on the basis of a prescribed probability of failure $P_{\sigma'}$, rendering this approach a semi-probabilistic one.

Performance measures may be defined according to the specific target at hand, as for instance fatigue performance (see Table 4), and they draw from the indicators that have been aggregated in the database provided by Working Group 1 of the Action.

Table 4. Performance Measures for the Ultimate Limit State

Goal	CL: Component Level SL: System Level	Performance Measures	Comments
Fatigue Safety	CL	Fatigue Safety Level $\mu_{fat} = \frac{\Delta\sigma_c / \gamma_{Mf}}{\gamma_{Ff} \cdot \Delta\sigma_{E,2}}$	γ_{Ff} , $\Delta\sigma_{E,1}$, $\Delta\sigma_c$ and γ_{Mf} are given in current codes & standards, such as (IIW, 2003).
Fatigue Safety	CL	Damage Accumulation $D = \sum \frac{n_i}{N_i} \leq 1$	n_i : #cycles at stress range $\Delta\sigma_i$ N_i : #cycles at fatigue strength for stress range $\Delta\sigma_i$
Fatigue Safety	CL	Stress Intensity Factor (from Fracture Mechanics) $\Delta K = Y \cdot \Delta\sigma \cdot \sqrt{\pi \cdot a}$ Fatigue life $N = \int_{a_0}^{a_c} da / C \Delta K^m$	see (Gurney, 1968) for further details
Structural Adequacy	CL/SL	$\gamma_G G_k + \gamma_Q Q_k \leq R \left(\frac{X_k}{\gamma_M}, a_{nom} \right)$	see (Imhof, 2004; EC 0, 2001) for further details
Load-Resistance Factor Design (LRFD)	CL/SL	$\sum_i \eta_{Di} \eta_{Ri} \eta_{Li} \gamma_i Q_i \leq \phi R_n$	AASHTO (1998) takes ductility, redundancy and the operational importance into account (Imhof, 2004)

Probabilistic Assessment

As pointed out in (Heidkamp and Papaioannou, 2011) the end goal in structural assessment lies in control of the inherent uncertainties, which may be attributed to the variation of material properties with environmental effects and deterioration processes; the hardly measurable inputs (loading); the construction and manufacturing processes themselves; as well as inadequacy of the employed structural models. In view of the above, it becomes evident that a probabilistic approach is necessary. To this end, the Joint Committee on Structural Safety, JCSS, has developed a full probabilistic model code [JCSS, 2001].

In contrast to the deterministic approach, which accounts for uncertainties through the use of partial safety factors, the JCSS code adopts probabilistic distributions to quantify uncertainties. Therefore a prescribed probability of failure is set as:

$P_f = P\{R - S \leq 0 < P_d\}$, on the basis of which the reliability index β is introduced to quantify safety, and a threshold in the form of a target reliability index β_t is set for assessment. As illustrated in Figure 5, the reliability index may be interpreted as a measure of the distance from the most likely realization point to failure, and is typically computed through First and Second Order reliability Methods (FORM/ SORM) or numerical simulation methods (e.g. Monte Carlo)

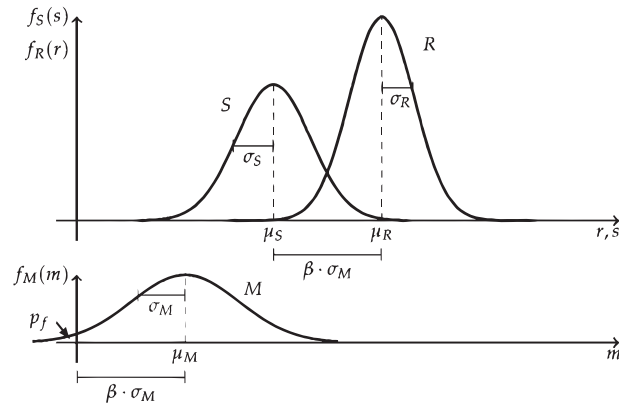


Figure 5. Derivation of the reliability index when R and S are independent and normally distributed, with $M = R - S$ denoting the safety margin. Figure reused from the PyRe 5.0.2 documentation.

The obtained reliability levels depend on the age of the bridge and on the planned remaining lifetime. These may be prescribed from the time of inspection t_{insp} to the end of the lifetime T , or it is further possible to define these with respect to shorter intervals, as for instance the time between inspections (Koteš and Vičan, 2012).

Table 5. Recommended target reliability indices for structures to be designed, related to the specified reference periods at the ultimate limit state.(fib Model code for concrete structures 2010)

Ultimate Limit State	Target β	Reference period
Low consequence of failure	3.1	50 years
	4.1	1 year
Medium consequence of failure	3.8	50 years
	4.7	1 year
High consequence of failure	4.3	50 years
	5.1	1 year

The probabilistic assessment is not limited to the aspects of reliability, but further extends to risk-based design yielding further PIs, such as for instance the robustness index, that according to Baker et al., could be defined as:

$I_R = \frac{R_D}{R_{ID} + R_D}$, where R_D is the risk due to direct consequences, linked to damages in the constituents of the system for a given exposure loading event, and R_{ID} corresponds to the risk due to indirect consequences. Further risk indicators may be defined on the basis of the loading of the structure (exposure), the strength of the components of the structure (vulnerability) and the redundancy, ductility, effectiveness of condition control and maintenance (robustness) (Faber, 2009).

3.1.3 Assessment at the Serviceability Limit State

Serviceability limit states are defined in EN 1990:2002 as those limit states that concern the functioning of the structure or structural members under normal use; the comfort of people; or the appearance of the construction work. EN 1990:2002 poses a further distinction between reversible (e.g. elastic) and irreversible (e.g. plastic) serviceability limit states (Bouassida et al., 2010).

The serviceability, or working condition, of the bridge is normally verified by visual inspection or monitoring, which should deliver quantitative indicators such as stresses in concrete and steel, crack widths, deflections and vibrations calculated for service (operational) loads. Use of such information allows for adoption of similar assessment procedures to those implemented in the ULS in terms of performance measures. For instance, the reliability index may be calculated for the serviceability limit state, as summarized in Table 6.

Table 6. Recommended target reliability indices for structures to be designed, related to the specified reference periods at the serviceability limit (fib Model code for concrete structures 2010)

Serviceability Limit State	Target β	Reference period
Reversible	0.0	Service life
Irreversible	1.5	50 years
Irreversible	3.0	1 year

3.1.4 Methods of Structural Assessment

An obvious bottleneck in the procedure foreseen by codes and standards is the treatment of either inadequate or subjective information (e.g. on critically of defects to the overall capacity). An objective assessment need instead be put in place, which largely relies on the availability, ease of implementation and resolution of available monitoring and inspection methods. The information stemming from monitoring and inspection may be exploited in three major ways:

- i) For direct measurement of PIs, where this is feasible (e.g. use of strain gauges for monitoring cracking).
- ii) For the updating and calibration of structural models to reflect the true state of the bridge system. Recalculation of the relevant indicators is then possible with increased confidence.
- iii) For the extraction of novel data-driven (or information-driven) condition indicators. This is particularly relevant in the case of information obtained from monitoring systems (Limongelli et al. 2015)

Visual Inspection

Visual inspection forms the “de-facto” tool of structural assessment in both Europe and the rest of the world. In the following figures and tables a background of the existing Bridge Management Systems is given which was originally published within ASCAM project in 2012.

The Austrian evaluation system of the maintenance condition, implemented in the new guidelines for the condition assessment of road bridges is based on the rating system with 5 condition levels, presented in Table 7. The evaluation of the object is performed according to the Table 8.

There is no clear correlation between component (element) condition rating and object (structure as a whole) condition rating, which means in the end there is no objective way of checking the final condition grade of the object. The influence of the importance of a certain component on structural stability, safety and durability is not defined, as well as the influence of the amount of certain condition caught on certain component. This means the final evaluation of the object as whole needs to be performed from really experienced bridge engineer, who can prove his decision about certain rate.

Table 7: General rating for the condition in Austria [RVS 13.03.11]

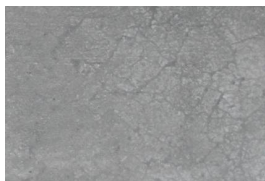


Grade	Condition
1	Very good
2	Good
3	Satisfactory
4	Faulty
5	Bad



Table 8: Object (structure as whole) evaluation system [RVS 13.03.11]

Grade	Description
1	No or very minor damages. Faults stem from the construction phase, e.g. from discrepancies in dimensions, aesthetic flaws. No reduction in load bearing capacity, serviceability or durability. No maintenance measures required.
2	Minor, light damages; faults stem from the construction phase and show no signs of deterioration. No reduction in load bearing capacity and serviceability. In case of no intervention, limitations to the serviceability/durability will only arise in the long-term. Corrections recommended in the course of regular maintenance or repair works.
3	Medium damages which do not affect the load-bearing capacity. Signs of a reduction in serviceability and durability can be found. Maintenance should take place in the mid-term in order to raise the serviceability and durability back to desired level.
4	Heavy damages which currently do not affect the load-bearing capacity. A reduction in serviceability and durability is clearly noticeable. Repair measures should be planned in the short-term in order to raise the serviceability and durability to the desired level. Maintenance interventions can be substituted by another assessment/special assessment within a prescribed deadline (shortening the inspection interval).
5	Very heavy damages which result in a reduced load-bearing capacity and/or serviceability unless renewal/repair takes place. Repair/renewal works should be initiated immediately.

In Croatian asset rating system is based on degradation categorization given in Table 9. Final categorization is expressed in percentage of the entire surface area of certain element, and indicates how much surface of the concerned element should be repaired and to what extent based mainly on the visual inspection and no quantitative assessment of the bridge condition as a whole.

Table 9: Croatian condition rating system

Damage category	Type of damage	Main performance indicators	Example
0	No damage.		
I	Smaller defects resulted from the construction process.	<ul style="list-style-type: none"> - Surface imperfections - Small cracks (shrinkage cracks) 	
II	Smaller defects resulted from the exploitation.	<ul style="list-style-type: none"> - Surface cracks - Delamination of surface cement paste film - Evaporation of $\text{Ca}(\text{OH})_2$ 	
III	Defects that in long term decrease durability of the structure. Repair is needed.	<ul style="list-style-type: none"> - Network of cracks in concrete cover - Contamination of concrete cover (chloride, pH) - Concrete loss due to frost and de-icing salts damage 	

IV	Defects that can, in the foreseeable future, decrease the reliability of the structure. Repair is needed now.	<ul style="list-style-type: none"> - Delamination, spalling of concrete cover (partially) - Honeycombs in concrete - Corrosion of steel visible - Loss of steel cross section due to corrosion 	
V	Defects that present a serious danger for safety of the structure. Intervention is needed emergently, and if necessary limitation or shutdown of traffic.	<ul style="list-style-type: none"> - Delamination and spalling of concrete cover (full) - Advanced corrosion of steel, - Significant loss of steel cross section 	

Slovenian National Building and Civil Engineering Institute started to develop BMS (named MOST) at the beginning of '90s, and the system is in operation from 1996. Beside the manual and guideline for the usage of MOST, Slovenia has no official regulation for carrying out the inspections and procedures for the assessment of the condition of existing bridges.

The condition rating of the bridge is performed in a quantitative form. Final assessment code is given in Table 10. Bridge condition is calculated as a sum of individual elements damage rating:

$$R = \sum RF_i \quad (\text{Eq. 3.1})$$

Table 10: Bridge condition rating [MOST, Slovenia]

Condition class	Definition	Condition rating R
5	Very good	$0 < R < 5$
4	Good	$1 < R < 15$
3	Satisfactory	$10 < R < 30$
2	Bad	$20 < R < 50$
1	Critical	$R > 40$

Individual elements damage rating is calculated as follows:

$$RF_i = B * K_1 * K_2 * K_3 * K_4 \quad (\text{Eq. 3.2})$$

Where individual factors mean:

- B - type of damage, in the range of 1 to 5
- K_1 - importance of the defect for the particular element (0.3, 0.7, 1.0)
- K_2 - damage level (0.4, 0.6, 0.8, 1.0) corresponding to (I, II, III, IV)
- K_3 - damage extend (0.5, 0.8, 1.0) corresponding to (A, B, C)
- K_4 - seriousness (threat) of the damage to the element (1, 3, 5, 10)

Examples for (K_2)

- cracks in concrete

Crack width (mm)	0.1-0.2	0.2-0.5	0.4-1.0	>1.0
K_2	0.4	0.6	0.8	1.0

- reinforcement corrosion

Corrosion (%)	0-2	2-10	10-20	>20
K_2	0.4	0.6	0.8	1.0

Defect extend (K_3)

(%)	do 10	10-40	>40
K_3	0.5	0.8	1.0

In Norwegian BMS there are 4 levels (from 1 to 4) of condition rating and this does not include no damage.

Defects are categorized by using a system of letters and numbers meaning:

M=Environment

B=Load capacity

T=Traffic safety

V=Maintenance cost

This is combined with a number meaning:

1=Small damage, no repair needed

2=Medium damage, Repair needed in 4-10 years

3=Large damage, Repair needed in 1-3 years

4=Critical damage, Repair now

Then these two are combined two for all the bridge elements. An example: V3 means damage with consequence for maintenance cost, need repair within 1-3 years.

The element condition is related to the number of years before maintenance is needed, and the condition rating is not levelled. The structure condition is quantified by calculating a character using condition from the element condition.

In French BMS there are 5 levels in condition rating on the element level; 1, 2, 2E, 3 and 3U. Each element's part is evaluated and classified (1,2,2E,3 or 3U). During the inspection of an element all defects are listed and each of them includes a classification of the element. The final class is the maximum of all the defects. For the structure level condition rating number of levels and categories are the same as for element level: 5 levels, 1, 2, 2E, 3 and 3U, and the structure level is the maximum of all element levels.

There is a catalogue of defects for each kind of bridge, and defects are according to the IQOA Methodology. Depending on the nature of the defect and of the structure, defect is appointed according to either the material or the structure.

In Table 11 information presented in the above tables about bridge condition rating is summarized in one table for different countries.

Table 11: Bridge condition rating in different countries

Different bridge condition rating in different countries									
Austria		Croatia		Slovenia (MOST)		Norway		France	
Grade	Condition	Class	Condition	Class	Condition	Class	Condition	Class	Condition
1	Very good	0	No damage	5	Very good	1	Small damage	1	Good overall state
2	Good	I	Smaller defects from construction period.	4	Good	2	Medium damage	2	Minor structural damage. Non urgent maintenance needed
3	Satisfactory	II	Smaller defects from exploitation period.	3	Satisfactory	3	Large damage	2E	Minor structural damage. Urgent maintenance needed.
4	Faulty	III	Defects that in long term decrease durability	2	Bad	4	Critical damage	3	Structure deterioration. Non urgent maintenance needed
5	Bad	IV	Defects that in foreseeable future can decrease reliability	1	Critical	Defects are categorized by using the following system of letters and numbers and combined with the above classes: M=Environment, B=Load capacity, T=Traffic safety, V=Maintenance cost.			
		V	Defects that present serious danger to safety of traffic						
No clear correlation between component (element) condition rating and object (structure as a whole) condition rating.		Structure level condition is not determined from the above classes but from the influence of each elements functionality on traffic safety, mechanical resistivity, stability, durability and general condition of element. Structural level is then determined by combining maximum elements level grades.		Bridge condition is calculated as a sum of individual elements damage rating.		The element condition is related to the number of years before maintenance is needed, and the condition rating is not levelled. The structure condition is quantified by calculating a character using condition from the element condition.		For the structure level condition rating number of levels and categories are the same as for element level: 5 levels, 1, 2, 2E, 3 and 3U, and the structure level is the maximum of all element levels.	
PREDICTION OF BRIDGE AGEING									
Probabilistic (details given in Task 1 report).		None		None		None		Probabilistic in nature (Poisson's function is used)	
Grades/Classes and conditions are not comparable between themselves for different countries in this table.									

As a further example in Switzerland a 5 scale rating is adopted in terms of 1:good, 2:acceptable, 3:deteriorated, 4:bad and 5:alarming. This may be translated into an “equivalent age” (Adey et al., 2006) for the bridge system, which is further linked to intervention and maintenance costs.

The frequency of such inspections, as further reported in the WG1 report (COST TU1406, 2016) the basis of any kind of assessment is a detailed inspection process. Such inspections may be classified into four main categories:

- Visual inspections, e.g. yearly basis.
- Simple checks, e.g. 3 years after every main inspection.
- In-depth examinations or main inspections, e.g. every 6 years.
- Special inspections, following exceptional occurrences or incidents.

Destructive Testing

Detailed structural inspections may be carried out, by extracting samples from the operating system, in order to conduct laboratory tests (e.g. strength of materials, corrosion ingress) that provide information on structural integrity. Core samples, half-cell and structural integrity testing may be conducted at this level, albeit with an invasive character.

Non-destructive Testing (NDT)

NDT methods aim in providing information on structural performance, without harming the structure itself, i.e., in a non-invasive manner. NDT technologies may be applied on components of the bridge system, often as a result of the inspection process, for detection of a flaw or defect, or when deterioration is suspected. A wide variety of non-destructive technologies are available for bridge structure. As an example, the Ground Penetrating Radar (GPR or géoradar) methodology is able to map information on concrete elements (rebar, post-tensioning and fiber optic cables, conduits); highway elements (voids, pipes, pavement thickness) reporting in this way the soundness of individual structural components. It may further be used for the detection of zones with increased chloride contents and moisture, in this way providing a warning on deterioration through corrosion. Thermographic methods may also be adopted for similar purposes. Acoustic emission comprises a further NDT methodology for the tracking of initiation of destructive processes (e.g. cracking) and their propagation inside a structure. Similarly to the principles of acoustic emission the magnetic flux leakage method has also been used for structural/mechanical damage detection, with potential in the detection and localization of fracture of reinforcing steel bars. This list is not exhaustive and the interested reader is refer to the work of (Ayswarya et al., 2016).

Proof-Level Load Testing

Load testing may be exploited when it is critical to assess the current condition of an existing system in the serviceability or ultimate limit state. Static tests using loaded trucks constitute one of the standard means of structural testing for determining structural capacity. However, such tests come with severe shortcomings, associated not only with their usually prohibitive cost but additionally with practical implementation details. Such details are related to the maximum size of truck, limitations in the maximum load these are allowed to carry, the rather long interval within which the operation of the bridge needs to be suspended and so forth. Oftentimes, these tests are conducted for fully operational specimens where the desire is to quantify the deformation of the bridge under increasing static load without necessarily going into the nonlinear range, thereby possibly damaging the structure. In the context of this project an alternative approach is suggested for obtaining an estimate of the deflection of the bridge when discussing the linear load range. Such a procedure may provide significant information on how the capacity of the bridge complies with the estimations of the design process (BD 21/01).

Structural Health Monitoring (SHM)

The use of monitoring systems for bridge structures is lately becoming more and more established (Wenzel, 2009). The implementation of such systems can be classified in two main categories depending on the duration of the instrumentation, which may vary from short term (typically up to few days) or mid-term (few days to few weeks), to long term (few months to few years), and perhaps throughout the lifespan of the structure (Glisic et al., 2007). A noteworthy example of short term monitoring for condition assessment and immediate decision making processes is the non-destructive dynamic field testing (from vibration response data) conducted in three Cincinnati bridges for the rating of those specimens (Aktan et al., 1994). The testing methods utilized in that case included impact tests as well as proof-load level truck-load tests.

A main issue in damage identification and condition assessment through monitoring data is the fact that environmental effects also play a major role in the properties of the system. In this sense, long-term monitoring (from cradle-to-grave) is advisable for continually tracking the evolution of the system's properties under environmental, operational and deterioration effects. Long-term monitoring systems have already been implemented on a number of bridges in Europe (Casciati, 2003) the United States (Pines and Aktan, 2002) and elsewhere. An example of a state-of-the-art implementation scheme is the long term monitoring system deployed on the Tsing Ma bridge in Hong Kong Chung et al. (2003), involving a network of more than 350 sensor channels including GPS and Fiber Bragg Grating (FBG) sensors. A further pioneering monitoring initiative is the one initiated by the Californian Department of Transportation (Caltrans) and the California Strong Motion Instrumentation Program (CSMIP) for instrumenting Caltrans bridges throughout the state, recording their response during earthquakes. This data is assimilated with a larger data stream from further infrastructure components, for identifying the areas of greatest potential damage for use by the Office of Emergency Services and other emergency response personnel in the event of a damaging earthquake.

Although still relatively rare, such schemes are becoming more and more available. As the necessary technology becomes increasingly cheaper and software systems become more and more spread, such schemes are envisioned as the future of monitoring, eventually to be required by code to accompany traditional assessment methods such as visual inspection.

3.1.5 Combined Assessment – KPIs

An obvious challenge in what is outlined above lies in the extraction of indicators at the system level, since many of the performance measures are defined on the level of individual components and across different limit states. Summing, averaging or maintaining the most adverse of ratings amongst components could be an option, however no uniform approach is recommended across standards.

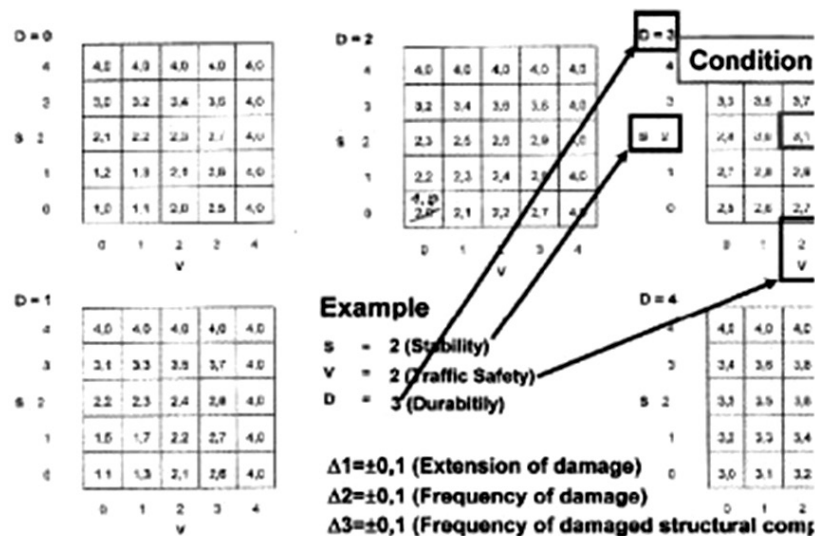


Figure 6. SIB-Bauwerke Condition Index Extraction (reused from Bridge Evaluation Quality Assurance in Europe, US Department of Transportation, March 2008)

For example, in Germany, assessment is facilitated via a dedicated computer program, SIB-Bauwerke Release (Figure 6). The inspector yields an assessment on the influence of a defect or damage with respect to structural stability, traffic safety, and durability on separate grade scales from 0 to 4. On the basis of these ratings, the software calculates a condition index for the structural element or structure, which further offers guidance with respect to the maintenance and repair actions that may be required (Table 12).

Table 12. Recommended target reliability indices β for structures to be designed, related to the specified reference periods at the serviceability limit state.

Grade	Description
1.0–1.4	Very good structural condition
	The structural stability, traffic safety and durability of the structure are given. Continuous maintenance is required.
1.5 –1.9	Good structural condition
	The structural stability and traffic safety and durability of the structure are given. In the long term, the durability of the structure may be negatively affected to a small degree. Continuous maintenance is required.
2.0–2.4	Satisfactory structural condition
	It is possible that, in the long term, the durability of the structure may be negatively affected. An expansion of the damage or consequential damages which, in the long term, would lead to considerable deterioration of the structural stability and/or traffic safety and increased wear and tear is to be expected. Continuous maintenance is required. Maintenance is required in the medium term. Measures to eliminate the damage or warning signs to maintain traffic safety might be necessary at the short notice.
2.5–2.9	Unsatisfactory structural condition
	The structural stability of the structure is given. Traffic safety might be negatively affected. The durability of the structure may be negatively affected quite a bit. An expansion of the damage or consequential damages which, in the medium term, would lead to considerable deterioration of the structural stability and/or traffic safety and increased wear and tear is to be expected. Continuous maintenance is required. Maintenance at short notice is required. Measures to eliminate the damage or warning signs to maintain traffic safety might be necessary at short notice.
3.0– 3.4	Critical structural condition
	The structural stability or traffic safety of the structure is negatively affected. Possibly, durability of the structure is no longer given. An expansion of the damage or consequential damages may, in the short term, lead to the fact that structural stability and traffic safety are no longer given. Continuous maintenance is required. Immediate repairs are required. Measures to eliminate the damage or warning signs to maintain traffic safety or restrictions in its use might be required as soon as possible.
3.5 – 4.0	Inadequate structural condition
	The structural stability or traffic safety is negatively affected quite a bit or is no longer given. Possibly, durability of the structure is no longer given. An expansion of the damage or consequential damages may, in the short term, lead to the fact that structural stability and traffic safety are no longer given and that it will result in an irreparable deterioration of the structure. Continuous maintenance is required. Immediate repairs or renovations are required. Measures to eliminate the damage or warning signs to maintain traffic safety or restrictions in its use might be required immediately.

As a further example, this time from the UK, the BridgeStation software, made available by the London Bridges Engineering Group (LoBEG) allows for the recording of inspections and the automatic generation of condition indicators. There, Element condition indices may be entered as a result of the inspection procedure, with the Bridge Condition Index calculated on the basis of all recorded conditions on so-called “critical elements”, by propagating the most adverse scenario.

As noted in (Wenzel, 2009), monitoring information may also be incorporated in assessment systems. As an example, the BRIMOS® (Bridge Monitoring System) rating is a classification based on the assessment of 1000 structures on the basis of their vibrational signature, as obtained via monitoring cam-

paigns. Structural integrity and the corresponding risk level are assessed based on measured dynamic properties (eigenfrequencies, mode shapes, vibration intensity and static as well as dynamic vertical displacements), visual inspection, finite element model updating and reference data. The result is a factor (index), related to a predefined risk level.

However, indicators (PIs) may also be regarded individually in the decision-making process via appropriate methods, such as multi-criteria analyses (Chapter 5), Decision/Event Trees, Markov-Decision Processes (Masovic et al. 2015), or Bayesian Decision Analysis methods (prior, posterior, or pre-posterior analysis (Gouleta et al. 2015)).

3.2 Seismic assessment

Seismic assessment of existing bridges in Europe often requires dealing with structures that have been not been designed for seismic prone areas (due to a previous different mapping of the seismic hazard at the site) or have been dimensioned according to old design codes based on approaches not anymore consistent with the current requirements.

In the following is reported a brief overview of the philosophy underlying the definition of the performance goals together with a discussion of the two approaches (Force-Based and Displacement-Based) currently used for the seismic assessment of bridges. The link between the performance parameters describing the current state of the structure and the performance goals is also briefly discussed.

As will be discussed the KPIs related to safety and reliability are considered in seismic assessment at the system or element level while availability is checked at the network level.

3.2.1 Seismic design and assessment philosophy

The philosophy underlying the design and assessment under seismic action is quite different from the approach used for more frequent actions that are not supposed to damage the structures.

Structures in seismic prone areas are required to survive small to moderate earthquakes with no or limited damages and to survive to strong seismic events, even with severe damages, to ensure the safety of users. By adopting this approach the energy imparted by the earthquake can be dissipated, thus avoiding collapse. The ductility of the bridge should be such that plastic deformation can occur without significant loss of strength. As a result of the nonlinear behavior, the structure is subjected to forces that are lower with respect to those that would act if the structure behaved linearly.

The ability of a structure to withstand a strong earthquake is thus strongly related to its energy dissipation capacity that depends on its *ductility* that is the capability to undergo large displacements in the inelastic range without sensible loss of strength. For this reason the most straightforward way to assess the capability of a structure to withstand a strong earthquake would be the comparison between the demand and the capacity in terms of ductility (see Figure 7 **Error! Reference source not found.**):

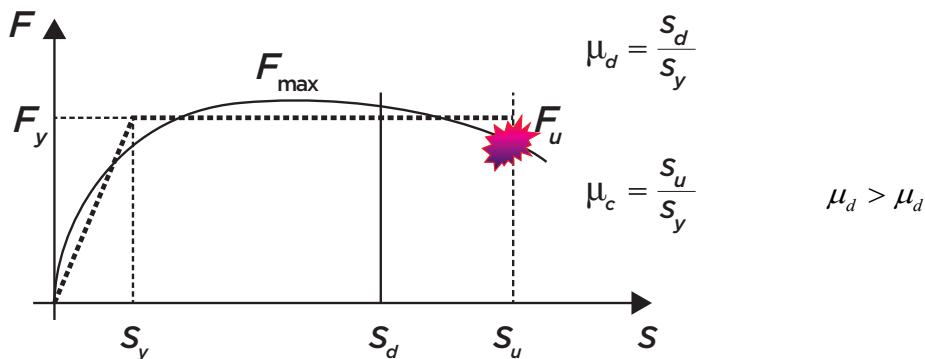


Figure 7. Displacement ductility capacity and demand

being μ_c and μ_d respectively the displacement ductility capacity and demand, s_d is the maximum displacement induced by the earthquake and s_u is the displacement at collapse.

The traditional approach to seismic design and assessment has always been 'Force-Based' that is the comparison between capacity and demand is carried out in terms of Resistance and Actions on the

structure. The design forces are computed reducing the elastic forces using ‘reduction factors’ that are related to the ductile capacity. The assessment of the actual ductility capacity, both at element and at structural level, is not carried out directly but entrusted to the compliance with a series of prescriptions. Many of these prescriptions come from the capacity design principles, introduced in New Zealand about 40 years ago (Park and Paulay, 1975), and which are today integrated in most of the design codes (EN 1998). According to the capacity design principles, the design of the structures (or the upgrading for existing ones) has to be performed so as to enhance the energy dissipation capacity. This is achieved by:

- a) allow the formation of flexural plastic hinges in order to develop a stable collapse mechanism, and thus dissipate a large amount of energy. For a bridge this means for example the formation of flexural plastic hinges in as many piers as possible and not in the deck.
- b) avoiding brittle failures (for example shear failures) by making the resistance with respect to brittle collapse mechanisms (shear failure) higher with respect to the resistance to ductile mechanisms (bending failure) or the resistance of the superstructure is lower than the resistance of the foundations.

3.2.2 Seismic performance goals

The philosophy informing the approach to seismic design and assessment outlined in the previous section is translated in the following **performance goals**: the structure must be able to withstand (ATC, 1978):

- minor or frequent earthquake shaking without damage,
- moderate levels of shaking with only non-structural damage and
- severe shaking without collapse and a threat to life safety

This philosophy (thus the definition of the performance goals) is common to both traditional *prescriptive approach* to seismic design and assessment and to modern *performance-based approaches*. The main difference between the two approaches is that in the second the performance goals are explicitly defined and assessment consists in checking that such *performances are met*. In the traditional approach specific prescriptions are enforced implicitly assuming that compliance with them ensures achieving the performance goals. In this second case the assessment consists in checking that the *prescriptions are met*.

Current design codes, namely the Eurocodes, have a mixed approach: they define performance goals in terms of Limit States but the achievement of these goals is entrusted to the satisfaction of a number of prescriptions in terms of the structural strength capacity/demand ratio or prescription regarding the member detailing e.g. the compliance with capacity design principles.

3.2.3 Traditional prescriptive approach

Many European bridges in seismic areas have been designed according to old codes based on the prescriptive approach in which the **performance goal** is the achievement of given demand to capacity ratio referred to the preservation of life and safety of the users. Other **performance goals** related to the limitation of damage, maintenance of functions or provision for easy repairs were not at all considered in this traditional approach.

The assessment is carried out based on structural reliability methods and Capacity and Demand are expressed in terms of Resistance and Actions on the structure. The Actions on the structure are fixed by the codes usually through a response spectrum.

These methods do not address explicitly the probability of occurrence of the earthquake or the costs of the consequences since these are implicitly taken into account in the definition of the actions on the structure (response spectrum) and of the behavior factor (used to reduce the actions on the structure based on its dissipation capacity).

3.2.4 Modern performance-based approach

The performance goal of the modern approach in seismic design is the achievement of a certain level of performance taking into account the related consequences. In contrast to the traditional methods, performance-based assessment should be carried out making decisions based on the desired level of performance.

As previously noted, whilst the first generation of Performance based approaches, contained for example in the current version of the Eurocodes, performance levels are defined, but the achievement of the performance goals is still entrusted to a number of prescriptions. In other words even if the performance goals are explicitly defined through the limit states, they are considered to be achieved if some prescriptions are verified. For example two performance levels are specified in the Eurocodes (EN 1998):

no collapse (at Ultimate Limit State) implying that *'the bridge shall retain its structural integrity and adequate residual resistance, although at some parts considerable damage may occur'*.

minimization of damage (at Service Limit State) implying that the seismic action *'may cause only minor damage to secondary components and to those parts of the bridge intended to contribute to energy dissipation. All other parts of the bridge should remain undamaged'*

These requirements are assumed to be satisfied if the structure complies with a number of prescriptions regarding ductility (at component or structural level), capacity design rules and values of the strength reduction factors.

In the last generation of Performance-Based Approaches, risk methods are applied considering exposure and hazard (instead of actions), vulnerability (instead of capacity) and also consequences that, in prescriptive design, are not explicitly accounted for. In the definition of the seismic hazard the probability of occurrence of the seismic event is explicitly taken into account and related to vulnerability through the fragility curves that describe the probability of a given state of the system (no damage, low damage, high damage,....collapse) as a function of the seismic hazard level. The fragility curves can be developed at the component level and at the system level and research efforts are currently being devoted to the computation at the network level. The fragility curves are then used to obtain the probabilities of different damage states (levels) and the risk is computed by considering the costs of different consequences (e.g. casualties or monetary losses) related to the considered performance level. Decisions about possible interventions of the structure are then made based on the costs of different performance levels.

Going back to the current (mixed) perspective to PBA two different approaches can be followed in the assessment of the structure: the Force-Based Assessment (FBA) and the Displacement Based Assessment (DBA). In the following sections some more details about the two approaches and the relevant methods for the assessment of the performance goals are given.

The main difference between FBA and DBA is that in the first one the assessment of the performance goals is carried out in terms of forces namely by comparing Resistance and Actions while in DBA the displacement capacity is compared to the displacement demand.

Since the structure is allowed to behave non linearly under a strong earthquake, its non-linear behavior should be necessarily considered for the computation of the capacity.

For both approaches the most suitable seismic tool to assess the seismic performance would be a nonlinear dynamic analyses using a numerical model of the structure. This type of analyses theoretically allows capturing with high accuracy the real behavior of the structure by explicitly modeling and evaluating post-yield ductility and energy dissipation under an earthquake ground motions.

Unfortunately this type of computation is not common practice yet and presents several drawbacks that make it feasible only in the case of very particular structures. First of all the uncertainties connected with **performance indicators** that affect the modeling of the structures (material parameters, state of damage or degradation, etc) may strongly affect the results as well as a high dispersion in results can be obtained changing the input chosen to model the seismic excitation. Furthermore the analyses could be quite time consuming, the extensive output can be difficult to interpret leading to neglect the most important aspect of the seismic response. Last but not least the computational cost of such analyses could be quite high for large structures and not justified by the amount of the uncertainties connected with the modeling.

For these reasons simplified methods have been proposed for both the approaches and are currently included in several design codes.

3.2.5 Force-Based Assessment (FBA)

In the FBA simplified methods based on elastic analysis (response spectrum) or on static analysis (equivalent) are allowed by the codes (EN 1998) and often employed in the case of ordinary structures.

In these simplified methods the seismic actions are obtained reducing the values of the elastic response spectra using the 'behaviour factor' which is a strength reduction factors dependent on the ductility capacity thus on the other **performance indicators** (regularity, redundancy, material,...) of the structure. The codes give prescriptions on how to compute the behavior factor q . In Figure 8 the response spectrum for two values of the behavior factor are reported ($q=1$ is the elastic response spectrum and $q=3$ the response spectrum to use for a structure with behavior factor equal to 3).

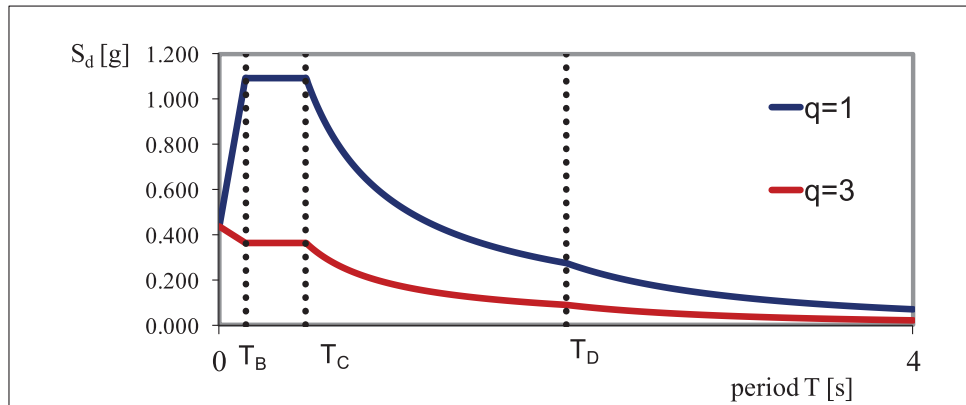


Figure 8. Response and design spectrum from EN 1998

Two most generally used force based methods for the seismic assessment of bridges, which are also implemented in several seismic codes are the:

- lateral force analysis and
- multi-modal response spectrum analysis.

When the ratio between the demand D_i obtained from the seismic load combination and the capacity C_i of the considered i -th ductile element of the considered bridge is :

$$\rho = \frac{D_i}{C_i} > 1.0, \quad (\text{Eq. 3.3})$$

as well as if the ratio does not exceed:

$$\frac{\rho_{max}}{\rho_{min}} > 2.5, \quad (\text{Eq. 3.4})$$

as suggested in the Eurocode 8/3 (CEN, 2005b), the lateral force analysis with elastic spectrum can be taken into account for the assessment of such structure.

This method can be generally applied only to regular bridges, for example when the differences in the exploited level of the capacity in different columns are small (see EC8/2 for details) or if the influence of higher modes are considerable. It is therefore rarely applicable to existing structures; especially older ones built before the modern principals of seismic engineering have been established. In such cases the multi-mode response spectrum analysis using the elastic response spectrum should be used.

In the multi-mode response spectrum analysis the natural modes of vibrations (eigenmodes), the natural periods (eigenvalues) and modal participation factors are computed at first by using modal analysis. Afterwards, the modes which significantly contribute to the global response of the bridge are identified. This limitation is in Eurocode 8/1 (CEN, 2005a) defined by the requirements, that the sum of the effective modal masses should exceed or be equivalent to 90% of the total mass of the structure as well as that all the modes having the effective modal masses greater than 5% should be considered in the response spectrum analysis. After the identification of the significant modes the results from acceleration response spectrum are combine, generally using the complete quadratic combination or CQC rule.

3.2.6 Displacement-Based Assessment (DBA)

In DBA the equivalent static nonlinear analyses (pushover analyses) is employed to take into account the nonlinear behavior of the structure avoiding a heavy computation. It should be mentioned that

Displacement-Based Assessment (DBA) procedures are now well-established for buildings (Kappos 2010), however for bridges, besides its numerous advantages, the procedure still has some disadvantages. Their application is mainly limited to bridges that can be modeled by single degree of freedom for calculating seismic demand and its applicability is in the scope of preliminary design. Reasoning behind this lies in the importance of the higher modes in the transverse response of bridges even of some relatively short ones (Paraskeva and Kappos, 2010), which complicates the proper assessment of the displaced shape of the bridge and the target displacement. In this case not one target displacement is required but a target displacement profile.

Following is reported a description and some comments about pros and cons of Displacement-Based methods for seismic assessment.

Nonlinear static analyses (Pushover) is an effective tool to evaluate the expected non-linear behavior and consequent failure pattern in different components of the bridge. ATC-40 recommends nonlinear pushover analysis procedure with various levels of the seismic demands. ATC-32 gives recommendations for the pushover analysis procedure applied to the bridge structure. This approach is recommended as well by the current design codes in Swiss as well as in the European design codes (SIA 2018 2004, Eurocode 8, 2003). Pushover method compares the earthquake's action (target displacement w_t , as indicated in Figure 7 to the deformation capacity of the structure.

As the pushover analyses takes into account the material nonlinearity of structures, it usually gives a more realistic behavior of the structure exposed to earthquake action provided the parameters of the model are reliably calibrated.

The goal of the static pushover analysis is to evaluate the overall strength, typically measured through the base shear V_b , yield, and maximum displacement δ_y and δ_u , as well as the ductility capacity μ_c of the bridge structure. The pushover analysis can examine the sequence of limit states, formation of plastic hinges, and redistribution of forces throughout the structure, with the increment of the lateral loads or displacement demand. The pushover curve (force vs. deformation) of the bridge also allows identifying any softening behavior of the entire structure due to material strength degradation or P- Δ effects.

This represents a displacement controlled method and the pushover method is conducted to a specified limiting displacement value in order to capture the softening behavior of the structure by monitoring the displacement at a point of reference, such as one of the column's top nodes or the center of the superstructure span.

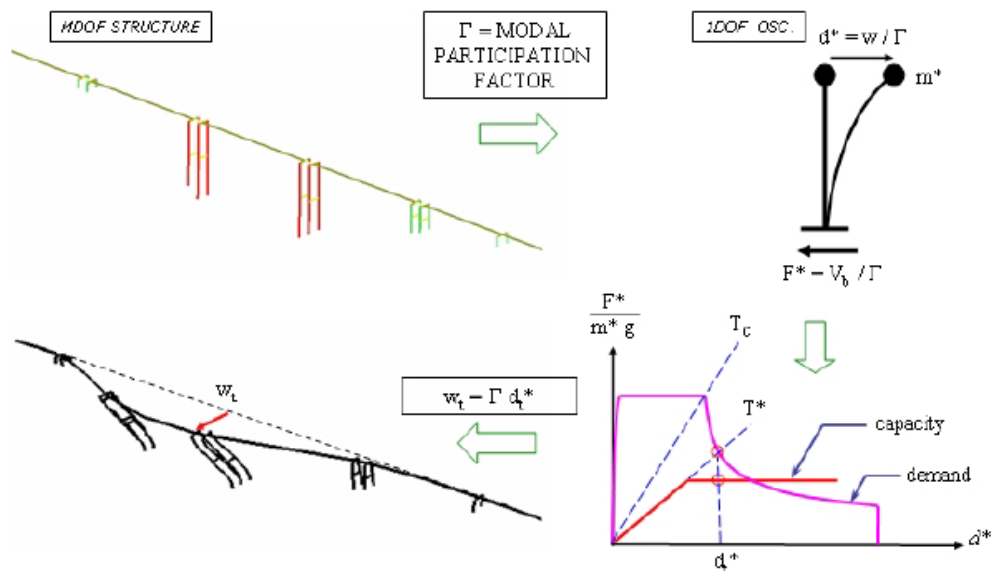


Figure 9. Target displacement computation (Commend et al).

The lateral load pushover analysis is conducted in several directions, including longitudinal, transverse, and at a certain angle with respect to the principal directions or axes of the bridge. The values for maximum displacements are computed for each direction of analysis, in accordance with the structural system considered.

The method is formed in the way that the lateral load is imposed on structure gradually with a definite pattern (e.g. uniformly distributed, triangular load, mode shape etc.) and the structure is allowed to yield gradually (continuous yielding of various components). The incremental application of the loading is continued till the displacement of a reference point of the structure achieves the target displacement. Normally, the top of structure is used as the reference point and analysis is stopped at the achievement of the roof target displacement, In the case of complex bridge structures, the criteria for the choice of an appropriate force pattern for pushover analysis of bridge structures including more than one mode shape (see Figure 10) are proposed by Isakovic, Fischinger, and Fajfar (1999).

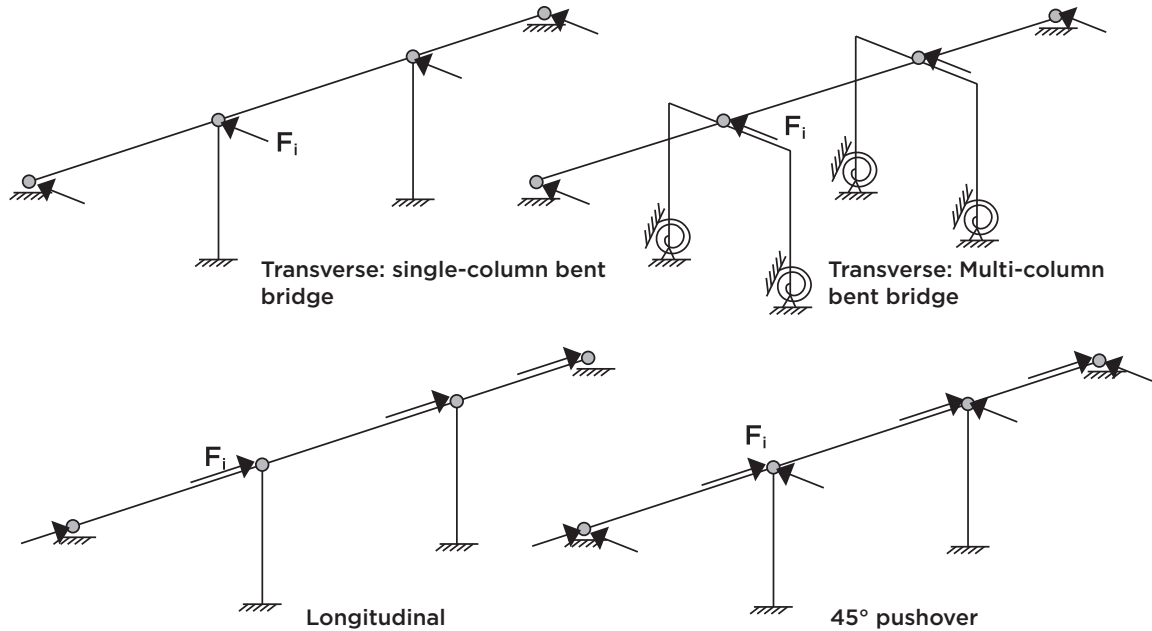


Figure 10. Force pattern for pushover analysis

Once the analysis is finalized a pushover curve is obtained (Figure 11), where the total base shear and displacement capacity of the bridge are determined.

In order to capture the changing properties of structures in pushover analysis adaptive pushover procedures have been developed and proposed by various researchers (Antoniou and Pinho, 2004; El-nashai, 2001).

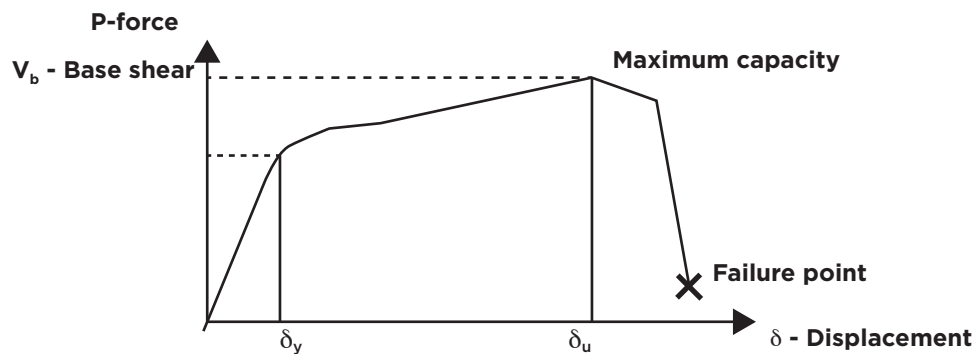


Figure 11. Typical pushover curve

Pushover analysis in many cases will provide much more relevant information than an elastic static or even dynamic analysis. In some cases it can provide misleading results (Krawinkler, 1996). Higher modes are rather important for bridges structures. In that respect multi-mode pushover analysis procedures have been proposed to consider the contributions of the higher modes. Modal pushover analysis procedure (MPA) was proposed by Chopra and Goel (Chopra and Goel, 2002). In the procedure, a pushover analysis is conducted for each mode separately, and then total seismic responses are computed by combining the responses due to each modal load.

As well in an adaptive pushover procedure, the lateral force pattern is updated according to the time-variant stiffness distribution of the structure. Displacement based adaptive pushover procedure

was proposed by Antoniou and Pinho (Antoniou and Pinho, 2004). Here a set of updated lateral displacements are imposed on the structure. Pushover procedures with adaptive lateral force patterns can provide more accurate dynamic response evaluations of structures. The disadvantage is that they are conceptually complicate and computationally demanding for routine application in structural engineering practice. The advantage of the nonlinear static pushover analysis is that it can check out the deformability of structure, find out the weakness of structure, control the damage degree of structure under intensity earthquake and it is favorable among engineers (Akhavissy, 2012; Forcael, 2014).

All types of non-linear static analysis briefly described above should be used with care when torsionally sensitive bridges are assessed, e.g. bridges supported by the very short central and higher side piers (Table 13).

Table 13. Recommended types of inelastic analysis with respect to the type of the bridge (Kappos et al. 2012)

Type of the bridge	Single mode methods	Non adaptive multi-mode methods	Adaptive multi-mode methods	Nonlinear dynamic analysis
Short bridges on moderate to stiff soil, pinned at the abutments, and not supported by very short columns	X			
Short bridges pinned at the abutments, supported by the side and long central columns	X	X		
Long bridges (or curved) without very short central columns		X	X	
Short bridges with roller supports at the abutments			X	
Short or long bridges supported by the very short central and higher side columns				X

The specific dynamic properties of a structure can give misleading results in case of even more sophisticated pushover analysis. In such cases higher degree of accuracy is required. This can be obtained by using the non-linear dynamic analysis, provided the parameters of the model the nonlinear behaviour of the material and the input are correctly modelled.

Nonlinear dynamic time-history analysis can take into account nonlinearities and strength degradation of different elements of the bridge is proper material characteristics and parameters are modelled. It is usually the ground acceleration which is taken into account as loading induced on the structure or foundation displacement. The design displacements are directly determined through dynamic analysis using suites of ground motion records.

For complex three-dimensional structures such as curved bridges, the direction of the earthquake that produces the maximum stresses, in a particular member or at a specified point, is not apparent. In that respect different earthquake motions at various input angles are applied to assure that all the significant modes are excited and the critical earthquake direction is captured, producing the peak response and estimating accurately the seismic demand on the structure.

The advantages of the nonlinear dynamic time-history analysis are first of all the ability to describe nonlinear inelastic response as well as to represent the time-history of the seismic response of the structure. The main disadvantage of the time history analysis method is the high computational and analytical effort required and the large amount of output information produced. However, it should be stated that recent development of computer hardware has allowed to reduce the required computational time and made it more practical to run many time history analyses for complex bridge structures. As a disadvantage one can take as well the fact that the results can be extremely sensitive to the input time history and structural models.

3.2.7 Role of performance indicators in the seismic assessment of bridges

In the procedure of seismic assessment **at system or element level** the performance indicators enter

through the model used to compute the structural response with the aim of assessing the performance goals.

The modeling of the structure requires the knowledge of the **performance indicators** (geometric, mechanical and related to the damage state) that govern the structural behavior and that can be directly or indirectly measured on the structure to assess. The wider and more precise the knowledge of the measured **performance indicators**, the more reliable the assessment of the performance goals.

For example in reinforced concrete bridges the knowledge about the cracked state of cross sections can be used to reduce the elastic stiffness of the corresponding sections or results from static load tests can be used to calibrate the elastic modulus of the materials. In this context vibration-based **performance indicators** such as modal parameters (frequencies and mode shapes) or Frequency Response Functions play a key role in the calibration of the numerical models allowing to inform the model with the real characteristics of deformability of the bridge, not related to a specific location (such as the above mentioned performance parameters) and able to describe its global behavior in the elastic range under any combination of loads. Furthermore, monitoring systems installed on a structure that undergoes a strong seismic event may allow improving the understanding of the actual nonlinear behavior under the dynamic loads caused by earthquakes.

3.2.8 Seismic assessment at the network level

At a network level, the consequences of seismic actions have to be analyzed taking the seismic vulnerability of bridges due to limitations in the previous design approaches and to the exposure to external environment and aggressive agents that, over time, can influence their seismic capacity thus increasing their fragility.

The approach to seismic assessment at the network level is to minimize the consequences induced by the occurrence of possible future earthquakes. Several performance indicators are computed in order to design a specific retrofit network plan aimed at reducing the overall network vulnerability. This is achieved by prioritizing the interventions that minimize the global consequences at a network level.

In this context, the performance goal is not the compliance with a threshold assigned for the performance indicators (as in the case of the assessment at the element or structure level) but rather the minimization of such performance indicators at the occurrence of the event.

This requires a comparative analysis of different alternative plans of seismic risk mitigation at a network level (e.g. a set of seismic retrofit interventions implemented on a subset of bridges belonging to the network) aimed to identify the one leading to the most beneficial effects in terms of reduction of the performance indicator values.

The requirement of a roadway infrastructure network with respect to the occurrence of a hazardous event like an earthquake has two main scopes, mainly associated with the time dimension: in the immediate aftermath of a strong seismic event, the network has not to allow rescue and evacuation procedures access to the affected area, whereas during the medium-to-long term recovery period it has to guarantee a suitable flow capacity in relation to the traffic demand. Both criteria can be included in the "Availability" KPI, in accordance with RAMSHEEP criteria.

Regarding the first situation the performance indicator is the accessibility, that can be quantified in a probabilistic way with a value ranging between 0 (not-accessible) and 1 (fully-accessible). Accessibility is a property of the site, but is strictly influenced by seismic fragility of the components of a network. Namely the aim of the accessibility analysis is to evaluate if, in case of an hazardous, in a roadway network - represented as a system of nodes and links - the connection between different pairs of nodes is still ensured. Different types of modeling strategies can be performed when dealing with disaster accessibility, Ertugay et al. (2015) where a practical example was also developed considering a system of health and shelter services in the Municipality of Thessaloniki, Greece. Accessibility can be assessed by modeling an area with zone-based; isochronal-based or raster based techniques. Focus in such cases is on evaluating if each link can fail in case of earthquake due to the failure of one or more structural systems belonging to it (e.g. bridges) or interacting with it (e.g. jutting buildings in historical centers, see as example Argyroudou et al. 2015, Zanini et al. 2016). In such a way, assuming only bridges as seismically prone systems within a network link, the vulnerability of the link, i.e. the probability of failure of a connection is a function of the seismic fragilities of the bridges belonging to it (see Augusti et al. 1994; Augusti et al. 1998; Zanini et al. 2013).

Regarding the medium-to-long term consequences, seismic damage to bridges at network level reflects in reductions of flow capacity for some links of the infrastructure. This implies a redistribution of the traffic flows that is a function of the origin-destination matrix, strictly related to the type of traffic assignment model identified. Such type of traffic redistribution can have serious impacts on the overall network functionality, causing not-negligible travel delays. Different performance indicators can be used for quantifying this type of consequence: among others, network resilience can represent the overall functionality reduction during the restoration process (Alipour and Shafei 2016). The network resilience is defined as the level of functionality of the network over time and can take values ranging between 0 (disrupted) and 1 (full functionality). However, for an economic quantification of consequences at the network level associated with the occurrence of an earthquake, it is usually preferred to compute the total travel delay (Alipour and Shafei 2016) as the difference between the total travel time on the network in the pre-event conditions and the total travel time in the damaged configuration. The increased travel time in the seismic damage scenario to bridges, with respect to the normal conditions, represents the total travel delay. A modified version of this parameter can be computed by considering the environmental impacts associated with the increase of travel time, as an equivalent (fictitious) increase of the travel time.

3.3 Scour assessment

In recent years, the number of bridge failures induced by natural hazards has drastically increased. The reason for this is a combination of; aging networks, limited maintenance funding and increased hazards (e.g. climate change leading to larger, more frequent flood events). Flooding affects the foundations of bridges crossing rivers, causing scour at the base of pier foundations set in the river bed, See Figure 12. Scour is the excavation and removal of material as a result of flowing water and is the primary cause of bridge failure in the United States where approximately 600 bridges have failed in the last 30 years (Briaud et al. 2001, 2005; Melville and Coleman 2000), and is a significant problem in the United Kingdom (Maddison 2012) and central Europe (Tanasic 2016).

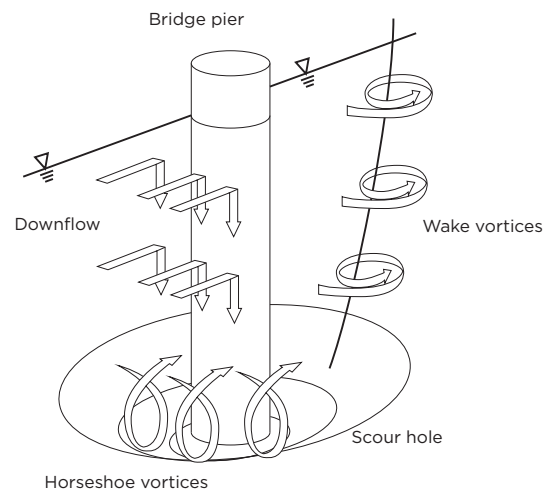


Figure 12. Schematic of the scour process (from Prendergast and Gavin 2013)

Scouring effects can be particularly significant for masonry arch bridges which are common across Europe's road networks because these structures are normally founded on shallow foundations (Zanini et al. 2016).

The assessment of scour around critical bridge infrastructure to date has broadly been undertaken using visual inspections of the foundation condition, sometimes through the use of trained divers. Generally visual inspections of bridges typically involve rating a structure based on the perceived condition, whereby the rating denotes the necessity for intervention or the time before the next inspection. A variety of non-scour related defects are widely measured in this way such as cracking, water ingress, concrete spalling, etc., see (Irish National Roads Authority 2008a; Irish National Roads Authority 2008b). Visual inspections for scour aim to assess the nature and magnitude of a scour hole around a critical foundation element such as a pier or abutment foundation, and alert bridge managers when a scour hole is deemed to have surpassed some pre-determined threshold condition. Although no specific rating schemes are globally in existence for this, a number of individual rating systems

have been implemented (by universities and local authorities) on various networks throughout Europe. However, it is widely recognized that visual inspection is expensive and can have limited effectiveness since it cannot be performed during flood conditions when the risk is largest. As scour holes typically refill with loose material when flood waters recede, visual inspection might underestimate the impact of floods and suggest the foundation is stiffer than in reality. To address these deficiencies a number of mechanical and electrical devices have been developed to provide direct measurement of scour. The method of operation and a brief review of the advantages and disadvantages of a range of commercial systems are summarized in Table 14 (after Prendergast and Gavin 2014).

Table 14. Overview of some instruments capable of direct scour measurement

Type	System	Primary Operation	Advantages	Disadvantages
Single-use device	Float-Out Device	Buried at specific depth and floats out when scour reaches level	Simple operation method, indicates float-out by triggering a switch	Require expensive installation, have only a single use, can only detect scour at installation depth
Pulse/radar device	Time-Domain Reflectometry (TDR)	Uses changes in the dielectric permittivity constants between materials to determine a depth of scour at a particular location	Easy to read results – provides direct view of scour depth	Requires long probes installed in riverbed, measurement accuracy affected by channel temperature variation
Drive/Buried Rod Systems	Magnetic Sliding Collar	Physical probe positioned around a rod augured into the soil moves downward with scour progression closing magnetic switches	Easy to read data, direct measurement of scour	Only detects scour condition at location of sensor, may miss global effect
Sound-wave devices	Sonic Fathometer	Fixed to bridge pier, these emit sonic pulses to continuously establish the water-sediment interface	Continuous scour measurement in vicinity of bridge pier with easy to read data output	Susceptible to measurement error from entrained air in highly turbulent flow

The performance of bridges under scoured conditions is dependent on the nature of the situation, such as type of bridge, nature of traffic, foundation type, redundancy, and others. New bridges are designed with scour in mind whereas older bridges on the network may require retrofitting to reduce the scour risk. New bridges are designed with a design scour depth calculated based on a given flow condition, see (Kirby et al. 2015; May et al. 2002). These broadly empirical methods, for example the Colorado State University, CSU method (Arneson et al. 2012), derive a design scour depth based on the geometry of the pier, the depth of flow upstream, the flow velocity, the flow angle to the bridge, and the nature of the bed material among other variables. The bridge geometric parameters are then optimized to reduce the design scour depth below some threshold value, such as 2.3 times the bridge pier diameter for example. In Italy guidance is provided by the Italian Code for Constructions (NTC 2008). When designing new river bridges, it is necessary to provide a hydrological report and an associated hydraulic one mainly focused on the design solutions adopted against scouring phenomena. The Code suggests to avoid, where possible piers in the river bed, indicating when necessary a distance between consecutive piers of at least 40 m. At European level, Eurocode 1 in Section 4.9 suggests how to take into account actions without giving specific formulations for the assessment of scour depths.

3.4 Joint seismic and scour assessment: current research trends

Fragility curves describe the relationship between a certain intensity measure and the probability of failure. Ter Huene (2014) describes the development of fragility curves for bridge scour, the form is shown in Figure 13.

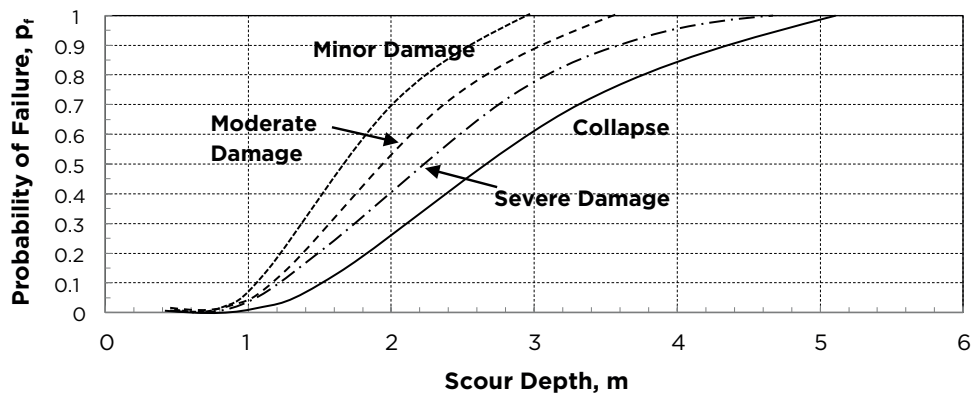


Figure 13. Development of fragility curves for the effect of scour on bridges

The approach is particularly useful for considering separate damage (limit states), in this case the impact ranging from minor damage to complete collapse to the probability of failure of a bridge due to scour depths of up to 6m. Whilst the approach is commonly applied to single hazard analysis, and is particularly useful in considering the effect of hazards such as earthquakes, it has only recently been applied to consider joint multiple hazards occurring simultaneously or with some small time-lag.

The importance of analyzing multiple-hazards, both natural and man-made is widely recognized. An approach that calculated the reliability indices of a bridge to combined effects from earthquakes, wind, scour and vessel collision over some time interval ΔT was developed by Ghosn et al. (2003). Prasad and Banerjee (2013) note that flood induced scour is not in itself a load event, rather it is the result of a flood event and its effect is to amplify the impact of other load events on the bridge performance. Therefore, superposition of individual load events is not a reliable means of assessing impact; instead a hazard-specific analysis is required to consider the bridge performance. The authors performed assessments of the effects of combined scour and earthquakes hazards on the dynamic response of four reinforced concrete bridges with spans. The bridge supports were piled and the spans varied from 2 to 5 and the response of the pile groups used at support locations were modelled using p-y springs acting on a single equivalent pile. Scour was modelled by incrementally removing springs. Five damage states were considered, ranging from no damage to complete collapse, the boundaries between damage states were based on the displacement ductility. Modelling was performed based on the assumption that some flood induced scour precedes an earthquake event. The study found that non-linear changes of the seismic fragility characteristics of the bridge occurred as scour progressed. Changes were rapid during the early stages of scour and for the systems considered became negligible for scour depths greater than 3m. The diameter of the pile foundation was seen to have an effect on the response with larger piles mitigating the effects of scour, whilst the bridge length was also shown to impact the fragility response.

Wang et al. (2014) also modelled the effect of earthquakes following a scour event on the seismic response of three forms of reinforced concrete bridges; single frame box-girder, a three span simply supported girder and a three-span continuous girder bridge. Finite element models were developed for each of the bridges and scour was induced by removal of lateral (p-y) and axial (t-z) soil springs. The analyses showed that the periods of the first few vibration modes increased with scour for all bridge types considered. The degree of change in natural frequency was dependent on the bridge type, with the three-span simply supported structure being more sensitive to change in scour depth. Interestingly when considering component analysis of the bridge columns, in some cases (for the three-span bridges) scour had a beneficial effect on the column response due to the much longer vibration periods that result.

Gehl and D'Ayala (2015) propose a component based approach to multi-hazard fragility analysis of road bridges. Damage dependent fragility curves are derived at a component level and a Bayesian Network approach is used to assemble these component level curves into a system level assessment. As a result multi-variation fragility functions can be derived wherein each input variable represents an

intensity measure for a specific hazard. The hazards considered included earthquakes, ground failures and fluvial floods.

3.5 Implementation of Structural Health Monitoring

Some information on the adoption of SHM techniques is already provided in Section 3.1.4. The term Structural Health Monitoring is usually adopted within the context of low-cost technologies, typically installed over long-term periods for the monitoring of structural assets. However, short-term and periodic measurements are also possible, while Non Destructive Evaluation (NDE) methods may also often be regarded as part of the SHM concept. While long-term monitoring is informative on the behavior of the bridge over its complete life-cycle, short-term monitoring and NDE may be requested as a result of a preliminary assessment, e.g. from visual inspection, when a flaw or damaged element is detected, or deterioration is suspected.

The *Table 15* below offers some examples of main monitoring tools and the context within which these may be adopted, without making a reference to Non-Destructive testing tools, since these come with a clearly specified purpose. NDT options for bridges are further covered in Section 3.1.4.

Table 15. Monitoring & Inspection Technologies and Corresponding Use for bridges.

Class	Measured Performance indicator	Technology	Type of Assessment / Performace goal
Dynamic Response/ Vibration-based Monitoring	Accelerations	Accelerometers (diverse sensitivities: MEMs, epi-sensors)	SLS under ambient loads: <i>Simulation-driven approach:</i> Updating of a FE model of the system in terms of the elastic properties of the system, as well as boundary conditions. <i>Data-driven approach:</i> Extraction of normalized condition indicators (against environmental/ operational effects) for prediction damage & deterioration. ULS under extreme events (e.g. earthquakes): <i>Simulation-driven approach:</i> Updating of a FE model of the system, also in the nonlinear (plastic) range. <i>Data-driven approach:</i> Detection of residual deformations. Detection of damage from appropriate statistical indicators.
	Dynamic Strains	Strain gauges FBGs Distributed measurements based on Rayleigh back-scatter on Brillouin Scattering	
	Dynamic Deflections	Robotic Total Station LVDTs Laser Sensors Interferometric Radar Laser Vibrometer	
	Tilt	Tiltmeters (typically MEMs)	
Static Response	Strains	Strain gauges FBGs Distributed measurements based on Rayleigh back-scatter on Brillouin Scattering	SLS under ambient loads: <i>Simulation-driven approach:</i> Updating of a FE model of the system in terms of the properties and loads of the system. <i>Data driven approach:</i> Assessment of long-term effects, such as creep & shrinkage ULS: <i>Simulation/data driven approach:</i> Estimation of fatigue accumulation from measured strains. <i>Data driven approach:</i> Detection of residual effects under extreme loads.
	Deflections	Laser Scanner Total Station/Tachymeter Leveler	

Environmental Readings	Chloride Content	X-ray fluorescence Tiltration Photometry	Critical for Assessment of Corrosion and Scour effects
	Temperature Humidity Moisture	Weather Station	Critical for condition normalization, particularly in case of long-term monitoring
Loads	Tensioning Force	Load Cells	Assessment of condition of tendons, cables and anchors
	Traffic	Weigh-in-Motion System (WIM)	Assessment of actual loads on the bridge and updating of the corresponding models and calculated Pls.
	Wind	Weather Station Anemometer Meteorologic Mast	

The purpose of the monitoring system, aligns well with the goals prescribed by the FHWA (US), under the Long-Term Bridge Performance (LTBP) Program. More specifically, monitoring and detailed Inspection should aim in:

- Improving knowledge on condition/performance of the bridge
- Refine or validate the results of preliminary (visual) inspection.
- Advance research in deterioration and predictive models.
- Support cost analysis and quality control plans.
- Further the advancement of technology used for the assessment of critical but hidden bridge elements and components.
- Support development of improved design methods and maintenance/bridge preservation practices.
- Quantify the effectiveness of various maintenance, repair, and rehabilitation strategies.
- Improve the operational performance of bridges with the potential to reduce congestion, delay, and accidents.
- Promote the next generation of bridge and bridge management systems.

Whilst detecting damage to a bridge superstructure using dynamic response from SHM is a relatively well researched field, the application of the approach for scour detection is less well-developed. Research has considered the use of natural frequency, mode shapes and curvature thereof, covariance of acceleration signals and changes in the RMS of acceleration signals have all been investigated as scour indicators (See Briaud et al. 2011, Chen et al. 2014, Elsaid and Seracino 2014 and Klinga and Alipour 2015). Foti and Sabia (2011) used the modal parameters (natural frequency and mode shape) to investigate the response of a five-span bridge following flood induced scour and after remediation. They recognised that scour affecting one side of the pier resulted in asymmetrical dynamic behaviour that could be detected using an array of accelerometers placed along the foundation in the direction of flow. Elsaid and Secarino (2014) describe laboratory testing on a scale-model bridge supported on piles and finite element analyses that revealed horizontally displaced mode shapes were particularly sensitive to scour progression as a result of the reduced flexural rigidity of the pile foundations. Briaud et al (2011) investigated the dynamic response of a model bridge with either shallow or piled foundations in a laboratory flume. They found that the frequency response in the flow direction and the RMS values of the accelerations showed the greatest sensitivity to scour and could be practical indicators. An attempt to implement the system on areal bridge was unsuccessful because of logistic constraints with regard to the instrumentation and logging set-up.

Prendergast et al. (2015) describe a field test to investigate the dynamic response of pile in dense sand to scour. The test pile, See Figure X was fitted with accelerometers that were monitored at a frequency of 1000 Hz. A modal hammer was used to produce an initial lateral excitation at the top of the pile. The hammer was calibrated to excite low frequency resonances in the structure (heavy tip mass, soft impact tip) so the fundamental mode of vibration could be more easily obtained. Scour was then induced in stages by removing 200 mm deep soil layers and repeating the frequency response measurement at each stage, with the modal hammer hitting the same location on the pile during each test. In total, nine scour levels were considered, the final scour depth being 1.8 m or 5.3 D bgl.

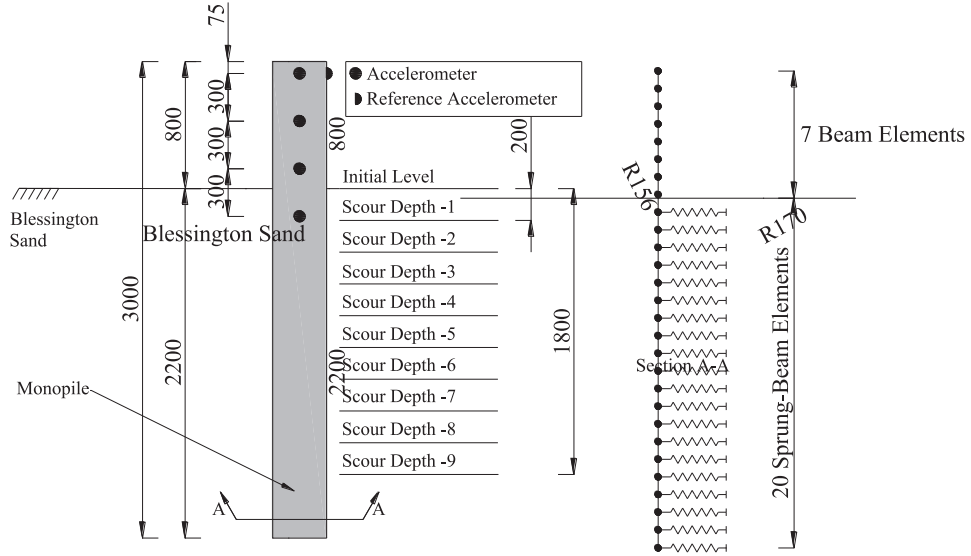


Figure 14. Test layout and numerical model

A 1-D finite element numerical model of the soil-structure system was developed in MATLAB to investigate the effect of soil stiffness on the structural response of the experimental pile. The soil-structure model consists of Euler-Bernoulli beam elements supported by lateral (p-y) springs, See Figure 14. The elements are assembled together into global mass and stiffness matrices. The undamped system natural frequencies are obtained from the model by solving the eigen problem shown in Eq. (3.4) (Tedesco et al., 1999).

$$[[\mathbf{D}]-\lambda[\mathbf{I}]]\{\mathbf{A}\}=\{\mathbf{0}\} \quad (\text{Eq. 3.4})$$

where \mathbf{I} is the Identity matrix and \mathbf{D} is a square matrix of order n and is called the characteristic matrix of λ . λ is the eigenvalue and \mathbf{A} is the associated eigenvector. \mathbf{D} is the system matrix formed in Eq. (3.5).

$$[\mathbf{D}]=[\mathbf{M}_G]^{-1}[\mathbf{K}_G] \quad (\text{Eq. 3.5})$$

where \mathbf{M}_G and \mathbf{K}_G are the global mass and stiffness matrices for the soil-structure model respectively, and include the contribution from the soil springs in the model. The eigenvalues of \mathbf{D} can be obtained as the roots of the characteristic equation, obtained by expansion of Eq. (3.6).

$$[[\mathbf{D}]-\lambda[\mathbf{I}]] = 0 \quad (\text{Eq. 3.6})$$

The system matrix was created in MATLAB for the soil-structure system and the eigenvalues and eigenvectors (natural frequencies and mode shapes) were obtained using in-built functions available in the MATLAB programming environment. Scour was modelled as the removal of springs from the numerical model. An eigenvalue analysis was undertaken for each spring removal phase and a profile of frequency against scour was generated. In the numerical model the largest uncertainty with respect to input parameters relates to the spring stiffness used to model the soil stiffness in the p-y springs. Since the dynamic loading applied to establish the natural frequency of the structure induced very small displacements, the small-strain stiffness of the soil can be used to represent soil displacements. Two values were considered, those obtained from in-situ measurements of the shear wave stiffness and value obtained from correlation with in-situ Cone Penetration Tests, CPT results. The experimental results of system frequency are compared to predictions using the numerical model in Figure 15, where the model is seen to produce excellent predictions of the system frequency and is able to track changes in frequency due to scour.

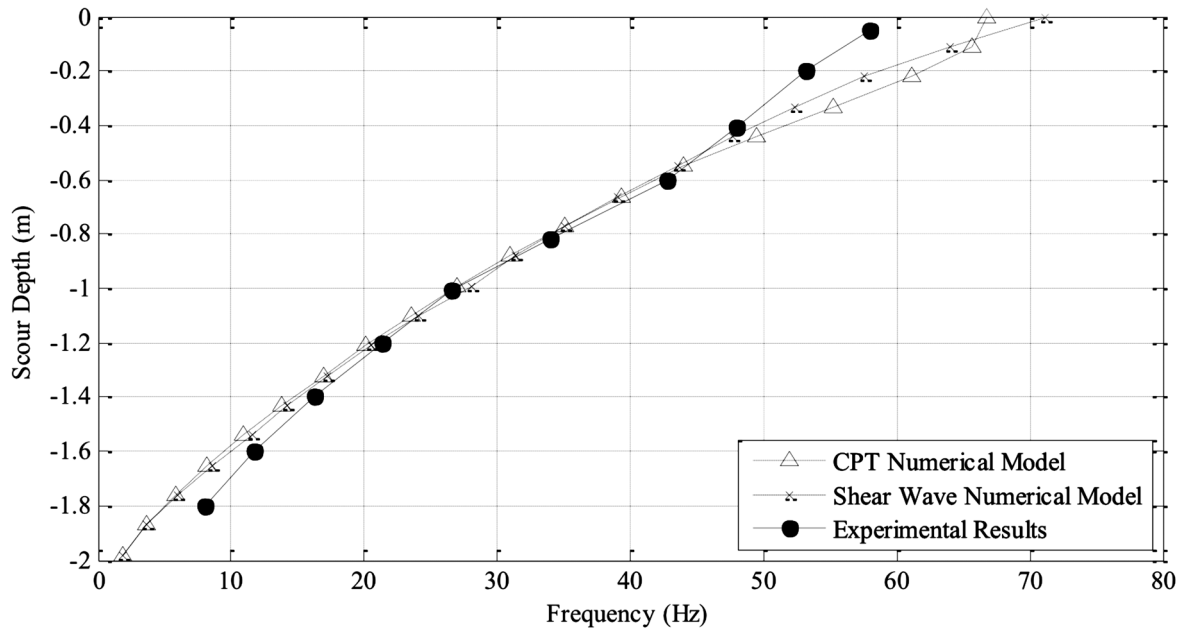


Figure 15. Comparison of Experimental results with numerical prediction of frequency change with scour

Prendergast et al. (2016) extended the numerical model to include a bridge structure (a two-span integral bridge) to ascertain if it was possible to detect bridge scour using acceleration signals generated by traffic loading, See Figure 16.

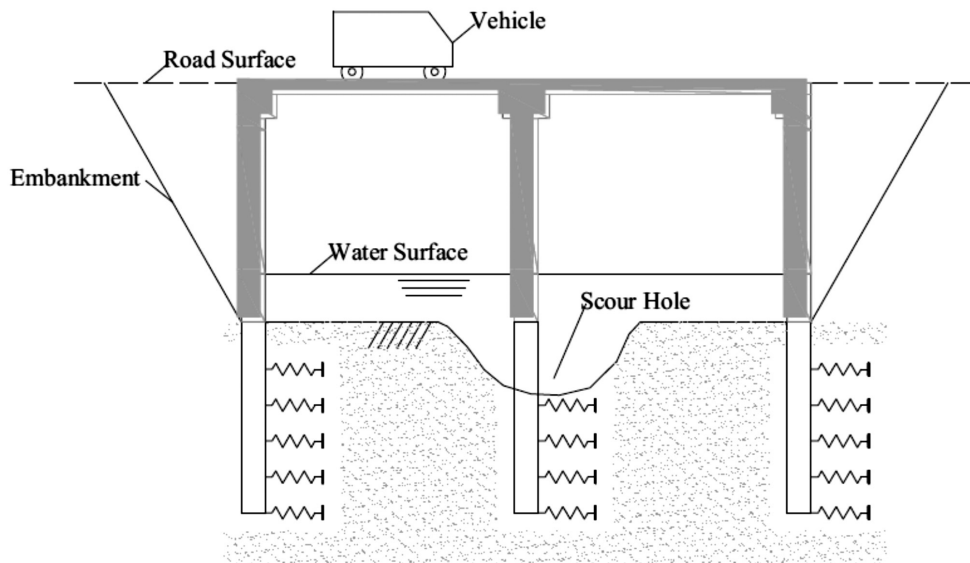


Figure 16. Schematic of extended model including bridge structure and traffic loading

The method was shown to detect measureable changes of acceleration at the top of the bridge piers for a range of soil types under loading from a two-axle truck.

4 ECONOMY, SOCIETAL AND ENVIRONMENTAL PERFORMANCE GOALS

4.1 Introduction

Optimal use of financial resources has become an essential part of managing infrastructure. In the past decade it has become very clear that infrastructure management decision making based only on low-cost does not meet modern sustainability requirements. It is essential that all costs and benefits are acknowledged and taken into consideration so whole life cycle costs of a certain structure and certain through life scenario of a structure can be optimized. Concrete structures like bridges are vital links in many transport networks and represent a big capital investment for both governments and taxpayers. They have to be managed in a way that ensures society's needs are optimally met.

While agencies in certain countries already regularly recognise indirect costs that structures (all activities linked to these structures) lay upon the society and environment, in many countries this is still yet hardly ever mentioned. But even when taken in consideration LCC in many Bridge Management Systems has mainly been applied within the bridge operation phase, even though it has several useful applications within the bridge entire life, from cradle to grave, and can support investment decision making. Economical parameters are still easier to distinguish while for societal and environmental parameters to be taken into consideration more detailed analysis with multi-objective optimization has to be performed especially on the network level. This chapter will give an overview of cost components which may be used to build LCC models / tools in order to assist decision-makers at all levels and within all phases in selecting the most cost-effective alternative from an array of applicable alternatives. Figure 17 shows investment phases through whole life of a bridge from initial idea, construction, maintenance and end of life.

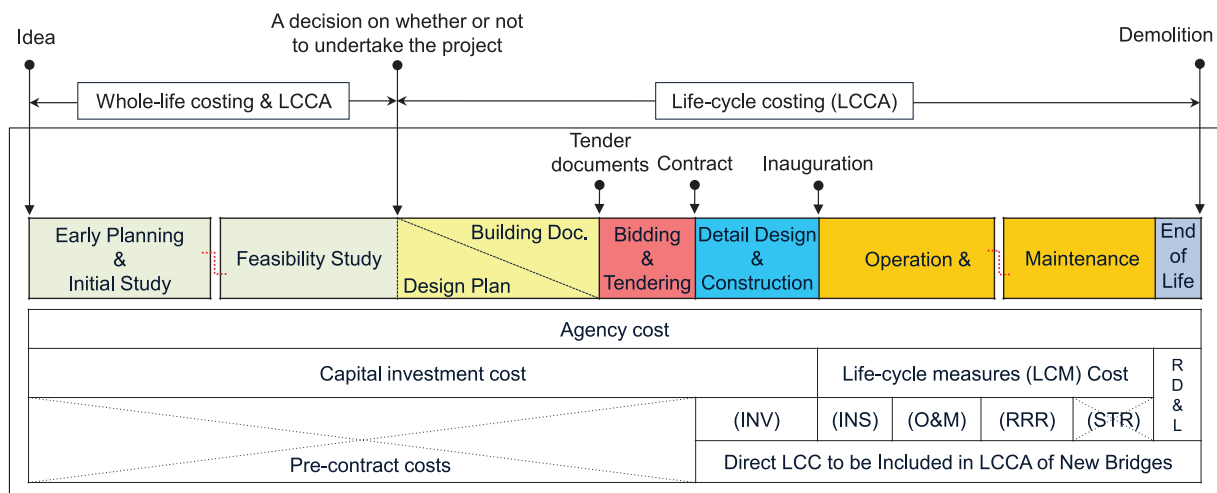


Figure 17. Bridge investment phases and LCC categories; INS = inspection; O&M = operation and maintenance; RD&L = recycling, demolition, and landscaping; RRR = repair, replacement, and rehabilitation bridge investment phases, the possible LCC applications and saving potential (Safi, 2013)

Life cycle cost analysis is a method that enables one to compare different investment alternatives based on the total costs that are associated with that alternative. Not only initial investment costs but also all costs that develop throughout the objects life cycle are taken into account. This entails costs made during operation as well as end of life costs. Although by definition LCC seeks to include all costs that arise during the lifetime of an object, not everyone agrees on the precise identification of these costs (Woodward, 1997). Based on literature on LCC of infrastructure, the following cost elements of interest will be taken into account: agency costs, user costs, society and environmental costs.

For infrastructure objects a traditional view of LCC would be the costs incurred by government bodies for realisation, maintenance and disposal of the object in question, so called agency costs. (Chandler, 2004).

A more holistic approach as for example used by the U.S. Department of Transportation (2002) includes the costs incurred by the users of the object, namely user costs, which can be seen by this definition.

“Life-cycle cost analysis (LCCA) is an evaluation technique applicable for the consideration of certain transportation investment decisions. All of the relevant costs that occur throughout the life of an alternative, not simply the original expenditures, are included. Also, the effects of the agency’s construction and maintenance activities on users, as well as the direct costs to the agency, are accounted for.” (U.S. Department of Transportation, 2002)

Thus, user costs are the financial effects of construction and maintenance activities that are incurred by the user of the infrastructure object in question. This includes costs generated by the fact that during construction and maintenance users are spending more time on the road (traffic delay and vehicle operation costs). But this can also include costs based on the fact that during road work activities traffic accident rates increase (accident costs).

An even broader interpretation of LCC can be made by also including social costs. For example: (Ehlen, 1997) mentions the importance of including ‘third party costs’, such as costs incurred by surrounding businesses and environmental costs. Murphy (2013) also distinguishes social costs as a third cost element that has to be accounted for. He includes aesthetic and cultural value and environmental costs in this category. He does not however describe a method to monetize these social costs. Keoleian et al. (2005) also include some environmental costs for LCC calculation but the methods and data used are not clearly explained.

There are different ways to monetize environmental effects. One of these is the so called ‘revealed collective preference method’ (Davidson & Wit, 2003). Environmental life cycle assessment on for example different bridge designs of different materials are largely dependent on assumptions that are made regarding production and disposal methods.

The three cost categories that make up the total LCC – agency costs, user costs and society costs – will be presented in the following chapters. Each chapter will describe the needed input variables and the necessary calculations (equations) that may be used to develop the LCC calculation tool.

$$LCC = AC_{disc} + UC_{tot, disc} + SC \quad (\text{Eq. 4.1})$$

The total LCC are calculated by summarizing the different cost categories: Agency costs (AC), User costs (UC) and Society costs (SC). Agency costs represent direct costs incurred during the asset life cycle, and user and society costs are indirect costs which occurred due to the usage of the asset.

4.2 Economy performance assessment

Bridge Management Systems are designed for managing groups of bridges (can include thousands of structures) with limited financial resources and are used as a tool as part of a broader bridge management strategy. Bridge Management System (BMS) is defined as a rational and systematic approach to organizing and carrying out all the activities related to managing a network of bridges, including optimizing the selection of maintenance and improvement actions in order to maximize the benefits while minimizing the LCC (Hudson et al., 1992). Depending on the bridge inventory, constraints and societal demands, a road agency will lay out a strategy aiming to achieve optimal results within the limitations posed by the constraints.

Mainly there are three aspects addressed by BMSs found in literature: condition assessment, modelling future degradation and optimisation of maintenance, repair and rehabilitation (MR&R) decisions and actions. These aspects are then analysed on project level and network level. Both levels are interrelated and should not be analysed separately.

One of the main factors in making maintenance decisions is the deterioration level of an object. With regard to maintenance planning, two opposing approaches may be applied, a corrective versus a preventive maintenance strategy. In a corrective strategy, maintenance is planned after a certain amount of damage has occurred, whereas a preventive approach aims to plan maintenance so as to prevent unaccepted damages from occurring. In practice, often a combination of both strategies is followed, aiming for an optimal balance between costs and performance, see Figure 18. A good understanding of bridge condition and future degradation is necessary for this optimisation.

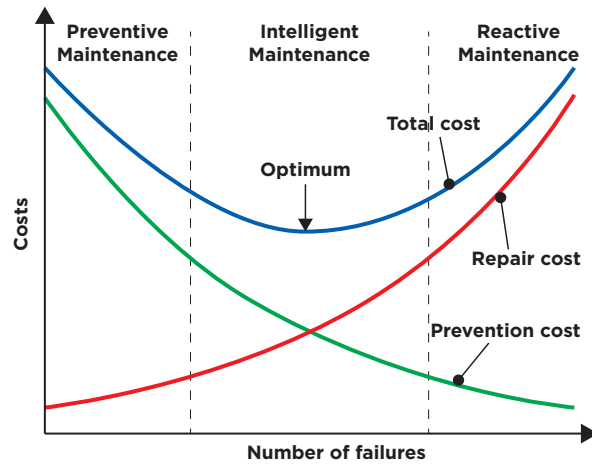


Figure 18. Optimization of Maintenance Costs

A maintenance plan should be based on a decision making system which enables choosing the best repair option and timing considering multiple performance aspects, for example safety, durability, functionality and economy. A first maintenance plan could be made up based on the design of the structure and be periodically updated during the service life based on inspection and maintenance data. Figure 18 presents a general framework for the life cycle of concrete structures, where the strategy of interventions (maintenance, repair, rehabilitation) is consisting of:

- Defining the set of requirements to be fulfilled by the structure;
- Performing a technical and economic analysis through the use of requirements and performance indicators, for example by a life-cycle cost analysis (LCCA); and
- Selecting among the options identified the optimal repair or strengthening method.

Finally, the repair has to be executed and the maintenance needs of the repaired structure have to be established. Life-cycle cost analysis, or LCCA, is the most suitable methodology for comparing different repair options, but is very often not feasible because of lack of experience or of reliable data. (Andrade & Martinez, 2009; CONRENET 2007, Andrade & Izquierdo 2005; Frangopol et al. 2001). Due to these reasons the process of choosing the best repair option is often expert based which is an outdated approach and should be avoided in modern decision making process.

4.2.1 Agency costs

Agency costs are the costs that are directly born by the owner during the entire life span of an asset. They are usually divided into three subcategories, namely: design and construction costs, maintenance costs and end of life costs. The method of calculating the Agency costs is based on the works of Chandler (2004). Basically this means breaking down the total structure into different elements and multiplying these elements with an estimated unit cost per element. The construction costs are those costs that the agency must cover in order to build an asset. This construction costs are related to the acquisition of a new asset that for instance, belongs to a new construction plan or to a service demand. This cost can also be related to the replacement of an asset that has already reached the end of its service life. The construction costs include labor cost, material cost, equipment costs and, indirect costs. It is call indirect costs to those costs that are not directly account to an object, this costs include personnel, administration and security costs.

The total discounted agency costs are the sum of the three sub cost categories and therefore calculated by:

$$AC_{disc} = ICC + \left(\sum_{t=0}^T \frac{MC_{T,nom}}{(1+r)^t} \right) + \frac{EoLC_{T,nom}}{(1+r)^T} \quad (\text{Eq. 4.2})$$

Where in:

ICC = Initial construction costs (€)

MC_{t, nom} = nominal maintenance costs for year t (€)

EoLC_{nom} = nominal end-of-life costs (€)

t = year in life cycle from 0 until end of life cycle T

T = year in which life cycle ends

r = discount factor (%)

4.2.1.1 Design and Construction costs

The design and construction costs are the costs that the agency will have to make for realising the construction of the object. These costs include direct construction costs: costs of material, labour and equipment. Usually indirect construction costs, such as risks, profit, general costs, execution costs and one-off construction costs are also taken into account. Besides direct and indirect construction costs there are also unassigned object risks, engineering costs and other additional costs.

The direct construction costs are calculated by first dividing the designed object into separate construction elements. The next step is determining the unit cost of a particular construction element and multiplying it by the amount that that element occurs in the design. This results in the total costs of that particular element in the total object. Doing this for every construction element and summarising these costs will yield the total assigned construction costs.

The rest of the initial construction costs are calculated by taking a percentage of the direct construction costs (plus if applicable the one-off, execution and general costs). The percentage and the value with respect to which that percentage is taken should be based on the empirical / statistical data of the owner.

All these costs are made at the beginning of the life cycle of the object. Therefore there is no need for any discounting. The needed calculation for the initial construction costs are given by:

$$ICC = \sum_{i=n}^m CUC_i \times Cq_i \times (1 + \chi) \quad (\text{Eq. 4.3})$$

where in

ICC = initial construction costs (€)

i = construction element n until element m

CUC_i = construction unit cost of element i (€/unit)

Cq_i = the quantity of construction element i present in the design (unit)

χ = an additional percentage to cover unassigned, indirect, engineering and other costs.

4.2.1.2 Maintenance costs

The maintenance costs is another cost category that can contribute significantly to the total life cycle costs. The maintenance costs will be calculated in a similar manner to the initial construction costs. First the maintenance scenario that most accurately describes the estimated required maintenance over the life cycle of the object has to be determined. This means determining the different necessary maintenance activities, their accompanying frequencies and their estimated unit costs. Next, the unit cost of a certain maintenance activity (AUC_i) is multiplied by the quantity of units related to that activity (Aq_i). The resulting yearly maintenance cost for that activity is attributed to all the years in the life cycle of the object in which that maintenance activity takes place (based on the frequency attributed to that activity (t=?)). This creates a maintenance schedule with which the total maintenance costs of every year in the life cycle can be calculated.

The maintenance costs for one specific year are therefore calculated by:

$$MC_{t,nom} = \sum_{i=n}^m AUC_i \times Aq_i \quad (\text{Eq. 4.4})$$

Wherein:

MC_{t,nom} = nominal maintenance costs for year t (€)

i = activity n until m

AUC = activity unit cost of activity i (€/unit)

Aq_i = quantity of units for activity i in year t (unit)

Summarizing the maintenance costs of every year in the life cycle of the object gives the total nominal maintenance costs of the object. Because the maintenance costs are made in the year the maintenance takes place, the future cash flows have to be discounted to create a present value.

The total discounted maintenance costs may be increased by a certain percentage that cover the *unassigned costs, indirect costs and unassigned object risks* (but not *engineering costs and other additional costs*), if that is the owner's practice.

The total maintenance costs for the object during its life cycle is therefore calculated by:

$$MC_{tot,disc} = \sum_{t=0}^T \frac{MC_{t,nom}}{(1+r)^t} \times (1 + \chi) \quad (\text{Eq. 4.5})$$

Wherein:

MC_{tot} = the total maintenance costs during the life cycle of the object (€)

$MC_{t,nom}$ = maintenance costs for year t (€)

t = year in life cycle from 0 until end of life cycle T

r = the discount factor (%)

χ = an additional percentage to cover unassigned, indirect, engineering and other costs

4.2.1.3 End-of-life costs

The third sub category in the category agency costs are the end-of-life costs. These include the costs of demolition and disposal minus the residual value. In this model the end-of-life costs will be calculated the same way that the initial construction costs are calculated. Namely by assigning the construction elements with a unit cost for end-of-life costs and multiplying the amount of a certain building element with the corresponding end-of-life unit cost. In this research it will be assumed residual value is equal to zero. The resulting equation is shown in:

$$EoLC_{nom} = \sum_{i=n}^m DUC_i \times Cq_i \quad (\text{Eq. 4.6})$$

Wherein:

$EoLC_{nom}$ = nominal end-of-life costs (€)

i = construction element n until element m

DUC_i = demolition and disposal unit cost for element i (€/unit)

Cq_i = the quantity of construction element i present in the design (unit)

Because the end-of-life costs take place at the end of the life cycle of the object the costs will have to be discounted. This is done using:

$$EoLC_{disc} = \frac{EoLC_{T,nom}}{(1+r)^T} \quad (\text{Eq. 4.7})$$

Wherein:

$EoLC_{disc}$ = are the discounted end-of-life costs (€)

$EoLC_{T,nom}$ = are the nominal end-of-life costs at the end of the life cycle (€)

T = year in which life cycle ends

r = discount factor (%)

4.2.2 User costs

The second cost category that is included in the life cycle costs is the category *user costs*. The user costs are the costs that are born directly by the user of the bridge. These costs can be divided into several sub-categories of which the most common and most significant are taken into account in this study, namely: *vehicle operating costs*, *traffic delay costs*, and crash costs or *traffic accident costs*.

These costs are all a result of the work zones that are associated with the construction and maintenance of the bridge during its life cycle. It is therefore essential for the determination of these costs that the amount of maintenance (frequency and duration) that is needed and the results of this maintenance on the traffic flow and traffic safety is estimated carefully. The maintenance scenario that is used for the determination of the maintenance costs will therefore also have to include information about the effect of the maintenance activity on the traffic flow (i.e. resulting extra travel time per vehicle and the duration of the activity) and the effect on the traffic safety (i.e. the number of extra accidents).

The equations used for determining the vehicle operating costs, traffic delay cost and traffic accident costs are based on the work of Sundquist & Karoumi (2012). There are other methods available for determining the vehicle operating costs such as the National Cooperative Highway Research Program (NCHRP) Report 133 method, the Texas Research and Development Foundation (TRDF) method, and the Federal Highway Administration's (FHWA) HERS-ST Method (FHWA's office of operations, 2015). While these methods are more extensive and thus may yield more accurate results, these methods also require more data for input. Data of which it is deemed unlikely (and not considered part of the scope

of this study) that the researcher was able to determine the needed values. The method of Sundquist & Karoumi (2012) is therefore chosen as the most appropriate method in this case. How the different sub-categories are determined exactly will be discussed below.

The total user costs are a summation of the three sub-categories; vehicle operating costs, traffic delay costs and traffic accident costs. Because the user costs are made during the life cycle of the bridge, future cash flows will have to be discounted to determine a total present value.

The total discounted user costs are determined using:

$$UC_{tot, disc} = \sum_{t=0}^T \frac{VOC_{t, nom}}{(1+r)^t} + \sum_{t=0}^T \frac{TDC_{t, nom}}{(1+r)^t} + \sum_{t=0}^T \frac{TAC_{t, nom}}{(1+r)^t} \quad (\text{Eq. 4.8})$$

Wherein:

- $UC_{tot, disc}$ = total discounted user costs (€)
- t = year in life cycle from 0 until end of life cycle T
- r = discount factor (%)
- $VOC_{t, nom}$ = nominal vehicle operating costs in year t (€)
- $TDC_{t, nom}$ = nominal traffic delay costs in year t (€)
- $TAC_{t, nom}$ = nominal traffic accident costs in year t (€)

4.2.2.1 Vehicle operating costs

The vehicle operating costs are the costs that are associated with the operation of the vehicles that pass the work zone (e.g. fuel costs, lubricant costs, overall vehicle wear, depreciation, etc.). Because of the extra travel time that is caused by the maintenance work zones – either by a limited allowed driving speed along the work zone or by a (potentially imposed) detour route taken by vehicles – the vehicles passing the work zone road section endure longer operation times and thus more operating costs. These costs will differ for different sizes of passenger cars and different sizes of freight traffic. To avoid the need of having to know the exact traffic composition an average value is taken based on certain amount of freight traffic. In case of reduced maximum allowed driving speed due to the presence of a work zone the extra travel time can be determined by:

$$ETT = \frac{L}{S_a} - \frac{L}{S_n} \quad (\text{Eq. 4.9})$$

Wherein:

- ETT = extra travel time (hours)
- L = length of the work zone (km)
- S_a = adjusted average traffic driving speed during work zones (km/h)
- S_n = normal average traffic driving speed outside work zones (km/h)

If the maintenance activity in question requires a detour (so that the above equation is not applicable) the extra travel time can be estimated and entered as a direct time value (DTT). This gives:

$$ETT = DTT \quad (\text{Eq. 4.10})$$

Wherein:

- ETT = extra travel time (hours)
- DTT = detour travel time (hours)

The vehicle operating costs can then be determined using:

$$VOC_t = ETT \times ADT_t \times V_{oc} \times N_t \quad (\text{Eq. 4.11})$$

Wherein:

- VOC_t = the vehicle operating costs for year t (€)
- ETT = extra travel time per vehicle (hours)
- ADT_t = the average daily traffic in year t (passenger car equivalent (PCE)/day)
- V_{oc} = is a monetary value for operating costs expressed (€/hour)
- N_t = the duration of a certain maintenance activity expressed (days)

4.2.2.2 Traffic delay

The traffic delay costs are the costs that represent the valuable time of the road user itself. This economic value of the user's time is dependent on several factors. The type of traffic (passenger vehicle or freight traffic), the amount of persons/cargo per vehicle and the type of cargo/person (business/leisure). The model should not become too complicated and the necessary data needed to use as input for the model should be obtainable by the user of the model. Therefore a general average user time value is taken. This value is estimated by assigning a certain percentage of the traffic as freight traffic and the rest of the traffic as passenger vehicles. Average time values for these types of traffic are estimated from used values in the literature. The time value for freight traffic and passenger traffic is then proportionally averaged to a general average time value for the road section under investigation. The traffic delay costs can be determined in a similar fashion as the vehicle operating costs this is done by:

$$TDC_t = ETT \times ADT_t \times V_{UT} \times N_t \quad (\text{Eq. 4.12})$$

Wherein:

TDC_t = traffic delay costs for year t (€)
 ETT = extra travel time per PCE (hours)
 ADT_t = the average daily traffic in year t passing the bridge in question (PCE/day)
 V_{ut} = is a monetary value for the users time (€/hour)
 N_t = the duration of a certain maintenance activity (days)

4.2.2.3 Traffic accident costs

According to the FHWA's office of operations (2015) numerous studies indicated that there is an increase (of between 20% to 70%) in crash rates along work zones. The traffic accident costs are the costs that are a result of the death, injury or material damages due to this increase in crash rates. This study differentiates between two types of accidents and their resulting costs; traffic accident costs with resulting deaths and traffic accident costs of accidents with severe injury. In this last category of accident costs the costs of accidents with minor injury and mere material damages are also included by an increase in the value used for crash costs per accident that represents these types of accidents. This is based on the data provided by Stichting Wetenschappelijk Onderzoek Verkeersveiligheid (SWOV, 2014). Both the crash costs of accidents with resulting deaths as the crash costs of accidents with severe injury can be determined using the same equations. The total traffic accident costs are a summation of the two.

The extra accidents per vehicle can be determined by:

$$EAV = L \times (A_a - A_n) \quad (\text{Eq. 4.13})$$

Where in:

EAV = extra accidents per PCE (# of accidents/PCE)
 L = the length of the work zone (km)
 A_a = the adjusted crash rate during work zones (# of accidents/PCE km)
 A_n = the normal crash rate outside of work zones (# of accidents/PCE km)

The traffic accident costs are then determined using:

$$TAC_t = EAV \times ADT_t \times V_{AC} \times N_t \quad (\text{Eq. 4.14})$$

Wherein:

TAC_t = traffic accident costs for year t (€)
 EAV = extra accidents per PCE (# of accidents/PCE)
 ADT_t = average daily traffic (PCE/day)
 V_{ac} = average cost value per accident (€/accident)
 N_t = the duration of a certain maintenance activity (days)

4.3 Societal performance assessment

The third category which may be included in the LCC is the category of society costs. These are costs borne by society at large.

The goal of assessing the environmental effects is to compare different design solutions.

4.3.1 Society costs

Society costs are costs borne by society at large. Some of the structures, such as bridges or special buildings might be seen as monuments and icons which the citizens may relate with the cultural heritage of a certain area. This atmosphere and the will to identify certain area and its values with an icon may motivate for bold and spectacular solutions. Some alternatives have exceeded all cost estimates but they have been chosen as aesthetically the best. Certainly, there is a hidden value behind the external shape of structures, such as bridges in some special locations. The inclusion of this value in the evaluation process leads to eliminating the worst aspects of bridge design and encourages the best. This value should be computed for the different feasible proposals in fair-bases and converted to a measurable value to be able to include it in the LCC model. (Safi, 2012)

4.3.2 Environmental costs

The environmental costs are the costs that are caused by use of energy and resources during the construction, maintenance and end-of-life phase of the bridge and the accompanying emissions to the environment. There are a couple of general steps associated with environmental impact or life cycle assessment (LCA) studies. ISO 14040 is an international guideline on how to perform an LCA and suggest the four steps shown in the life cycle framework in Figure 19.

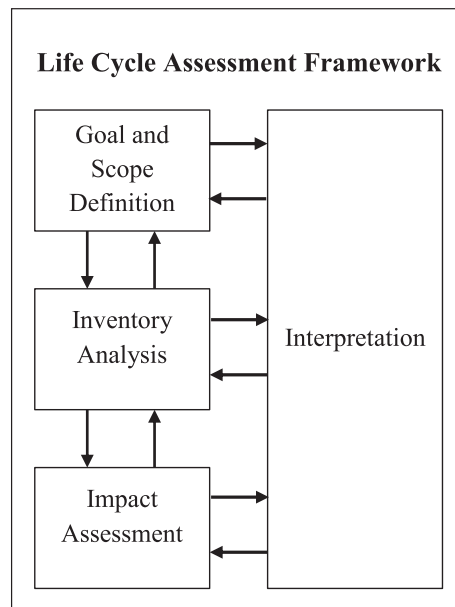


Figure 19. LCA framework of ISO 14040 (ISO 14040)

One issue, brought up in the construction of a new structures nowadays, is the impact this structure will have on the immediate environment. Beside traditional requirements for the concrete structures, a trend toward extended attention to the environmental impacts due to different designs, and different maintenance, repair and rehabilitation strategies, is growing stronger. Requirements for long term considerations regarding renewable and non-renewable resources for infrastructure construction are missing or unclear. For specific bridge location, different bridge types can be used, however, each of them produces different environmental impact. Based on the results of an environmental study of BECO (2013) it is determined that most of the environmental effects (about 90%) are caused by the use of construction material during initial construction and during maintenance. It is therefore decided that - for the sake of simplicity of the model - only the determination of environmental effects caused by material use is included. This means that environmental effects due to transportation, construction activities and end of life scenarios are not taken into account.

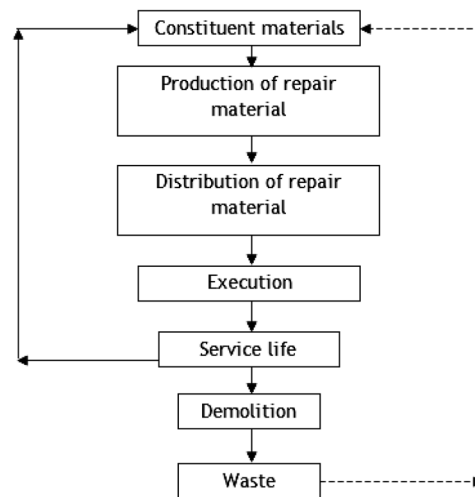


Figure 20. Life cycle steps in the ecology and health evaluation (2004, REHABCON)

Life cycle costs in the ecology (natural environmental effects e.g. water pollution, resource consumption) and health evaluation (air pollution, dust generation, carbon monoxide) is shown in Figure 20. (2004, REHABCON)

For example the environmental impacts of the bulk of materials in the design can be assessed. These materials are estimated to be: steel; concrete; polyester; glass fiber; epoxy; carbon fiber; asphalt; gravel; PVC.

The environmental effect categories of the CML-2001 method may be used since this is an often used method in Europe and implemented in different software solutions. In the case study presented in this report, for the LCA calculations GaBi software was applied. This software allows to estimate the relevant environmental indicators (based on the CML-2001 method (see Thinkstep, 2015)). Using the 'revealed collective preference method' the environmental costs can then be determined.

In Appendix 1 the environmental impact per kg of material j for impact category i are displayed. These have been determined with the help of LCA software GaBi or other literature sources if needed. Values can be found in Table A1.

Also, for these effect categories the Dutch agency Rijkswaterstaat (part of the Dutch Ministry of Infrastructure and the Environment) published a list of corresponding shadow prices determined by the so called revealed collective preference method (TNO-MEP, 2004). These shadow prices are a way of monetizing environmental effects. For an explanation and in-depth discussion the author refers to the report by CE Delft (2003). The different environmental effect categories and their corresponding shadow prices are presented in

Table 16. Environmental effect categories and shadow prices (TNO-MEP, 2004)

Environmental effect category	Shadow price (€/kg equivalent)
Abiotic depletion elements (ADP) (€/Sb eq)	0.16
Abiotic depletion fossil (ADP) (€/Sb eq)	0.16
Global warming potential (GWP) (€/CO ₂ eq)	0.05
Ozone depletion potential (ODP) (€/CFK-11 eq)	30
Photochemical ozone formation potential (POCP) (€/C ₂ H ₂ eq)	2
Acidification potential (AP) (€/SO ₂ eq)	4
Eutrofication potential (EP) (€/PO ₄ eq)	9
Human toxicity potential (HTP) (€/1,4-DCB eq)	0.09
Freshwater aquatic ecotoxicity potential (FAETP) (€/1,4-DCB eq)	0.03
Marine aquatic ecotoxicity potential (MAETP) (€/1,4-DCB eq)	0.0001
Terrestrial ecotoxicity potential (TAETP) (€/1,4-DCB eq)	0.06

This is where the method used in this research differs from the framework of the ISO 14040. Instead of first determining the life cycle inventory (LCI) of one complete product life cycle and then determining the resulting impact via life cycle impact assessment (LCIA), this study first determines the environmental impact of one kg of material as a basic parameter of the model and then uses those values to calculate the total environmental impact by multiplying it with the amount of material present in the design (which is determined during the entry of the different bridge elements into the calculation tool). The total environmental costs (in this case the total society costs) can then be determined using Equation 15. Environmental costs incurred during the life cycle of the bridge are not discounted as recommended by (Hellweg, Hofstetter, & Hungerbuhler, 2003).

$$SC = \sum_{i=n}^m EE_i \times SP_i \quad (\text{Eq. 4.15})$$

Wherein:

SC = society costs (€/functional unit)

EE_i = environmental effects for impact category *i* (kg of impact category equivalent (ICeq)/functional unit (one bridge))

SP_i = the shadow price for environmental effect category *i* (€/kg of ICeq)

i = environmental impact category *n* until *m*

The environmental effects per impact category can be determined using Equation:

$$EE_i = \sum_{j=n}^m EE_{i,j} \times Mq_j \quad (\text{Eq. 4.16})$$

Wherein:

EE_i = environmental effects for impact category *i* (kg of impact category equivalent (kg ICeq)/functional unit (one bridge))

EE_{i,j} = environmental effect for impact category *i* per kg of material *j* (kg ICeq/kg material)

Mq_j = material quantity per functional unit for material *j* (kg material/functional unit)

j = the different materials *n* until *m*

4.4 Combined economy and environmental performance assessment

Asset managers need to be able to evaluate both the economic costs and the environmental consequences of diverse maintenance and rehabilitation options so that an informed and optimised choice can be made, taking into account both perspectives. Furthermore, a thorough understanding of any trade-off between economic and environmental implications, generated by the options available, is required to ensure cost-effectiveness whilst reducing the environmental consequence to a minimum. This section presents such decision support model for evaluating the combined economic and environmental performance of diverse maintenance strategies for maximising the service life of bridge infrastructure assets.

The developed framework can be used for appraising the economic cost and environmental performance of maintenance strategies that have different working life extension horizons. It provides a consistent evaluation method for comparing the performance of the strategies based on unequal life extension periods. The model also incorporates an optional step for evaluating and comparing a temporary delayed strategy with a 'maintain now' base scenario by introducing a novel time-weighted function, which will be discussed in detail below. Figure 21 presents the methodology framework in terms of a flowchart.

With respect to the LCC assessment, the input parameters are the main direct costs of the construction, maintenance and end-of-life of a maintenance system; they can be categorised into materials and components, labour, equipment and plants and indirect costs. The assessment output is the life cycle cost for the agency. The LCA assessment main flows are materials, energy and water for production, construction, maintenance, demolition and disposal of the maintenance system, whilst the output flows are the emissions to air, water and land including waste.

Once the economic and environmental analysis are performed, the decision evaluation is carried out using a form of 'scoring and weighting' multi-criteria analysis (MCA) approach.

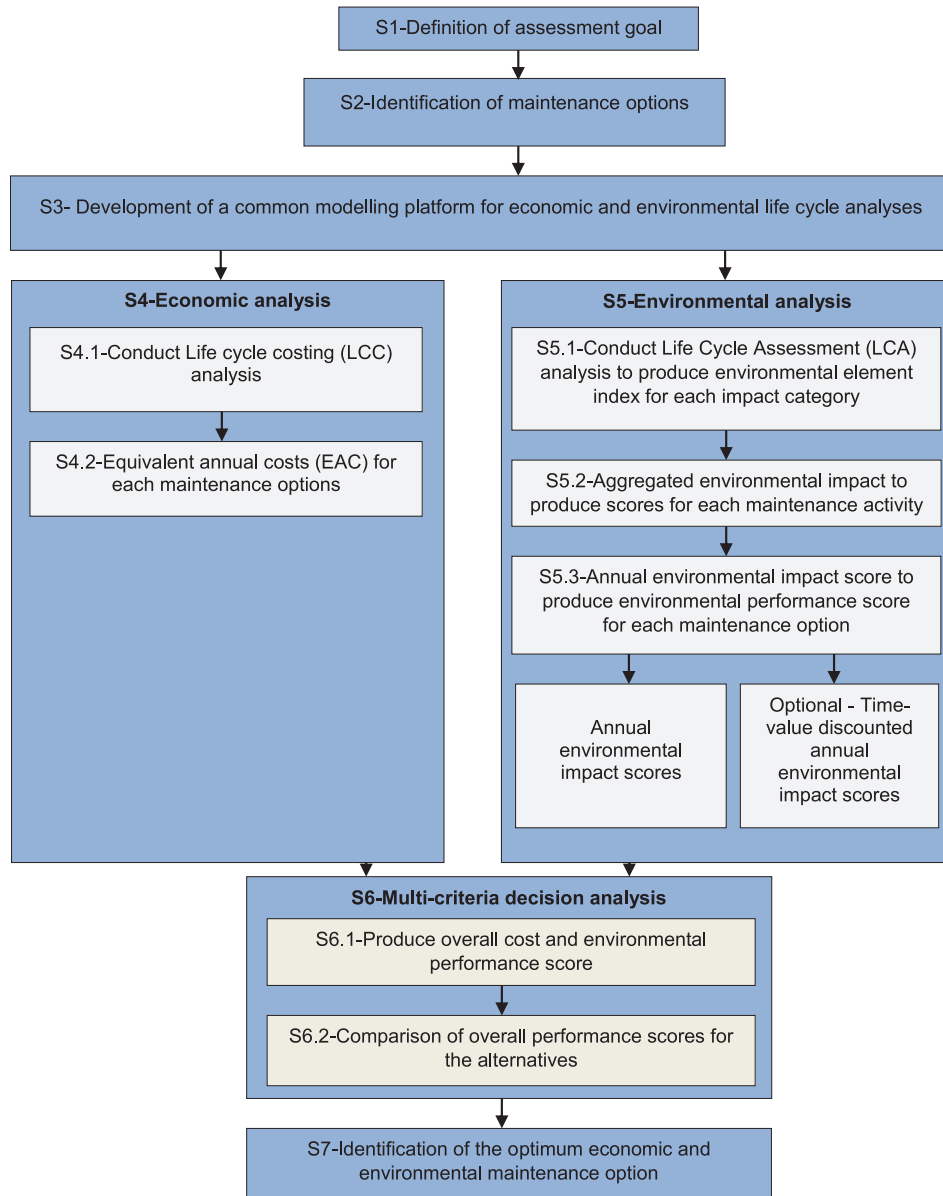


Figure 21. Flowchart of the combined economic and environmental assessment methodology framework

In order to evaluate the economic performance of maintenance activities through an LCC assessment, the annual life cycle cost of the competing maintenance strategies can be estimated using the Equivalent Annual Cost (EAC) method to address the cost comparison between maintenance strategies with dissimilar service lifespans. The EAC converts all the costs occurring over a period of time and presents the LCC cost as an equivalent uniform annual amount (Riggs, 1997). The EAC expression is presented in Equation 1 below for estimating the annual performance cost of a single maintenance system. The alternative with the lowest EAC cost is the most cost-effective.

$$EAC = (NPV_{activities, year j} + NPV_{activities, year k} + NPV_{activities, year m}) \times AF \quad (Eq. 4.17)$$

Where

$NPV = \left(\frac{C_n}{(1+r)^n} \right)$ is the Net Present Value of the total activity cost occurring at year = $j, k, m...$ etc.;

$AF = \frac{r}{1 - \frac{1}{(1+r)^t}}$ is the Annuity Factor, used to convert the total NPV into a uniform annual cost equivalent

is the total extended service period of the utilised maintenance intervention or strategy system;

is the total activity cost; and

is the discount rate in percentage.

In terms of the environmental analysis, an aggregated environmental impact obtained from the LCA analysis is converted into an annual aggregated environmental impact (Ann. Agg, EI) for determining the environ-

mental performance of maintenance alternatives with different life extension capabilities. Ann. Agg. EI represents the life cycle impact generated for an additional service year extended by a maintenance. Basically, the lower the Ann. Agg. EI index value, the better the environmental performance of an alternative system. In addition, a time-value weighting option is utilised to model time-preference, so that the temporary impact delay can be assessed as one of the potential alternatives. The temporary impact delay can be adopted as a measure to help the transport infrastructure industry in managing their environmental burdens such as delaying high polluting and resource consuming maintenance projects to slow down the environmental emission rate or 'buying time' for deploying cleaner technologies and higher eco-efficient designs.

Equation (2) below presents the general expression for the Ann. Agg. EI. It denotes the impact level of a maintenance alternative for prolonging the structure service life by an additional year.

$$\begin{aligned} \text{Ann. Agg. EI (in unit/ year)} = & (\sum \text{Agg.EI}_{\text{prod,year } i} \times \text{Tw}_{\text{year } i}) + \\ & (\sum \text{Agg.EI}_{\text{constr,year } j} \times \text{Tw}_{\text{year } j}) + (\sum \text{Agg.EI}_{\text{use,year } k} \times \text{Tw}_{\text{year } k}) \\ & + (\sum \text{Agg.EI}_{\text{end,year } m} \times \text{Tw}_{\text{year } m}) \end{aligned} \quad (\text{Eq. 4.18})$$

Where

$\sum \text{Agg.EI}_{\text{xxx, year } i}$ is the sum of aggregated environmental impacts for the 'material production', 'construction', 'use' and 'end-of-life' activity modules that occur on years 'i' to 'm', respectively

$\text{Tw}_{\text{year } x}$ is the time-value weighting factor for years 'i' to 'm'.

The time-value weighting factor (Tw) can be expressed as follows:

$$\text{Tw} = \text{Discount Factor} \times \text{Annuity Factor} = \frac{1}{(1+r)^n} \times \frac{r}{1 - \frac{1}{(1+r)^T}} = \frac{r(1+r)^T}{(1+r)^n[(1+r)^T - 1]} \quad (\text{Eq. 4.19})$$

where r is the environmental discount rate in percentage,

n is the aggregated impact occurred due to activity at a specific year i, j, or k,

T is the total residual service life duration of the structure; $T = t + t_d$, where t = extended years duration and t_d = delayed year duration

The combined economic and environmental performance score is proposed for measuring the overall cost and environmental performance of the maintenance alternatives, as presented by Equation 4:

$$\text{CEE score} = (W_{\text{eco}} \times \text{EAC}) + (W_{\text{env}} \times \text{Ann. Agg.EI}) \quad (\text{Eq. 4.20})$$

where CEE Score is the combined economic and environmental score

W_{eco} is the economic weighting factor

EAC is the Equivalent Annual Costs

W_{env} is the environmental weighting factor

Ann. Agg. EI is the Annual Aggregated Environmental Impact score

The derivation of the CEE score is based on the Simple Multi-Attribute Rating Technique using Swings (SMARTS) technique (Edwards and Barron, 1994). Scores and weighting for MCDA may be recommended and governments or the public sector for transport investment appraisals. The Economical Annual Equivalent Costs and the Environmental Annual Impact Scores are first converted into a single dimensional utility. The relative strength preference technique (Jin, 2007) is used to enable the costs and environmental scores to be measured from a same performance scale perspective, i.e. 0-100. The LCC costs and environmental scores of the alternatives are normalised into a 0-100 score performance, where 0 represents the most favourable and 100 the least favourable performance.

It should be noted that a second alternative instead of carrying out an MCA analysis, as described above, and assessing the different maintenance strategies in terms of scores, may be to monetise the environmental impacts and combine them together with the life-cycle costs into a single monetary value. Environmental impacts, or environmental performance indicators, that can be considered in such an analysis can include air pollution (CO₂ emissions), waste, water use and pollution, hazardous substances, depletion of non-renewable resources, noise and vibration, ecology, land pollution.

5 MULTI-OBJECTIVE OPTIMIZATION MODELS

Implicit in any decision-making process is the need to construct, either directly or indirectly, the preference order, so that alternatives can be ranked and the best alternative can be selected. For some decision-making problems, this may easily be accomplished. For example, in case of a decision based on a cost-minimization rule (where the lowest-cost alternative is chosen), the preference order is adequately represented by the natural order of real numbers (representing costs). Hence, in such a case, the preference order need not be constructed explicitly (Patidar et al., 2007).

Multi-criteria decision-making (MCDM) provides a systematic approach to evaluate multiple conflicting criteria in decision making. Conflicting criteria are typical in evaluating options: cost or price is usually one of the main criteria, and some measure of quality (performance level) is typically another criterion, usually in conflict with the cost. MCDM is used to identify and quantify decision-maker and stakeholder considerations about various (mostly) non-monetary factors, in order to compare alternative courses of action (Patidar et al., 2007). Alternatively, the multiple performance criteria can be combined into a so-called utility function, in which all the criteria are brought into a single scale.

In the course of developing network-level bridge programs, program managers typically face a variety of objectives and constraints. Examples of such objectives are given in Table 1, and constraints usually include a budgetary limitation that may not be exceeded or a minimum level of average bridge performance. The overlap between objectives and constraints is a key issue to practical multi-objective optimization of an asset management program.

An important class of decision-making techniques that attempt to construct the preference order by directly eliciting the decision maker's preference is predicated on what is known as utility theory. This, in turn, is based on the premise that the decision maker's preference structure can be represented by a real-valued function called a utility function. Once such a function is constructed, the selection of the appropriate alternative can be done using an optimization method. Broadly speaking, this technique involves three steps (Patidar et al., 2007):

1. **Weighting:** This assigns relative weights to the multiple criteria.
2. **Scaling:** Because the performance criteria can be of different units, scaling provides a common scale of measurement and translates the decision maker's preferences for each performance criterion on a 0-100 scale. This involves developing single-criterion utility functions.
3. **Amalgamation:** Amalgamation is combining the single-criterion utility functions using the relative weights into one measure based on mathematical assumptions about the decision maker's preference structure. This involves deriving the functional forms of multi-criteria utility functions.

Bridge maintenance planning is a process of deciding the scope, timing, costs, and benefits of future maintenance activities on a specific bridge. Optimization of maintenance activities regarding technical and economic requirements is essential for road owners to fulfil societal expectations. Due to the long life time of the road infrastructure, especially bridge structures (often longer than 50 years), the assessment of technical and economic performance is necessary in order to optimize budget expenditure. Life cycle cost (LCC) analysis and reliability based concepts are well established methodologies for identification and assessment of trade-offs.

As explained in previous chapters, bridge performance goals should be set as multi-objective system, taking into account different aspects of bridge and network performance.

Multi-criteria decision-making (MCDM) provides a systematic approach to combine these inputs with benefit/ cost information and decision-maker or stakeholder views to rank the alternatives. MCDM is used to identify and quantify decision-maker and stakeholder considerations about various (mostly) non-monetary factors in order to compare alternative courses of action (Kabir et al. 2014). Hierarchy structure for linking multi-objective bridge performance goals, covering most of the previously mentioned aspects with performance indicators is shown in Figure 3 in Chapter 2.3.2.

Alternatively, the multiple performance criteria can be combined into a so-called utility function, in which all the criteria are brought into a single scale.

In order to transform the various out into a single (mostly monetary) scale it is necessary to establish weight factor for the individual types of criteria. Some of the weight factors are available in some coun-

tries (for example weight factor for traffic delays, noise, injuries etc.), depending on the selection of criteria, some weight factor may still need to be developed. In the development of the weight factors the starting point can be taken in the qualitative approach mentioned above, from which the apparent relative weight can be deducted. Once the possible outcomes have been brought to a single scale, the best decision can be found as a formal optimised decision process, in which option with the maximum “utility” shall be selected as the recommended decision.

5.1 Analytic Hierarchy Process

5.1.1 Introduction

The Analytic Hierarchy Process (AHP) is a structured technique for organizing and analyzing complex decisions, based on mathematics and psychology. It was developed by Thomas L. Saaty in the 1970s and has been extensively studied and refined since then. It has particular application in group decision making, and is used around the world in a wide variety of decision situations, in fields such as government, business, industry, healthcare, shipbuilding and education.

Rather than prescribing a “correct” decision, the AHP helps decision makers identify the decision that best suits their goal and their understanding of the problem. It provides a comprehensive and rational framework for structuring a decision problem, for representing and quantifying its elements, for relating those elements to overall goals, and for evaluating alternative solutions.

The AHP aims to arrive at the relative weights for multiple criteria in a realistic manner while allowing for differences in opinion and conflicts that exist in the real world. The analytic hierarchy process can handle quantitative, qualitative, tangible, and intangible criteria. The process is based on three principles: decomposition, comparative judgments, and synthesis of priorities. It constructs a hierarchy and uses pairwise comparisons at each level to estimate the relative weights.

The procedure for using the AHP can be summarized as (Saaty, 2008):

1. Model the problem as a hierarchy containing the decision goal, the alternatives for reaching it, and the criteria for evaluating the alternatives.
2. Establish priorities among the elements of the hierarchy by making a series of judgments based on pairwise comparisons of the elements. For example, when comparing potential purchases of commercial real estate, the investors might say they prefer location over price and price over timing.
3. Synthesize these judgments to yield a set of overall priorities for the hierarchy. This would combine the investors’ judgments about location, price and timing for properties A, B, C, and D into overall priorities for each property.
4. Check the consistency of the judgments.
5. Come to a final decision based on the results of this process.

5.1.2 Case study

In this section, an example case is provided to demonstrate the multi-criteria optimization for the bridge maintenance planning. A road network has a number of bridges which might be exposed to deteriorating condition. Due to budget limitations, a decision has to be made regarding the selection of bridge(s) for maintenance. Therefore, the objective of this example is to illustrate the *decision making procedure for a bridge(s) selection, where the cost and downtime due to maintenance is aimed to be kept minimum.*

For the sake of demonstration, we have considered five bridges depicted as A, B, C, D and E. Each bridge on the network has a number of related attributes such as location, geometry, condition etc. Before assessing the bridge performance on the network level, it is assumed that for each bridge an optimal maintenance alternative has been selected, based on life cycle cost calculations for several alternatives, e.g. three alternatives: do nothing, minor repair, major repair. For the purpose of ranking the bridges in the terms of maintenance prioritization, the following performance indicators have been selected (see Table 17 marked bold): reliability level, maintenance cost, downtime due to the maintenance works, and importance on the network.

Table 17: List of performance goals associated with performance indicators (Allah Bukhsh et. al., 2017)

Performance goals	Performance Aspects	Performance indicators
To provide safe and reliable network	Reliability	<ul style="list-style-type: none"> Condition rating Reliability rating Number of casualties caused by traffic accident
To provide responsive and sustainable network	Availability	<ul style="list-style-type: none"> Availability of road (% of time) Downtime (caused by maintenance works)
To minimize agency cost	Economic	<ul style="list-style-type: none"> Owner cost (LCC, initial cost, maintenance, replacement)
To minimize negative impact on users, local communities and the environment	Societal and environmental	<ul style="list-style-type: none"> Importance on the network User delay cost Environmental impacts

Table 18. Representational data for illustrative case

	Reliability	Economy	Availability	Society
Bridges	Reliability level	Maintenance cost	Downtime	Importance on the network
Name	Score card	Euros	Hours	Traffic Intensity (# cars / day)
A: B101	3	500k	30	9000
B: B109	4	1000k	70	10000
C: B207	4	200k	60	13000
D: B307	5	800k	180	15000
E: B150	3	500k	40	5000

The related data for five bridges is provided in Table 18, where indicators are defined as follows: *Maintenance cost* is a direct (owners) cost which is expected to be incurred during the maintenance procedure. *Downtime* is defined as the unavailability of the bridge due to planned maintenance activity. The *reliability level* of a bridge is a constructed value based on the condition indexes. The reliability level only involves those bridges that are critical to perform their intended function. The score card depicting the reliability levels are shown in Table 19.

Table 19. Reliability level score card

Reliability Level	Description
1	Very Good (no faults)
2	Good (minor faults well within tolerance)
3	Fair (tolerable faults, no restriction in use necessary)
4	Poor (significant structural defects)
5	Very poor (seriously deficient, mitigation measures necessary)
6	Out of service (on high risk of failure, mitigation needed urgently)

Only those assets that are in the optimum range of serviceability, i.e. 1-5 can be considered in the maintenance planning procedure. Bridges having the reliability level of 6 are considered too critical to delay the maintenance actions. Finally, the network importance is defined by the intensity of traffic passing over the bridge each day.

Preference structure

Subjective judgment in the decision making methods plays vital role. The preference structure and procedures for this example is based on analytical hierarchy process proposed by Saaty (1988). To assign the ratio scales to the subjective judgments of comparison, a fundamental scale for pairwise comparison is used. By assigning the ratio scale values, the importance of one attribute (e.g. condition) become clear to another attribute (e.g. cost).

Based on the scale of pairwise comparison¹, a relative importance of each attribute to define the decision criteria is defined in Table 20. Considering the objective, the decision criteria are to select a bridge for maintenance that has minimum cost, minimum downtime, minimum reliability level, and minimum traffic intensity. Therefore, in comparison of maintenance cost and downtime the intensity of their relative importance is equal to 1. While, in comparison of maintenance cost and reliability level, the maintenance cost is moderately more important than reliability level with intensity of 3. It is important to notice that all the comparisons among attributes and their data is made for demonstration purposes only.

Table 20. Subjective judgment to ratio scales

Criteria		Importance & Intensity	
A	B	Imp	Intensity
Maintenance cost	Downtime	A	1: Equal importance
Maintenance cost	Reliability level	A	3: Moderate
Maintenance cost	Traffic Intensity	A	5: Strong
Downtime	Reliability level	A	3: Moderate
Downtime	Traffic Intensity	A	3: Moderate
Reliability level	Traffic Intensity	A	7: Very Strong

The subjective judgment to ratio scales provides a decision criteria matrix. The step of pairwise comparison is repeated four times to compare data values for each bridge to another. For instance, the maintenance cost for each bridge is compared to another and their relative importance is defined. The low maintenance cost will get high intensity value as compared to the higher maintenance cost. These pairwise comparisons of data values yielded four comparison matrixes.

To derive the final weighted scale, the Eigen vector method is used to first reduce the matrix into a value function ranging from 0 to 1 and then to find the largest Eigen value. Irrespective of matrix type either as decision criteria matrix or comparison matrix, the steps to reduce to Eigen vector are similar. The following steps are taken to find the best bridge for maintenance that incur minimal maintenance cost and downtime. To represent the calculation procedure, some steps are adopted from (Ma, Ma, Zhou, & Ma, 2015):

- a) normalize each matrix

$$\bar{e}_{ij} = \frac{e_{ij}}{\sum_{k=1}^n e_{ik}} \quad (\text{Eq. 5.1})$$

Where e_{ij} represents an element in the matrix M . \bar{e}_{ij} represents an element of normalized matrix which is resulted by dividing e_{ij} to sum of its column value. This step results into a normalized matrix.

- b) To calculate the geometric mean of each matrix, add all the elements in a row

Then divide the sum of values in a row to the number of rows

$$\bar{w}_i = \sum_{j=1}^n \bar{e}_{ij} \quad (\text{Eq. 5.2})$$

¹ Online link to scale: <http://bit.ly/2gEluQX>

This step represents the final weighted score for a matrix, represented as S_m . The calculation of geometric mean reduces the matrix size to 4*1 and 5*1 for decision matrix and comparison matrix respectively.

$$S_m = \frac{\bar{w}_i}{\sum_{j=1}^n \bar{w}_j} \quad (\text{Eq. 5.3})$$

- c) The final step is to calculate the largest value that represents the aggregated preference of reduced maintenance cost and downtime.

$$M_{jm} = (S_{mc}, S_d, S_{rl}, S_{ti}) \quad (\text{Eq. 5.4})$$

Where S_{sc} , S_d , S_{rl} and S_{ti} represent the geometric mean values for maintenance cost, downtime, reliability index and traffic intensity respectively. The final score is obtained by multiplying the judgment matrix represented as M_{jc} to the decision criteria matrix represented as M_{dc} .

The result shows the scoring of selected bridge maintenance alternatives for five bridges, when taking into account four different aspects, measured by cost, downtime, reliability level, and traffic intensity at the network level. Prioritization of bridges for maintenance will be done based on the objective to keep the maintenance cost and downtime minimum. The final obtained results are illustrated in Figure 22 and Figure 23 .

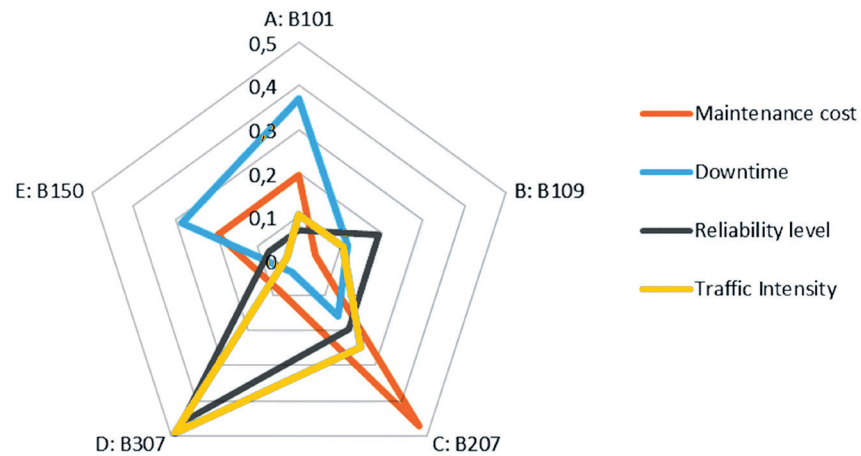


Figure 22. Comparison of performance aspects for five bridges

Before providing the final selection of a single bridge, Figure 22 graphically represents the equation 4 where the final score (S_m) is combined to form a judgment matrix (M_{jm}). The higher the value of an attribute the more it is preferable with regard to the defined objective. The graph shows that bridge C: B207 is most preferable selection in terms of reduced cost, while bridge A: B101 is best in terms of reduced downtime. Similarly, more conclusions can be drawn e.g. for the maximization of reliability level then bridge D: B307 is scoring the best.

It is important to notice that based on the attribute nature the applied function is defined. For instance, for the maintenance cost and downtime the minimization function is applied whereas for the reliability level and traffic intensity the maximization function is used.

Figure 23 shows the final prioritization result based on the AHP process for bridge maintenance planning, when the objective is set to minimize maintenance costs and minimize downtime. It is shown that bridge C: B207 is most preferred in terms reduced cost and downtime whereas B: 109 is least preferred bridge for maintenance.

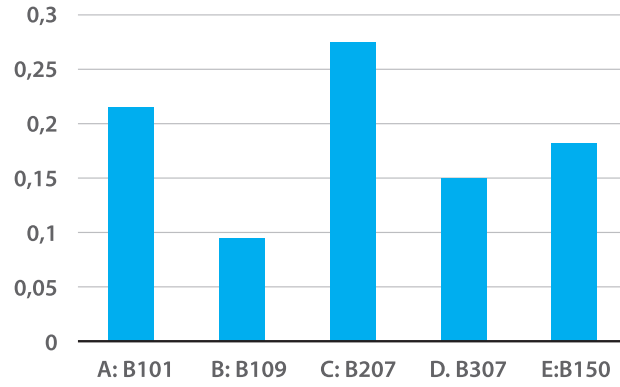


Figure 23. Bridge scoring based multi-criteria decision analysis

5.2 Multi-attribute Utility Functions

5.2.1 Introduction

Utility theory provides a measure of preferences of a decision maker over a group of alternatives (Ishizaka & Nemery, 2013). Based on the six axioms of utility theory, Multi-attribute utility theory (MAUT) is introduced by Keeney and Raifa (1993). MAUT provides a systematic approach to reduce the qualitative values of various attributes (i.e. performance indicators) into utility functions. The obtained utility scores are then aggregated based on the relative importance of attributes. The final score assigns a ranking to each alternative based on either minimization or maximization function. In other words, MAUT assigns the relative importance of performance indicators (e.g. condition, cost, etc.), while comparing number of bridges. These bridges are often referred as alternatives in MAUT.

MAUT involves the single decision maker who is willing to make certain trade-off among the performance indicators while exposed with uncertainty and risk (Keeney and Raiffa, 1993). The uncertainty is usually originated because of unavailable and dynamic nature of data, and involvement of number of stakeholders. For instance, in the bridge planning the exact estimation on number of users affected due to maintenance activity is difficult to define. MAUT integrates a body of mathematical utility models and a range of decision assessment methods in order to assist in decision ranking problem (Thevenot et. al., 2006). The single attribute utility function is calculated for each performance indicator, which reflects the risk attitude of the decision maker. The risk attitude is categorized into risk-taking, risk averse, and risk neutral. Figure 24 shows the resulting utility graph based on risk attitude of decision maker. The utility values can be calculated by plotting the attribute values in x-axis and utility values on y-axis ranging from 0 to 1.

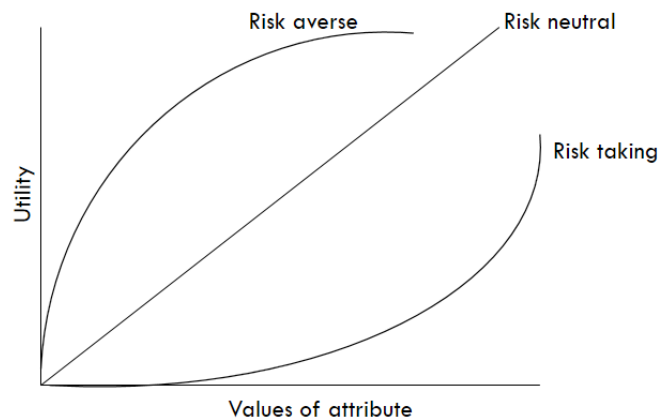


Figure 24: Risk Attitude of decision maker

The mathematical formulation of MAUT is represented as follows:

$$U(x) = k_1 U(x_1) + k_2 U(x_2) + \dots + k_n U(x_n) \quad (\text{Eq. 5.5})$$

Where

$U(x)$ is overall utility value of each alternative

k is a scaling constant that provides the relative importance of each performance indicator

$U_i(x_i)$ is a utility value of each performance indicator i for the alternative x

$$U_i(x_i) = A - B e^{(RT/x)} \quad (\text{Eq. 5.6})$$

Where

A and B are scaling constants

RT is risk tolerance

The general steps to apply MAUT on decision-making problem e.g. maintenance planning are summarized as follows:

1. Identify the decision objectives and define the attributes relevant to the problem
2. Quantify the attributes in a form that structures and represent the defined decision objectives and goals in utility functions
3. Calculate the single utility function for each attribute by estimating the indifference point(s) and risk attitude of a decision maker(s). This steps will establish a relationship between the attributes values and their utility scores based on preferences structures of the decision maker(s).
4. Determine the relative importance of attributes build on the weighting assigned by the decision maker(s).
5. Compute the aggregative utility score for each alternative by either multiplicative form of additive form. The total aggregative score will rank the alternatives, where an alternative that is the perfect fit in a realization of decision objective is ranked at highest.

5.2.2 Case study

In this section, an example case is provided to illustrate the application of MAUT for the bridge maintenance planning. The data considered in the example is exactly the same as mentioned in Section 5.1.2. The *objective* of this decision-making exercise is to rank the bridges alternatives where the cost and the downtime due to maintenance activity can be kept minimum.

The defined decision objective is defined by performance goals and is quantified into performance indicators (also called attributes). Among many performance goals, Table 17 shows the considered performance goals defined into performance indicators that are considered for this case study also. With the definition of performance indicators, the single utility function of each attribute is calculated by Equation 5.6. In this exercise, authors played the role of a decision maker to estimate the indifference point and the general risk attitudes. Here the calculation procedure to discern single utility function of owner cost is provided:

A decision maker is provided with a lottery question representing the minimum owner cost and maximum owner cost as shown in Figure 25. The median values between the maximum and minimum owner cost is called Expected Value (EV), which is 600 for owner cost.

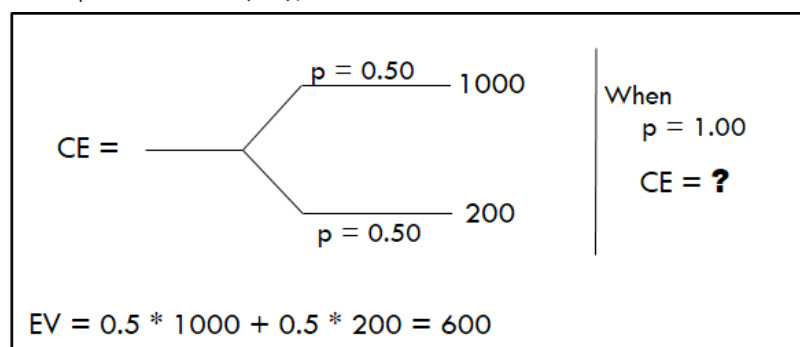


Figure 25. Lottery question to discern owner cost

In practice, an owner cannot always get the minimum cost as might desired. Therefore, MAUT has a concept of Certainty Equallient (CE) which is indifference point of a decision maker between the maximum (worst) owner cost and minimum (best) cost. In this case, the chosen CE is 400. Consider-

ing the risk tolerance value of 330, the Equation 2 becomes:

$$U(OC) = 1.0971 - 2.0113e^{(330/x)}$$

The single utility function (e.g. $U(OC)$) reduces the values from 0 to 1 representing the utility values of real numbers with respect to the defined objective. Figure 26 shows the graph of owner cost with respect to the assigned utility values.

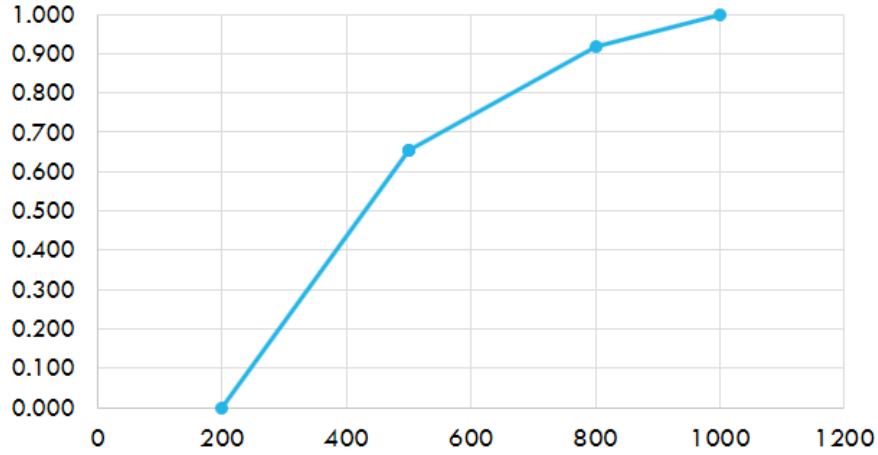


Figure 26. Single Utility Score of Owner Cost

Single utility scores of downtime, reliability rating and traffic intensity are computed in the same manner. Table 21 shows the actual data and the computed single utility value of each performance indicator.

Table 21. Attributes Data and Computed Single Utility Scores

	Owner Cost (OC)	Downtime (D)	Reliability Rating (RR)	Traffic Intensity (TI)	Single Utility Scores			
Bridges	Euros (k)	(hrs)	Score card	No. of cars/day	U(OC)	U(D)	U(RR)	U(TI)
A: Bridge 1	500	30	3	9000	0.6551	0.0000	0.0000	0.6369
B: Bridge 2	1000	70	4	10000	1.0000	0.4931	0.5987	0.7311
C: Bridge 3	200	60	4	13000	0.0000	0.3949	0.5987	0.9230
D: Bridge 4	800	180	5	15000	0.9191	1.0000	1.0000	1.0000
E: Bridge 5	500	40	3	5000	0.6551	0.1508	0.0000	0.0000

Finally, to obtain the total aggregative value of each bridge, the additive multiple attribute function shown in Eq. 1 is used. Considering the possibility of having the multiple performance goals, the relative importance of performance indicators is defined by k factor. Table 22 shows illustrative values of k factor for each performance indicator.

Based on the additive multiplicative values, the ranking of the bridges alternatives where the maintenance cost and downtime is kept at minimum is calculated. Since, the objective is to have minimum maintenance cost and minimum downtime; the bridge with least value is ranked higher. Table 23 and Figure 27 present the final aggregative value of each bridge along with their ranks.

Table 22. Trade-off values of performance indicators

Performance indicator	K (relative importance)
Owner cost	0.5263
Downtime	0.2632
Reliability Rating	0.1579
Traffic intensity	0.0526

Table 23. Ranking of bridge based on additive aggregation

Bridges	Additive	Rank
A: Bridge 1	0.396468	3
B: Bridge 2	0.802897	4
C: Bridge 3	0.251064	1
D: Bridge 4	0.957399	5
E: Bridge 5	0.384487	2

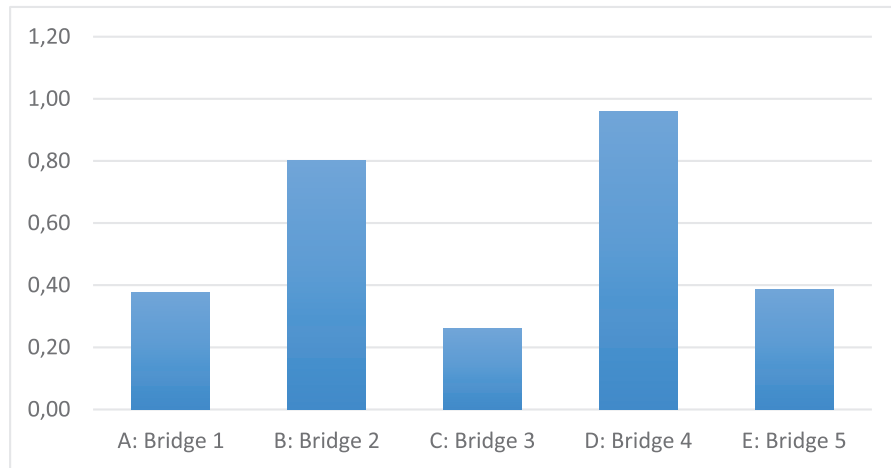


Figure 27. Ranking of bridges based on MAUT

5.2.3 Web-based tool

In WG2 of TU1406 COST Action, a [web-based tool](#) is developed to apply the multi-objective optimization. The developed tool have implemented one of the method of MCDA namely Multi-Attribute Utility Theory (MAUT) by using the R Utility package (Reichert, Schuwirth, & Langhans, 2013). The instruction for the application of the web-based tool is provided in Appendix 2.

Link to the tool is following: https://maut.shinyapps.io/application_of_maut/

5.3 Discussion and conclusions

One of the main challenges in future research is how to quantify performance goals other than technical, and how to link network level to the performance requirements on the object level. Network or even societal goals tend to be rather broad in their definition. Furthermore, there is often no exclusive relationship between performance indicators set at a lower level and goals at a higher level. An important notion is that in many countries, the main focus of bridge management is still the condition assessment of the particular objects or elements thereof.

The above mentioned case studies provide some guidance on how a program manager can implement multiple performance goals by establishing a multi-criteria decision making framework, based on the performance indicators linked to the performance goals and aspects. In this chapter, a set of performance goals is proposed, based on the existing practices and literature review, which are linked to the set of performance indicators that include reliability, availability, economy and societal aspects. For the purpose of showing how multi-criteria decision making can be performed, an illustrative case study has been done with the application of AHP and MAUT method. With these method it was possible to evaluate multiple performance aspects of multiple bridges in order to rank bridge maintenance activities on the network level.

6 CONCLUSION AND FUTURE DIRECTIONS

During the implementation of asset management strategies, maintenance actions are required in order to keep assets at a desired performance level. As the focus on an efficient delivery of network performance increases, so does the interest in the relations between societal goals, performance indicators for both the road network and bridges or bridge elements. The implementation of asset management should increase the integration of network and structure performance requirements. In doing so, bridge managers and road agencies face a number of challenges, these include:

- How to quantify the performance goals and related performance indicators?
- How to translate from network to the object level and vice versa?
- How to establish a complete set of performance indicators?

Network or even societal goals tend to be rather broad in their definition. Furthermore, there is often no exclusive relationship between performance indicators set at a lower level and goals at a higher level. An important notion is that in many countries, the main focus of bridge management is still the condition assessment of the particular objects or elements thereof.

The objective of the COST TU 1406 Action is to investigate the way bridge PIs and KPIs are collected and quantified, how performance goals are specified across Europe, and finally to produce guideline documents linking collection and quantification of PIs, KPIs, performance goals, standards, and practices to decision making processes. This report gives an overview of performance goals at different levels, from high-level strategic decisions to low-level, system-specific requirements. The report also tried to explain how other performance aspects, like traffic safety, availability, economy, environmental and societal impacts could be quantified and used for the multi-objective bridge performance goals assessment, also using it in the case studies.

Future developments should concentrate on the unification of:

- Standardization of the assessment procedures,
- Collection of PIs and quantification of KPIs,
- Development of maintenance optimization tools which can be applied in practice.

The report is a result of efforts within the Working Group 2 of the COST TU 1406 Action.

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8 APPENDIX 1 ENVIRONMENTAL IMPACT PER KG OF MATERIAL ($EE_{i,j}$)

In this appendix the environmental impact per kg of material j for impact category i are displayed. These have been determined with the help of LCA software GaBi or other literature sources if needed. Values can be found in Table A1.

Table A1. Impact per kg of material ($EE_{i,j}$)

Material	Steel	Concrete	Polyester	Glass fibre	Epoxy	Carbon fibre	Asphalt	Gravel	PVC
Impact category									
Abiotic depletion elements (ADP)	-4.93E-06	1.88E-07	4.47E-06	9.15E-05	3.26E-05	0.00E+00	5.96E-09	4.52E-10	1.71E-05
Abiotic depletion fossil (ADP)	6.54E-03	1.80E-04	3.66E-02	1.22E-02	5.79E-02	0.00E+00	9.00E-04	1.38E-05	3.07E-02
Global warming potential (GWP)	1.24E+00	1.21E-01	3.05E+00	1.97E+00	8.25E+00	0.00E+00	5.00E-02	2.28E-03	2.87E+00
Ozone depletion potential (ODP)	1.11E-08	1.26E-12	8.42E-11	9.66E-11	0.00E+00	0.00E+00	2.80E-08	6.74E-13	0.00E+00
Photochemical ozone formation potential (POCP)	5.49E-04	2.33E-05	1.66E-03	-1.69E-03	2.27E-03	0.00E+00	7.10E-05	1.53E-06	1.56E-03
Acidification potential (AP)	3.54E-03	1.83E-04	5.24E-03	1.10E-02	2.13E-02	0.00E+00	2.70E-04	1.47E-05	1.98E-02
Eutrofication potential (EP)	2.80E-04	2.57E-05	6.41E-04	1.38E-03	4.22E-03	0.00E+00	3.40E-05	2.42E-06	1.46E-03
Human toxicity potential (HTP)	2.01E-01	2.40E-02	1.07E-01	4.69E-02	4.87E-01	0.00E+00	3.80E-03	1.49E-04	6.29E+00
Freshwater aquatic ecotoxicity potential (FAETP)	1.13E-02	1.54E-04	1.88E-02	2.32E-03	4.31E-03	0.00E+00	9.40E-04	1.23E-05	1.15E+00
Marine aquatic ecotoxicity potential (MAETP)	3.05E+02	3.35E+00	1.11E+02	1.17E+02	3.05E+02	0.00E+00	1.50E+00	2.56E-01	2.04E+02
Terrestrial ecotoxicity potential (TETP)	4.92E-03	2.72E-04	1.82E-03	1.56E-03	1.08E-02	0.00E+00	4.90E-05	3.51E-05	9.51E-03
source:	GaBI	GaBI	GaBI	GaBI	GaBI	no data	(VWB Asphalt)	GaBI	GaBI

9 APPENDIX 2: INSTRUCTIONAL MANUAL FOR THE APPLICATION OF MAUT WEB-BASED TOOL

In WG2 of TU1406 COST Action, a [web-based tool](#) is developed to apply the multi-objective optimization. The developed tool have implemented one of the method of MCDA namely Multi-Attribute Utility Theory (MAUT) by using the R Utility package (Reichert, Schuwirth, & Langhans, 2013). The concepts of MAUT is explained in Chapter 5, Section 5.2 along with a simple case study. In the following, the instruction for the use of the web-based tool is provided.:

1. A user can only upload a *.csv file in the tool. The *.csv file can be generated directly from MS Excel by 'Save as' option. The uploaded file can have **maximum of four numeric columns**. There is no limit on number of text columns as well as on number of rows in the uploaded file. A sample data file is prepared and provided with this instructional manual.

Application of Multi-Attribute Utility Theory

Written and deployed by Zaharah Allah Bukhsh, PhD Candidate at Ur

Choose CSV File

No file selected

Make sure, uploaded file have four numeric columns

Figure 28: Interface to upload the *.CSV file

- As soon as the data is imported, the single utility function of each numeric is calculated. The tool provides *Data* and *Plot* tabs to visualize the computed utility scores. Moreover, the user can adjust his risk preference for each data attributes by using the slider. Figure 2 shows the interface with sample data that enable the user to adjust his risk preference. The utility score will be continuously updated by changing the risk preferences.

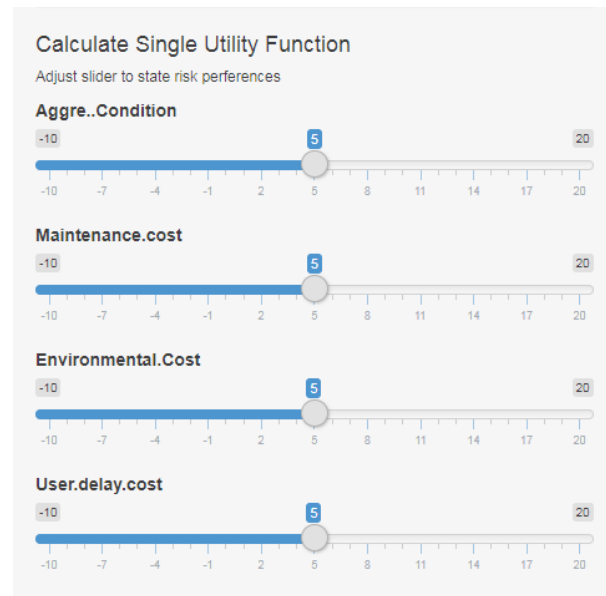


Figure 29: Interface to adjust the risk preference (Sample)

- Following the single utility, the aggregated utility score can be calculated by clicking on the checkbox. A user can assign different weights to each data attributes based on his preferences. Figure 3 shows the interface of tool for this purpose.

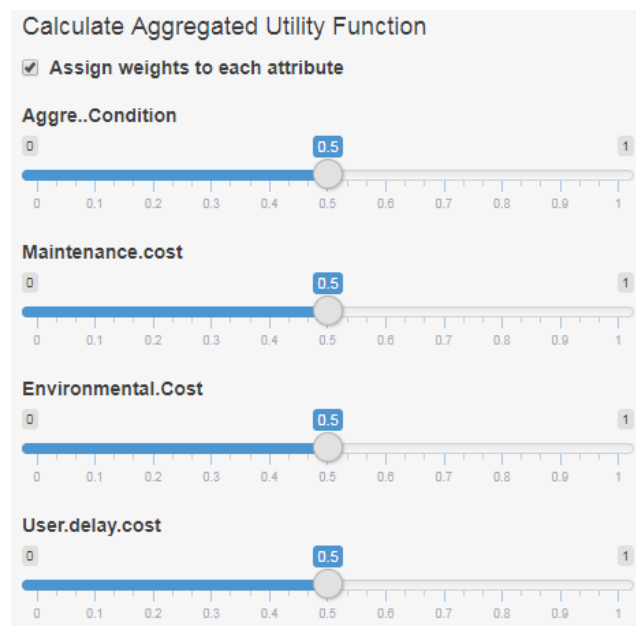


Figure 30: Interface to input the weights for each data point (sample)

- Finally, a user can download the file with computed single-utility score, weights, and additive aggregated value for the uploaded data. Please note, user must provide *.csv extension to file while downloading. Moreover, user can also directly crop-out the plots generated by tool.

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