TU1406

COST ACTION

Quality specifications for roadway bridges, standardization at a European level

WG3

Technical Report

Establishment of a Quality Control Plan

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GLOSSARY

Observation
It is a datum (i.e. piece of information) from a primary source, which may be acquired by human senses or by measuring/recording of some properties via adequate instruments. Observations can be qualitative i.e. only the absence or presence of a property is noted, or quantitative if a numerical value is coupled to the observed phenomenon by counting or measuring. The observation is a perception of human senses or data measured by instrument that is regarded as relevant within the context of the inquiry.

Indicator
It is something that shows what a situation is like. The "situation" depends on the context of an inquiry. The indicator can be qualitative (e.g. bad, good, etc.) or quantitative and is based on analysis of one or several observations.

Performance indicator (PI)
Following the above definition, the "situation" is understood here as performance. The term performance indicator stems from economics and measures the success of an organization or of a particular activity (such as projects, programs and other initiatives) in which it engages. The application of this term to physical objects is coupled to their fitness for purpose. The performance indicator measures fitness for purpose of a physical object such as bridge or its element. Since the fitness for purpose (i.e. quality) can change over time, so does the value of a performance indicator. Maintenance interventions can also change the value of performance indicator and therefore the performance indicators of physical objects also mirror the performance of the agency responsible for their maintenance. It is obvious that bridge performance relates to safety and serviceability, but other performance criteria can be useful as well.

Key performance indicators (KPIs)
Generally, there is no clear distinction between PIs and KPIs. In this project, the KPIs relate to a whole bridge and are as follows:

- Reliability is the probability that a bridge will be fit for purpose during its service life. It is the complement to the probability of structural failure (safety), operational failure (serviceability) or any other failure mode.
- Availability is the proportion of time a bridge is open for service. It does not include failure-related service outages but the ones due to planned maintenance interventions. Alternatively, the Availability can be measured as additional travel time due to an imposed traffic regime on bridge.
- Safety is the situation of life and limb being protected from harm during the service life of a bridge. Loss of life and limb due to structural failure is not included by this definition (since it would overlap with the Reliability).
- Economy is related to minimizing the long-term cost of maintenance activities over the service life of a bridge. Herein the user costs incurred due to detours and delays are not included.
- Environment is related to minimizing the harm to environment during the service life of a bridge.

Quality control plan
The quality control plan specifies all activities and tools, needed to ensure quality requirements related to bridge performance aspects (e.g. safety, serviceability, etc.). It defines the extent and the interval of inspections or investigations and data necessary to estimate key performance indicators and forecast their future development. Quality control plan also includes decision model that suggest maintenance action based on the forecast of key performance indicators. In this sense the quality control plan overlaps with the Strategic Asset Management Plan (SAMP) as defined in ISO 55000.

Failure mode
Failure modes are quasi-permanent or transient situations that violate code specifications or owner's/ operator's provisions. This includes but is not limited to overall bridge collapse. Some of these situations,
e.g. Ultimate Limit State and Serviceability Limit State, are specified in bridge design. Due to slow (deterioration) and sudden (e.g. natural hazard) processes, damages may occur that result in additional failure modes. Finally, owners/operators may define situations (e.g. spalling, corrosion traces, etc.) that are regarded as failure modes since they might comprise public perception of safety.

Vulnerable zones
These are the segments and or elements of a bridge structure in which damages have the largest impact on safety and serviceability. One vulnerable zone may be related to several failure modes.

Ontology
A set of concepts and categories in a subject area or domain that shows their properties and the relations between them. In this report, the Entity Relationship Diagram is used to describe the ontology of the Quality control framework.

Taxonomy
A classification – in this case different bridge types and main girder cross-sections are classified according to construction material, static system and geometry.

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1. Introduction

1.1 The importance of the road infrastructure

There is a broad consensus that the benefits of road infrastructure for the society cannot be overestimated. The investments in road infrastructure raise the growth potential of a national economy, which can be fully exploited by efficient utilization of the road infrastructure. It is difficult to quantify the economic benefit of road infrastructure but its lower bound is estimated to be between 4% (U.S. DOT, 2015) and 10% (*Kurte & Esser, 2008*), (*ARE, ASTRA, 2006*) of Gross Domestic Product (GDP). Apart from purely economic benefit, the road infrastructure enables road users to be involved in various activities that yield private, public and social benefits (*Frischmann, 2012*).

Maintaining these benefits on the long run in economically efficient, environmentally responsible and socially reconcilable manner, is the fundamental task of road authorities. They are bound to provide fast, safe, comfortable, and affordable travel.

Bridges are critical components of the road infrastructure as they ensure fast and safe passages over otherwise hardly surmountable obstacles. From the users' perspective, it is irrelevant whether a road is carried by a bridge or being in a tunnel or merely resting on a soil, so long it provides safe and fast travel from origin to destination. In this context, it is necessary to define what is meant by fast and safe.

There are codes of practice that apply to the design of a road infrastructure and they are related to clearance, speed and weight allowance. The design travel speed defines the minimum legal travel time on an arbitrary road link. In reality, this minimum travel time can be achieved only in the case of unrestricted traffic flow i.e. if road capacity is sufficiently higher than traffic volume. Based on the current or future traffic, one can specify some travel time – exceeding the minimum travel time - as the "fast travel". Clearly, the actual travel times will scatter significantly and the measure of this scatter needs to be considered when defining "fast travel". In this context, one often refers to travel time reliability.

The safe travel however is somewhat difficult to define since it does not imply absolute protection from undesired events. When travelling, there is always a probability of undesired events, which can harm life and limb, induce economic losses and damage environment. The sufficiently low probability of such undesired events defines the "safe travel".

The bridges can play significant role regarding the travel time as their posting can lead to detours and therefore to increased travel time. Furthermore, bridges or parts of them can fail, harming life and limb and inducing adverse consequences to economy and environment. A failed bridge can also cause detours and therefore impact travel times. In some cases, a failed bridge can render a region completely inaccessible with disastrous economic effects. It is therefore not surprising that the structural safety is the primary concern of bridge owners since it affects both safe and fast travel. Besides structural safety, the bridge owners care about serviceability which relates to user comfort that can be affected by deflections and vibrations of a bridge. If safety or serviceability requirements are not met, the bridge will be posted or even closed affecting adversely road users and the whole economy.

Following the definition of the "safe travel", a bridge is regarded as structurally safe if the probability of failure during its service life does not exceed some nominal value. Similar approach applies to serviceability in which the exceedance probability of some service limits (e.g. deflection, frequency, etc.) must be sufficiently low. Most modern codes of practices for bridge design have adopted this concept, e.g. (*CEN EN 1990, 2002*). In addition, the bridge riding surface must also fulfil the requirements applicable to pavements.

The owners need to ensure that their bridge inventory fulfils the safety and serviceability requirements during their service life and therefore maintain the benefits of road infrastructure. Considering the extent of the road infrastructure in developed countries, this is an increasingly challenging task. The prudent bridge owner needs to plan and execute timely intervention in order to cope with following challenges:

- Potentially unsafe bridges that are designed using bygone codes of practice
- Bridges with reduced resistance due to deterioration or mechanical damage
- Bridges exposed to natural hazards that are not or not adequately considered in design
- Increasing traffic volume and loads that can render some bridges unsafe
- Increased exposure of bridges to natural hazards due to climate change

The current approaches and tools available to bridge managers in practice seem to be hardly adequate to cope with these challenges.

1.2 Growing traffic and traffic loads

Safety and serviceability were always the primary concerns in bridge design. However, in course of time the requirements have changed significantly and existing bridges are characterized by several generations of design codes. Also, the codes of practices have changed both regarding actions and resistance models. It is therefore not surprising that the old, otherwise undamaged bridge may not fulfil safety and/or serviceability requirements for the current traffic loading.

The increasing traffic volume and traffic loads are probably the most significant challenge regarding the existing bridges. The traffic mix is shifting toward the larger share of heavy weight vehicles, effectively increasing the occurrence probability of the load situations that exceed current design loads. Furthermore, there is an increasing number of special transports, i.e. the ones that exceed legal limits and require special permits. In Switzerland, some of these transports cannot be regarded as accidental actions but rather frequent actions as passages occur on weekly or even daily basis. Similar situation is also in the USA as reported in (*Lou, et al., 2018*). This trend will increase in the future as the transportation industry is interested in using larger trucks with higher axle loads in order to improve economies of scale. The stiff competition will also lead to platooning i.e. to the trains of wirelessly coupled trucks. The wireless coupling allows to significantly reduce the safety distance between the trucks. This seems to result in a particularly aggressive load situation as these trucks also break simultaneously. The break forces that are mostly neglected in road bridges must be considered in the future and existing bridges need to be assessed for this load situation.

In Switzerland, the Federal Office for Spatial Development analysed several scenarios and estimates a yearly increase in total passenger-distance between 0.7 – 1.0% and in total tonne-kilometres between 1.0 – 1.5% until 2040 (*ARE, 2016*). The number of special (i.e. heavy) transports increased between 2008 and 2018 twofold.

In Denmark, the Danish Road Directorate estimate a 1.8 % yearly increase in total vehicle travel distance (kilometres) for the entire Danish network of roads from year 2016 to 2020 (*DRD, 2017*). In addition, the amount of transport work (tonne-kilometres) for Danish trucks on national roads has increased by 14% in the period 2000-2015. The same trend is seen when analysing the number of permits for special (i.e. heavy) transports. From 2013-2016 a 60% increase was found for the Danish state road network (*Petersen, 2018*).

In Portugal, traffic estimations are not available, but the increase of level of freight transport with GDP is observed (*Garcia, et al., 2008*). Growth estimates published by the Portuguese Central Bank point to a GDP increase of 1.7 % and 1.6 % in 2018 and 2019 respectively (*Bank of Portugal, 2017*). Taking into account the link between mobility (defined as the travel volume) to GDP (*Crozet, 2009*), a similar increase in traffic can roughly be expected.

Slovenia is one of the most transit countries in Europe as the 5th and 10th Pan-European corridors run through it. Consequently, 85% of the freight traffic is international according to the Slovenian statistical office (*Slo-stat, 2018*). Also, in the last four-year period (2014-2017) the average transport work (tonne kilometres) had a yearly increase of 9% and the amount of freight traffic increased by 6% yearly. The number of permits for special (i.e. heavy) transport yearly increased by 10% (*DRI, 2018*). The high traffic increase in the period under consideration is a consequence of stagnation in traffic between 2008 and 2013 due to financial crisis. Long-term freight traffic increase is directly correlated with the Slovenian and neighbouring countries GDP growth.

Between 2011 and 2016 the total passenger-distance in Serbia experienced a yearly decrease of ca. 1%, whereas the yearly increase of total tonne-kilometres amounts to ca. 10% (*Italferr S.p.A., IIPP, Nea, Witteveen+Bos, 2009*). This high increase is partly due the changes in the coverage of reporting units. If this change is not considered, the yearly increase is still higher than 5%.

1.3 Climate change

Safety and serviceability can be jeopardized by deterioration processes or sudden events. The resistance of deteriorated bridges can in time reach a level, at which there is an immediate danger of structural failure. In addition to it, the new insights (e.g. statistical analysis) in frequencies and magnitude of sudden events can render some bridges as unsafe and require posting or even their closure.

Climate change may lead to more frequent and intensive gravitational hazards, such as flooding, avalanches, landslides, rockfall, etc. By the end of this century, depending on their location in Europe the chances that an area will be exposed annually to at least one climate hazard with a current 100-year intensity will increase by a factor of between 6 and 15 times, as discussed in a report by (*Forzieri, et al., 2015*). Also, because of this increase in frequency of hazards, the joint annual exposure expectancy to multiple hazards rises much more sharply than for single hazards. In Southern Europe, 25% of the area could be annually exposed to at least two hazards with 100-year intensity by the end of this century, or nearly 250 times the baseline value. Key hotspots were identified at coastal regions and in floodplains in Southern and Western Europe, which are often highly populated and economically pivotal.

In relation to roadway infrastructure in Denmark, the amount of rain is of primary concern (higher temperature and wind velocity are secondary). The Danish Meteorological Institute has estimated an increase of 14% (+/- 6%) in the yearly amount of rain (scenario A1B) when comparing a reference period, year 1961-90, with projections to year 2100 (*Klimatilpasning, 2015*).

1.4 Decision-making for existing bridges

The decision-making process regarding the maintenance interventions (incl. rehabilitation and replacement) of existing bridges differs somewhat from the one regarding the construction of new bridges. These differences can be summarized as follows:

- Existing bridges already contribute to economy (see *Section 1.1*) and the maintenance interventions on them may result in their total or partial closure that incurs user costs. Opposite to design/construction of new bridges where these costs are not of pivotal importance, they need to be considered in cost/benefit analysis of maintenance interventions.
- In light of the user costs, extending the service life of existing bridges can be beneficial. In most cases it is better to invest into diagnostics of existing bridges, which may render their fitness for purpose. The safety and serviceability margins that apply in design need not to be applied for existing bridges, as they reflect the uncertainties in construction process. These uncertainties may be significantly reduced by simple measurements and sample testing and consequently the safety and serviceability margins can be narrowed.
- The design requirements are closely related to the design service life. If an existing bridge needs to be in service for a significantly shorter time, these requirements can be adapted accordingly.
- Extending the service life of an existing bridge is also environmentally beneficial. The consumption of resources for diagnostics actions cannot be compared to one of a replacement.
- It should be considered however, that the existing bridges are exposed to higher loadings than those considered when they were designed (i.e. due to traffic volume and traffic load increase) and this needs to be duly considered. Additionally, the climate change or new insight with in geology and weather patterns can render existing structures unsafe.

The above-mentioned aspects need to be adequately considered in decision-making process with regards to existing bridges and in preparation of quality control plan.

2. Terms and concepts

2.1 Definition of quality

There is no generally accepted definition of quality, but the most common definitions are:

- Quality is fitness for purpose
- Quality is a degree to which a set of inherent characteristics of a product or service fulfils requirements

Both definitions focus on customer's satisfaction. Applying these definitions to road infrastructure means that the service quality is given if performance goals from the users' perspective are fulfilled during the service life of a bridge. However, there is a broader definition of quality that includes also the service delivery process. In particular, the costs, societal and environmental aspects are included. According to this broader definition the quality is not given if, apart from customers' satisfaction further performance goals related to economic efficiency, environmental friendliness and social responsibility are not met. This broader definition is adopted by the Working Group 3 of the COST TU 1406 Action.

2.2 Definition of quality control

The term "quality control" can have two meanings. To control is both to verify, check or inspect but also to command, direct and rule. The former definition implies a passive task in which the quality is checked and reported. The latter definition is a broader one that includes undertaking all necessary action to ensure quality.

Consequently, the quality control plan specifies all activities and tools, needed to ensure quality. In case of road infrastructure, the quality control plan defines the extent and the interval of inspections or investigations and data necessary to estimate key performance indicators (KPI) and forecast their future development. Quality control plan also includes decision model that suggest maintenance action based on the forecast of key performance indicators. In this sense the quality control plan overlaps with the Strategic Asset Management Plan (SAMP) and Asset Management Plan (AMP) as defined in ISO 55000.

3. Quality of road bridges

3.1 Design and construction – acceptance criteria

When a new bridge is constructed and handed over from a contractor to an owner, it is assumed that it is built/designed according to the valid codes at that time and that all relevant loading cases and traffic demand are considered. The quality of a bridge at this point of time is at an adequate level since all the acceptance criteria e.g. structural safety, serviceability and traffic safety, are fulfilled. The acceptance criteria can be also extended to durability i.e. to fulfilment of structural safety, serviceability and traffic safety criteria during the whole service life. These criteria are broadly referred to as requirements, goals or standards that a bridge needs to meet, and in some countries, they are additionally verified in a „zero inspection", prior to commissioning.

3.2 Inspection and monitoring

After a certain period, an inspection/monitoring is performed on a bridge to determine if it meets the desired/required quality which can differ from the one at the time of commissioning. This is so-called static (snapshot) quality assessment and is performed in a variety of manners, either visually or with an aid of an equipment. If the results show unacceptable variations of quality at this instance, in-depth investigations or interventions may be triggered. Additionally, the results obtained in inspection/monitoring process represent a basis for adequate decision making on actions to ensure the required quality on a long-term. Here, planning is essential to establish a schedule, scope and optimal times between inspections. These are the tasks within a dynamic quality assessment, which is the core of bridge management.

3.3. Operation and maintenance

Bridge Management is not confined to bridge assessment following inspections but also includes maintenance planning & execution. The short-term maintenance planning is based on in-depth investigations and structural analysis and include detailed specification of interventions that are to be taken shortly thereafter. The mid- to long-term maintenance planning is a process, in which different intervention scenarios are developed. Here, there is a possibility to choose among preventative, corrective and operational actions. These interventions are not specified in detail and their costs are rough estimates backed by experience. The goal is to estimate financial and other needs well in advance thus avoid unpleasant surprises. Furthermore, early planning allows to choose the optimum time for interventions and reduce long-term costs.

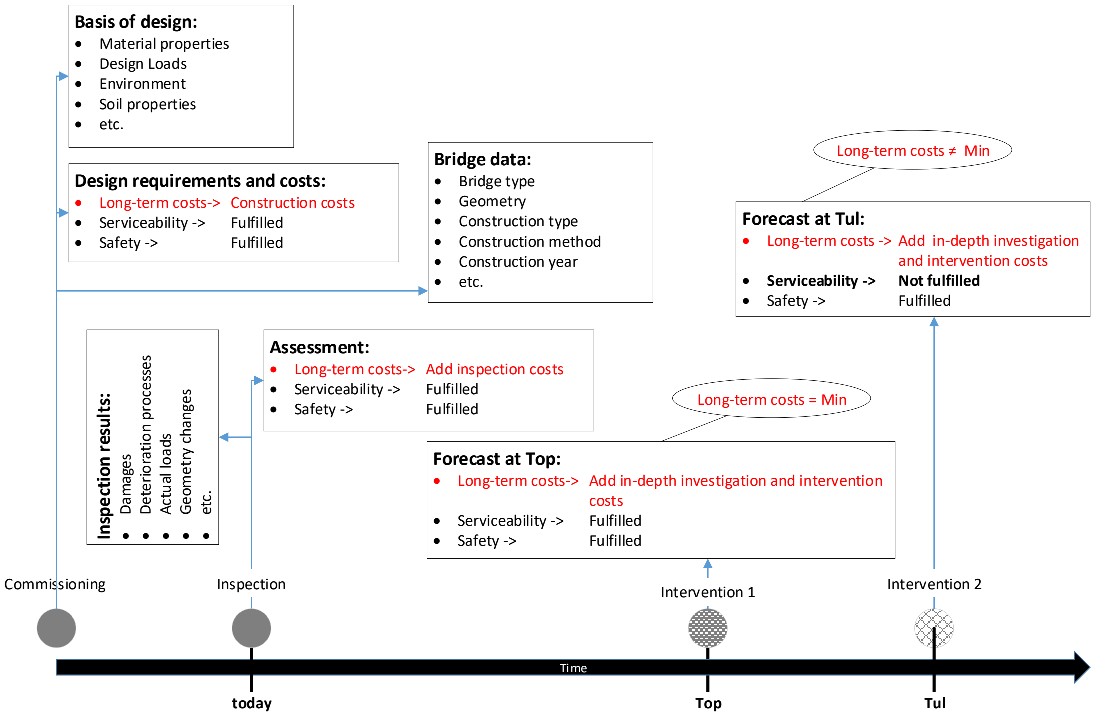


Figure 3.1 General approach to mid- to long-term maintenance planning

The general approach is presented in *Figure 3.1*, where it is assumed that an inspection is performed "today". The results from the inspection revealed some damages that in conjunction with the actual loads lead to worsening of the safety and serviceability levels that however still meet the requirements for existing structures. For mid- to long-term maintenance, planning forecasts for serviceability and safety are performed predicting that serviceability criterion will be not fulfilled at the time instance marked "Tul".

This means that the intervention needs to be executed no later than at that point in time, if serviceability requirements are not to be violated (i.e. a threshold criterion). However, it may well be that a scenario that includes an intervention at the time instance "Top", has lower long-term costs than the one with the intervention at the time instance "Tul". Thus, an extremal criterion related to long term costs have to be included to obtain an optimal solution in a decision-making approach. The *Figure 3.1* does not show any interventions after "Tul", but normally the ensuing interventions are considered in estimation of long-term costs.

The forecasts of safety and serviceability over time defines the time instance at which, at the latest, an intervention is necessary.

3.4 Key Performance Indicators for existing bridges

Following design provisions, the obvious choice for the Key Performance Indicators (KPI) for bridges would be safety and serviceability. Indeed, some suggestions for performance indicators (PI), e.g. in (*Brown, et al., 2014*) and (*Grischa & Sigrist, 2011*), include safety and serviceability but combine them with other performance indicators. As presented in *Figure 3.2*, the serviceability is combined with durability in performance category "Structural Condition" whereas safety is combined with stability to form the performance category "Structural Integrity". The performance category "Costs" include both agency and user costs. It should be noted that the user costs include delay, detour and accidents costs. Finally, the performance category "Functionality" include clearance, ride quality and load ratings and restriction on use.

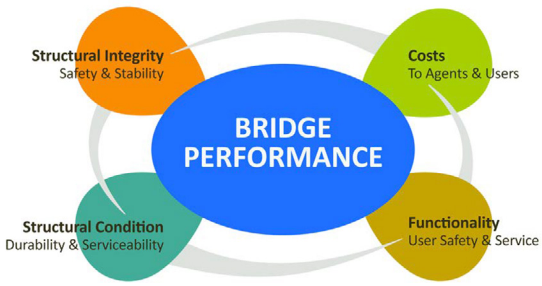


Figure 3.2 Bridge performance, (Brown, et al., 2014)

The most indicators relevant to the bridge performance are included in (*Brown, et al., 2014*), but the classification merits some further consideration. For instance, the structural integrity is related only to sudden events, mostly natural hazards such as earthquake, hurricane and fire. The observable deterioration processes, although they may compromise structural integrity, affect only durability and serviceability. The durability seems to be understood as a span of time in which neither safety nor serviceability is compromised and this understanding is adopted in this document as well.

Within the COST Action 1406, the Working Group 2 (WG2) elaborated the proposal for KPI based on the Dutch **RAMSSHE€P** approach (*Rijkswaterstaat, 2012*). This proposal was approved in the Workshop in Belgrade in March 2016. The following KPIs were defined:

- **S**afety, Reliability and Security (**S**, **R**, **S**) - a combined KPI
- **A**vailability and Maintainability (**A**, **M**) - a combined KPI
- **E**conomy € (i.e. Costs)
- **E**nvironment (**E**)
- **H**ealth and **P**olitics (**H**, **P**) - a combined KPI

One notes that whereas safety is directly considered, the serviceability is not. However, the serviceability is included in Availability. The decision upon the KPIs was followed by a second decision to evaluate KPIs qualitatively with an ordinal scale of one to five. The overall performance is represented with a "spider net" diagram as in *Figure 3.3*. The larger the area in the diagram enclosed by the KPI values, the better is a bridge performance. The most favourable value of a KPI is one, which means that if all KPIs values are in the green area of the diagram, the bridge performs excellently. Clearly, a similar diagram can be applied also for several bridges or even a whole bridge inventory.

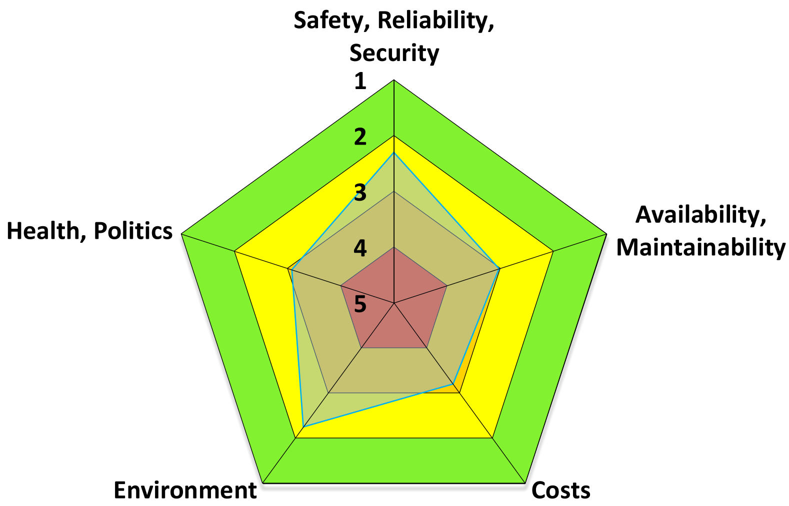


Figure 3.3 "Spider net" diagram, from (Stipanovic, et al., 2017)

The evaluation of KPIs from performance indicators (PIs) and mere observations is still matter of vivid discussions. In addition, it seems that the KPIs from *Figure 3.3* are not completely orthogonal. In particular, the Reliability and Availability seem to be overlapping. Furthermore, some KPIs are difficult to assess at least on a bridge level, notably KPIs Health and Politics but also the Maintainability. The Maintainability is the ease with which a product can be maintained in order to repair damages or their cause, repair or replace faulty components without having to replace still working parts and prevent unforeseen maintenance measures.

This can be understood as a design aspect and it is covered within Economy. The Security is degree of protection against vandalism and it is similar to the Maintainability. The protection against terrorism, which especially in the US belongs to security is not considered in this Action. The KPI of Health is absence of non-failure causes of illnesses (e.g. use of asbestos), which is in the most cases regulated. The KPI of Politics include elimination of causes for public outcry, image protection etc., it is a downstream performance goal i.e. fulfilled if RAS€E goals are met.

Given the latter, the following definitions of the RAS€E KPIs are adopted to comply with the WG3 framework:

- Reliability - the probability that bridge will be fit for purpose during its service life. It is the complement to the probability of structural failure (i.e. safety), operational failure (i.e. serviceability) or any other failure mode.
- Availability - the proportion of time a system is in a functioning condition. It is not reliability-related disruption of bridge users but originates from planned maintenance interventions (e.g. additional travel time due to an imposed traffic regime on bridge).
- Safety - related to minimizing or eliminating the harm to people during the service life of a bridge. The loss of life and limb due to structural failure is not included (see Reliability).
- Economy - related to minimizing long-term costs and maintenance activities over the service life of a bridge. Herein the user costs incurred due to detours and delays are not included.
- Environment - associated with minimizing the harm to environment during the service life of a bridge

Within a QC framework, the KPIs will be evaluated for different maintenance scenarios, looking for the most feasible one. Here, it must be underlined that KPIs of Reliability and Safety can be evaluated based on the inspection and/or investigation at a point in time (i.e. static quality assessment) but can be also be predicted over time (i.e. dynamic quality assessment). On the other hand, the KPIs of Availability, Economy and Environment can be only reasonably applied as a function of time.

4. Damage processes

4.1 Gradual observable and non-observable Damage Processes

Numerous processes can have a detrimental effect on a bridge. Those which may act singly or in combination to generate safety and serviceability problems are here referred to as damage processes. The information on damage processes are crucial for a performance prediction, planning of preventive maintenance as well as for planning of eventual rehabilitation. Some damage processes are gradual and observable (e.g. corrosion related to structural steel). These can be detected with a proper inspection strategy. Other damage processes are gradual and non-observable (e.g. corrosion of post-tensioning steel). These should be handled by a proper maintenance strategy. By means of reliable information on the processes, inspection and maintenance strategies can be optimized (e.g. reduce Life Cycle Costs and traffic disturbance).

In order to assess damage processes, damages should be graded with respect to their nature, intensity, extent and location. The gradation should be in accordance with the damage type, the cause of damage, and the material of an affected structural element. An example of a generic approach for quantitative modelling of bridge damages is explained in *Section 7.4*.

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

- Corrosion,
- Fatigue,
- High Temperature,
- Ponding,
- Overloading,
- Accidental Impact,
- Water retaining and accumulation

Also, the damage process for concrete were proposed according to *Table 4.1*. It should be noted that here, some of the processes are catalysing processes for an actual damage process. For instance, chloride contamination promotes the corrosion of reinforcement, a damage process which eventually causes failures. Other processes/ actions such as accidental damages, seismic activity, vibration, scour and vandalism, where considered out of the scope of DURATINET. For concrete, construction faults are referred to as a cause of early deterioration.

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

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

Specifically, for arch bridges (*Bień & Gładysz-Bień, 2016*) proposed the three main groups of the degradation mechanisms related with physical, chemical and biological phenomena. Degradation mechanisms considered by these authors were paired with terms from WG1 Report (*Strauss & Mandić Ivanković, 2016*) according to *Table 4.2*. A direct correlation is proposed for most of the terms. In some cases, several non-redundant damage process terms are matched to one degradation mechanism.

Table 4.2 Degradation Mechanisms and related Damage Processes in WG1 report

	Degradation Mechanisms (*Bień & Gładysz-Bień, 2016*)	**Damage Processes (WG1 Report)**
Physical	Accumulation of inorganic contamination	Aggradation (alluviation)
	Freeze/thaw actions	Freeze-thaw
	Erosion	Erosion /Abrasion
	Crystallization	-
	Extremal temperature influence	Temperature
	Rheological processes	Aging of material
	Overloading	Overloading of an element
	Leaching	-
	Fatigue	Fatigue
	Changes of geotechnical conditions	-
Chemical	Carbonization	Carbonation
	Corrosion	Pitting / Corrosion related to prestressing steel / Corrosion related to reinforcement steel / Corrosion related to structural steel / Corrosion related to equipment made of steel / Corrosion related to fixings, connectors
	Aggressive environmental impact	Sulphate reaction / Chemical action
	Reactions between material components	Alkali aggregate reaction
Biological	Accumulation of organic contamination	Biological Growth
	Influence of microorganisms	Biological Growth
	Influence of plants	Biological Growth
	Influence of animals	Biological Growth

In *Table 4.3*, the selected Damage processes from *Table 4.2* are related to structural material and their possible impact on the performance of a bridge. The three terms *leaching*, *crystallization* and *carbonization*, referred by (*Bień & Gładysz-Bień, 2016*), can be regarded as catalysing processes. The term *changes of geotechnical conditions* is accounted as changing geotechnical conditions. The overloading of an element was added to the list as a damage process that is induced by human activities.

Table 4.3 The proposed list of Damage Processes

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

4.2 Sudden events – natural hazards

The analysis of the impact that natural hazards have on bridges and transportation infrastructure is yet to be included in the future BMS. The older bridges are often not or not adequately designed for natural hazards and it is likely that the climate change has an adverse impact on frequency and intensity of gravitational hazards. The risk-based maintenance planning is actually developed for natural hazards and is being used by agencies world-wide, e.g. (*ASTRA, 2014*). It allows therefore to treat natural hazards in the same way as, for instance, traffic loads. The related failure modes and corresponding probabilities of failure (see *Section 6.4*) must be defined in the same manner as for traffic loads. This task can be more difficult than the one for traffic loads, since some bridges were not designed for extreme loadings. Furthermore, the probabilistic characterization for hazards is more difficult. The bridges must be examined for different frequency and intensity of hazard events. Based on this analysis, the probability of failure can be assessed as a function of hazard intensity.

This approach for local scour induced by flooding events as the predominant cause of bridge collapses is explained in (*Tanasic & Hajdin, 2017*). Herein, the failure modes of RC girder bridges are based on water-soil-structure interaction. This means that the resistance of a bridge and its ability to prevent collapse of river bed under the foundation is duly considered. Following the need to improve the behaviour of bridges in an earthquake event, a risk-based approach was developed by (*Padgett, et al., 2010*) for girder bridges.

Within the scope of WG3, the flood hazard and the related local scour are addressed (see *Section 10*).

5. Performance indicators and observations

5.1 Conceptual remarks

When a certain damage processes is initiated at a bridge, it will be eventually manifested by consequences that might be visible for bridge inspectors and recorded as observations. While observing and assessing the bridge, inspectors will try to figure out which damage processes could act on the bridge or its element. These observations and the knowledge about damage processes allows inspectors to determine a correct diagnosis. Here, the understanding of possible consequences is essential.

The observation is a perception of human senses or data measured by instrument that is regarded as relevant within the context of the inquiry. Observations can be qualitative i.e. only the absence or presence of a property is noted, or quantitative, if a numerical value is coupled to the observed phenomenon by counting or measuring.

A Performance Indicator (PI) measures the fitness for purpose of a bridge. For instance, a crack width larger than 0.4 mm can be a sign that the reinforcement yielded (at least once) and can be the indicator of an insufficient resistance or equally likely of one-time overloading. In this case, the same observation can indicate two different outcomes regarding reliability: one with an impact on reliability and one with no impact on reliability but on an irresponsible transportation company. In subsequent inspections, this dilemma can be cleared by investigating if the crack grows. So, there is a difference between observations and PIs, as the first are 'just the fact' and the latter is already an interpretation of its impact on a bridge performance.

The relationship between observations in bridge inspections and PIs is essential part of quality control for bridges. The definition of this relationship requires a deep understanding of the underlying damage processes that materialize as observations.

5.2 Damage Processes and Observations

A catalogue of possible observations may be established based on literature on RC girder and frame bridges:

- Bridges for Service Life Beyond 100 Years (SHRP 2) (*TRB, 2014*)
- BRIME Project (*BRIME, 2001*)
- Contectvet (*Contectvet, 2001*)
- COST 345 (*COST345, 2004*)
- Fib bulletins, e.g. no 22 on monitoring and safety evaluation of existing concrete structures (*fib, 2003*)
- Lifecon (*Lifecon, 2003*)
- Mainline (*MAINLINE, 2014*)
- Rehabcon (*Rehabcon, 2004*)
- Sustainable Bridges (*SustainableBridges, 2007*)
- Long Term Bridge Performance (*LTBP, 2016*)

It should be noted that some observations are symptoms, i.e. they have no direct impact on static (i.e. snapshot) KPIs (Reliability and Safety). If symptoms are interpreted in a dynamic context, i.e. for planning purpose, they may with very few exceptions have direct impact on relevant KPIs (Availability and Economy).

The survey and collecting of terms related with Performance Indicators (PIs) from different national documents, conducted by WG1 (*Strauss & Mandić Ivanković, 2016*), resulted in a list of more than 700 terms, later clustered and homogenized. From this process resulted a shorter list of 385 terms, grouped in 11 clusters from defects to rating and loads. The list was considered not as a final list of the PIs but as a list of terms that can be related with PIs.

Assuming that not all the 385 terms can be considered as PIs, a new categorization of the terms was suggested. For this purpose, the four categories were considered, corresponding to the common framework for the establishment of QCPs: Design & Construction, Observations, Damage Processes and their symptoms (see *Section 7.1*).

The category Design & Construction refers to certain properties or observations that influence to the original performance of the virgin (i.e. undamaged) state of the bridge. Usually these are not related with an interceptable damage process acting during the lifetime of the structure but can foster them. The selected terms are presented in *Table 5.1*.

Table 5.1 WG1 and WG3 correlation / Terms related with Design & Construction

WG1 Cluster	Design & Construction
Defects	Insufficient concrete cover
Related to material properties	Concrete quality insufficient
Related to material properties	Porous concrete
Related to material properties	Bad concrete compaction
Related to material properties	Aggregate segregation
Related to original construction and design	Formwork residuals
Related to original construction and design	Formwork settlement
Related to original construction and design	Mounting deficiency

From the list proposed by WG1, after extracting all redundancies and synonyms, twenty terms / Observations that may have a direct impact on performance (Reliability and Safety) were selected and presented in *Table 5.2*.

Table 5.2 WG1 and WG3 correlation / Terms related to Observations and Performance Indicators

WG1 Cluster	Observations/ Performance Indicators
Defects	Cracks
Defects	Crushing
Defects	Rupture
Defects	Delamination
Defects	Scaling
Defects	Spalling
Defects	Holes
Defects	Debonding
Defects	Obstruction/impending (e.g. of water flow)
Geometry changes	Displacement
Geometry changes	Deformation
Related to bearing capacity, structural integrity and joints	Wire break
Related to bearing capacity, structural integrity and joints	Presstresing cable failure
Related to bearing capacity, structural integrity and joints	Reinforcement bar failure/bending
Related to bearing capacity, structural integrity and joints	Stirrup rupture
Related to bearing capacity, structural integrity and joints	Tensioning force deficiency
Related to bearing capacity, structural integrity and joints	Loss of section (reduced section)
Related to bearing capacity, structural integrity and joints	Deteriorated mortar joints
Related to dynamic behaviour	Frequency
Related to dynamic behaviour	Vibrations/oscillations

Some simplifications were made to consider one term that generically define an observation. For instance, WG1 report listed several terms in the cluster labelled as “geometry changes”; whereas WG3 only use the terms “displacement” and “deformation” as they might outline all possible geometry changes that may take place in an element. Another simplification made is related with the cluster “equipment & protection”.

None of the terms proposed in this cluster were included since they are redundant with other more generic terms, independently from the element where they are observed (e.g. the term “approach slab settlement” is considered redundant with the generic term “displacement”, therefore only the latter was considered).

Also, generic terms were preferred instead of more specific terms related with one special type of structures. Taking as an example masonry arch bridges, where a common damage is the arch ring separation, one can consider that the generic term *crack* can define the observation while the specific reference to the *arch ring separation* is related with the type of a failure mechanism.

In order to access the impact of a certain observation on performance, the identification of active damage process/es is essential. The correlation of observable symptoms with potential damage processes may reveal what damages can be expected or what observation one might make in the future. *Table 5.3* summarizes most common drivers for each of the selected Performance Indicators. *Table 5.3*

Table 5.3 Common drivers for the selected Performance Indicators

Damage Process \ **Observations / Performance Indicator**	Cracks	Crushing	Rupture	Delamination	Scaling	Spalling	Holes	Debonding	Obstruction/impending	Displacement	Deformation	Wire break	Presstresing cable failure	Reinforcement bar failure/bending	Stirrup rupture	Tensioning force deficiency	Loss of section	Deteriorated mortar joints	Frequency	Vibrations/oscillations
Abrasion			•				•				•	•					•	•	•	•
Aggradation (alluviation)									•	•	•								•	•
Erosion	•		•		•		•			•	•	•		•	•		•	•	•	•
Changing geotechnical properties	•	•	•				•			•	•	•	•	•	•	•			•	•
Aging of material	•							•		•	•					•	•	•	•	•
Alkali aggregate reaction (alkali-silica reaction)	•			•						•	•			•	•	•			•	•
Sulphate reaction	•			•	•	•	•			•	•			•	•	•			•	•
Chemical attack				•	•	•					•	•	•	•	•		•	•		
Fatigue	•		•								•	•	•	•	•			•	•	•
Pitting corrosion	•		•		•		•					•	•	•	•		•		•	•
Corrosion related to prestressing steel	•	•	•										•				•		•	•
Corrosion related to structural steel	•		•		•												•		•	•
Corrosion related to reinforcement steel	•		•	•	•	•		•						•	•		•		•	•
Corrosion related to equipment made of steel	•		•		•												•		•	•
Corrosion related to fixings, connectors	•		•		•			•									•		•	•
Overloading of an element	•	•	•							•	•	•	•	•	•	•		•		
Biological growth	•	•	•				•	•	•	•	•							•	•	
Freeze-thaw	•			•	•	•	•	•			•						•	•		
High temperature				•						•	•					•		•	•	•

In addition to those observations that may have direct impact on performance, there is the group of observations that are merely symptoms of damage processes, not affecting a current performance (*Table 5.4*). However, the observation of gel exudation in a certain concrete element might be related with the occurrence of internal pressure (e.g. alkali silica), that in time will certainly affect future performance.

Table 5.4 Observations as Symptoms of a Damage Processes

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

6. Performance assessment

6.1 Current practice and its deficits

Performance assessment practice differs quite significantly from country to country, but the common denominator is that it relies on visual inspections. The visual inspections are – if performed by a qualified structural engineer – cost efficient and very valuable source of information. During the inspection, observations are recorded and evaluated. The result of inspections is a qualitative indicator, which is named differently from country to country as condition rating, condition state, condition class, etc. Whereas in the design phase the safety and serviceability concerns are addressed directly in quantitative manner, in the service phase, based on inspection results the condition state is determined, which is a qualitative indicator. The condition state is a vague measure for the deviation of inspected bridge from the "as new" condition. The direct assessment of safety and serviceability is regarded as not cost efficient since it is commonly assumed that it always requires an in-depth material investigations and structural analysis.

Based on a condition state, owners trigger often costly in-depth investigations or even maintenance actions. In practice, once an in-depth investigation – based on condition state is triggered, the maintenance intervention is very likely to follow, even if a bridge can still be used without restrictions. The reasons are different from country to country, but one is surely the visual appearance and related perception of safety that leads to maintenance interventions to remove all visible damages. In some cases, a maintenance action is triggered if a bridge fails structural safety and serviceability checks, with the load and resistance models for the design of new bridges. This is clearly inadequate and uneconomical, given the remaining service life and possibilities to reduce uncertainties on existing bridges. Some countries have introduced safety and serviceability checking formats for existing bridges in their code of practice, e.g. (*SIA, 2011*), and there are also model codes (e.g. (*fib, 2006*)) and national guidelines available (e.g. (*AASHTO, 2018*)).

In the design phase, the wealth of information about safety and serviceability for different load situations is created. This information is unstructured and mostly in paper form. After the commissioning of newly constructed bridge, the documents containing this information are handed over to the bridges' owners or operators that act further on as trustees of the bridges assigned to them. The documents are mostly in archives and in general not easily accessible. During the service life, inspections are performed with no consideration of safety and serviceability information produced during the design phase. It only within the in-depth investigation that the safety and serviceability are assessed again. There is a substantial gap during the service life of a bridge, in which decisions are made based on qualitative indicators, that are sometimes unrelated to the key concerns of bridge owners: safety and serviceability. In most countries, performance assessment is supported by databases, in which the results of inspections are stored, sometimes in great detail. The information from design phase i.e. critical load combinations, safety factors, assumed traffic loads is usually not stored in these databases. In some road agencies, there are load rating software that facilitate evaluation of special transports, but it is rarely used in conjunction with inspection results.

Even more surprising is that the relevant information on safety and serviceability is often not stored in the database after maintenance interventions. It can be assumed that the provisions of the current code of practice are fulfilled due to maintenance interventions, but it is not recorded if these are exceeded and by what margin.

It should be noted that within the in-depth investigations a substantial work effort is necessary to find information from the design phase or previous maintenance activities. In some cases, the information on existing bridges is lost due to negligence or some accident (e.g. fire, flooding). In some ways, the current performance assessment undergoes amnesia because

- relevant information from the design phase and/or in-depth investigation is not stored and/or
- information is stored only in paper form and is lost due to negligence or accidents

In the following chapters the methodology for assessment of structural safety and serviceability as ingredients based on inspection results and data from design phase is outlined. In the first step the safety and serviceability of the undamaged structure is evaluated based on the available data. In the second step the safety and serviceability are updated base on the inspection results. In the *Section 7.5* the methodology is further extended to other KPIs.

6.2 Safety and serviceability of an undamaged structure

The information on structural safety and serviceability margin of an undamaged (virgin) bridge is essential in the service phase. This information should be structured and include all relevant load cases, which would also allow owners to have a clear picture of possible failure modes that need to be observed in more detail (*see Sections 6.4 and 7.2*).

The current databases are not structured to accommodate the graphical presentation of structural systems and load situations. The material properties and load actions need to be included in the database as searchable data and coupled with graphical representations. The same apply to load models and provisions of current and previous codes of practice. A large effort is required to obtain and store information for all existing bridges, and this cannot be done within a short time period. Ideally, it could be done together with inspections or in-depth investigations and in this way, one can gradually fill the data-base.

Fortunately, there is a simplified method to assess safety and serviceability margins due to traffic loads. If the load model used originally for the design of a bridge is known, one can assume that the bridge is designed according to it. This means that the bridge resistance for the analysed limit states is in minimum as high as to sustain the internal forces due to the load model multiplied with the safety factor. In some sense, the originally used design load model is a proxy for the resistance (or a service limit) of the bridge. However, to obtain internal forces, one still need a structural system.

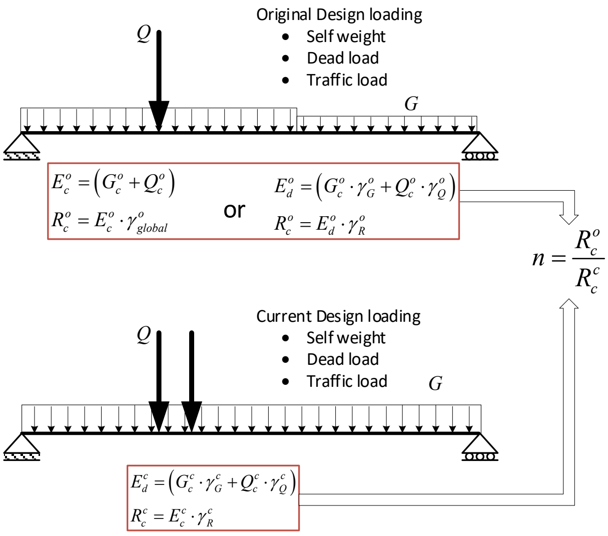


Figure 6.1 Assessment of safety and serviceability margins

Based on the experience from the load rating software (*Hajdin & Despot, 1999*) and (*BMVI, 2016*), the most non-landmark bridges can be simplified with a series of simply supported beams. Somewhat more sophisticated alternative is to model a bridge with a continuous girder as used in traffic simulations (*Freundt & Bönning, 2011*) and (*Meystre & Hirt, 2006*). The simplified model considers only the load transfer in longitudinal direction. The load transfer in transverse direction i.e. across the deck is indirectly considered by defining appropriate effective widths. If the bygone codes of practice did not require dimensioning in transverse direction, the presented simplification can be problematic. Furthermore, modelling of skew decks or girders with skewed support with simply supported beams is challenging and requires good understanding of the load carrying paths in these structures.

The current load model, i.e. the one that represent current traffic is to be applied on the same simplified structural system and maximum moment and shear forces are compared with the ones of the original design code. This concept is explained in *Figure 6.1* following the notation of the (*CEN EN 1990, 2002*). For the loading situation used in design, the load effect E_c^o is computed. For the bygone codes of design this value is multiplied with the global safety factor to obtain the characteristic value of required resistance R_c^o against the computed load effect. If the design is performed correctly one can assume that the real resistance value (i.e. allowable stress) will not be lower than R_c^o. For the newer code with partial safety factors, the load effect E_d^o need to be evaluated and this value is multiplied with the partial factor γ_R^o covering the uncertainty in the resistance model to obtain R_c^o. The current design loading is assumed to mirror the current traffic loading sufficiently well and therefore one can expect that the load effects from the real loading will not exceed the load effect of design loading E_d^c. The required resistance due to the current design R_d^c can be computed by multiplying E_d^c with γ_R^c. The degree of compliance n is the ratio of required resistances R_c^o and R_c^c. The values below one indicates that the bridge may have a safety/serviceability deficit.

Clearly, using this simplified method, safety and serviceability reserves that stem from bridge specific detailing and dimensioning decisions cannot be assessed. To this end one needs to perform a thorough structural analysis on more sophisticated structural system. Nevertheless, the simplified method can be efficiently used for screening purposes.

Owners can decide, based on their needs to store

- the actual structural system as used in analysis or
- a continuous girder model as used often used in simulation to obtain maximum load effect or
- a series of simply supported beams as in some load rating software

Independent of this choice, the results of a thorough structural analysis on an adequate structural system can be used to update the resistance (or service limits) of the model stored in the database. The safety or serviceability margins against the current design loads can be expressed therein either as:

- the degree of compliance n, that can be evaluated for each load situation as a ratio between the resistance/service requirements based on the total factored load effect and available resistance/service limit, or as
- the traffic load capacity factor with which the traffic load can be multiplied and still fulfil the safety and serviceability requirements

The latter seems to be more useful for owners to assess special transports or future traffic load.

6.3 Reliability

The modern codes define the safety and serviceability in terms of reliability i.e. the probability that a bridge will be fit for purpose during its service life, which is the definition also adopted within the WG3 approach. The partial safety factors in modern codes are calibrated to satisfy these reliability requirements. In (*CEN EN 1990, 2002*) the target annual reliability index for safety is 4.7 (corresponds to occurrence probability of $1.3 \cdot 10^{-6}$) and for serviceability 2.9 (corresponds to occurrence probability of $1.9 \cdot 10^{-3}$).

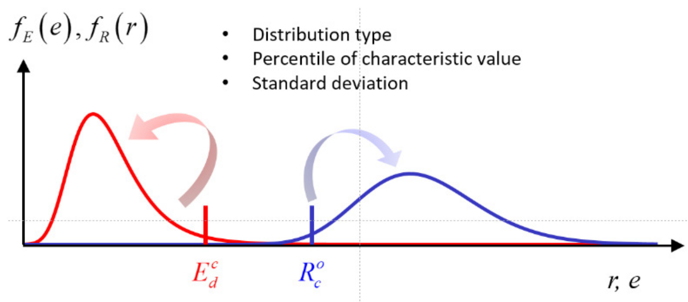


Figure 6.2 Estimating distribution of load effect e and resistance r

The bridge is considered as safe and serviceable if specified reliability indexes are not below these target values. If for an existing bridge the degree of compliance is below one, one can assume that the reliability index for safety exceeds the target value. However, the degree of compliance above one does not necessarily mean that the bridge does not meet reliability requirements. In (*Das, 1999*), traffic load capacity factor is evaluated based on reliability assessment for 15 similar concrete bridges in UK that are constructed in 1960's and 1970's to the same design requirements. Nevertheless, the traffic load capacity factors – derived from reliability assessment - vary between 1.9 and 5.2. Given that at that time the design was based on global safety factor, the results are not surprising. For the bridges that are designed or examined using modern codes of practice, the scatter is significantly smaller as demonstrated in (*Mertz, 2005*). However, even for these bridges the evaluation of reliability may be economically beneficial if existing bridges can still be used without restrictions.

Assessing the reliability of existing bridges can be tedious task as one needs to model all actions and material properties as stochastic variables. However, based on experience and available data, a simplified reliability assessment can be performed using the similar approach which was discussed in the previous section. The original design load situation can be used to assess the characteristic value of resistance against the chosen failure mode. For the example in *Figure 6.1*, the characteristic value of resistance is R_c^o. If the percentile of the characteristic value R_c^o and the type of its distribution is known, one can obtain the resistance distribution for a chosen failure mode. The recommendations for percentiles of characteristic values, standard deviations and distribution types based on construction materials can be found in literature, e.g. (*JCSS, 2013*). To compute load effects current design loading needs to be used. In general, several actions contribute to load effect and their characteristic values represent different percentiles of their distributions. In addition, the actions are modelled with different distribution types. For instance, the load effect due to the traffic load is normally modelled by extreme distributions (e.g. Gumbel max). This is supported by measurements and simulation that are performed within research projects all over the world, e.g. (*Freundt & Bönning, 2011*), (*Meystre & Hirt, 2006*), (*Mertz, 2005*) and (*Caprani, 2013*). After all, traffic load models are derived from probabilistic analysis based on traffic simulations and weigh-in-motion measurements, e.g. (*Calgaro & Sedlacek, 1992*). The distributions for the self-weight and dead load as well as for other actions can be derived based on percentile assumption of their characteristic values following the recommendations from literature, e.g. (*JCSS, 2013*). The derivation of distribution for load effect e and resistance r, for the chosen failure mode is illustrated in *Figure 6.2*.

The load effect can be determined on the simplified structural system and the reliability index can be computed. The reliability obtained in this manner can be regarded as a rough estimate as the material and action uncertainties are modelled based on literature and experience data. These results can be improved if reliability analyses would be performed on the relevant sample of the bridges of same type. These systematic detailed reliability analyses can be also used to update assumptions regarding distribution of stochastic variables.

The traffic load capacity factor can be also obtained based on reliability assessment as in (*Das, 1999*). It is a deterministic coefficient with which the stochastic effect of traffic load can be multiplied and still fulfil the reliability criterion.

6.4 Impact of observations - change from undamaged structure

The proposed methodology relies heavily on information from the design phase or from in-depth investigation and here it differs heavily from the approach described in *Section 6.1*. This includes following information that is indispensable if the effect of deterioration and damages is to be appropriately considered in assessment of bridges:

- Relevant failure modes defined based on the design documentation. These failure modes correspond to the critical load situations used in design and
- Vulnerable zones, for each failure mode (e.g. (*Linneberg, et al., 2017*) and (*NYSDOT, 1997*))

Vulnerable zones are those segments and/or elements of a bridge structure in which damages have the largest impact on safety and serviceability. One vulnerable zone may be related to several failure modes. Experienced inspectors know intuitively where the vulnerable zones are, but they can reassure themselves with readily available information (e.g. see *Section 7.2*). The damages outside vulnerable zones can also trigger failures, but for them to occur the extent of damages need to be significantly larger than in the vulnerable zones. If this seems likely, one needs to define an additional failure mode that can be triggered by the observed damages. This decision is up to owner/operator, for instance, a certain crack width or an extent of spalling can be chosen to be failure criteria, as they affect serviceability (i.e. KPI of Reliability) and KPI of Safety, respectively. It must be noted that failure modes related to serviceability cannot be readily combined with those related to structural safety (i.e. collapse) in the evaluation of the Reliability. For this, an adequate consequence analysis is necessary, i.e. a risk-based approach. Alternatively, an owner/operator can introduce own performance thresholds and assessment procedures (e.g. a weighted sum) to distinguish failure mode type importance in structuring of a QC plan (*Figure 6.3*). If the information stated above is available before inspections, the inspection procedures need not to change significantly.

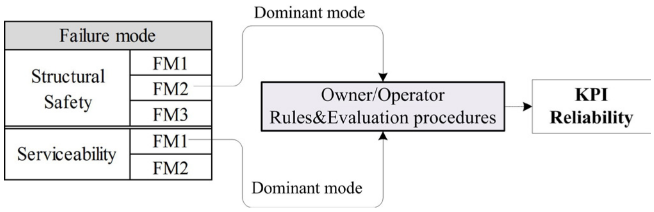


Figure 6.3 From failure modes to KPI of Reliability

For illustration purposes in this section, the failure modes are chosen to be collapse mechanisms. For a simply supported beam the vulnerable zones and corresponding failure modes FM1, FM2 and FM3, are presented in *Figure 6.4*. For each failure mode, the corresponding degree of compliance, if semi-probabilistic format is used, or reliability, if probabilistic assessment is required, is to be evaluated for an undamaged bridge. This can be done beforehand either using the simplified approach described in *Section 6* or by an in-depth examination. For illustration purposes of this procedure, a simply supported beam with the span of 10m is assumed to be loaded only by live load P, whose maximum annual value is normally distributed with a mean value of 100kN and standard deviation of 12kN (*Figure 6.5*)

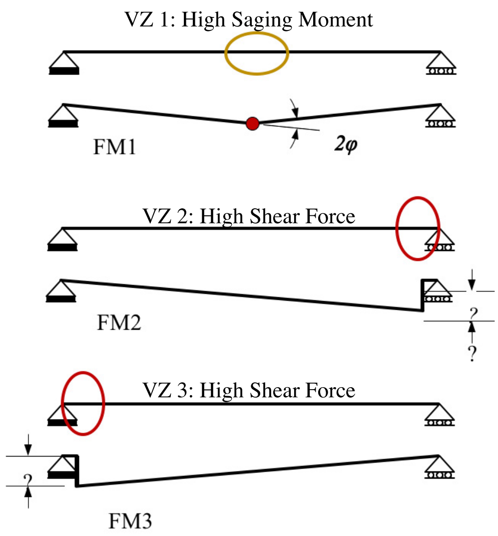


Figure 6.4 Vulnerable zones (orange and red ellipses) and related failure modes

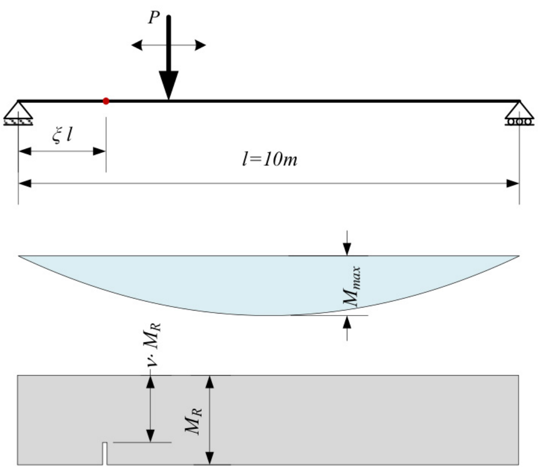


Figure 6.5 Load and resistance of a damaged simply supported beam

Permanent loads are neglected in this example. The resistance to bending moment is assumed constant along the beam and normally distributed with a mean value of 500kNm and standard deviation of 40kNm. Only failure mode FM1 is considered. Since both load effect and resistance are normally distributed, the reliability index and corresponding probability of failure can be computed as follows:

$$\beta = \frac{\mu_{M_R} - \dfrac{\mu_P \cdot l}{4}}{\sqrt{\sigma_{M_R}^2 + \left(\dfrac{\sigma_P \cdot l}{4}\right)^2}} = \frac{500 - 250}{\sqrt{40^2 + \left(\dfrac{120}{4}\right)^2}} = \frac{250}{40 \cdot \sqrt{\dfrac{25}{16}}} = \frac{1000}{200} = 5.0 \quad \Rightarrow P_f = 2.87 \cdot 10^{-7} \tag{1}$$

The à priori reliability i.e. the reliability of undamaged bridge meets the criteria according to (*CEN EN 1990, 2002*).

Once the observation has been collected, the impact of this observation on safety and serviceability is to be evaluated. Based on the observation the rough estimate of resistance reduction can be made, even if the uncertainty is large. If the safety margin of undamaged bridge is large, it can clearly withstand much larger damage than the one with a slim margin. It is therefore that the same damage i.e. the same condition state can lead to different decisions, based on the safety margin of the undamaged bridge.

In the illustrative example it is assumed that the observed damage lead to reduction of resistance, but there is uncertainty regarding the magnitude of this reduction. This uncertainty can be expressed by a discrete distribution as in *Table 6.1*.

Table 6.1 Likelihood of a resistance reduction – an example

Resistance reduction (1 - ν)	5%	10%	15%	20%
Likelihood	60%	20%	10%	10%

In most databases the location of damages is not recorded. This means that the location of damage is not known and therefore a uniform distribution along the whole must be assumed. The reliability index can be expressed as a function of the damage location (ξ = % of the span) as follows:

$$\beta(\xi) = Min\left(\frac{\nu \cdot \mu_{M_R} - \mu_P \cdot \xi \cdot (1-\xi) \cdot l}{\sqrt{\sigma_{M_R}^2 + \left(\sigma_P \cdot \xi \cdot (1-\xi) \cdot l\right)^2}}, 5\right) \tag{2}$$

With available information one can evaluate the probability of failure as follows,

$$P_f = \sum_{i=1}^{4} \vartheta_i \int_0^1 \Phi\left(-\beta(\xi)\right) \cdot d\xi \tag{3}$$

Where v_i represents the values of discrete distribution from *Table 6.1*. The resulting reliability index is

$$P_f = 3.15 \cdot 10^{-5} \Rightarrow \beta = 4.00 \tag{4}$$

which means that à posteriori (after inspection) reliability index does not meet the requirements of (*CEN EN 1990, 2002*). However, this assessment can be improved with some additional information: The reliability index can be plotted as a function of the damage location ξ as in *Figure 6.6*

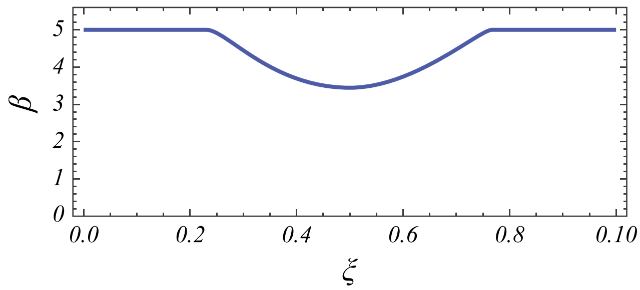


Figure 6.6 Reliability index β as a function of the damage location ξ

In this plot, the zone with an effect on reliability index is clearly visible. If the damage location is known, the reliability index can be updated. For instance, if the damage is in the far left quarter of the span, the reliability index is not affected i.e. the bridge meets the reliability criterion. On the other hand, if the damage is in the middle of the span the reliability index would be:

$$P_f = 1.66 \cdot 10^{-4} \Rightarrow \beta = 3.59 \tag{5}$$

This means that the bridge does not meet the reliability requirements. With this approach the inspection data can be rationally used in reliability assessment taking into account the uncertainty associated with it. The likelihoods as in *Table 6.1* can be refined as more data are collected.

In the example above, an isostatic system is chosen, which means that a local failure in one of the vulnerable zones is sufficient to trigger collapse of a bridge. For hyperstatic systems, e.g. frames, several local failures are required to trigger the collapse, thus it is necessary to associate vulnerable zones to a collapse mechanism (*Figure 6.7*).

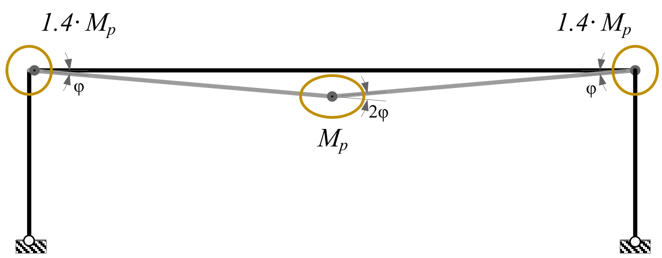


Figure 6.7 A collapse mechanism of a hyperstatic system and related vulnerable zones

It is assumed that the hogging plastic moments over the columns are 40% higher than the sagging plastic moment in the mid-span. Additionally, an inspection revealed that the damages in the mid-span lead to a plastic moment reduction of 15% and the ones over the columns are responsible for the plastic moment reduction of 5%. However, to assess reliability, one needs to estimate the reduction of resistance for the whole bridge and not only in vulnerable zones. In this case, the resistance is equal to the internal dissipation work and the resistance reduction can be obtained by comparing the dissipation work of the undamaged frame with the dissipation work of a damaged frame:

$$r = \frac{0.95 \cdot 2 \cdot 1.4 \cdot M_p + 0.85 \cdot 2 \cdot M_p}{2 \cdot 1.4 \cdot M_p + 2 \cdot M_p} = \frac{4.36}{4.8} = 0.91 \tag{6}$$

The resistance reduction amounts to 9%, which is significantly smaller that 15% plastic moment reduction in the mid-span.

Finally, it should be noted that the presented approach can be implemented as Bayesian net as in (*Isailovic, et al., 2018*). Bayesian nets are particularly useful in combination with Structural Health Monitoring (SHM). The monitoring data can be used on-the-fly to assess reliability of bridges.

If the results of an inspection raise doubts regarding acceptable safety and serviceability, the in-depth investigation should be triggered. The in-depth investigation should include all measures that may reduce uncertainties such as testing of material properties, checking of dimensions, stiffness measurements, etc. Furthermore, archive documents must be duly examined since they may reveal assumption made regarding load situations and material properties. For hidden structural elements such as foundations and reinforcements, old plans are the only source of information. Finally, the exposure of bridge to natural hazard needs to be investigated based on the newest observations (e.g. update of magnitudes, re-assessment of site conditions, etc.).

The structural analysis needs to be performed with nonlinear methods that address failure modes appropriately and yield realistic failure probabilities. This does not necessarily mean that sophisticated analysis is necessary, in most cases the skilful application of the limit theorems of the theory of plasticity is sufficient. The suitable structural analysis methods for evaluation of system behaviour have been discussed in (*SustainableBridges, 2007*).

The bridge should be examined for all relevant load situations and the results need to be stored in the database in a structured form. Gradually, by means of in-depth investigations, high quality information on all bridges will enter the databases allowing bridge owner to manage their inventory safely and efficiently. The growing experience will also allow for more accurate assessment of reliability based on inspections as Bayesian nets can be updated introducing new high-quality data.

7. Quality Control framework

7.1 General explanation of the Entity Relationship Diagram

The framework ontology for the most important entities has been established and presented in *Figure 7.1* as so-called Entity Relationship Diagram (ERD). Here, the “crow foot” symbolizes one-to-many relationship whereas the “crow foot with a circle” stands for one-to-zero or one-to-many relationships.

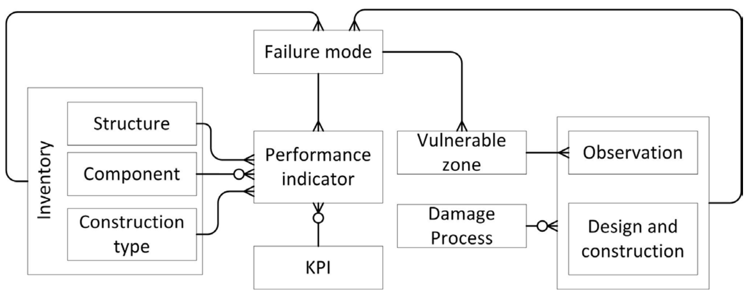


Figure 7.1 The framework ontology

The entity “Structure” includes all bridge types (e.g. girder, frame, arch bridges) and the entity “Element” all element types (e.g. beams, decks, piers). In the entity group Inventory, there can be other entities apart from “Construction type” such as “Geometry” and “Construction method”. The entity “Design and construction” include several other entities related to original design such as construction year, design loads, soil characteristics, etc. The entity “Observation” comprises damages, geometry changes, etc.

The diagram can be interpreted as follows: There is an observation (e.g. a crack) with a certain property (e.g. crack width), on an element of a certain type (e.g. a beam), with the location in a vulnerable zone (see *Section 7.2*) that is related to a specific failure mode of a structure (e.g. a girder bridge). With an influence of other data (e.g. the construction year), this observation will have an impact on a defined KPI expressed by a performance value. The entity level defines the impacted level (e.g. a structure).

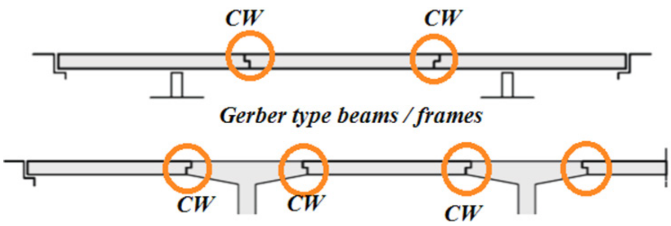
The damage process is derived based on observations and on original design and construction data. It governs the development of the observed damages in the future and allows the forecast of performance indicators.

The methodology how to account for all impacts of various observations in evaluation of one value of a KPI is further discussed in *Section 7.5*.

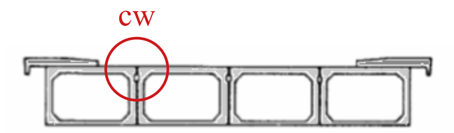
7.2 Vulnerable zones

7.2.1 Conceptual weaknesses

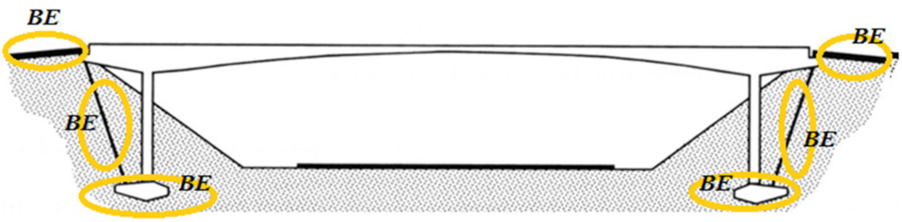
History has shown that some design concepts do not perform well, i.e. there are critical elements related to a bridge type or a bridge cross-section type. Those elements should be carefully evaluated before intervention strategies are formulated. A typical example is a Gerber hinge in a girder or frame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



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



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



Further categorization of the girder and frame bridge elements can be made in relation to their exposure to damage processes and sudden events. For example, middle piers may be exposed to impact from vehicles & vessels depending on the location and type of underpass traffic, or local scour if the substructure foundations are not protected from soil erosion due to flooding waters.

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

7.2.2 Vulnerable zones related to the superstructure

Nearly all damage processes related to concrete structures might affect any part of a concrete bridge. It should however be emphasized that not all parts of the bridge are equally important with respect to consequences. Considering for instance load bearing elements of a bridge, there are some regions/zones that are highly vulnerable, and should be treated with special care, which is further discussed. It is useful to relate failure modes to structural subgroups (see *Table 8.1*). In general, superstructure might fail in a bending or a shear failure mode.

The proposed segmentation of the superstructure in the longitudinal direction (partitioning of an element into regions with different vulnerability) is based on the (*NYSDOT, 1997*) and (*LTBP, 2016*):

High moment regions

- Sagging (label **HMS** region)
- Hogging (label **HMH** region)

High shear regions (label **HS region**)
Construction joint (rigid type) (label **CJ region**)
Shear key (label **SK region**)
Hinges (label **HG region**)
Anchorage zones (label **AN region**)

Example of such segmentation is presented in *Figure 7.2* for a Gerber type bridge girder.

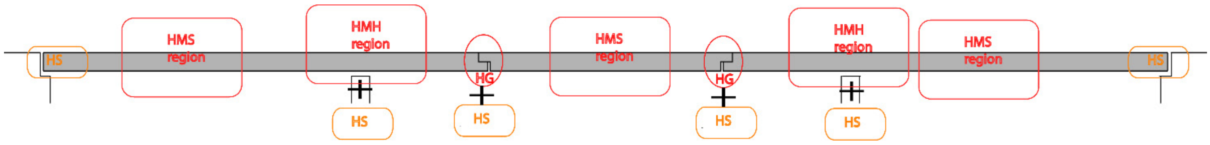


Figure 7.2 Example of segmentation for a Gerber type girder bridge

Typical locations of vulnerable zones in girder and frame bridges are presented in *Figure 7.3* along with the relationship to a failure mode (bending, shear).

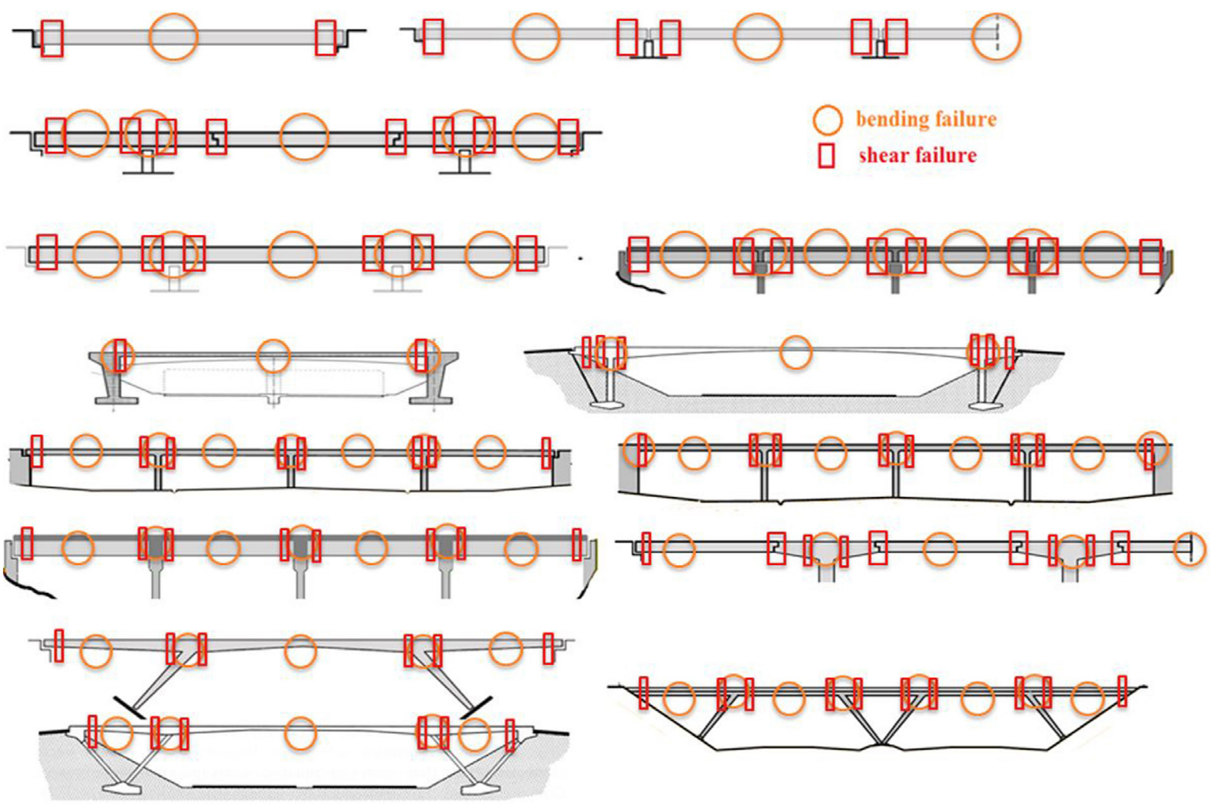


Figure 7.3 Vulnerable zones for different types of girder & frame bridges

When a damage is observed in those regions, it should be evaluated. This might require a structural assessment. For structural safety, a failure mode is a collapse and it may be associated with one or several vulnerable zones (i.e. for hyperstatic systems) which should all be inspected. For serviceability, the failure modes are related only to one vulnerable zone (e.g. specific crack width, deflection). For the latter, the checks that need to be performed are similar as in design, i.e. at a specific section.

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

In *Figure 7.4*, a few examples of observations that affect vulnerable zones are provided. Further examples may be found in various literature incl. inspection manuals, e.g. (*DRD, 2014*), (*Correia, et al., 2012*), (*Breysse, et al., 2012*) and (*Ghosn & Yang, 2014*).

a)  b) 

Figure 7.4 Examples of observations that affect vulnerable zones
a) Chloride induced corrosion in a half-joint, (CIRIA, 2017); b) Delamination due to ASR in a deck slab (affecting the shear resistance), (Larsen, 2009)

7.2.3 Vulnerable zones related to substructure

It should be noted that elements of substructure support the superstructure, implying that their failure might lead to a total collapse. In general, a substructure might fail in crushing or buckling failure mode. In addition, bearing areas are exposed to splitting forces. Pier cups (if they exist) are generally exposed to high shear stresses making them particularly vulnerable. Elements of substructures are mainly exposed to sudden events e.g. impact, scour and earthquake. Typical locations of vulnerable zones related to piers are presented in *Figure 7.5*along with their relationship to an anticipated failure mode.

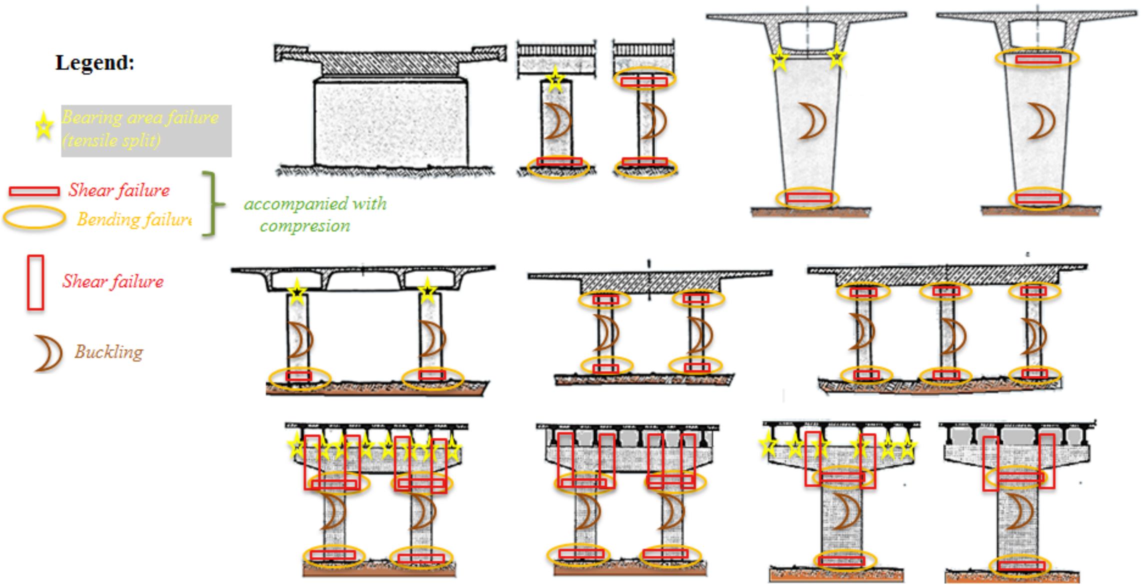


Figure 7.5 Vulnerable zones related to substructures

7.2.4 Damages related to equipment

Elements related to equipment are related to nearly all bridge types. It should be noted that malfunction of these components might jeopardize load bearing elements of a bridge (serviceability issues) and/or impose severe consequences themselves. Level of service generally depends on adequate function of these elements (traffic safety issues). In the following, checklists that may be used during inspections are provided.

Bearings, (*Ramberger, 2002*):

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

- Condition of sliding or rolling surfaces
- Condition of the anticorrosive protection, against dust, and of the sealings

Please refer to specific literature e.g. (*TRB2, 2014*), (*TRB, 2014*) and (*Austroads, 2012*) for more information on bearings.

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

Expansion joints, (*Ramberger, 2002*):
- Damage of the anticorrosive protection
- Cracks due to fatigue in steel members
- Damage to seals
- Workability of the linkage (proper function)
- Obstruction or damage of the drainage system

Please refer to specific literature such as ref. (*TRB2, 2014*), (*TRB, 2014*) and (*Austroads, 2012*) for more information on expansion joints.

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

Waterproofing is usually not visible, i.e. the condition has to be assessed from possible consequential damage on neighbour components such as:
- Leaking decks / wet spots beneath superstructure
- Finding of protective concrete wash out
- Swelling of pavement
- Cracking of pavement

Pavement/Overlay:
- Cracks, unevenness, holes and swelling
- Rutting
- Lack of friction
- Joint failure
- Improper drainage

Barriers, windscreens and signs:
- Damages from impact
- Condition of the anticorrosive protection
- Missing or loose bolts
- Condition of the anchorage
- Condition of concrete

in addition, signs should be checked for:
- Visual appearance (readability, reflection, lighting etc.)

Installations typically comprise lighting (typically light poles), electro-mechanical dehumidification systems (primarily on landmark bridges), Structural Health Monitoring Systems (primarily on landmark bridges), hydraulic opening arrangement and possible utility lines fixed / fastened on the bridge. They shall be evaluated case by case.

7.2.5 Hidden defects/damages

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

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

- Superstructure
    - Concrete body of an element
    - Reinforcement
    - Prestressing wires/stands and anchorages
    - Voided and cellular structures
    - Half-joints
    - Obscure surfaces
    - Concrete hinges
    - Temporary works
- Bearings and expansion joints
    - Poor access
    - Inspection at the 'wrong time'
    - Uninspectable items
- Drainage
- Waterproofing
- Substructure

In the light of previous discussions (see *Section 6.4*), the effect of a possible hidden defect/damage on the bridge performance should be duly considered even if these are not located in vulnerable zones. These damages should be treated in a same way as a natural hazard.

7.3 Vulnerable zones specific for arch bridges

In following, the vulnerable zones for four types of arches (*Section 9.1*), are discussed. Independently from the used construction material i.e. masonry, concrete or steel, arches transmit the loads from the superstructure diagonally to the substructure – an exception must be made to tied-arches as explained in *Section 9.1.6*. True arches are in pure compression, although most arch bridges resist a load combination of axial compression, bending moment, and shear.

Despite this similarity, the load paths vary depending on whether it is an open or close deck arch, a through arch or a tied arch. Load path determines regions with different vulnerability, further analysed according to the arch typology and material, with special emphasis on the superstructure. Floor systems, understood as a conjugation of beams, stringers, deck and eventually bracing, as well as bridge equipment, shall be analysed according to *Sections 7.2.2 and 7.2.4* respectively.

7.3.1 Close Spandrel Deck Arch

7.3.1.1 Masonry close spandrel deck arch

Similarly, as for other bridge types, vulnerable zones in masonry arch bridges are those locations/areas of the structure where the existence of certain damages has the highest probability to cause an undesired behavior of the structure. The correlation of the damages and the areas where their occurrence should raise concerns about the bridge structural performance is briefly explained below. The main deficiencies in masonry arch bridges are broadly classified as damages to foundations and superstructure.

The most common observations related to foundations include local undermining, differential settlements and masonry dislocations due to loss of mortar joints. The main problem in identifying damage at foundations is the difficulty of inspecting these underground structures. The material discontinuities (joints between blocks, joints between infill material and masonry components, and in the infill granular materials) allow to absorb/accommodate foundations settlements with visible deformations (i.e. joint opening).

Superstructure damages are presented in *Figure 7.6* and they can be a result of:

- Processes causing loss of bricks, loss of mortar joints and salt efflorescence in bricks. These mostly occur due to inadequate rainwater drainage, freeze-thaw cycles and penetrating vegetation (*Figure 7.6a*)
- Arch barrel deformations with longitudinal or transverse cracking; opening of arch joints and separation between brick rings in multi-barrel vaults (*Figure 7.6b*)
- Spandrel wall movements: sliding and bulging or detachment from the barrel. These occur due to the low resistance of spandrel walls to out-of-plane displacement caused by orthogonal pressures due to the weight of infill, traffic loads and horizontal transverse seismic action
- Fractures/cracking in piers and wing walls (*Figure 7.6c*)

a)

b)

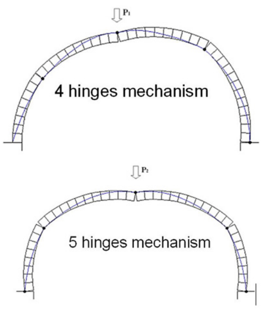
c)

Figure 7.6 Typical defects of masonry arch bridges, from (Zampieri, 2014)
a) Degradation and loss of bricks, loss of mortar joints and salt efflorescence; b) longitudinal cracking;
c) penetration by vegetation, fractures in piers and wing walls

Damage assessment, understood as the identification of causes related with the observations should consider the following:

- Cracks in the arch transverse joints can be associated with the development of hinge mechanisms in the arch (*Figure 7.7*). It can be a sign of low material strength, excessive loading or occurrence of foundation settlements/ rotations.

a)



b)

Figure 7.7 Arch hinge mechanisms
a) Four and five hinges mechanisms (Costa, 2009); b) Experimental testing (Page, 1987)

- Longitudinal cracks in the arch, along the spandrel-arch connection, are likely related with the structural response in the transverse direction, typically influenced by the interaction between arches, spandrels walls and infill material (*Figure 7.6b*)
- Leaning and bulging of the spandrel wall as well as detachment between spandrel-arch connection (opening and slipping), are also likely related with the structural response in the transverse direction (*Figure 7.6b*)
- Longitudinal cracks in the arch, distributed along the arch can be also associated with the structural response in the transverse direction and foundation settlements.
- Diagonal cracks in the arch intrados are frequently associated with settlements (*Figure 7.8a*).

- Cracks in the intrados of piers and abutments can be associated with foundation settlements and with its different configuration representing the different cracks' orientation (horizontal, vertical and diagonal cracks are likely related with larger vertical settlements in the central zone, in both external zones or in one external zone involving rotation, respectively) (*Figure 7.8b*).
- Detachment between cutwaters and piers, as well as loss of stones and total or partial collapse of cutwaters are normally caused by the mechanical action of vegetation growth and by the river flow (with eventual solid material transported). Shallower foundations of cutwaters are also common, causing differential settlements between these elements and the pier itself (*Figure 7.8c*).
- Localized cracking in masonry units (blocks) can be related with settlements and crushing due to (local) overstress.
- Material deterioration of blocks and loss of mortar in the joints are frequently related to aging and/or caused by water flowing through the structure (*Figure 7.6c*).

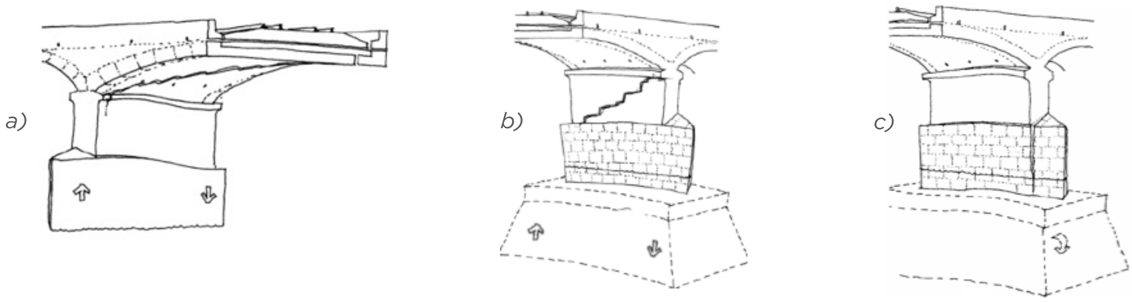


Figure 7.8 Typical cracks in masonry arch bridges
a) Diagonal cracks in the arch intrados; b) Cracks in the intrados of piers and abutments settlements; c) Detachment between cutwaters and piers, adapted from (García-Catalán & Álamo, 2006)

Concrete close spandrel arch bridges can be assessed in a similar way to masonry close spandrel arches. In fact, although the material used is different, structural elements are still mainly subject to compression. Even if present, steel reinforcement has a small contribution to the overall strength of these structures.

7.3.2 Open Spandrel Deck Arches

7.3.2.1 Concrete open spandrel deck arch

A segmentation regarding the definition of regions with different vulnerability include *Bearing Areas, Shear Zones, Tension Zones* and *Compression Zones, Equipment, Foundation Settlement*:

- Different *Bearing Areas* can be potential vulnerable zones according to their load magnitude, where the arch/skewback interface has the greatest value. Longitudinal cracks at the arch indicate an overstress condition and spalling of concrete would indicate loss of cross section of the reinforcement bars.
  The second greatest bearing load magnitude occurs at the arch/spandrel column interface. Here possible existence of horizontal cracks in the columns within several meters from the arch would be seen indicating excessive bending in the columns, caused by overloads and differential arch rib deflection.
  The third greatest bearing load magnitude belongs to the spandrel column/cap interface. Existence of diagonal cracks in columns, beginning at the inside corner and propagate upward, with possible cause in differential arch rib deflections (*Hartle, et al., 2002*).
  The superstructure supports with thin sheets of another material (e.g. rubber, cooper) between the concrete elements, where main longitudinal girders resting directly on the piers (*Figure 7.9*), can caused heavy cracking both in the anchor zones (i.e. hinge zones) of the girders and in the pier cross-beams supporting them (*Šavor, et al., 2009*).
- At the ends of the spandrel bent caps, as well as other high shear zones in the floor system shear cracks can be formed due to high shear forces in these regions.
  Struts connecting arch ribs are also subject to torsional shear stress. Cracks in this region indicate excessive differential deflection in the arch ribs (*Hartle, et al., 2002*).
- *Tension Zones* exists in spandrel bent caps, spandrel columns and in the floor system. At spandrel bent caps maximum tension generally occur in the midspan at the bottom and, if exist, at the cantilevered ends at the top.
  Transverse cracks in the arch (oriented perpendicular to the arch member) may indicate tension and therefore an overstress condition.
  Spandrel walls are also vulnerable due to tension therefore the *arch/spandrel wall interface* can experience cracks, movement, and general deterioration of the concrete (*Hartle, et al., 2002*).

- *Compression Zone* exists throughout the arches and spandrel columns where buckling forces and bending moments can cause excessive surface stresses resulting in cracks (*Hartle, et al., 2002*).
- *Position of all equipment* enables the proper functioning of the bridge and should be regularly inspected and replaced when it is necessary. Defects or damages of a drainage systems, pavement or railings reduce traffic safety on a bridge. Inadequate drainage of roadway and/or inside the arch box, but also damaged waterproofing or deteriorated expansion joints can cause deterioration of the structure due to combination of water retention and several processes e.g. carbonization, chloride induced corrosion, freeze/thaw actions, etc. Decayed bearings and expansion joints can cause unplanned structural movements and additional strain in concrete (*Šavor, et al., 2009*), (*Kušter Marić, et al., 2016*).
- Arch bridges are vulnerable to *foundation settlement* hence movements and rotation of the arch abutments should be regular monitored, especially if the foundation soil has low bearing capacity.
- Except conditions of an aggressive maritime environment or high seismic activity of the area, one of the most vulnerable zone of arch bridge as well as girder and frame bridges is bridge deck: due to direct exposure of the traffic and aggressive substances (de-icing salts, water, CO_2, etc.).

a)

b)

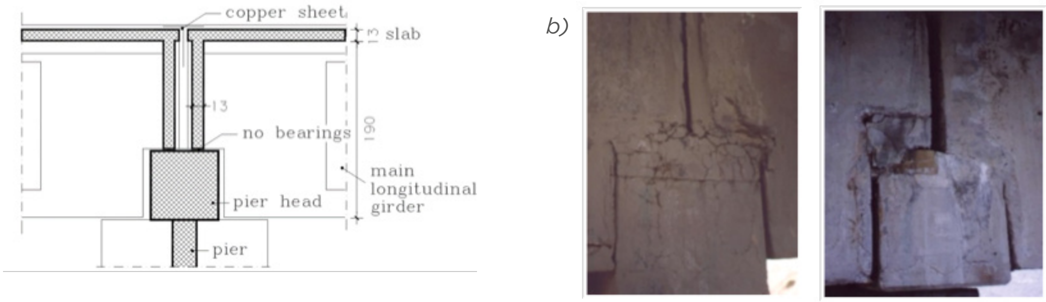


Figure 7.9 Details of superstructure resting on the pier
a) without bearings (Šavor, et al., 2009) and b) cracking of girder and pier head (HIMK-Croatian Institute for bridges and structures, 2002)

7.3.2.2 Steel open spandrel deck arch

As the arch is the superstructure main load-carrying part, all its members can be considered as vulnerable. In addition to the arch members, vulnerable zones can be considered as the spandrel members in the deck arch.

- Regarding *arch members* global alignment should always be checked for any movement, deflection or distortion. Alignment of bearings can denote global movements of the arch, as well as arch ribs buckling and crippling, and arch rib splice plates. Shear stress and overloading should be dismissed through the observation of defects/damages in bearings on each of the supports. On flexural zones, tension and compression flanges can denote signs of overstress, as well as the existence of cracks near connections in tension members. Web areas over the supports, including bearing stiffeners, jacking stiffeners and diaphragms, should be checked for buckling, crippling and loss of section.
- In deck arches the end connection of spandrel columns and spandrel girders should be examined for cracks and loose fasteners. Signs of section loss and buckling damage are also important if observed in spandrel girders, spandrel caps or spandrel columns. In these elements signs of flexure overstress should also be examined.
- Orthotropic steel plates, which are lighter than concrete decks, are often used for long-span bridges and viaducts in urban area which are subjected to topographical conditions or other factors. On bridges where an extremely large number of heavy vehicles travel, however, in recent years there have been several observed cases of deck damage due to fatigue. Fatigue of orthotropic steel decks may occur in locations that do not allow easy discovery of the precursors or progression thereof.

The *Figure 7.10* shows an example of fatigue crack in an orthotropic steel deck found by a check after a deformation was spotted in the pavement. This is something that has to be avoided (a crack being generated without being noticed) and this represents a vulnerable zone of a steel arch bridge.

a) b)

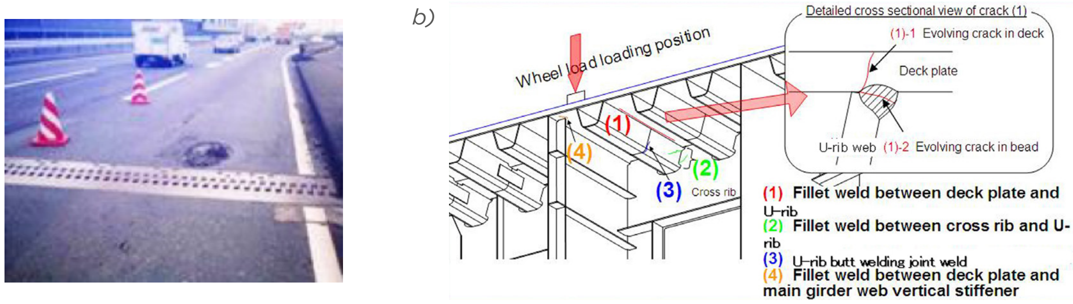


Figure 7.10 Fatigue of orthotropic steel deck
a) Example of damage to pavement caused by fatigue of orthotropic steel deck; b) Types of orthotropic steel deck fatigue crack (Public Works Research Institute, s.d.)

- Another vulnerable location is Stringer-to-Floor-Beam connections. In many existing bridges, mechanically fastened stringer-to-floor-beam connections are made using double angles riveted or bolted to the web plates of both members. The common engineering practice has been to design these connections to take account of shear forces alone (their load-carrying function being to transfer the stringer end reactions to the floor beam). While this assumption may be adequate for the ultimate limit state design, the behaviour of these connections under moderate loads might differ substantially. Connections of this type have displayed high vulnerability to fatigue cracking. A large number of damage cases in which fatigue cracks were detected in the connection angles have been reported (*Figure 7.11*). In old riveted connections, rivet failures are also very common. Fatigue cracking in stringer-to-floor-beam connections is generated by secondary effects which are deformation induced in nature. Like stringer-to-floor-beam connections, connections between floor beams and main load-carrying elements or system (main girder, main trusses, arch ties, etc.) have also displayed numerous fatigue problems.

a) b)

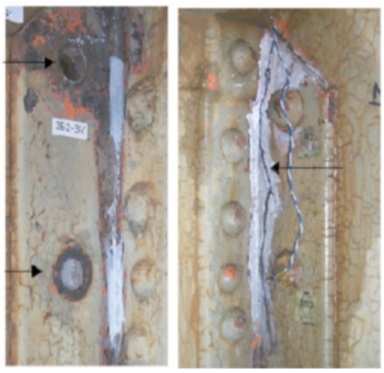


Figure 7.11 Example of fatigue damage in stringer-to-floor-beam connections (Al-Emrani, 2002)
a) at the junction between the rivet head and shank; b) at the external leg of the connection angle

7.3.3 Through Arch and Tied Arches

In addition to the arch members, vulnerable zones can be the *bracing* (through and tied arches), and hangers (through and tied arches).

7.3.3.1 Steel Through Arch and Tied Arches

- In through and tied arches *bracing* should be inspected for buckling and crippling, or loss of section, as they might be subject to tension or compression. The web members should be inspected as any other truss. The top rib chord should be inspected as a compression member.
- *Tied Girders* are critical due to the large tensions they are subjected to.
- *Hangers*, present in through and tied arches, are also subject to large tensions therefore may be critical. Alignment of the hangers, collision damage due traffic, corrosion and cracks near both ends are signs of distress. Any welded connection between hangers and attachments should be carefully observed for cracks. Vertical hangers in steel arch bridges are usually designed to take care of the axial forces. Details of hanger connections to arch and bridge deck are generally designed to ensure a moment-free connection. For this reason, the hangers are often assumed to be pin-connected at both ends. Several cases have been reported

in which fatigue cracking at the connections of bridge hangers were observed. In most cases, a combination of two different mechanisms has contributed to fatigue cracking in these details:

- *Vibration*: The slender hangers usually have very low bending stiffness, which makes them very sensitive to resonance. The cables can be excited by traffic loads on the bridge and/or wind loads.
- *Secondary stresses due to connection stiffness*: Ideal moment-free pin connections do not exist in reality. Even when designed as such, a connection will always acquire some rotational stiffness inherent to detailing or gradually during the service life of the bridge, due to corrosion (so-called freezing).

Figure 7.12 shows an example of an arch bridge in which the hangers have developed fatigue cracks at their connections to the steel arch.

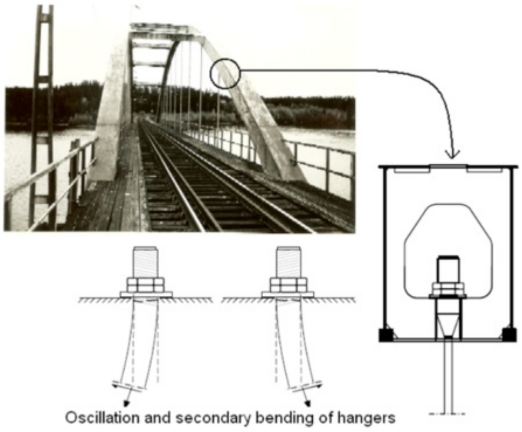


Figure 7.12 Fatigue cracking of hangers, from (Kesson, 1991).

7.4 Quantitative modelling of bridge damages

Modelling of bridge structures in computer-based systems in supporting bridge evaluation is of great importance for the efficiency of the performance assessment process. Precision of the numerical representation of the structure geometry influences accuracy of the description of technical parameters incl. damage location on the bridge.

For each damage, the following measures should be described (non-dimensional → three-dimensional):

- Damage intensity, $I \rightarrow (x, y, z)$
- Damage extent, $R \rightarrow R(x, y, z)$
- Damage location, $L \rightarrow L(x, y, z)$

If damage intensity and extent are homogeneous, the following relations applies (three-dimensional shown for illustration):

$$I \rightarrow I(x,y,z) = \begin{cases} I \; for \; L(x,y,z) \neq 0 \\ 0 \; for \; L(x,y,z) = 0 \end{cases} \tag{7}$$

$$R \rightarrow I(x,y,z) = \begin{cases} 1 \; for \; L(x,y,z) \neq 0 \\ 0 \; for \; L(x,y,z) = 0 \end{cases} \tag{8}$$

Non-homogenous models can also be applied.

In most Bridge Management Systems, non-dimensional models of bridge structures are applied. This means that the structure geometry is represented by a set of non-dimensional points modelling each bridge component, e.g. support no. 1, span no. 1, etc. Characteristics of each component (dimension, material data, inspection data etc.) are not oriented in the space but only assigned to the "name tag" of the component. Such modelling of a bridge geometry does not enable very precise spatial orientation of the collected information.

As an example of the abovementioned concept, the approach in (*SustainableBridges, 2007*) is presented in the following. The concept uses the term segment as a part of a construction component artificially separated and individually described. In the case of a combined slab and beam bridge, each T-beam will be a segment. In the case of a pure slab bridge, a division into minimum 10 segments is proposed.

Damage extent R, is described in percent i.e. 0% (no damage) and 100% (a damage covers the entire component). For deformations, destruction, discontinuity and displacement, the damage extent can be evaluated as:

$$R = \frac{m}{n} \cdot 100\% \tag{9}$$

where, m is the number of segments where damage can be noticed and n is the total number of segments of the evaluated component. For protection damages and contamination, the damage extent can be evaluated as a quotient of the damaged area divided by the visible area:

$$R = \frac{1}{n} \cdot \sum_i \frac{\Delta A_i}{A_i} 100\% \tag{10}$$

where:ΔA_i is the area of segment "i" covered by the damage type and A_i is the visible area evaluated component *Figure 7.13* shows an example of an abutment divided into segments with indication of contaminated areas. The damage extent can be evaluated as ($\Delta A_4 = 1.0m^2$, $\Delta A_5 = 2.5m^2$, $\Delta A_6 = 0.8m^2$, $\Delta A_{10} = 3.0m^2$, edge segments $A_1 = A_{10} = 8.0m^2$ and intermediate segments $A_i = 4.0m^2$):

$$R = \frac{1}{n} \cdot \sum_i \frac{\Delta A_i}{A_i} 100\% = \frac{1}{10}\left(\frac{1.0}{4.0} + \frac{2.5}{4.0} + \frac{0.8}{4.0} + \frac{3.0}{8.0}\right) 100\% = 14.5\% \tag{11}$$

Damage intensity I, can be based on the following principles:

- Damage intensity I_i of a segment "i", is determined for the most damaged cross-section
- If damage intensity varies between segments, separate values of damage intensity I_i and damage extent R_i shall be evaluated for each segment
- The damage intensity measure can be different for each damage type and it can be described with numerical values (destruction, discontinuity, losses and protection damages) or linguistic values (deformations, displacements and contamination).

As an example, an intensity of concrete losses can be determined as:

$$I_{b,i}^u = \frac{\Delta F_{b,i}^u}{F_{b,i}} \tag{12}$$

Where $\Delta F_{b,i}^u$ is the loss of concrete cross-section area in segment "i" and $F_{b,i}$ is the designed concrete cross-section area in segment "i" For other damage intensities, refer to (*SustainableBridges, 2007*)

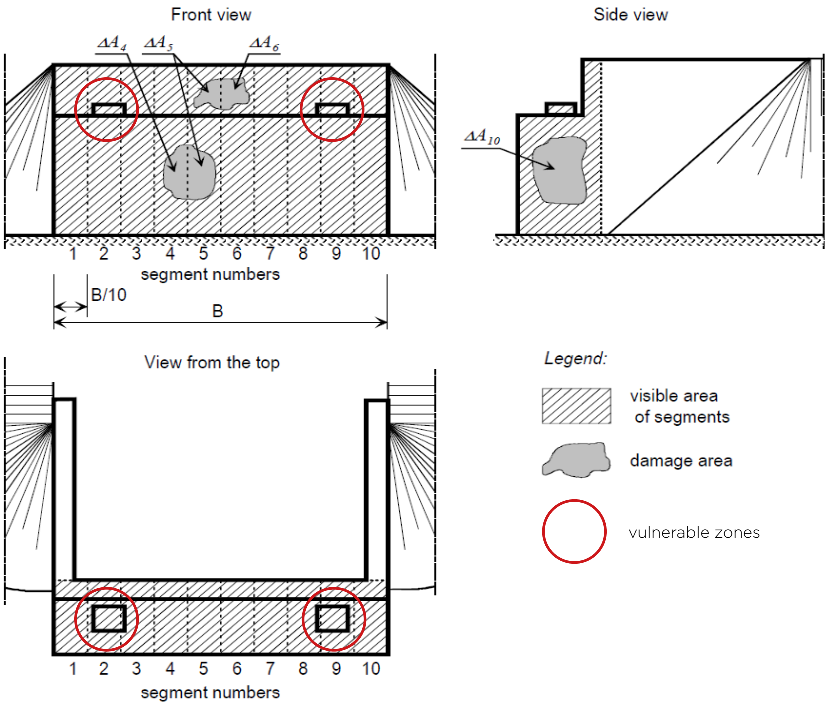


Figure 7.13 Example of the division into segments and registration of contamination damage (vulnerable zones shown by red circles), adapted from (SustainableBridges, 2007).

Damage location for non-dimensional geometry can only be described with accuracy to the specific component or element. This may be acceptable for estimation of agency costs, given that all damages will be repaired. However, the spatial location is important as the damages in vulnerable zones affect structural safety more severely than similar damages outside the vulnerable zones. Naturally, this is important for prioritization of maintenance efforts due to budget constraints of the owners. Reporting of location in one, two or three dimensions should be acknowledged and recommended by the inspector, based on his observation and knowledge of the underlying damage process.

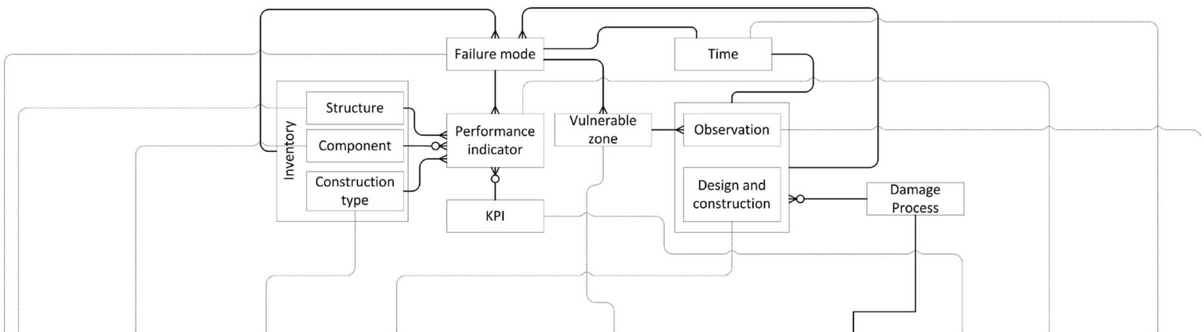
Detailed description of damage location will become more straightforward in the coming years, as Building Information Modelling (BIM) using 3D models has become more widespread as part of Bridge Management Systems (not only for landmark bridges). Damage location in vulnerable zones should be treated by a structural reassessment in a phase-wise process with increasing complexity, assisted by inspections, testing and monitoring also with increasing complexity.

7.5 Derivation of KPIs from PI

7.5.1 Evaluation of KPIs based on "engineering judgement"

To show the variety of QCPs for concrete girder and frame bridges, the methodology of the project ContectVet, (*Contectvet, 2001*), should be highlighted. The ContectVet provides a framework for the three dominant damage processes for reinforced concrete bridges: corrosion, frost and alkali silica reaction. In comparison to COST1406 which uses KPIs, this framework only addresses a bridge performance related to structural safety via Simplified Index of Structural Damage (SISD). Also, the ContectVet performs verification according to available codes, while the WG3 approach relates the current (i.e. observed) resistance reduction to the virgin (damage free) state of a bridge. There are three QCP steps in ContectVet: inspection, assessment, and prognosis, which account for an urgency of intervention, i.e. a necessity for more detailed investigations. The QCP in WG3 includes both static and dynamic quality control where it accounts for diverse maintenance scenarios and KPI development over time (see *Section 7.10*).

Based on the WG3 framework ontology (*Section 7.1*), an example of a performance evaluation has been given in *Figure 7.14*. The necessary information and related connections have been structured in a table, where the key point is the relationship between failure modes/vulnerable zones. So far, within BMSs this information has been considered as "engineering judgement" and is not related to various bridge types and crucial observations. In the example, the observations related to the main girder and the deck, which are in vulnerable zones (*Section 7.2.2*), have a certain impact on PIs. It is on a judgement of an Owner/Operator to assess this impact (e.g. maximum value, weighted sum, etc.) in terms of urgency of intervention, as well to predict a time interval in which the related KPI value will reach a predefined threshold for an intervention (e.g. *Section 8.5*).



Girder bridge	Main girder	Reinforced concrete	1970	Bending failure mode	HMS	Corroded reinforcement	Corrosion	Reliability (Structure safety)	3	4	10 years
	Main girder	Reinforced concrete	1970			Crack					
	Main girder	Reinforced concrete	1970			Spalling					
	Deck	Reinforced concrete	1970		HMH	Corroded reinforcement	Corrosion				
	Deck	Reinforced concrete	1970			Corroded reinforcement					
	Deck	Reinforced concrete	1970			Spalling					
	Deck	Reinforced concrete	1970			Efflorecences					
	Deck	Reinforced concrete	1970	Shear failure mode	HSS	Crack	Overloading		4		
	Deck	Reinforced concrete	1970	Falling chunks		Spalling	Corrosion	Safety (Life and limb)	2	4	15 years
	Curb	Granite	1970	Traffic & pedestrian accident		Damaged	None		4		

Figure 7.14 An example of a protocol for performance evaluation - derivation of the KPIs from PIs

7.5.2 Evaluation of KPIs using Bayesian Nets

Bayesian nets may be applied to evaluate the value of Reliability KPI. An example of the simplified Bayesian network for à priori reliability assessment is presented in *Figure 7.15*. As risk-based assessments are considered as an advanced method, some of the input parameters may also be more advanced, e.g. load effects based on actual traffic data. More complex Bayesian Nets may be found in e.g. (*Schubert & Faber, 2008*).

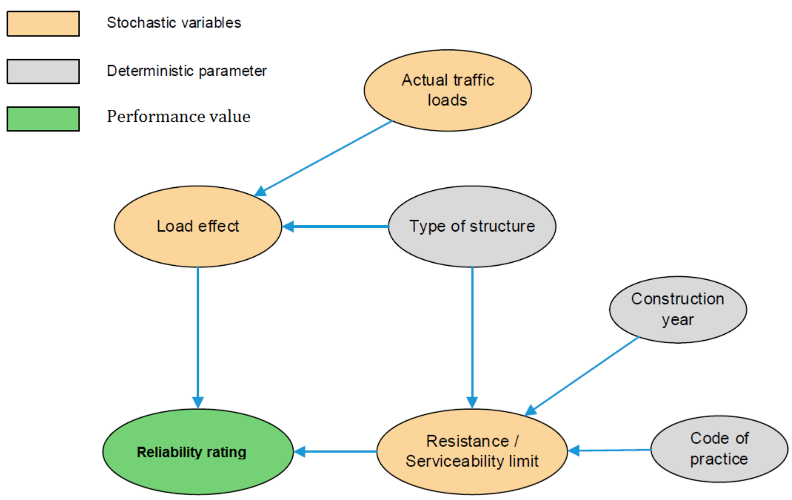


Figure 7.15 Simplified Bayesian network for á priori reliability assessment

The à posteriori assessment of reliability is performed after an inspection or detailed investigations. The qualitative à priori values are updated based on the observations. Please note that the node, Actual traffic loads, have been excluded in *Figure 7.16* and *Figure 7.17* for the sake of simplicity.

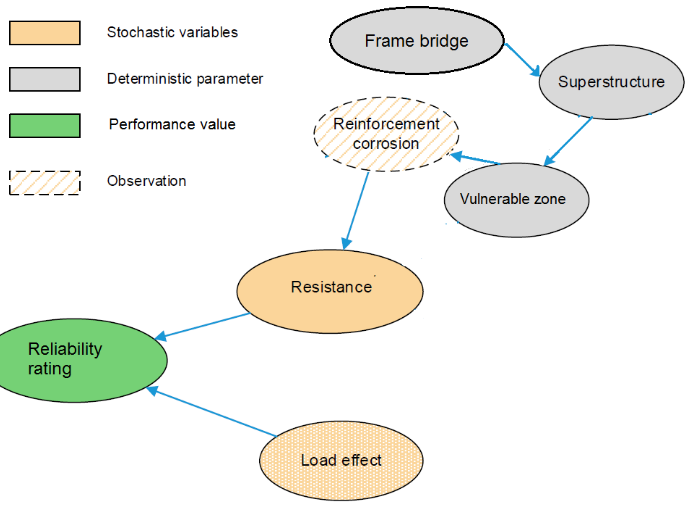


Figure 7.16 Simplified Bayesian network for qualitative à posteriori assessment of reliability

To evaluate resistance in course of time, in order to forecast reliability, available deterioration models related to the observed damage process might be applied (*Figure 7.17*). The deterioration models can be derived from physico-chemical deterioration processes (*Roelfstra, et al., 2004*) and (*Alnaggar, et al., 2017*). The most deterioration models in current BMS are based on statistical analysis of past condition data, e.g. (*Lethanh, et al., 2017*) and (*Hajdin & Peeters, 2008*).

Many BMS use Markov chains to model deterioration as it supports the discrete scale for condition states and transition probabilities can be easily derived from condition data. However, Markov chains are mostly employed for elements or components, whereas their applicability for individual damages waits to be proven. Alternatively, Dynamic Bayesian Net (DBN) might be used, e.g. (*Chatzi, et al., 2017*)

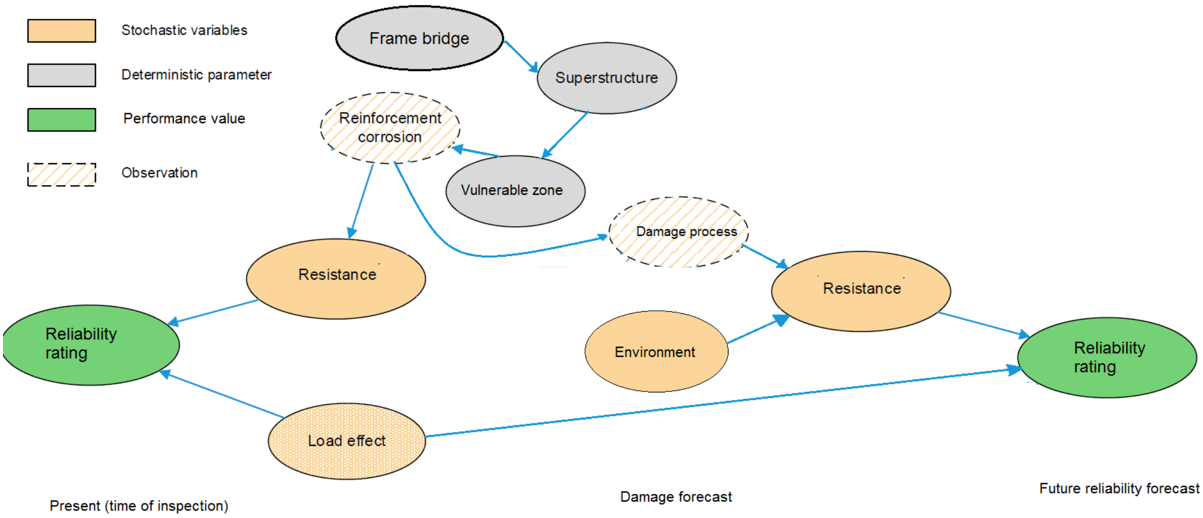


Figure 7.17 Simplified influence diagram with a damage forecasting

7.6 Normalization of KPIs

Following the decision made at the Belgrade Workshop in March 2016, all KPI are scaled from one to five, one being the best value and five being the worst value. For Reliability and Safety this has been illustrated in *Section 7.5*. The other KPIs such as Availability, Environment and Economy need to be expressed in native units and then scaled from one to five. For instance, the maximum yearly costs that are expected can be regarded as five and the costs in other years can be scaled appropriately.

To avoid the bias between the bridges, the maximum yearly costs for all analyzed bridges need to be set to five. Similar reasoning applies for Environment, which can be also expressed in native units (e.g. kg CO2) or in monetized terms.

As defined in *Section 3.4*, the Availability is the proportion of time a system is in a functioning condition. In this case the value can be only zero or one for each time instance. In the practice the bridge may be partially serviceable. It can be posted to a weight or clearance limit and accordingly certain number of vehicles will deviated with some economic impact.

However, a lane on the bridge can be closed resulting in congestions and rerouting of vehicles. The Availability can be therefore measured by additional travel time for each vehicle category. This additional travel time can be monetized as user costs. In this case the same reasoning applies as for Economy.

The evaluation of additional travel time is not trivial and required a valid traffic model. If such a model is not available one can establish a qualitative Availability value, based on the importance of the road and possible alternative routes.

The normalization procedure of all KPIs (except the Environment) is illustrated by examples in *Sections 8 and 9*.

7.7 Development of KPIs over time

The KPIs can be conveniently visualized using a „spider net diagram" (e.g. *Figure 3.3*). Here, each of the KPIs are given on a separate axis, and when their development over time is of interest, the time axis can be appended orthogonally on the plane of the diagram. In this manner, the "performance tube" can be generated.

It is presented in *Figure 7.18*, where there are five axes corresponding to the adopted KPIs within WG3.

As an example, the linear change of the KPIs` values in time is adopted here. In general, the necks in the diagram represent the time intervals of low performance, whereas the areas with "full" pentagon cross-section are the time periods of high performance. Alternatively, volume between the "full" pentagon and the "performance tube" can be regarded as performance deficit that is to be minimized.

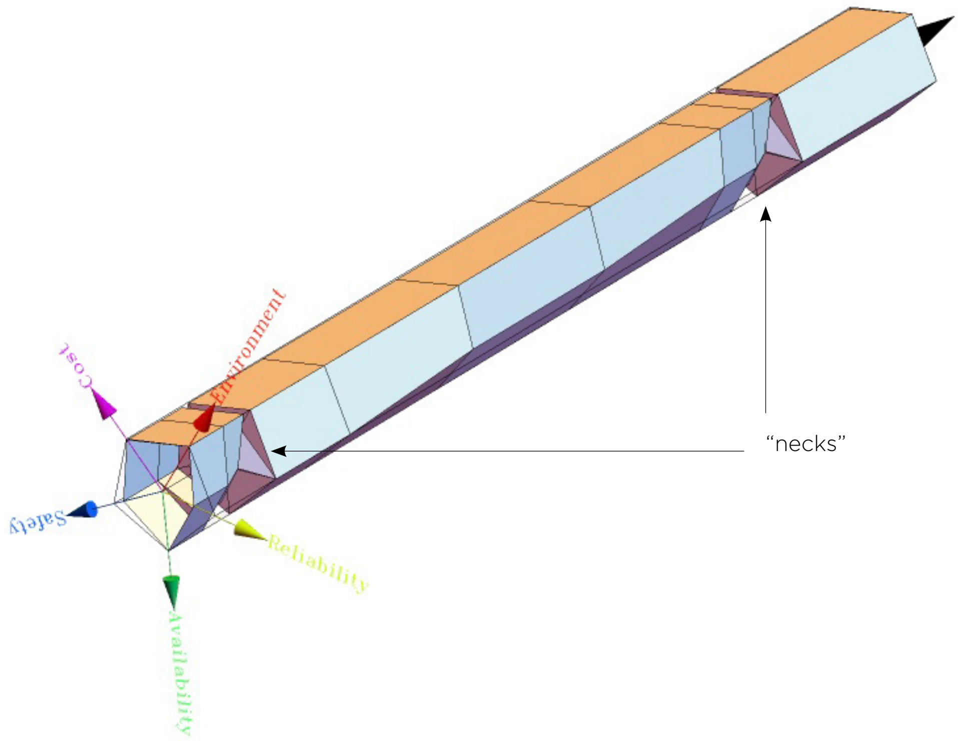


Figure 7.18 3D Spider net - Performance Diagram

The Availability, Economy and Environment axes are deterministic and reflect a chosen maintenance scenario over course of time. The Reliability and Safety axes represent the occurrence rate of the chosen failure mode. It must be noted that there can be more than one Reliability axis i.e. that failure modes related to either serviceability or safety can be separately assessed. This is convenient for the case when analyzing decisions on maintenance and would like to account both for the failures due to severe deterioration and the failures due to a hazard.

7.8 Time preference of KPIs

The question arises how to evaluate impacts of future events and compare them with present events i.e. what is more important - a reliable bridge today or at some point in the future? For costs or cash flows there is an established procedure – discounting. The future expenditures are discounted to present - Net Present Value (NPV), e.g. with the discount rate of 2% the expenditure of €102 in a year is equal to €100 today. For a time-dependent cash flow, NPV can be calculated as in *Figure 7.19*. Here, the values a and b are expenditures related to the start and the end of a time interval i.e. t_s and t_e, respectively. It is however not clear how should be dealt with non-monetized KPIs. There are quite a few studies on social preference of non-monetized properties such as individual emotions and values. This can hardly apply here as the KPIs for bridges have some economic impact. It is therefore decided to deal with the KPIs Reliability, Availability and Safety in the same manner as with the cash flow i.e. to discount them in the same manner as the expenditures for maintenance interventions. The value of these KPIs is more important today then e.g. in one, two or 10 years. Thus, it is regarded that the interventions on the short term are more expensive but the benefits are also more valuable. If a scale, e.g. one to five is adopted for all KPIs, the NPV is already directly comparable.

$$NPV = \frac{\{[r\cdot(t_e-t_s)-1]\cdot b+a\}\cdot e^{-rt_e}+\{[r\cdot(t_s-t_e)-1]\cdot a+b\}\cdot e^{-rt_s}}{r^2\cdot(t_e-t_s)}$$

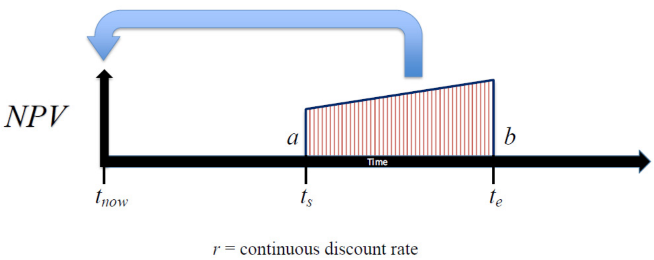


Figure 7.19 Evaluation of the Net Present Value for time dependent KPIs

In order to reduce the KPIs to the same scale as for any time instance, the normalization is performed i.e. the NPV is divided with the NPV calculated if all KPIs were equal to one over the whole investigation period. These values can be regarded as "average" long term KPIs. The decision, which maintenance strategy is to be chosen based on the "average" long term KPIs can be made using methods developed in (*Stipanovic, et al., 2017*).

7.9 Outlook – Risk as performance indicator

The evaluation of the native values of KPIs (e.g. additional travel time, tonnes of CO2) for different scenarios would allow a different approach to performance assessment and decision making. To this end the consequences of various failure modes need to be estimated, so that the axes of Reliability (potentially several axes) and Safety (potentially several axes) can be aggregated into a single monetized Risk axis. Clearly, the monetized KPIs of Availability and Environment as well as Economy can be added to each time instance. The results would be long-term costs for different maintenance scenarios. The one with the lowest long-term costs is the optimum maintenance scenario. The problem of time preference would be automatically solved as there would be only one cash-flow.

7.10 The steps in the QC Framework

The quality control framework is envisioned to have two stages - static and dynamic. In general, the first one comprises the preparatory work, inspection tasks and snapshot assessment of the KPIs. The second mode implies assessment of remaining service life, KPI development over time and finding an optimal maintenance scenario i.e. decision making.

The steps for a static (snapshot) quality control comprise:

1. Preparatory work
   - Study an inventory information
   - Identify conceptual weaknesses of the original design
   - Identify the material weaknesses
   - Compare the current traffic loads to traffic load model used in the original design
   - Define the vulnerable zones
   - Evaluate à priori reliability
2. Inspection on site
   - Identify damages (e.g. cracks, spalling, deformations, etc.)
   - Measure on site material properties
   - Collect samples
3. Lab test (e.g. carbonatization depth, chloride ingress, etc.)
4. Assessment of the Reliability KPI
   - Qualitative assessment of resistance reduction based on observed damages
   - Qualitative assessment of reliability (structural safety and serviceability)
5. Assessment of the Safety KPI

The steps for a dynamic quality control comprise:

1. Assessment of a remaining service life
   - Assessment of the speed of active damage processes
   - Damage forecast
   - Reliability and Safety development over time
2. Maintenance scenario
   - Reference scenario –intervention at the end of service life
   - Preventative scenario
   - Estimate long term costs for all scenarios
   - Estimate Availability for all scenarios
   - Estimate an effect of maintenance on Reliability and Safety
3. Decision making
   - Preform multi-attributive or multi-objective optimization
   - Monetize non-monetary KPIs
   - Determine the optimum scenario

The two stages of a QC plan and related steps are further described in the illustrative example given in *Section 8.5*

8. Application of the Quality Control framework to girder and frame bridges

8.1 Ontology

As detailed in the previous sections, a vast amount of knowledge about bridges (materials, deterioration processes, damages, costs, etc.) enters into a QC plan. In many countries, this knowledge comes primarily from engineering judgement by experienced specialists. Recently, bridge knowledge management using ontologies has been studied in (*Helmerich, 2016*), where it has been used for modelling and organizing bridge engineering knowledge from a domain expert's view. The QC plan ontology has been proposed in *Section 7*. A bridge inventory, which comprise the data on: bridge structure type, type of construction and a decomposition into a hierarchy of elements, has all necessary information. Performance indicators shall be evaluated from this data together with on-site observations and design & construction data, where the latter is sometimes referred to as a 'birth certificate'. Guidance on collecting such data is thoroughly elaborated in e.g. (*FWHA, 2016*). In order for the inspector to estimate a performance indicator, profound knowledge on vulnerable zones associated with the type of bridge and critical failure modes is essential.

Furthermore, to understand the impact and development of the observation made on-site (i.e. a performance prediction), knowledge about the underlying damage processes as well as demands (traffic volume etc.), is required. Based on the described input, a performance indicator (PI) can be evaluated relating to one or a several Key Performance Indicators (KPI), i.e. Reliability and Safety during inspections (as a snapshot in time) paired with Availability and Economy, when planning future inspections and maintenance interventions.

8.2 Taxonomy

8.2.1 Bridge elements and their condition rating

Within WG3, a taxonomy for different bridge types, such as girder and frame bridges (named by their structural system), has been developed. A survey performed within the FP7 project SeRoN (*SeRon, 2011*), involving 14 European countries and overall 45.896 bridges, reveals that approximately 88% of bridges throughout Europe are girder (64%) or a frame (24%) structures. Truss bridges are much less represented (approx. 4%). Most of the bridges with short spans (< 30 m) are built using concrete as the main construction material. Medium span bridges (30 – 100 m) are mostly built as composite structures (steel superstructure with a concrete deck) or pure steel structures. Bridges with long spans (over 100 m) are predominately made of steel. According to the survey, there is a substantial number of small bridges < 30 m (86%). The most common material used for bridges in Europe is reinforced and pre-stressed concrete (86%). Therefore, the focus in this section is set on the concrete bridges as they represent the vast majority of girder and frame roadway bridges in Europe.

The current practice for bridge evaluation divides the bridge structure into constitutive elements/components that can be commonly clustered in groups: superstructure, substructure and equipment. Here, the elements which are subjected to bending due to traffic load are in the superstructure group, while in the substructure group are elements mainly subjected to compression. The additional elements, in the group 'equipment', provide protection either to the structure or the users. Also, those elements may provide comfort to the users. The list of bridge elements mainly depends on a bridge structure type and should be defined in the bridge inventory list. It should be noted that some elements/components do not exist for all bridge types, e.g. integral frame bridges do not have bearings and expansion joints.

Girder bridge – the main groups of elements: The main groups of elements for girder bridges are shown in *Figure 8.1*. The connection between the superstructure and substructure is provided by bearings.

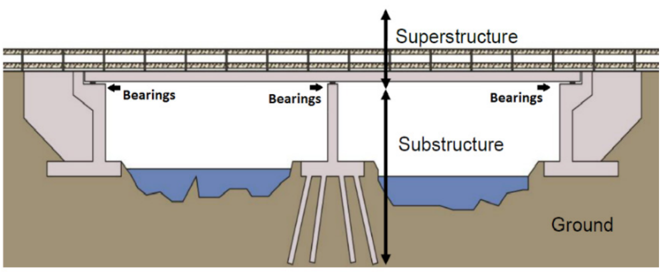
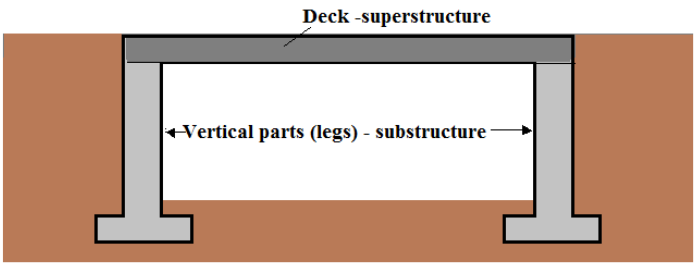


Figure 8.1 The main groups of elements for girder bridges

Frame bridge – main groups of elements: Different to girder bridges, a monolithic connection between superstructure and the substructure exist in frame bridges. There is some ambiguity in defining superstructure and substructure for frame bridges because of the monolithic connection, and here adopted is the definition in *Figure 8.2*.

a)



b)

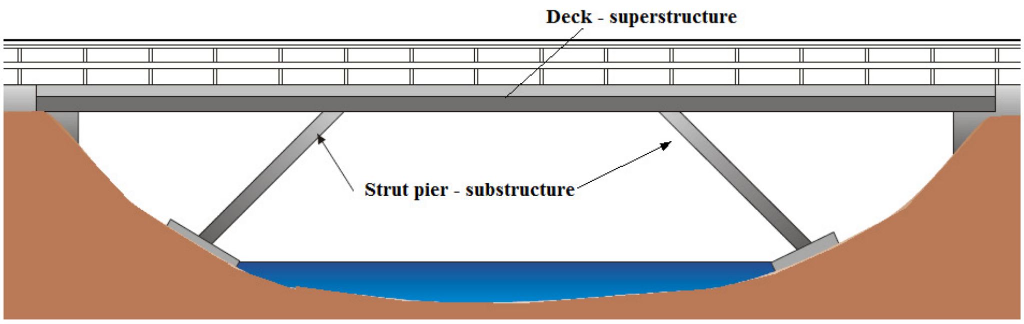


Figure 8.2 Definition of super- and substructure for frame bridges
a) Portal frame; b) Strut frame

According to the Bridge Inspector's Reference Manual (*FHWA, 2012*), the frame beams or slabs and frame legs are considered as the superstructure, while the abutments are considered as the substructure. For consistency, that approach is not adopted here and the distinguishing between superstructure and substructure is presented in *Figure 8.2*. A segmentation might be useful in creation of event trees and fault trees when analysing bridge performance. In *Table 8.1*, the main components of each group are listed. Some components can be further subdivided, e.g. foundations.

Table 8.1 Decomposition of girder and frame bridges into main components (#-presumptive number).

Group	Components (#-presumptive number)	Primary function
Superstructure	Deck slab (1)	Load bearing
	Main girder (0 to n)	Load bearing
	Cross beam/diaphragm (0 to n_1)	Load bearing
	Construction joints/Hinges (0 to n_2)	Load bearing
Substructure	Abutments incl. wing walls (2)	Load bearing
	Piers (0 to k)	Load bearing
	Foundations (2 to 2+k)	Load bearing
Equipment	Bearings*(0 to n·(k+2))	Articulation/load bearing
	Expansion joints (0 to j)	Articulation
	Drainage (0-1)	Protection
	Run-on slab (0 to 2)	Comfort
	Waterproofing (1)	Protection
	Pavement/Overlay (1)	Protection and comfort
	Barriers and wind screens (2 to 5)	Protection and comfort
	Signs (0 to i)	Protection and comfort
	Installations (0 to m)	Comfort

*In some countries, the bearings are considered as load bearing elements due to their function. That approach is not adopted here considering that bearings are mainly mechanical replaceable parts of the structure. Prestressing cables and ducts are considered as a subcomponent of the component where they are located.

Most countries use so called ‘condition rating’ mainly on an element level, as a measure of deviation from the ‘as new’ condition. Commonly, this condition rating is given without a more detailed explanation, and thus the information on specific damages are lost. The global condition rating of a whole bridge structure is commonly calculated by summing up the weighted contributions of the element condition ratings. The adopted weighting related to elements may lead to inconsistent results between two bridges which are broadly in a similar condition but have a very different number of elements. The “weight” of each element might be calculated by dividing the element cost by the total cost of the structure.

This weighting is appropriate for a cost-effective maintenance planning where the aim is to avoid that costly elements become too much deteriorated, but regarding the structural failure, the importance of an element would be better reflected by its contribution to the bridge safety. To the elements that have the greatest influence on the structural safety, the highest weights can be assigned. This approach is more appropriate than weighting the inspected elements in terms of costs. It should be noted that the worst condition rating is usually a proxy for an unacceptable probability of a failure of the assessment unit (usually a component).

There was a considerable effort to improve bridge assessment by introducing so-called system factors to account for load path redundancy. Those factors were obtained for typical bridge types and configurations of superstructure cross-sections (*Ghosn & Moses, 1998*), (*Liu, et al., 2001*), (*Ghosn & Yang, 2014*). In addition, a method is proposed to consider structural robustness in condition rating based on visual inspections (*Anitori, et al., 2014*).

8.3 Reaching the worst condition of girder and frame bridge components

The proposed segmentation (*Table 8.1*) is helpful when providing performance predictions for deteriorating bridge elements. This can be done using historical data, e.g. using a condition rating as a PI, in discrete-time Markov chains. Here, the distribution between condition ratings can be estimated, using Markov transition probabilities. If other relevant data is available for a given component type (e.g. environmental exposure), this can be used for grouping of components under similar circumstances (exposure, material properties, geometry etc.).

Given that the component is considered to fail when it reaches its worst condition or another performance goal, then the survival is defined as a condition where the performance goal is not violated. The survival of bridge components as function of bridge age has been studied in (*Masovic & Hajdin, 2013*), using the data from the Serbian Bridge Information Database (*Figure 8.3*).

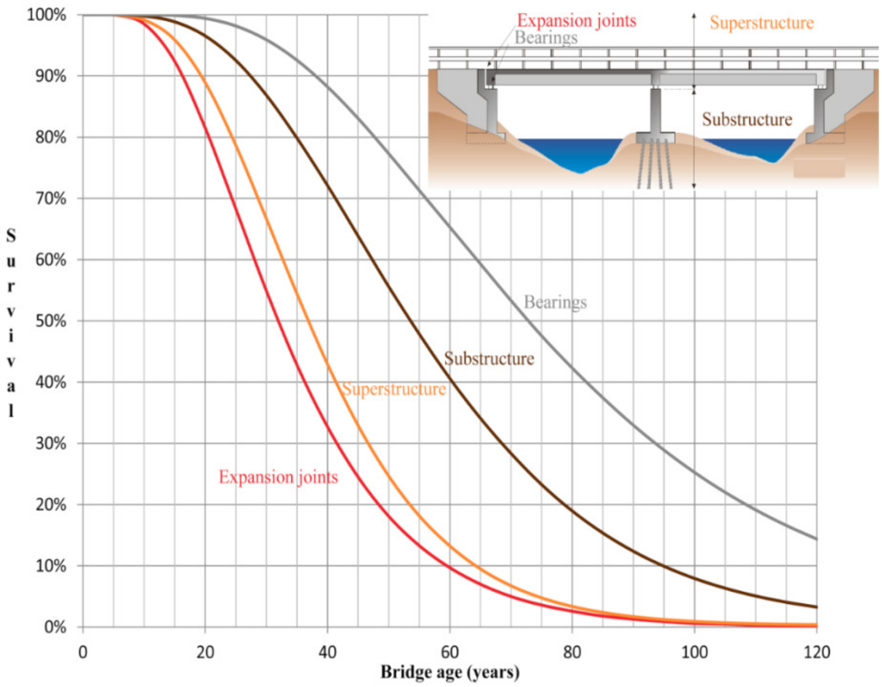


Figure 8.3 The survival of bridge components (Masovic & Hajdin, 2013)

From *Figure 8.3* it is observed that expansion joints are the most vulnerable bridge component with a 50% survival probability at approximately 30 years. As a comparison, the Danish Road Directorate has published typical service lives under normal operation establish as a joint effort between Owners, Consultants and Contractors (*Table 8.2*).

Table 8.2 Typical service lives for bridge expansion joints under normal operation (DRD, 2014)

Expansion joints	30-50 years (general)
	50-100 (cast-in parts)
	15-25 (sealers)
	10-15 (joints)

When estimating survival probabilities in *Figure 8.3*, other data that might affect deterioration were not used. It is expected that the survival probability of e.g. expansion joints would change if conditioned by the Annual Daily Traffic (ADT).

It should be noted that the presented survival probabilities are based on a condition rating, that already exist in various BMS, but cannot provide information concerning structural reliability. In addition, in some countries the condition rating is not related to a specific damage process but rather present the lump sum of various damages on a bridge element. Also, in modelling of deterioration, the probabilities of the Markov chain model are commonly estimated using statistical calculation rather than modelling of relevant physical or chemical processes. Nevertheless, the model can be very useful since it enables employment of Markov decision process in bridge management.

In general, a correlation between the condition rating and the probability of failure is implicitly assumed. It should be noted that elements of the substructure support the superstructure implying that failure of the substructure might lead to a total collapse. It is useful to relate failure modes to structural groups. In general, superstructure might fail in a bending or shear failure mode, while the substructure might fail in crushing or buckling failure mode. Although condition ratings give insufficient information concerning failure modes, *Figure 8.3* presents that historical data in broad terms confirm that elements of the substructure deteriorate rather slowly in comparison to superstructure elements. Equipment failure would affect the level of the service that the bridge provides, but it also affects further deterioration of structural elements.

8.4 Bridge types

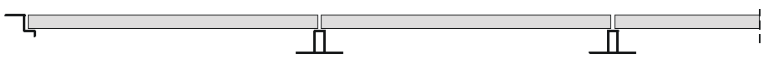
Labelling of components has been performed with reference to the BRIME project (*BRIME, 2001*), which was partly funded by the European Commission. In the following, most of the figures are taken from this project with some adjustments. Sketches like these should be incorporated in inspection protocols in order to simplify and harmonize observations.

Most common structural systems for girder and frame bridges:

Girder bridges (label G):

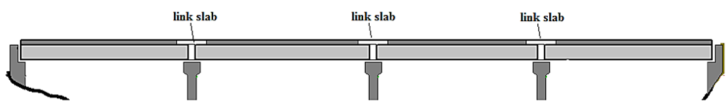
Single span (label **GA**)

Multiple span simple beams (label **GA1**)



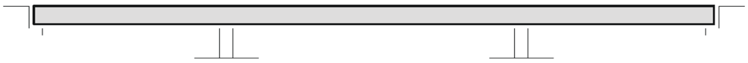
Gerber type girders (label **GG**)

Semi-continuous type girder (label GCS):

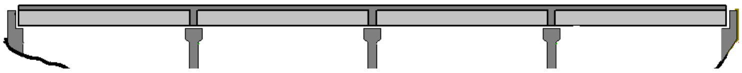


Continuous girders:

Monolithic (label **GC1**)



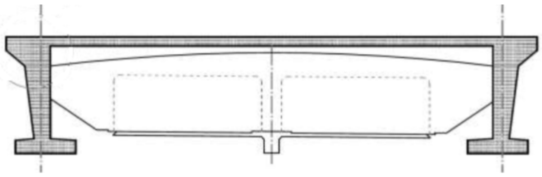
Made continuous from precast elements (label **GC2**)



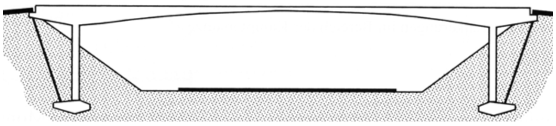
Frame bridges types (label F):

Vertical piers:

Single span (*integral type*) (label **FA**)

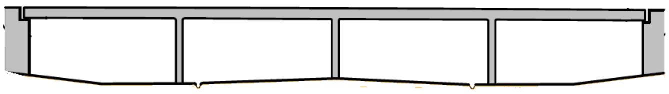


Rigid frame with tie (label **FB**)

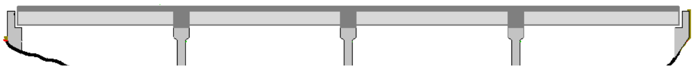


Multiple spans (semi integral type):

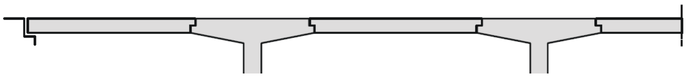
Monolithic (label **FC1**)



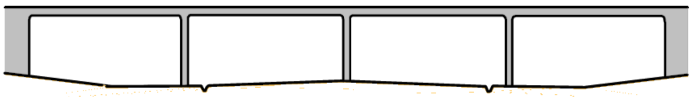
Made from precast elements (label **FC2**)



Gerber type frame (label **FD**)



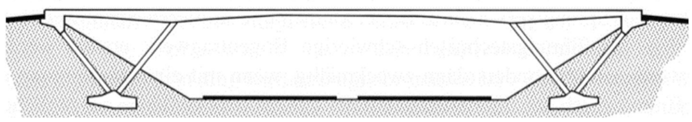
Integral bridge (label **FE**)



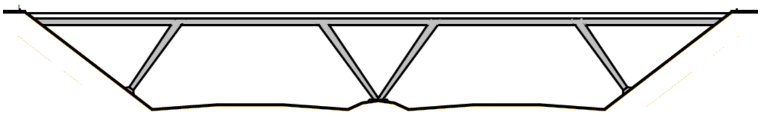
Inclined piers:

Inclined leg frame (label **FF**)

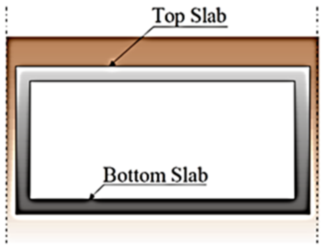
Inclined leg frame with tie (label **FG**)



Frame with "V" piers (label **FI**)



Box-culvert bridge type frame (label **FBC**) – it should be noted that in some countries the culverts are considered separately from bridges

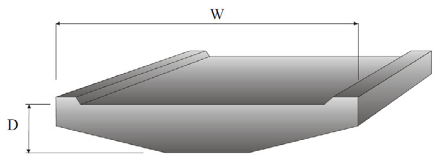


8.4.1 Superstructure

The most common superstructure cross sections related to the aforementioned bridge types are:

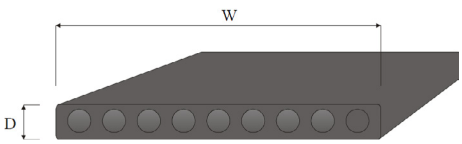
Slab

Solid slab (Label SA1)



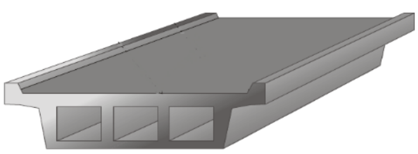
Pseudo slab (Label SA3)

Void slab (Label SA2)



Multicellular:

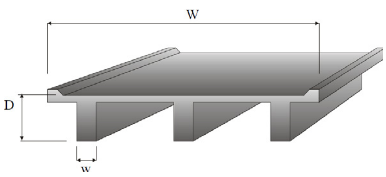
Cast in place (Label SB1)



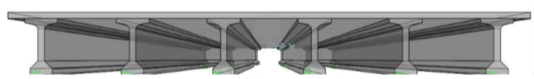
Precast (Label SB2)

Slab on beams:

Cast in place (Label SC1)



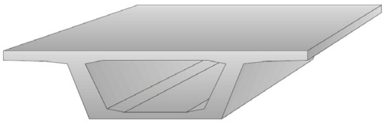
Precast (Label SC2)



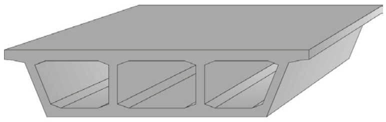
Steel beam and slab (Label SC3)

Box section:

Single cell (Label SD1)



Multicell (Label SD2)



Steel box and PC slab (Label SD3)

Proposed segmentation related to the cross sectional (deck) type SA, SB, SD (top and bottom slab):

Similar segmentations can be performed for other cross section types.

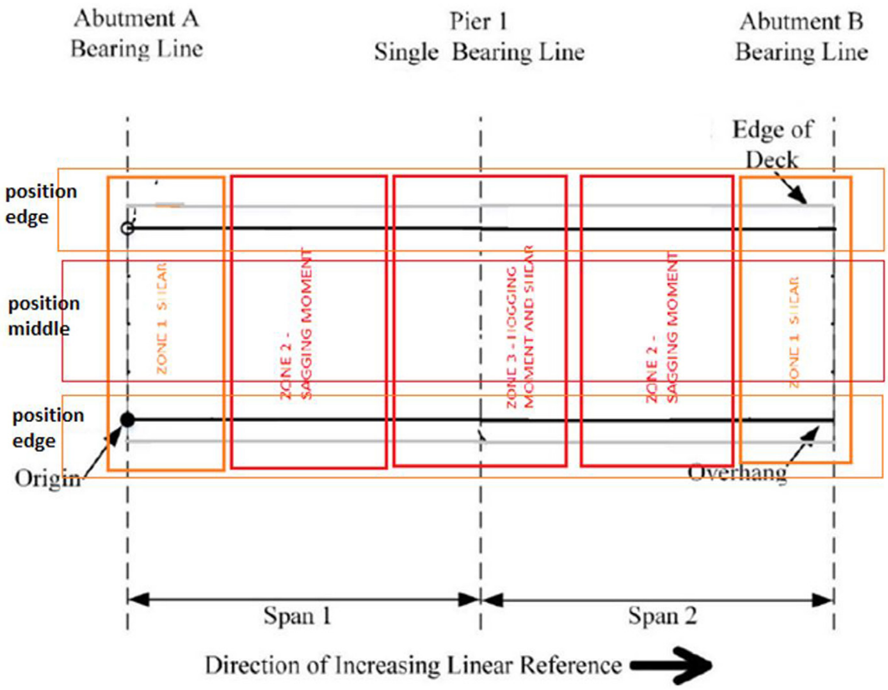


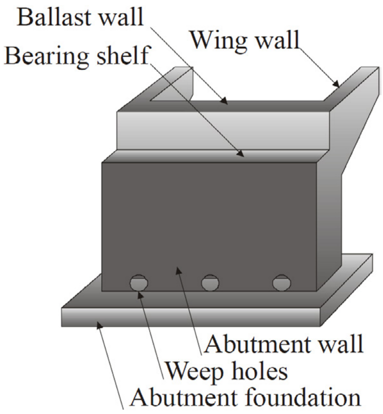
Figure 8.4 An example of segmentation for a two-span bridge, adapted from (LTBP, 2016)

8.4.2 Substructure

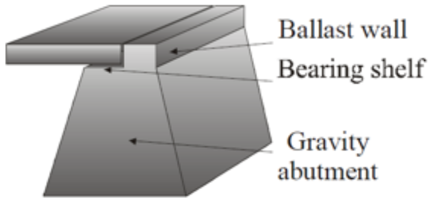
A similar review for the elements of the substructure can be performed. Types of abutments, piers and foundations can be listed and labelled for further reference. Regarding embankments and retaining walls, e.g. in (*BRIME, 2001*).

Abutments *(label* ***AB****): refer to the girder and frame bridges except for integral abutment bridges*

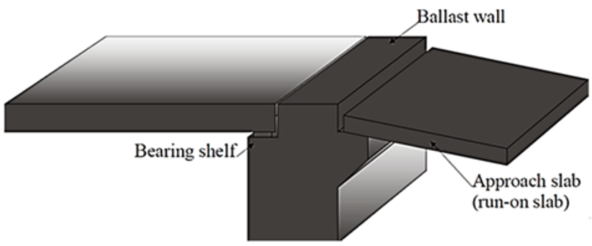
Cantilever Abutments (Label **AB1**)



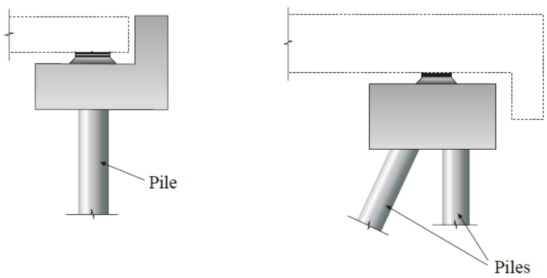
Gravity Abutments (Label **AB2**)



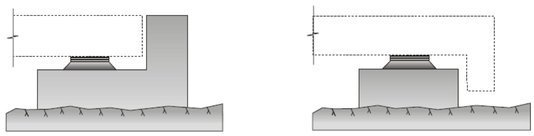
Typical accompanying elements to the abutments



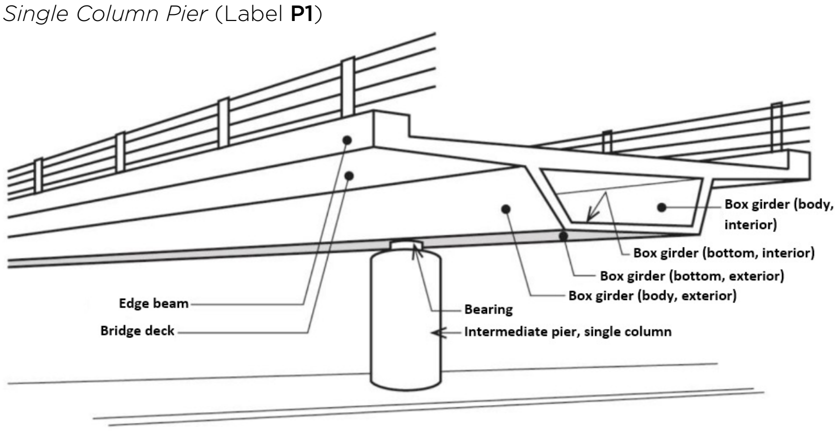
Pile-Abutments (Label **AB3**)



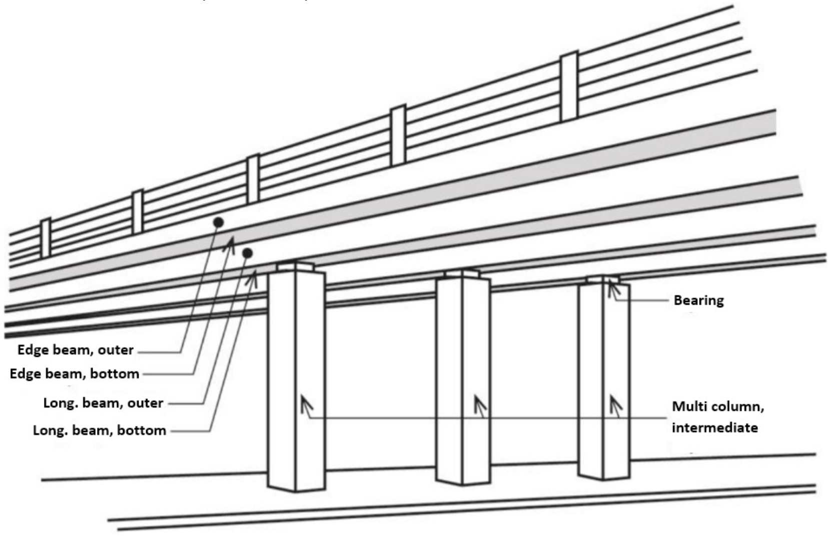
Bank-Seated Abutments (Label **AB4**)



Piers (Label **P**): *refer to the girder and frame bridges*
Single Column Pier (Label **P1**)



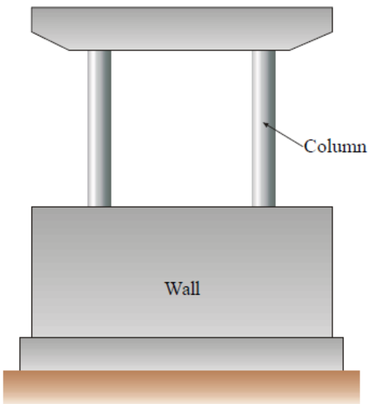
Multi Column Pier (Label **P2**)



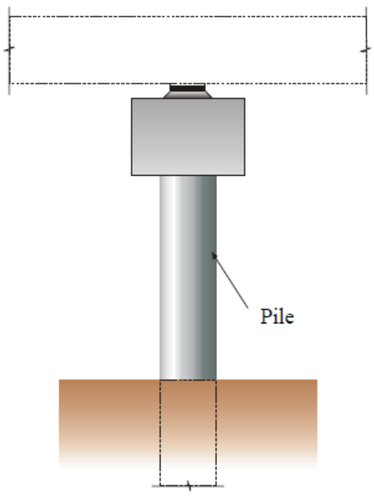
Wall Pier (Label **P3**)

Gravity Pier (Label **P4**)

Column/wall Pier (Label **P5**)

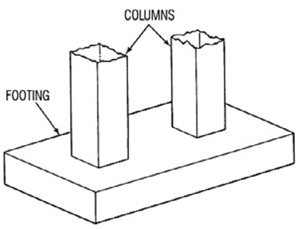


Pile Pier (Label **P6**)

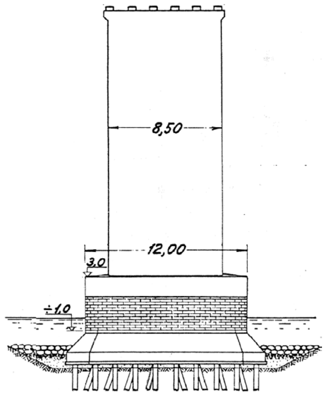


Foundation (Label **FU**): *refer to the girder and frame bridges*

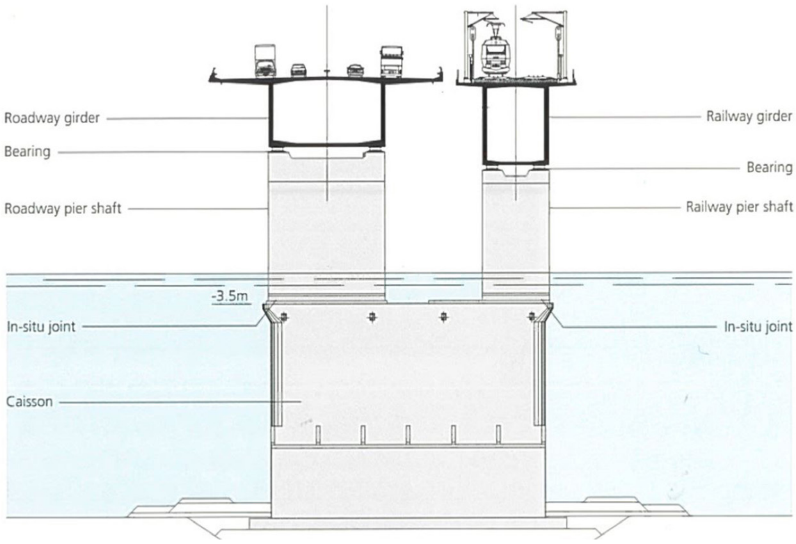
Spread footing (Label **FU1**)



Piles (Label **FU2**)



Caissons (Label **FU3**)



8.4.3 Equipment

A similar labelling for the components in the equipment-category (bearings, expansion joints, drainage, waterproofing, pavement/overlay, barriers and windscreens, signs and installation) can be performed based on equipment specific literature listed in *Section 7.2.4*.

8.5 Illustrative Example

In this section, the protocol for a quality control plan (*Table 8.3*) is used to demonstrate the presented WG3 methodology. Consistent with current inspection practices, the values of KPI are evaluated as a snapshot in time. The inspection should be carefully planned before visiting the site. As discussed, the locations of vulnerable zones depend on a structural system, and those locations in the example are acknowledged in *Figure 8.5*. During a visual inspection, those zones should be carefully examined. The orange cycles indicate zones where bending failure is possible (i.e. a ductile failure mode). The red cycles indicate more critical zones, where shear failure is possible (i.e. a brittle failure mode).

Any information regarding previous inspections and/or interventions is very important and attention should be paid to the locations were previous inspection records reveal damages, especially if those are in the vulnerable zones. If the damages were repaired, a current inspection would reveal the effectiveness of the repair measure, and if not, it can indicate the speed of the damage process.

Static quality control - Step I: Preparation for the inspection (office work)

1. Review of the inventory information:
    a. RC frame bridge;
    b. Original blueprints available; - see *Figure 8.5* with marked vulnerable zones
    c. Construction year 1963;
        i. No particular weaknesses of original design;
        ii. No particular material weaknesses are known - steel bars do not have any ductility problems
    d. Widened in 1977;
        i. The obvious weakness is the longitudinal joint connecting the old and the new part of the bridge
        ii. The bridge was recalculated in 1977;
        iii. Code of practice has changed since widening (no information concerning prior reliability index);
    e. Compare the current traffic load to traffic load model used for previous calculation;
    f. Estimate prior ''virgin'' reliability index (it was estimated as 3.8 for this example)
2. Other relevant information:
    a. Estimated current traffic on the bridge (AADT is 10.000);
    b. A local road passes beneath the bridge (uncertain AADT on the local road);
    c. No particular natural hazard;
    d. Location is city periphery;
    e. Climate is continental;
3. Review of the previous inspections/interventions:
    a. 2001. Condition rating – fair (an intervention was suggested);
    b. 2008. Condition rating – poor (an intervention was suggested);
    c. 2014. Condition rating – serious (estimation of the load rating was suggested);
    d. No data available concerning previous interventions (type, costs, etc.).

Static quality control - Step II: On-site inspection

1. Study of previous inspections and inventory information may suggest that on-site material properties should be investigated. Collection of samples for lab test may be performed.
2. Damage identification (location):
    a. Previous damages in comparison to the previous inspection records (if any);
    b. New damages in comparison to the previous inspection records (if any);
    c. Evidence of previous repair (if any, either recorded or not).
3. Assessment/measurement of damage extent and intensity;
4. Identification of damage processes;
5. Qualitative assessment of resistance reduction based on observed damages. Preliminary (rough) assessment of resistance reduction on structural level (reliability). Is it necessary to perform in- depth investigations?
6. Assessment of Safety (life and limb)

Dynamic quality control steps (office work):

1. Model the damage process
2. Estimate the remaining service life
3. Define various maintenance scenario
4. Compare the scenarios / determine the optimum scenario

For each element, observations on the element level are recorded and underlying damage process is recognised. That information is of essential importance regarding prediction of the future bridge performance.

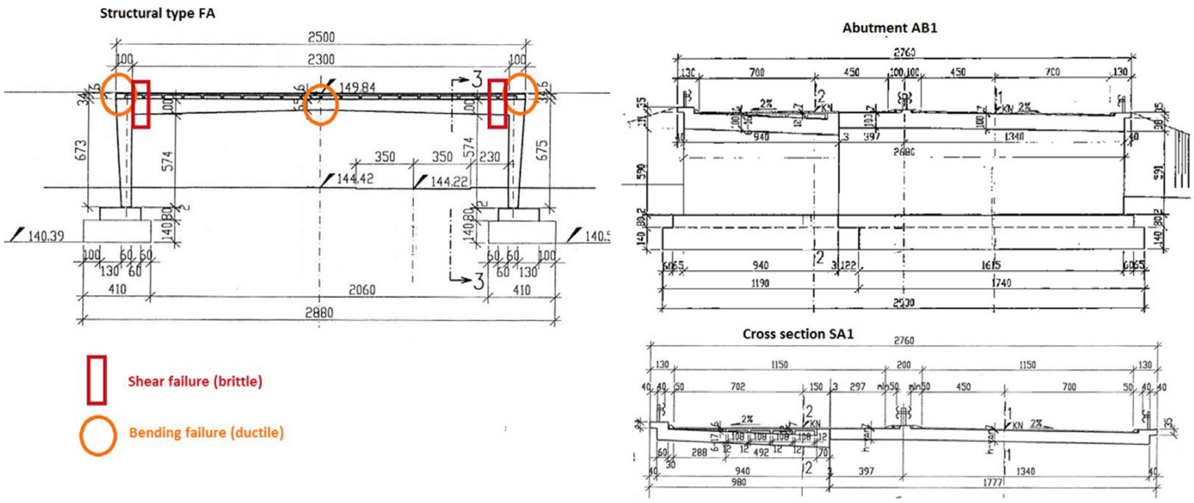


Figure 8.5 The bridge cross section and longitudinal section with labelled vulnerable zones

The case when some of the vulnerable zones are not accessible for visual inspection should be noted in the inspection report, which can trigger further investigations. In this example, those zones are high hogging moment regions (the orange cycles at the frame corners in *Figure 8.5*). Alternatively, evaluation of issues that may be present in these zones should be based on an engineering judgement (e.g. by observing deflection under the current traffic load). Related reasoning should be stated in the inspection report as well.

Photos from visual inspection are incorporated in *Figure 8.6*. In the vulnerable zones, observations are the following:

1. No active cracks or spalling at „red zones"; (an uncertain cause and development of diagonal crack is observed, but it was repaired);
2. Severe spalling with loss of reinforcement section in the „orange zone" related to sagging moment region;
3. "Orange zones" related to hogging moment region is inaccessible.

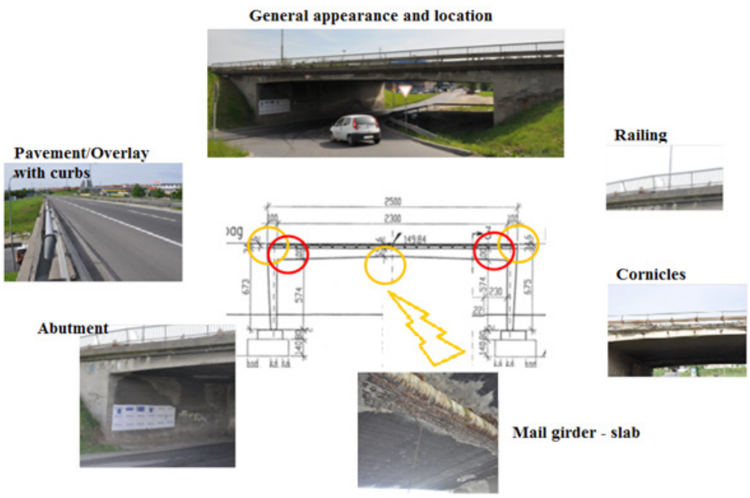


Figure 8.6 The main observations from the visual inspection

The proposed protocol is shown in *Table 8.3*. The table is intended to present current observations regarding present visual inspection. Note that other relevant data (e.g. obstacle type, exposure to hazards, ADT, etc) are not shown here, although they are should be the part of a bridge inventory list (see *Section 12.1*).

Table 8.3 Protocol for qualitative evaluation of KPIs Reliability and Safety for the Illustrative example

Structure	Group	Element/ Material	Observations			Damage Process	Vulnerable Zone	Failure Mode	Primary KPI	Element Evaluation	KPI value	
			Damage	Closer Specifications							Reliability	Safety
				Quantity for Repair	Position Within the Element							
Frame bridge type FA	Structural Elements Equipment	Deck old SA1/RC	Crack	Repaired	Diagonal	Not active	HS	Shear failure mode	(R)	2	3	2
			NA	NA	NA	Susceptible corrosion	HMH	Bending failure mode	(R)	3		
			Reinforcement corrosion	10%	Bottom	Corrosion	HMS	Bending failure mode	(R)	4		
			Spalling	15m2	Bottom	Corrosion	HMS	Falling pieces	(S)	2		
			Efflorescence	5%	Bottom	-	HMH	-	symptom	-		
		Deck new SA1/RC	NA	NA	NA	Susceptible corrosion	HMH	Bending failure mode	(R)	2		
			Reinforcement corrosion	5%	Bottom	Corrosion	HMS	Bending failure mode	(R)	3		
			Spalling	8 m2	Bottom	Corrosion	HMS	Falling pieces	(S)	2		
			Efflorescence	5%	Bottom	-	HMH	-	symptom	-		
		Abu.1 AB1/ RC	Spalling	0.5m2	Abutment front	Corrosion	-	-	(R)	2		
		Abu.2 AB1/ RC	Spalling	0.8m2	Abutment edge	Corrosion	-	-	(R)	2		
	Equipment	Curb	Missing peace	1%	-	Abrasion	-	-	(S)	2		
		Railing	Deformation	5%	-	-	-	Falling from the bridge/	(S)	2		
		Railing	Flaking	10%	-	Corrosion	-	-	(S)	2		
		Asphalt Overlay	no damage	-	-	-	-	-	(S)	1		
		Cornices	Spalling	4.5m2	80%	Corrosion	-	Falling pieces	(S)	2		

The performance value for the KPI of Safety is evaluated with attention towards other relevant data, such as the traffic under the bridge (not shown in this table). Safety issues are evaluated regarding user's safety, and these relate mainly to non-structural components i.e. equipment. It should be noted that spalling from the deck slab and cornices implies the risk of injuries due to chunks of concrete falling and potentially hitting by-passers or vehicles under the bridge. The related qualitative performance scales are enclosed in Appendix (*Section 12.2*). The evaluation of the waterproofing is omitted from the table since it is not directly available for a visual inspection.

The evaluation of the components is also presented in *Table 8.3* (the column Element evaluation), as such data already exists in most BMSs. This is done to distinguish from the previous practice, which is mainly cost oriented and usually disregards a system behaviour. The component condition might be based on replacement costs (weighted sum).

Alternatively, if there is more than one condition rating for one element, because different types of deterioration have been found on this element, the maximum of these condition ratings will be adopted for this element. In the given example (*Table 8.3*), it would mean that the deck should be in condition four, while the abutments should be in condition two. There is no information of the overall bridge condition, but it can be rated by the worst condition among the load bearing elements. It should be noted that this approach is conservative.

As the final result of the proposed methodology, the performance value on the system level is not the highest performance value on the structural component level. The reasoning for this should be further elaborated.

Given that the system is statically indeterminate and bending failure is anticipated, it is possible that a load redistribution might occur. It should be noted that there is high uncertainty concerning hogging moment region. Since no issues concerning deflection under current traffic on the bridge is observed, it was concluded that those zones still have the adequate resistance. Based on experience and elementary statics, the resistance reduction can be assessed as approximately 10%, which can be used for an update of the Reliability KPI via reliability curves (*Figure 8.7*). More information on the curves and qualitative performance scales are enclosed in the Appendix (*Section 12.2*).

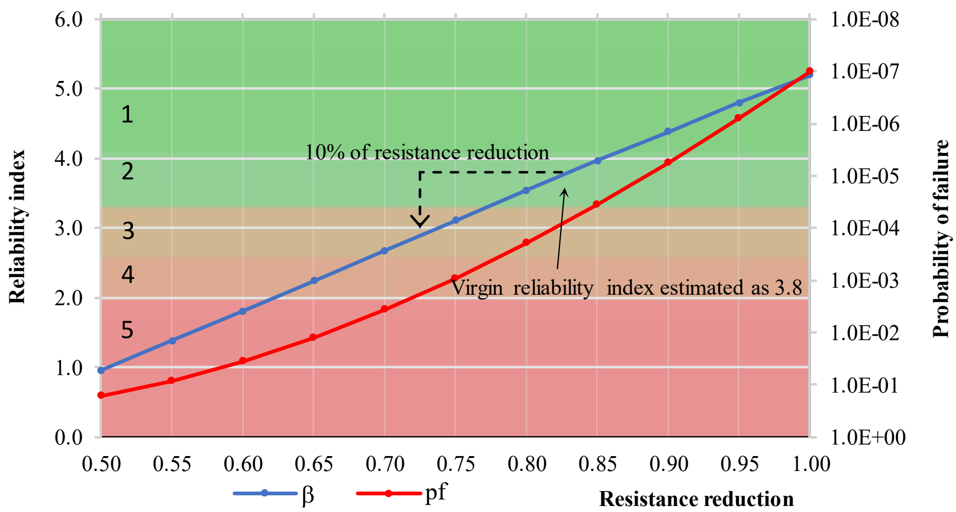


Figure 8.7 An update of the qualitative Reliability based on a reliability curve, virgin reliability index and inspection

Nevertheless, the assigned value of KPI (R) on the structural level might require qualitative structural assessment. Structural elements are evaluated mainly with the respect to the reliability and inclusion of the system behaviour i.e. anticipated failure mode would improve the assessment. In the proposed protocol, all observations including present irrelevant damages and symptoms should be recorded for the future reference.

Only observations related to vulnerable zones, regarding structural reliability are given in *Figure 8.8*, while the most critical observations are highlighted (orange colour). Anticipated failure mode is also noted. The damage process is omitted since it is of no importance for the snapshot assessment.

The presented KPI values are evaluated as a snapshot in the time of the inspection, but other relevant data regarding previous inspections and interventions are of importance. In this example, the crack in the HS region was obviously repaired, but there is no record when such intervention was performed. Since the crack does not seem to be active, it was concluded that the shear failure is not an issue. Also, it seems somewhat confusing that the previous observations indicate damage of the waterproofing while the pavement is in a good condition. This indicates that repavement was performed and previous intervention data should have confirmed that.

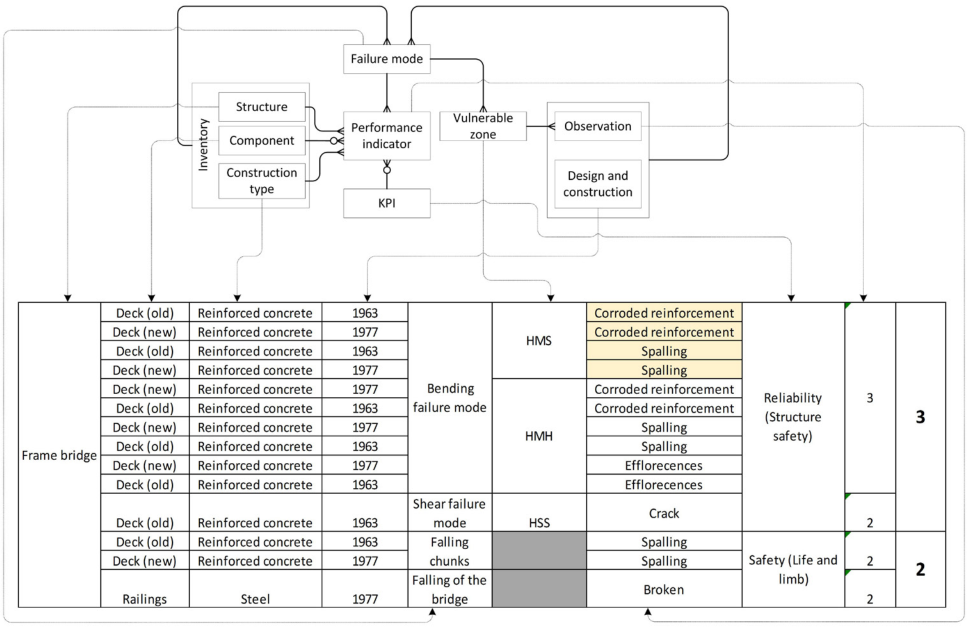


Figure 8.8 The protocol (static, i.e. snapshot in time – concise version)

Previous observations (in inspection records, if any) at the same location of the structure can indicate the rate of a damage process and are very valuable for the performance prediction. For example, the presented bridge was inspected in 2014 and the loss of section of reinforcement bars at the midspan was similar to the present observations, indicating that the process is either slow or stopped.

In this case, the bridge is a part of an important highway (e.g. AADT > 10000) and the required KPI (A) could affect previous (as well as present) decisions.

In *Figure 8.9*, the ''time'' entity is added representing the remaining service life, i.e. the point in time at which Reliability or Safety will reach some threshold value (i.e. value five according to the adopted scale). The proposed KPI (R) is based on reduction of structural resistance, qualitatively estimated on a simplified structural system (e.g. a simple beam). The remaining service life (i.e. reaching an unacceptable return period for a failure) is estimated based on the foreseen rate of deterioration and anticipated damage process (omitted in *Figure 8.9*). If possible, the latter should be backed up by inspection records or other verified models.

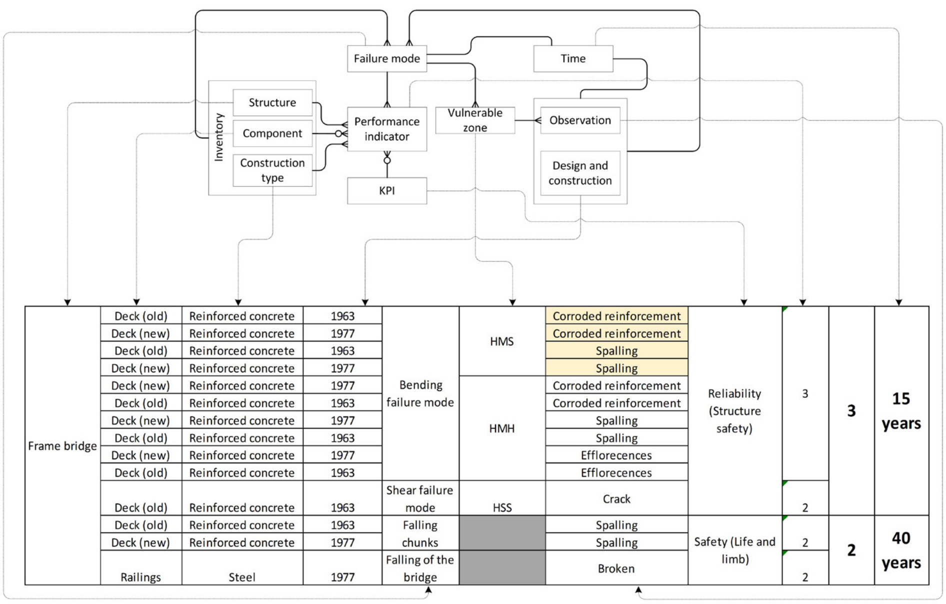


Figure 8.9 The protocol with time scale, i.e. dynamic

For example, a linear decrease of Reliability is assumed (given that the resistance reduction is governed by a loss of section of reinforcement bars. The related reference maintenance scenario is presented in *Figure 8.10*. This scenario comprises full repair at year 15 and year 105.

The deterioration rate has been assumed linear between plateaus at each Reliability level, and that accelerates significantly from the Reliability level three to level five.

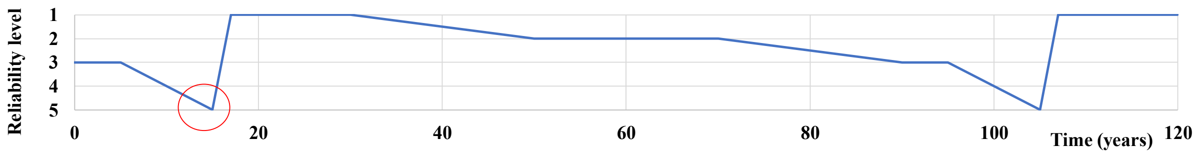


Figure 8.10 Reliability as function of time.

For each Reliability level and with the respect to time preference linked to the damage process, each country might establish maintenance scenarios.

For example, several maintenances scenarios for various Reliability levels are proposed in *Table 8.4*. For the present case study, strengthening to Reliability level one (reliability index above four, see *Section 12.2*) is more expensive than repairing to the 'virgin' reliability level (reliability index 3.8).

Table 8.4 Examples of maintenance scenarios for each Reliability level.

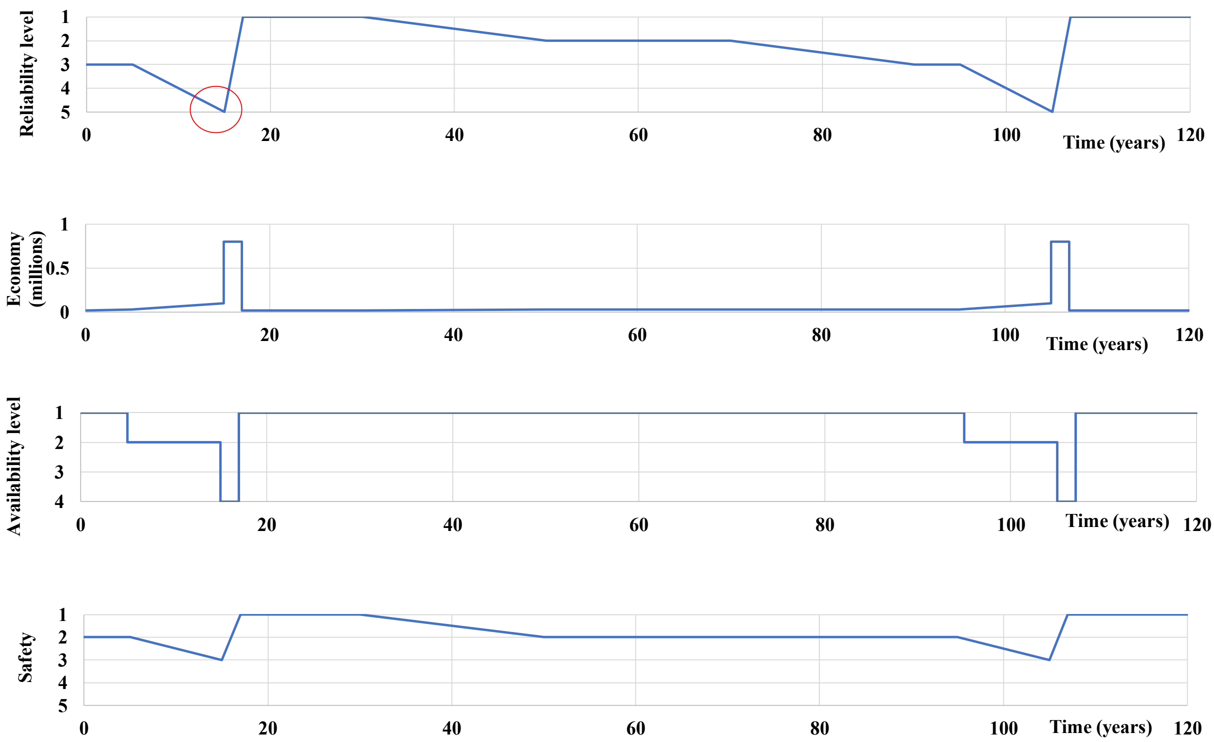
Reliability level	**Scenario**	**Measures***
1	Reference	Do nothing (schedule for the next inspection in five years)
2	Reference	Do nothing (schedule for the next inspection in e.g. five years)
	Preventive basic	Strengthen to establish Reliability level one
3	Reference	Do nothing (schedule for the next inspection in e.g. five years)
	Preventive 1	Do nothing (schedule for the next inspection in e.g. three years)
	Preventive 2	Repair to establish as design 'virgin' reliability
	Preventive basic	Strengthen to establish reliability level one
4	Reference	Do nothing (schedule for the next inspection in e.g. three years)
	Preventive 1	Do nothing (schedule for the next inspection in e.g. one years)
	Preventive 2	Repair to establish as design 'virgin' reliability
	Preventive basic	Strengthen to establish reliability level one
5	Reference	Strengthen to establish reliability level one (mandatory!!!)

*5 years is understood as a regular time between inspections (might vary between countries), three or one years are crudely assumed given the damage process.

When the inspection is scheduled with shorter time intervals than regular (e.g. three or one year), it should be followed by the reliability assessment.

For each scenario, graphs for each KPI (R - Reliability, E – Economy, A – Availability and S – Safety) can be elaborated. This has been performed for the illustrative example as presented in *Figure 8.11*. "Reference scenario" is a "do-nothing" scenario from *Table 8.4*. It should be noted that Availability is established on the network level in a scale one to four, while Economy has a monetized scale

a) Reference scenario



b) Preventative scenario

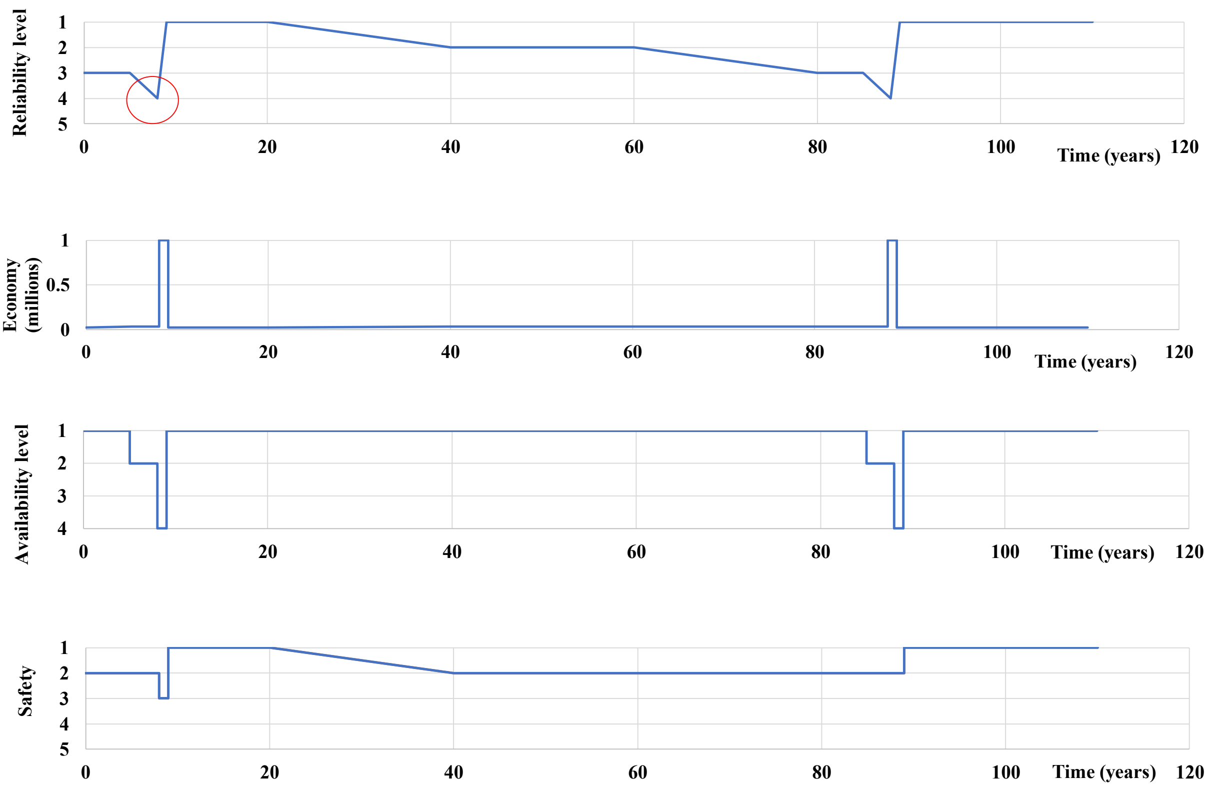


Figure 8.11 Comparison of KPI's for two maintenance scenarios for the bridge in the illustrative example

Comparison of various scenarios might be performed in many ways, e.g. monetization is widely adopted method. However, that approach is not chosen in this COST action.

When all KPI are expressed on the scale of one to five (one is the best, five is the worst) (see *Section 7.6*), the spider diagram over time can be generated. In *Figure 8.12*, a 3D spider diagrams are given as an example.

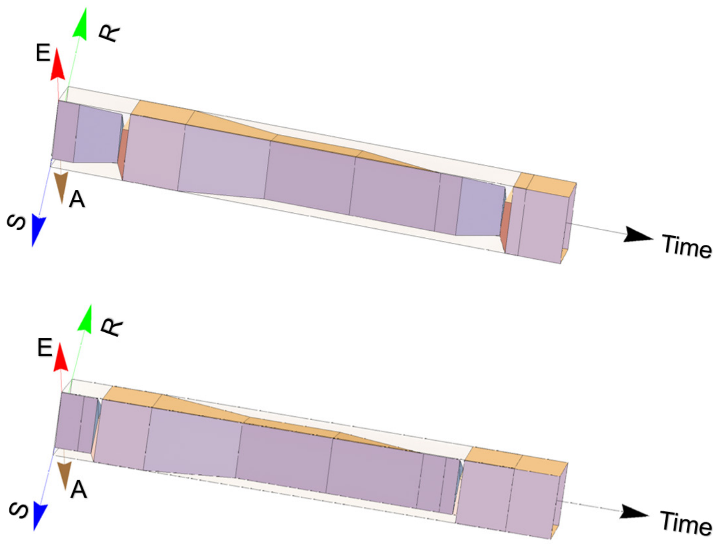


Figure 8.12 Comparison of KPI development as function of time for two maintenance scenarios

In order to account for the time preference, discounting is established for future expenditures. It is directly applicable to KPI of Economy (E). If the same procedure is applied to other KPIs (R, S, A), then the 'average' or the net present KPI for each scenario can be found and compared.

For decision making, the net present KPI in form of spider diagram is presented in *Figure 8.13*.

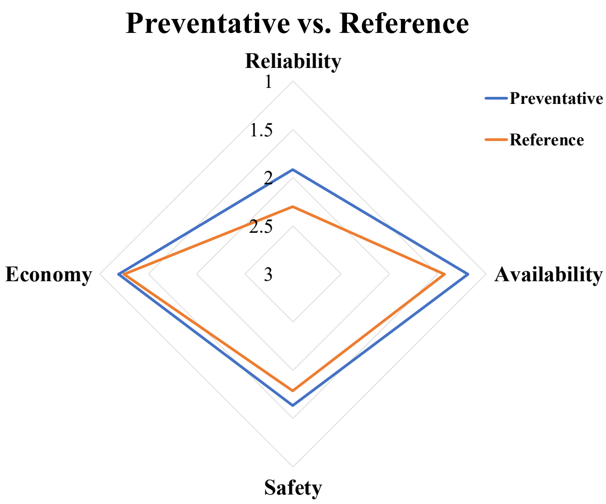


Figure 8.13 Comparison of the net present KPI for the two maintenance scenarios

9. Application of the Quality Control framework to arch bridges

9.1 Taxonomy & Ontology

Arches bridges may be grouped according to the following parameters:

- The position of the deck and the nature of the load transfer from the deck to the arch (*Figure 9.1*);
- The structural articulation - the arch can be fixed (*Figure 9.1a,b*) or hinged, either with one (*Figure 9.1c*), two (*Figure 9.1d*) or three hinges (*Figure 9.1e*) incorporated into the arch rib;
- The shape of the arch (*Figure 9.2*);
- The materials in use.

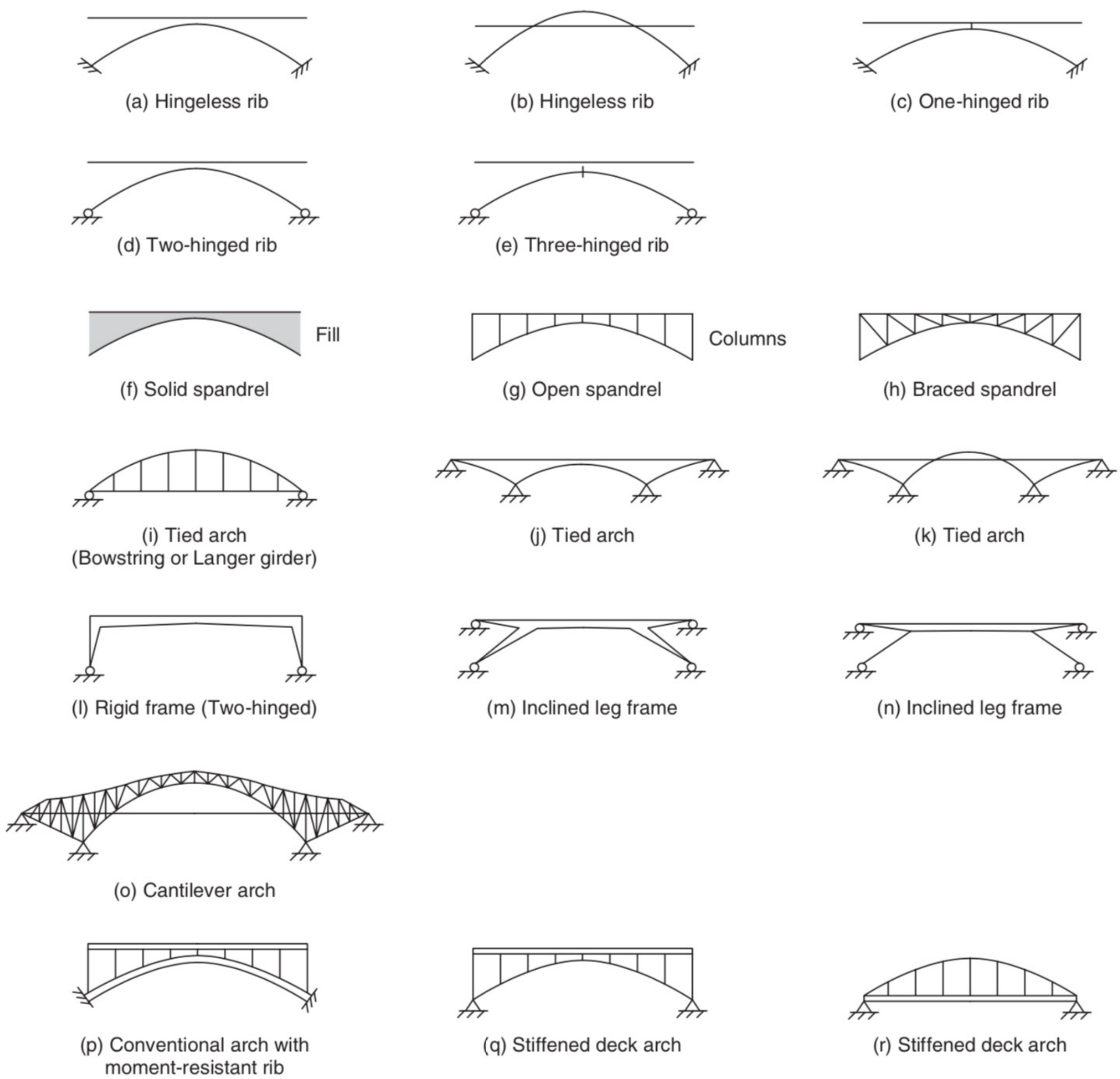


Figure 9.1 Type of arch bridges according to (O'Connor, 1971), cited by (Parke & Hewson, 2008)

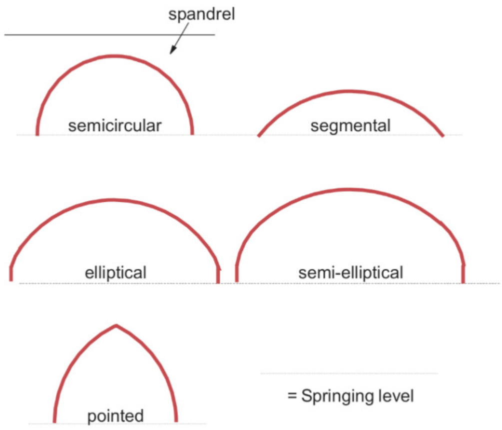


Figure 9.2 Different arch shapes, from (Ryall, 2001)

Element identification depends mainly on a bridge structural type and secondly on the used material. The four main structural types are the Close Spandrel Deck Arch, Open Spandrel Deck Arch, Through Arch and Tied Arch. Based on (*Hartle, et al., 2002*) the following definitions and element identification, for concrete and steel bridges, can be considered:

- *Close Spandrel Deck Arch*: an arch bridge with the deck above the top of the arch. Closed spandrel arches receive traffic loads through the fill material contained by spandrel walls. These spandrel walls are usually aligned with both sides of the arch barrel. The arch (in these cases called ring or barrel) diverts loads and weight to abutments or large piers. In some cases, fill material is replaced by several spandrel walls that distributes the loads over the barrel. It is also common to use small openings in the spandrel walls to reduce the weight of the fill material and to allow water to pass through in a case of high water flow.
- *Open Spandrel Deck Arch*: an arch bridge with the deck above the top of the arch. The load is transferred to the spandrel columns, which are in compression. The arch supports the spandrel column and transfers the compressive load to the ground at the supports. Concrete and steel structures are designed to resist a load combination of axial compression and bending moment with stringers or floor beams resisting the traffic load in bending. In masonry structures, secondary arches are used to transfer the load to the spandrel columns.
- *Through Arch*: an arch bridge in which the deck passes between the arches. The through arch is constructed with the crown of the arch above the roadway and the arch foundations below the roadway. The deck is hung from the arch by wire rope cables or other tension members. Traffic loads are supported by a deck, which resists the loads in bending. The load from the deck is transmitted to the stringers (if present) and then to the floor beams. The stringer and floor beams resist the traffic load in bending. The load is transferred to the hangers, which are in tension. The arch supports the hangers and transfers the compressive load to the ground at the supports.
- *Tied arch*: The tied arch can be considered a variation of the through arch. In a through arch, the horizontal thrust of the arch reactions is transferred directly to the foundations while the tied arch transfers the horizontal reactions through a horizontal tie which connects the ends of the arch together. The tie is therefore a tension member responsible to keep the arch in compression. Since tied arch bridges redistribute the horizontal loads to the tie girders, the piers for tie arch bridges can be smaller than the piers for through arch bridges. Traffic loads are supported by a deck, which resists the loads in bending. The load from the deck is transmitted to the stringers (if present) and then to the floor beams. The stringer and floor beams resist the traffic load in bending. The load is transferred to the hangers, which are in tension. The arch supports the hangers and transfers the compressive load to the ground at the supports. The horizontal component of the arch rib thrust is then transformed to the tie girders.

Element identification regarding the above mentioned structural types will be analysed according to the most common material – masonry, concrete and steel. Timber is not referred because it is usually restricted to small spans and it is particularly unusual in roadway bridges.

9.1.1 Close Spandrel Deck Arch

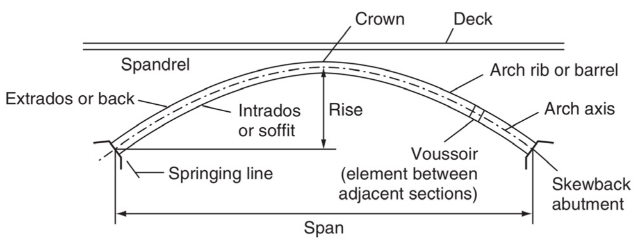
Close spandrel arches make use of the fill material (granular) to receive traffic loads and to distribute them through the arch ring. The structural elements are mainly subject to compression, therefore the most common materials for the arch, spandrel walls, abutments and piers are masonry (i.e. stone or brick) or concrete (light weight/non-reinforced).

These bridges usually have no separate bridge deck, but the arch carries side walls which retain the earth fill on which the wearing surface is laid. The arch slab in these bridges is often strengthened with a concrete slab (*Lindbladh, 1996*).

A taxonomy for this type of arch bridges include (*Figure 9.3* to *Figure 9.5*):

- *Arch barrel*: the load-bearing part of the arch. It contains a single thickness of voussoir stones or several rings of brickwork or random coursed rubble (varied stone size).
- *Arch Rings*: A single ring of bricks or stones of approximately even size formed to an arch profile.
- *Spandrel walls*: walls that sit on the edge of the arch barrel and that limits the extent of and retains the backfill.
- *Fill material*: regularly shaped body made of dried clay, usually incorporating straw for better cohesion.
- *Backfill*: material (usually low-quality fill) used to give support behind a structure. For a masonry arch bridge, backfill material is placed in the spandrels between the arch barrel and the road surface and retained laterally by the spandrel walls and/or wingwalls. It normally consists of granular material e.g. gravel or building debris, which may have been excavated for the foundations or is waste from the construction.
- *Deck*
- *Roadway pavement*
- *Bridge parapets*
- *Piers*: an intermediate support between adjoining bridge spans or a thickened section located at intervals along a wall to strengthen it.
- *Cutwaters*
- *Intrados*: the inner (concave) curve of the arch barrel.
- *Abutments*: a body, usually of masonry, which provides the resistance to the vertical forces and the thrust of the arch.
- *Wing walls*: a wall at the abutment of a bridge, which extends beyond the bridge to retain the earth behind the abutment.
- *Embankment*: transition between the bridge and surrounding ground, often providing horizontal and vertical support for the abutment foundation.
- *Skewback*: The inclined surface of the course of masonry located at the extremity of an arch, which transmits the stresses of the arch to an abutment or pier; surface of an inclined springing.

a)



b)

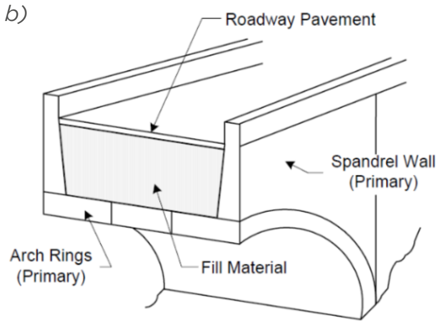


Figure 9.3 Elements in a Close Spandrel Arch bridges
a) Front view, from (Hartle, et al., 2002); b) Cross view, from (Parke & Hewson, 2008)

9.1.2 Masonry close spandrel arch

With special emphasis in masonry structures, the arch ring can be further subdivided in areas or smaller elements such as:

- *Voussoirs*: a wedged-shaped masonry unit used to make an arch or vault (voussoirs can be flat or irregular).
- *Springer* or *springing stone*.
- *Keystone* or *Crown*: the highest and last-placed stones in an arch. In the arch barrel of a bridge there are a series of keystones at the crown, across its width, which are often left projecting on side elevations.
- *Haunch*: The side of an arch, between the crown and the pier.
- *Bed joint*: a joint between masonry courses.
- *Bedding plane*: a plane of stratification in natural sedimentary stone.

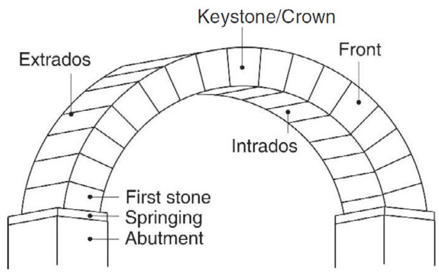


Figure 9.4 Terms of stone arch (vault), in (Koch, 1998) cited by (Proske & van Gelder , 2009)

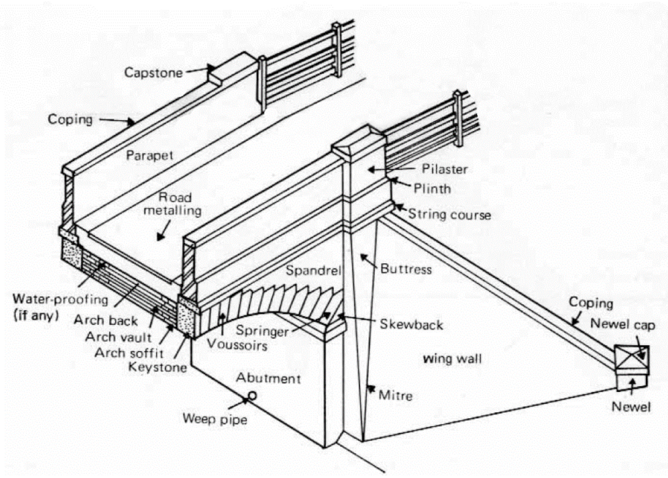


Figure 9.5 Terms of masonry arch bridge, from (McKibbins, et al., 2006)

Structural elements may have diverse geometry such as the various configurations of the arches, spandrels, piers and cutwaters. One single structure may include different solutions, mirroring its permanence over different periods of history and knowledge, as illustrated in *Figure 9.6*.

Figure 9.6 Geometry of stone arch bridges (Costa, 2009)

9.1.3 Concrete close spandrel arch

The early concrete closed spandrel arches mimicked stone bridges (*Figure 9.7*). In concrete structures and precast concrete culverts, where the arch can be identified as culvert barrel, joints might also be considered as elements.

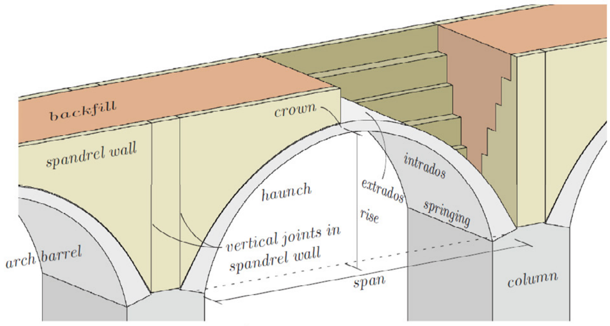


Figure 9.7 Terms in a concrete arch bridge, (Andersson, 2011)

Another type of close spandrel concrete arch is rib arches, with the ribs supporting solid spandrel walls, which in turn support the bridge deck (*Figure 9.8*).

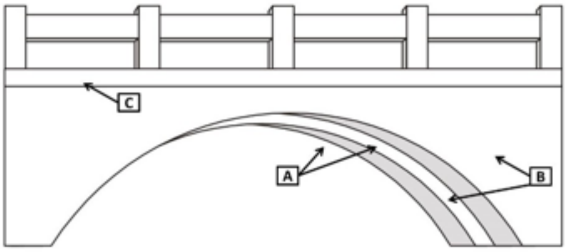


Figure 9.8 Concrete closed spandrel arch bridge
A – arch rib, B – spandrel wall, C – deck, (Miller, et al., 2000)

9.1.4 Open Spandrel Deck Arch

This type of structures substitutes the fill material by spandrel bents or spandrel columns that receive traffic loads, resulting in lighter structures. The spandrel may be open with columns and/or hinges used to transfer the deck loads to the arch.

9.1.4.1 Concrete open spandrel deck arch

A common taxonomy can be used based in the following elements (*Figure 9.9* to *Figure 9.11*):

- Deck girders
    - Longitudinal girder
    - Cross beam
- Deck slab
- Arch
    - Arch ribs: single, double, multiple; separated or connected by bracing system
    - Cross-section: solid, box: single, double or multiple cell
    - Arch strut
    - Bracing system
- Piers
  Location:
    - Spandrel piers, piers at the springings (portal piers), piers founded on soil
  Type:
    - Solid wall pier
    - 2 or more columns connected at the top by a head-beam
    - 2 or more columns connected by cross-girders
    - Multiple columns without head-beam
- Abutments
- Arch hinge

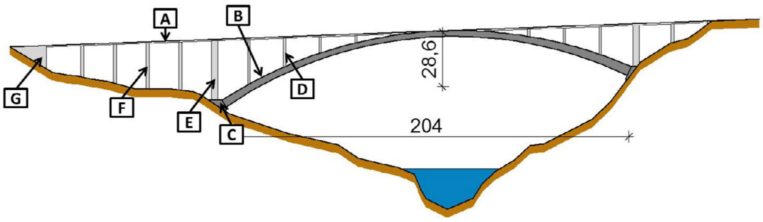


Figure 9.9 Elements in an Open Spandrel Concrete Arch
A – Superstructure, B – Arch, C – Arch abutment/springing, D – Spandrel pier (above arch), E - piers at the springing (arch abutments), F – piers founded on soil, G – abutment, adapted from (Radić, 2009)

a)

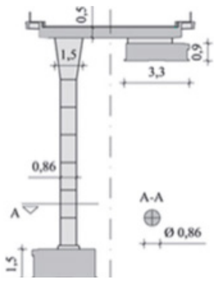
b)

c)

d)

Figure 9.10 Pier types
a) wall pier (Wölfel, 2013); b) two columns connected at the top by a head-beam, (Bleiziffer, et al., 2011);
c) double columns without head-beam, (Radić, et al., 2016);
d) two columns connected by cross-girders, (Beslać, et al., 2008)

a)



b)

Figure 9.11 Open spandrel arch types
a) separated solid arch ribs, (Franetović et al., 2013);
b) arch ribs connected by bracing system, (Radić, et al., 2016)

9.1.4.2 Steel open spandrel deck arch

Steel arch bridges are commonly distinguished by the rib type, with arches being classified as solid ribbed, braced ribbed, or spandrel braced (*Hartle, et al., 2002*), (*Han, et al., 2016*). These types of bridges are usually used for long spans.

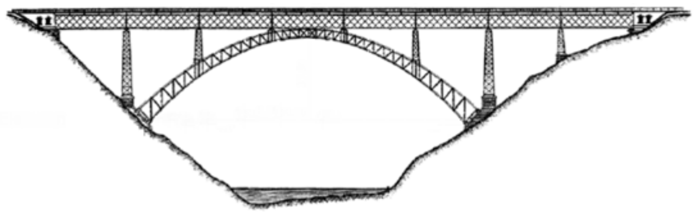


Figure 9.12 Open Spandrel Steel Arch Bridge, adapted from (Gentile & Saisi, 2013)

Independently from the rib type the following taxonomy can be defined):

- Arch ribs
- Spandrel columns
- Spandrel girders
- Deck girders
    - Longitudinal girder
    - Cross beam
    - Bracing
- Deck slab
- Stringers (if present)
- Sway bracing
- Upper lateral bracing (bracing in the floor system)
- Lower lateral bracing (bracing in the arch rib)

a) b)

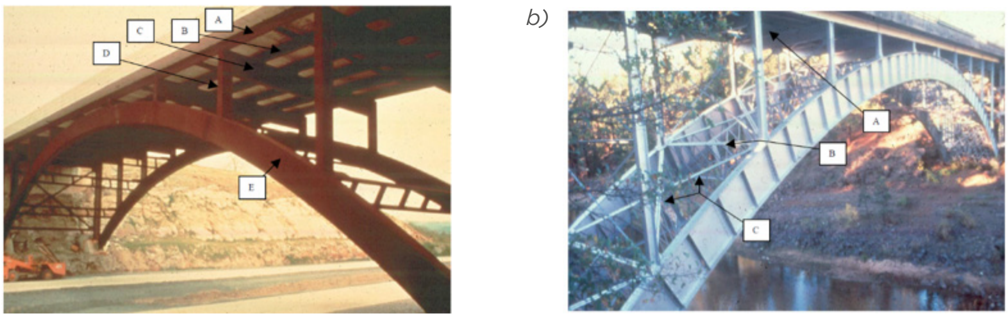


Figure 9.13 Elements in an Open Spandrel Steel Arch, adapted from (Hartle, et al., 2002)
a) A - Spandrel girders; B - Stringers; C – Floor beam; D - Spandrel columns; E - Arch rib; b) A - Upper lateral bracing; B - Sway bracing; C - Lower lateral bracing

Since the curved rib of the arch bridge is subject to a high axial force, the chance of a failure due to buckling of the rib cannot be ignored and must be accounted for. For this reason, slender arches are often connected by bracing. *Figure 9.14* shows some types of bracings.

a) b) c)

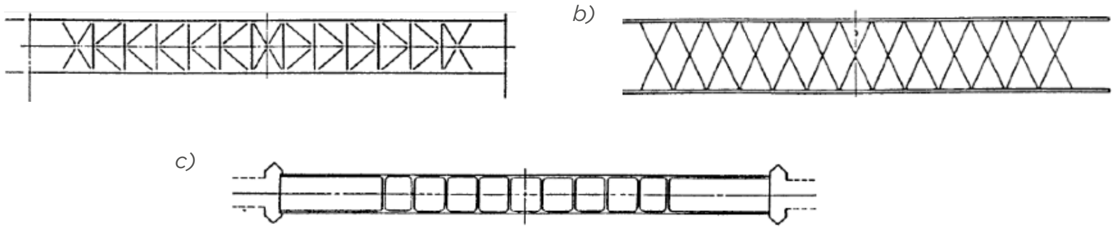


Figure 9.14 Different types of bracing (Chen & Duanbridge, 2000)
a) K-type of bracing; b) Diamond type of bracing; c) Vierendeel type of bracing

The steel deck can be made as orthotropic plate, and there are two basic types of longitudinal ribs: open ribs (flat bars, bulb sections, inverted T-sections), and closed ribs of a trapezoidal or rounded cross section (*Figure 9.15*). The characteristic difference between the open and the closed ribs is in their resistance to torsion; the torsional rigidity of the closed ribs is considerable, while that of the open ribs is very small.

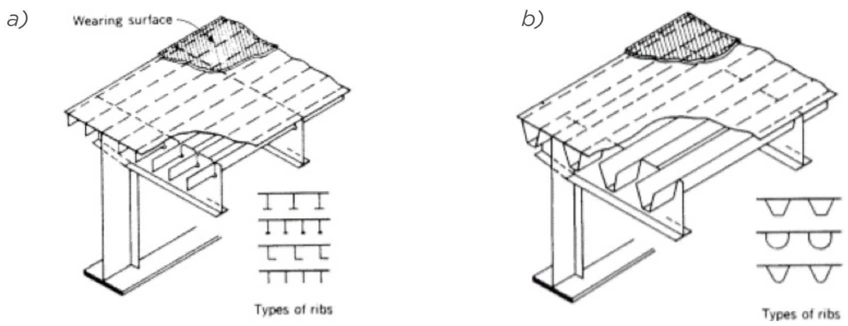


Figure 9.15 Two basic types of steel plate bridge deck, (Chen & Duanbridge, 2000)
a) Deck with open ribs; b) Deck with closed ribs

In a stiffed-arch system, the steel plate deck, acting in conjunction with the stiffening girders, will contribute significantly to their flexural rigidity and thus increase their participation in carrying the loads. For a given stiffness desired, the deck contribution to the rigidity of the bridge cross section may permit making the stiffening girders shallower than they would have to be without the deck participation. Thus, a slender appearance may be obtained without sacrificing rigidity.

9.1.5 Through Arch Bridges

In addition to the elements described in previous sections 0regarding open spandrel deck arches, in a through arch bridge part of the deck is hanged from the arch. Therefore, and as illustrated *Figure 9.16* the trough arch bridge's deck is supported by two different types of elements:

- *Suspenders*: support the bridge deck when it is situated below the arch.
- *Spandrel columns*: transmit the load from the bridge deck to the arch rib when the deck is situated above the arch.

Figure 9.16 Trough arch bridges, (Lindbladh, 1996)

Further differentiation can be made according to the deck position, considering a through arch bridge, when the deck is located at the springline of the arch, and a half-through arch when it is located between the springline and the crown of the arch. Special type of Half-Through bridges are the Fly-Bird: a half-through arch bridge with two cantilever half-arches. This bridge type has a large spanning capacity, with most of these structures built in China. Cantilever half-arches are illustrated in *Figure 9.17*.

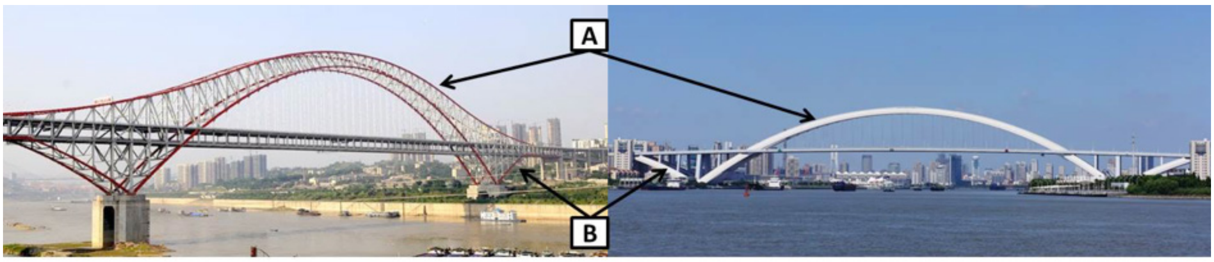


Figure 9.17 Elements in Fly-Bird type of Through Arch
A – Arch ribs, B - Cantilever half-arch ribs, adapted from (Radić, et al., 2016)

Arches of almost all through and half-through bridges are made of steel. Concrete through arch bridges are uncommon due to the presence of members in tension, namely those responsible for hanging the deck from the arch above.

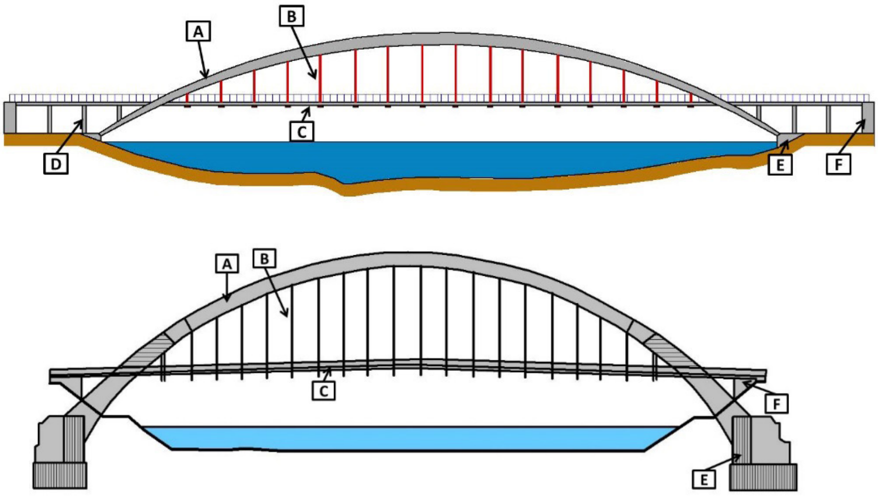


Figure 9.18 Elements in concrete Through Arch
A – Arch ribs, B – Hanger, C – girder/deck, D – pier, E – arch abutment, F – abutment (adapted from (Radić, 2009)

9.1.5.1 Steel through arch

A common taxonomy can be used based on the following element identification for steel bridges:

- Arch ribs (if present consisting of top and bottom rib chords - *Figure 9.20*)
- Rib chord bracing
- Hangers
- Deck girders
  - Longitudinal girder
  - Cross beam
  - Bracing
- Deck slab
- Sway bracing
- Lateral bracing (top and bottom rib chords)
- Lateral bracing (floor system)
- Cantilever half-arch ribs (for fly-bird type only)

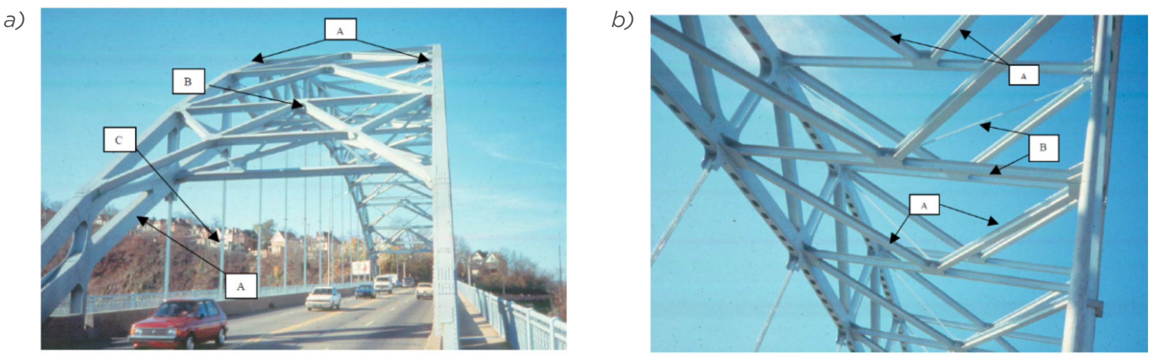


Figure 9.19 Elements in a Through Arch
a) A - Top and bottom rib chords, B - Rib chord bracing, C - Hangers; b) A - Lateral bracing (top and bottom rib chords), B - Sway bracing, adapted from (Hartle, et al., 2002)

9.1.6 Tied Arch Bridges

In an attempt to minimize the horizontal thrust on the abutments, the deck may be used to 'tie' the arch. The tie bar is a structural tensile element used to provide restraint, typically comprising steel rods installed transversely through a bridge, and attached to pattress plates, to provide restraint to the spandrel walls (*McKibbins, et al., 2006*). The stiffening girder distributes concentrated loads from the traffic over a greater length of the arch (*Figure 9.20*). The suspender dampers are damping devices which counteract oscillations/vibrations/fatigue in the suspenders. They are in most cases provided in the form of horizontal connectors between the suspenders (*Lindbladh, 1996*). As variation of the through arch, concrete tied bridges are even rare due to the existence of members (tie members) subject to considerable tension efforts, therefore only steel bridges are considered in this report.

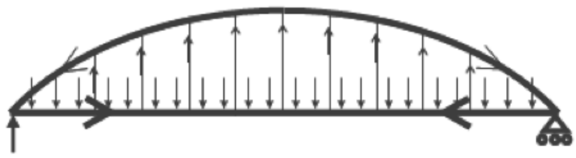


Figure 9.20 Diagram of a tied arch bridge structural behaviour, from (British Constructional Steelwork Association, 2015)

9.1.6.1 Steel tied arch

A tied arch, as shown in *Figure 9.21*, is one where the reactive horizontal forces acting on the arch ribs are supplied by a tension tie at deck level of a through or half-through arch. The tension tie is usually a steel plate girder or a steel box girder and, depending on its stiffness, is capable of carrying a portion of the live loads.

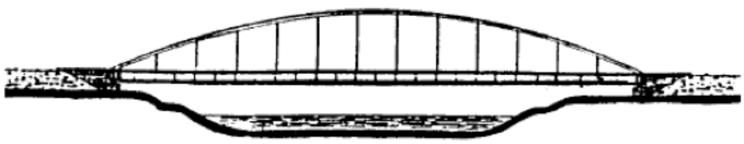


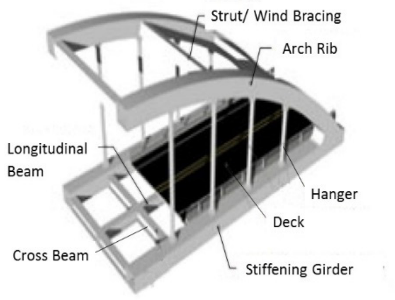
Figure 9.21 Steel tied arch bridge, (Chen et al., 2000)

A taxonomy can also be derived from the element identification for steel bridges (*Figure 9.24*):

- Arch ribs: This compression member is typically a welded box girder erected in sections spliced together. For shorter spans it is not uncommon for the rib to be a curved rolled I-section or for concrete the rib is typically having a rectangular cross section.
- Tie-girder: This element connects the ends of the arch and is either a welded box girder or an I-section such as plate girder. Tied arch bridges can be categorized into two main categories depending on the action in the tie-girder. Structures with a relatively flexible tie-girder are termed a "bowstring" arch as the tie-girder carries mainly axial tension and has a small cross-section, hence the bowstring moniker. Tied arch bridges having a stiffer, larger tie-girder are defined as a Langer girder system. In this system, the tie-girder carries significant flexural demand. This has the benefit of nearly eliminating flexural demands on the arch rib so that it is very nearly in pure compression. This element is singularly most responsible for the tied arch being categorized as fracture critical.
- Hangers: There are three commonly used hanger systems for tied arch bridges. Vertical hangers are referred to as a "Langer" system (*Figure 9.23a*). The "Nielson" system having inclined hangers intersecting once (*Figure 9.23b*), and the "Network" system where hangers intersect at least twice.
- Floor system: Over time the deck or floor systems of tied arches (and trusses also) have evolved into transverse floor beam systems supporting longitudinal stringers, which in turn support a transversely spanning mild reinforced concrete deck slab. And while other deck systems are, of course, possible, the majority of truss and arch bridges use this basic arrangement. Thus, for greater differentiation of elements, it may be considered:
  - Deck girders
    - Longitudinal girder
    - Cross beam
  - Deck Bracing
  - Deck slab

• Wind bracing: transverse beams connecting the arch ribs.

a)



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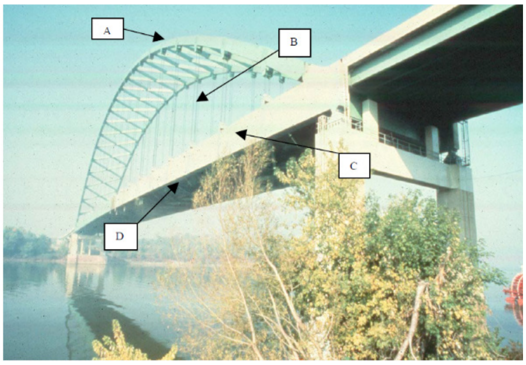
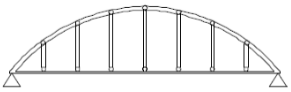


Figure 9.22 Elements in a Tied Arch
a) adapted from (Han, et al., 2016); b) A – Arch ribs, B – Hangers, C – Tie members, D – Deck girders, slab and deck bracing, adapted from (Hartle, et al., 2002)

a)



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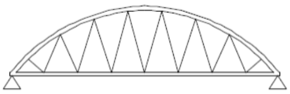


Figure 9.23 W Hanger systems, adapted from (Han, et al., 2016)
a) Langer and b) Nielsen

9.2 Illustrative Example

In this section, the quality control framework is used on the Formigosa bridge, a roadway infrastructure located at km 114 + 284 of EN13 close to Valença, dating back to 1864 (*Infraestruturas de Portugal, 2006*). Its structural typology consists of a structural system based on a single stone masonry arch supported by two abutments (*Figure 9.24a*), which is very common in the European roadway network particularly in Portugal. The bridge has a straight longitudinal profile of about 13 m long, 13.6 m wide and 5.4 m high.

The structure consists of a single segmental arch with a clear span of about 6.1 m and a rise of 1 m supported in two abutments. The arch has regular voussoirs of 0.5 m thick, in granite stone masonry with mortared joints. The bridge has four abutment-walls made of mortared granite masonry with variable length, ranging from 6.0 to 8.1 m long, which extend beyond the arch abutments and spandrel walls, sustaining the embankments. The most obvious vulnerable zones for this type of structures are the arch barrel, the spandrel walls and the abutments.

Two principal inspections were carried in 2007 and 2015 (*Infraestruturas de Portugal, 2007*) (*Infraestruturas de Portugal, 2015*). Several interventions were reported to be carried out in the bridge. The inspection report of 2007 mentions a (previous) bridge widening (unknown date) to its actual width, creating a weakness in the longitudinal joint between both construction phases. In the same report, a bridge intervention was suggested, which design and implementation was completed before the 2015s inspection (*Infraestruturas de Portugal, 2012*).

According to the Portuguese bridge rating system SGOA, the global condition score assigned by the bridge inspectors in the first inspection (in 2007, before rehabilitation) was four (zero to five rating scale, where the zero denotes the best condition and five the worst condition) (*Freire & Amado, 2015*). The assessment from the inspection in 2015 (after rehabilitation), corresponds to global condition rate of one.

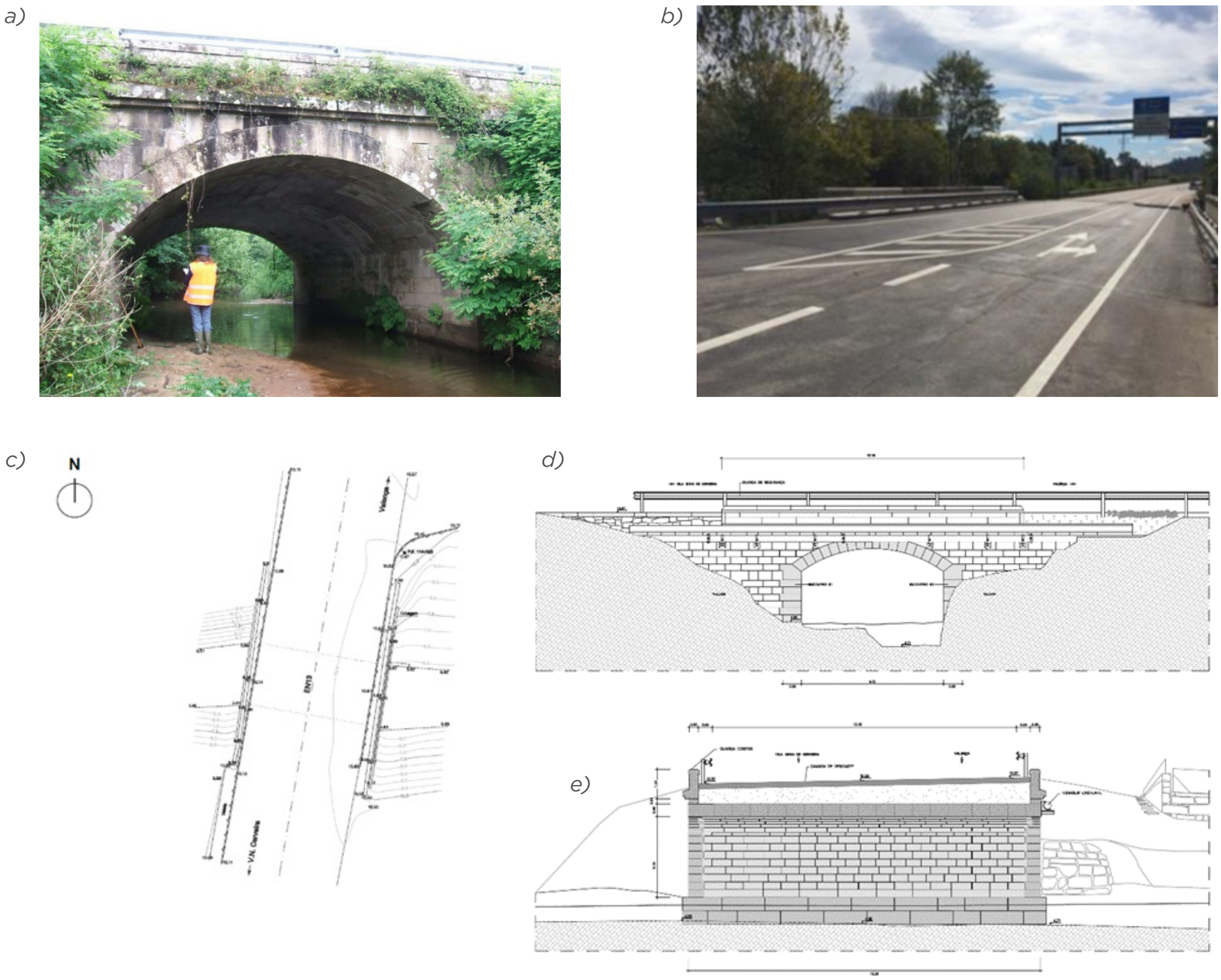


Figure 9.24 Formigosa bridge
a) side view, (Infraestruturas de Portugal, 2006); b) deck view, (Infraestruturas de Portugal, 2015); c) elevation; d) plan view; e) cross section, (Infraestruturas de Portugal, 2012)

9.2.1 Principal inspection of Formigosa bridge carried out in 2007

According to the observations made in 2007, two types of damages can be differentiated: the damages related to environmental and chemical actions and the damages related to physical actions. The first type is present more broadly in the structure, being generally independent of the structural behaviour, involving problems associated with the presence of water, biological pollution and erosion as shown in *Figure 9.25*. It was observed that in general the bridge facing stones have vegetation, moss and lichens, efflorescence and water flowing resulting from insufficient maintenance and inadequacy of the drainage system as shown in *Figure 9.26*. Other more localized damages can also be noted from the report, namely lack of stone blocks and lack of mortar in the joints.

Figure 9.25 Damages – environmental and chemical actions, (Infraestruturas de Portugal, 2007)
a) Water flowing, efflorescence, stalactites and vegetation, on the arch intrados; b) Vegetation and black films (the southwest face on the spandrel wall); c) Moss and lichens (the northwest face on the spandrel wall)

Figure 9.26 Obstruction of the drainage system at the deck level, (Infraestruturas de Portugal, 2007)

The damages related to physical actions are in a form of cracks, crushed stones and deformations resulting from either from settlements (e.g. caused by scour, changing geotechnical properties) or excessive loading. There are noteworthy problems such as: (i) diagonal crack in the spandrel walls of the northern part of the bridge, (ii) settlement of blocks in the intrados of the arch, (iii) joint opening in the arch crown, (iv) block fracture and crushing in the arch intrados, (v) cracking in the abutment (intrados) along the central axis of the bridge, (vi) cracking at the joints on the interface between the mortar and blocks or within the mortar and (vii) joint opening. In *Figure 9.27* examples of the mentioned defects are shown.

Figure 9.27 Damages associated with mechanical actions, (Infraestruturas de Portugal, 2007):
a) Diagonal crack in the spandrel walls (the north-western part of the bridge); b) Diagonal crack in the spandrel walls (the north-eastern part of the bridge); c) Displacement of blocks (the intrados of the arch eastern part); d) Joint opening in the arch crown; e) Block fracture and crushing (the north-eastern part of the arch intrados; f) Cracking at the joints in the abutment zone.

Apart from the previous observations on the principal structural components of the bridge, from which the inspectors identified the evidence of resistance reduction due to previous settlement phenomena and excessive loading, other observations regarding non-structural elements as pavement and parapets are also mentioned in the inspection report, highlighting Safety concerns.

In *Table 9.1* a reinterpretation of the data collected in the 2007`s inspection is proposed, according to the presented WG3 methodology. When no correlation is suggested between Observations and Failure Modes, one can consider that the related observations are not yet affecting the performance of the bridge but only pointing to the existence of a certain Damage Process.

Table 9.1 Protocol for qualitative evaluation of Formigosa bridge (related to the Principal inspection performed in 2007.)

Structure	Group	Element/ material	Observations: Damage	Observations: Closer specifications	Damage process	Vulnerable zone	Failure mode	Primary KPI	Element evaluation	Performance value: Reliability	Performance value: Safety
Stone Masonry Arch Bridge	Structural elements	Arch	Cracks	Joint opening in the arch crown	Changing geotechnical properties/ Overloading	Arch intrados	Longitudinal behaviour/ Arch decompression	(R)	3	4	4
			Displacement	Displacement of blocks in the crown zone of the arch intrados	Changing geotechnical properties/ Overloading	Arch intrados	Longitudinal behaviour/ Arch decompression	(R)	4		
			Rupture	Block fracture	Changing geotechnical properties/ Overloading	Arch intrados	Longitudinal behaviour/ Excessive compression	(R)	4		
			Crushing	Crushed stones	Changing geotechnical properties/ Overloading	Arch intrados	Longitudinal behaviour/ Excessive compression	(R)	4		
			Deteriorated mortar joints	Lack of mortar	Abrasion/ Erosion	Arch intrados	-	(R)	4		
			Vegetation	-	Biological growth	Arch intrados	-	symptom	-		
			Wet spots	-	-	Arch intrados	-	symptom	-		
			Efflorescence	-	-	Arch intrados	-	symptom	-		
		Spandrel walls	Cracks	Diagonal cracks along the spandrel above the arch	Changing geotechnical properties/ Overloading	Spandrel wall – north side	Longitudinal behaviour	(R)	3		
			Deteriorated mortar joints	Lack of mortar	Abrasion/ Erosion	Spandrel walls – all	-	symptom	-		
			Vegetation	-	Biological growth	Spandrel walls – all	-	symptom	-		
			Wet spots	-	-	Spandrel walls – all	-	symptom	-		
		Abutments	Cracks	Cracks at the joints in the intrados zone of the abutment	Changing geotechnical properties	Abutment - north	Excessive compression	(R)	3		
			Rupture	Block fracture	Changing geotechnical properties	Abutment - north	Excessive compression	(R)	3		
			Deteriorated mortar joints	Lack of mortar	Abrasion/ Erosion	Abutments – all	-	symptom	-		
			Vegetation	-	Biological growth	Abutments – all	-	symptom	-		
			Wet spots	-	-	Abutments – all	-	symptom	-		
	Equipment	Parapet	Loss of section	Lack of stone blocks due to a vehicle impact	-	-	Parapet collapse	(S)	4		
			Inadequate geometry	Insufficient high	-	-	-	(S)	4		
		Pavement	Cracks	-	Changing geotechnical properties/ Overloading	-	-	(S)	3		

Regarding experience and the reliability index (β) for this type of bridges, it is assumed for the virgin state that β equals 5.2 for a reference period of one year. Although the bridge was not designed with the notion of design working life, it seems appropriate to assume at least 100 years, given its longevity. Considering the inspection performed in 2007 and relaying on expert judgement, a qualitative assessment of resistance reduction based on observed damages is estimated at approx. 20%. The influence of resistance reduction on reliability is given in *Figure 9.28*. This chart was plotted for the present case study adopting a ratio of the live/dead load of 0.05 (see *Section 12.2*). For the estimated reduction, the related KPI(R)=4. Considering the urgency of intervention proposed in *Table 12.1*, a reassessment and possible intervention should be performed shortly after an inspection, which corresponds to the action that had been pointed out in the bridge inspection report.

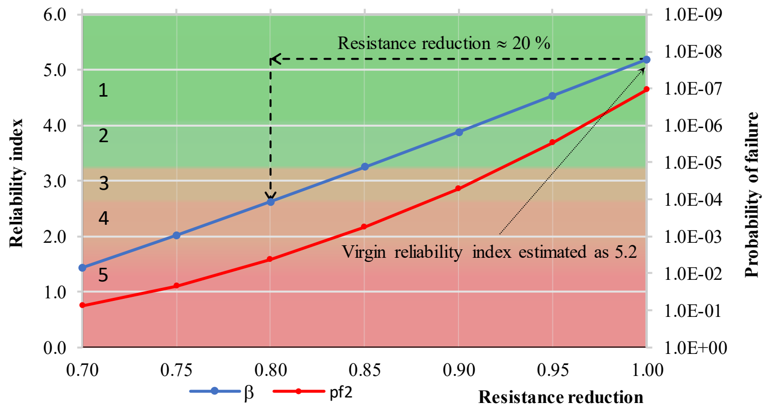
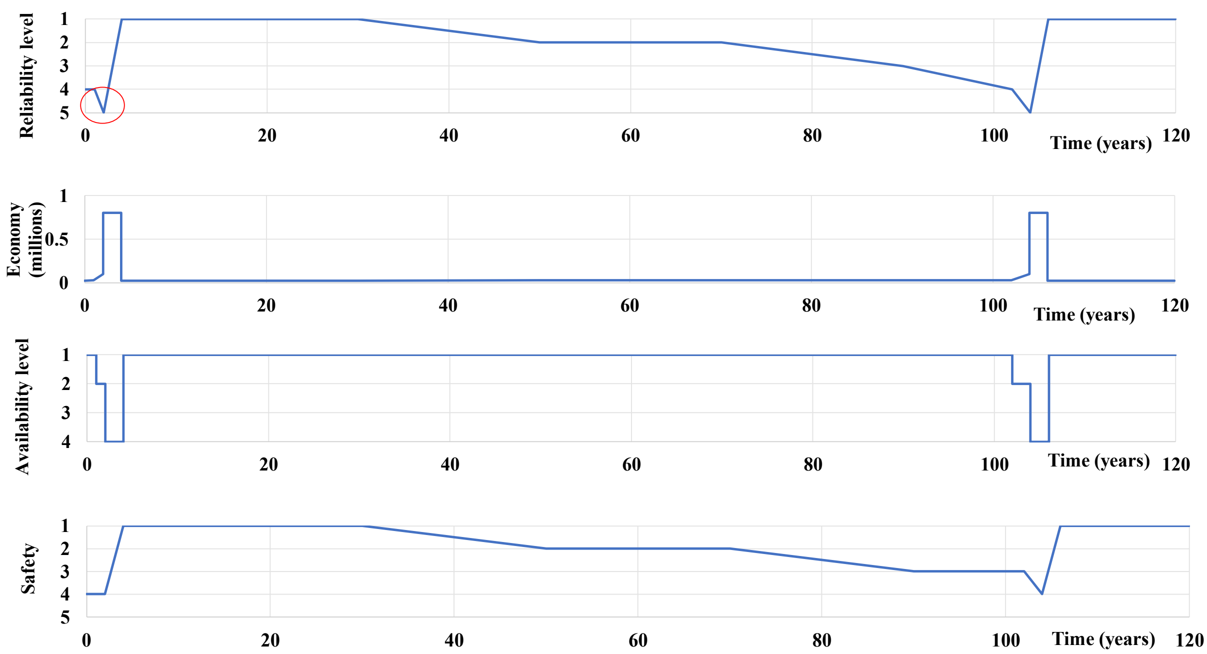


Figure 9.28 Influence of a resistance reduction on reliability index & probability of failure

Regarding the KPI of Safety (S), the observations regarding insufficient height of the parapets (inadequate geometry) justify the grade 4 (*Table 12.3*), as it is likely that a person could get injured and the intervention shall be performed shortly after inspection. The damages observed in the arch intrados, considered as a vulnerable zone, point to possible failures modes in the longitudinal plane of the arch and arch decompression (in addition to local excessive compression). Remaining service life was not directly accessed, but this entity is correlated with the rating system used for the inspections (SGOA). The assigned rating of four (the SGOA scale) is defined by the BMS as correlated with a qualitatively estimated remaining service life (understood as a threshold for serviceability) of two years. Here the two maintenance scenarios were compared (*Figure 9.29*), similarly to the example in *Section 8.5*. The reference maintenance scenario i.e. the "do-nothing" scenario (see *Table 8.4*) comprise full bridge repair at year two and 104 (i.e. when the bridge reaches reliability level five). The preventative scenario implies immediate rehabilitation intervention on a bridge. The maintenance costs were roughly estimated (Economy) and the Availability is established on the network level in a scale from one to four.

a) Reference scenario



b) Preventative scenario

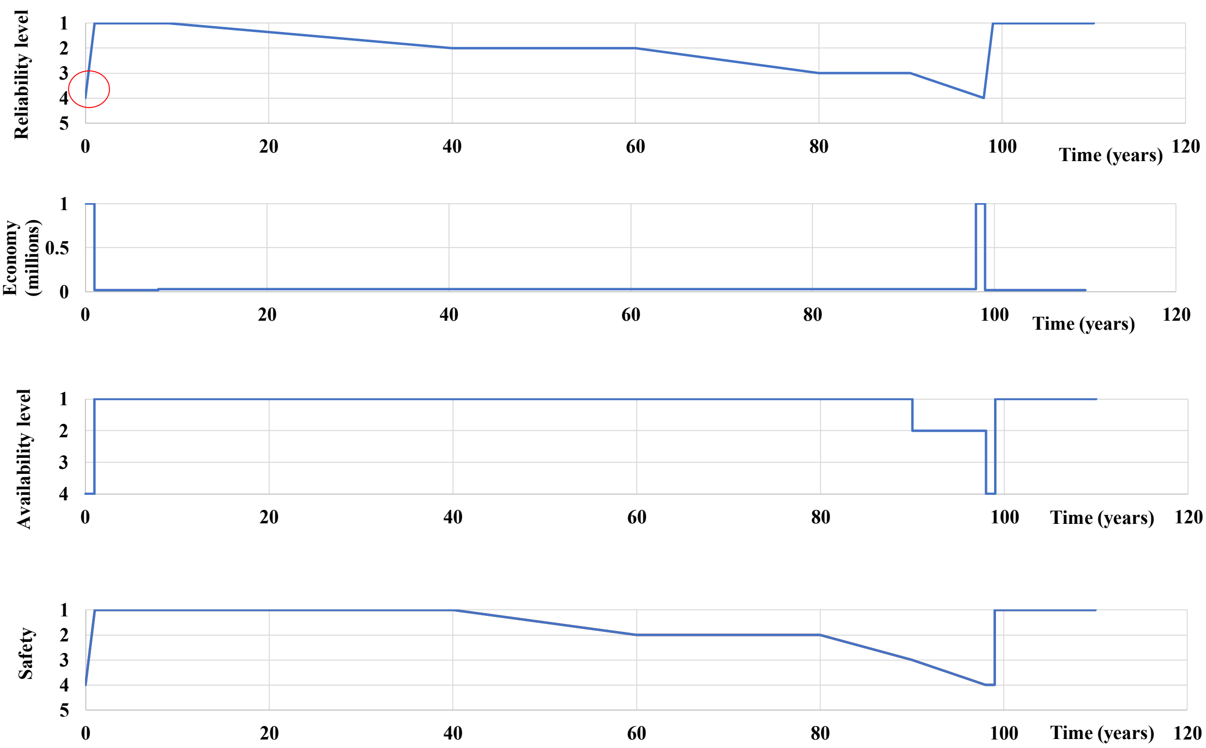


Figure 9.29 Comparison of KPI's for two maintenance scenarios for the arch bridge example

After the normalization of the KPIs, the net present KPI in a form of the spider diagram is presented in *Figure 9.30*.

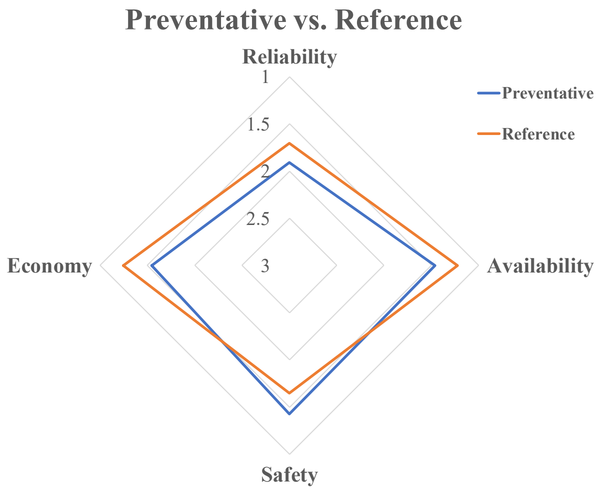


Figure 9.30 Comparison of the net present KPI for the two maintenance scenarios

9.2.2 Rehabilitation intervention of Formigosa bridge carried out in 2012/13

The performed intervention on the bridge comprised casting of a 9.0m long concrete slab over a thin layer of extruded polystyrene placed over the infill. At the both bridge ends, two rows of micropiles support the concrete slab.

The arch and abutments were strengthened by transverse ties and all facing zones of the masonry structure were consolidated by injecting and repointing the masonry joints with appropriate mortar. *Figure 9.31* shows a longitudinal cross section of the strengthening solution implemented for the Formigosa bridge (*Infraestruturas de Portugal, 2012*).

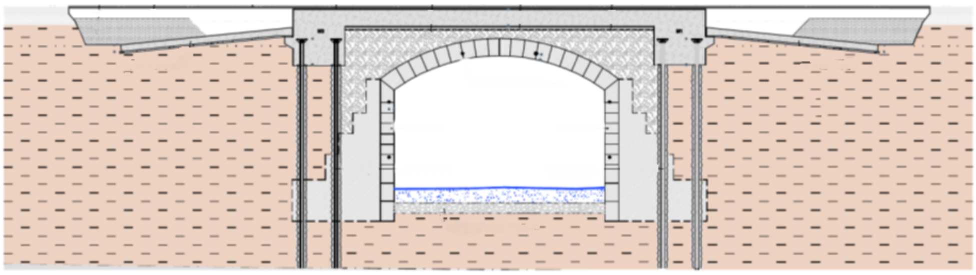


Figure 9.31 Strengthening solution of Formigosa bridge. Longitudinal cross section (Infraestruturas de Portugal, 2012).

A new principal inspection was carried out in 2015. According to the observations made the bridge is in good condition with some of the old damages, as joint opening and block displacement, still visible but stabilized and consolidated as result of the bridge intervention. Minor observations were pointed out as lack of material in the embankment due to erosive action of the river flow. The corresponding KPI of Reliability is one. A general overview of the bridge after the rehabilitation can be observed in *Figure 9.32*.

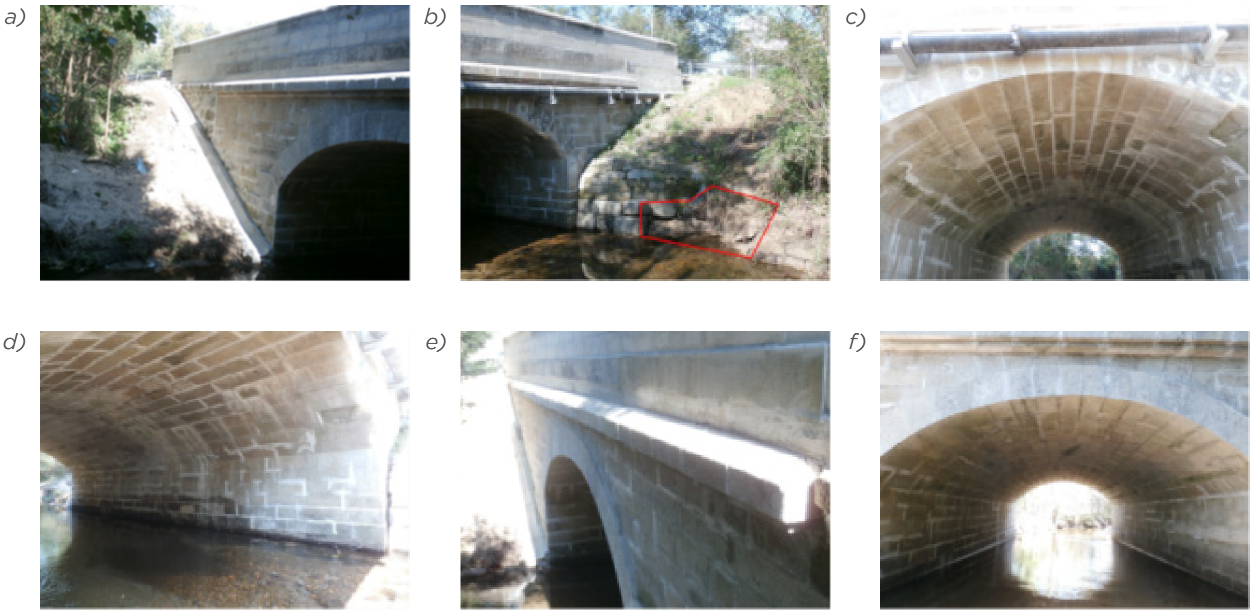


Figure 9.32 The bridge elements/sections after the rehabilitation (Infraestruturas de Portugal, 2015): a) spandrel walls (the north-western part of the bridge); b) spandrel walls (the north-eastern part of the bridge) and lack of material in the embankment; c) intrados of the arch view from east; d) north-eastern part of the arch intrados and abutment; e) southwest view of the bridge; f) west view of the arch intrados.

10. Application of the Quality Control framework to sudden events

10.1 Sudden events in bridge management practice – flooding and local scour

The sudden events acting on bridges are regarded as observable, non-interceptable processes i.e. those which do not leave ample time for an adequate mitigation action. Based on their genesis, there are two groups of events, anthropogenic events (e.g. accidents, explosions) and natural hazards (e.g. gravitational, earthquake). It is necessary to account for the dynamics and uncertainties of sudden events for structuring adequate quality control (QC) plans. The main challenges, with respect to each event group, that need to be addressed are: return period, magnitude, the area/point of impact on a bridge structure, induced damage (e.g. a failure mode) and related total consequences.

In focus of this report is the flooding hazard with related local scour at substructures, since it is the most frequent and the most widespread culprit of inadequate bridge performance in the world, as discussed in (*Faber, 2007*), (*Imhof, 2004*) and (*Sullivan, 2005*). The local scour represents a hydraulic erosion process that entails lowering of a riverbed by flowing water at bridge substructures and their foundations (*Figure 10.1*). Eventually, this may lead to bridge failures, and the main concern is that local scour in most cases can be regarded as a sudden process, which does not leave ample time for an adequate mitigation action.

a) *b)*

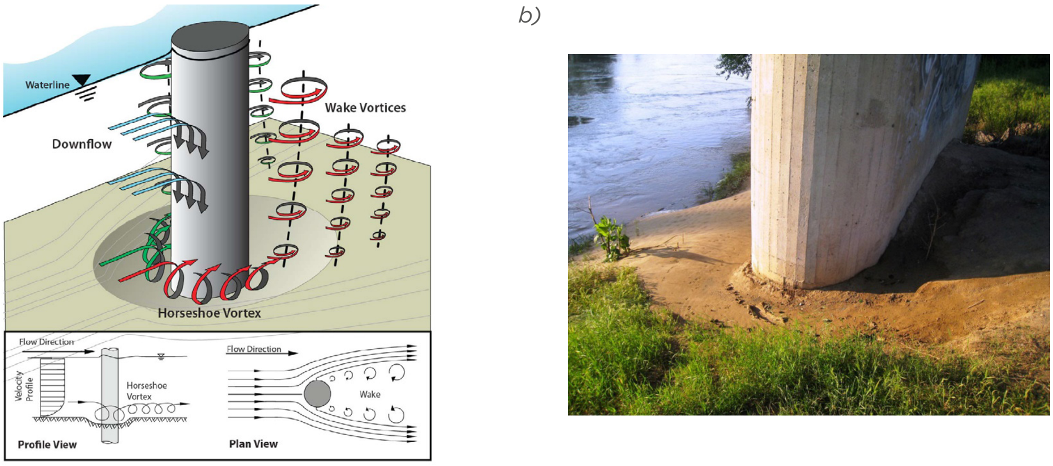


Figure 10.1 Local scour at a pier
a) Schematics of the vortices at a cylindrical pier (FDOT, 2010);
b) On-site evidence of the local scour following a flood in Serbia 2014

In the state-of-the-art BM practice, the natural hazards are tackled with risk-based approaches, e.g. (*FEMA, 2007*), (*Pearson, et al., 2002*) and (*ASTRA, 2014*). Here, the magnitude of a hazard and the related structure exposure are accounted for, but the resistance of a structure to a failure in the event is rarely considered.

In the most cases, the direct costs of a failure, e.g. loss of life and limb and repair & reconstructions are assessed, while an adequate evaluation of the indirect consequences of bridge failures, e.g. travel time loss due to hazards, is rarely performed. The methodologies which account for natural hazard impact on transportation infrastructure still await to be dully incorporated in the modern Bridge Management Systems (BMS).

10.2 The scope

The current approaches in European BM practice related to mitigating the threat of oncoming flooding events are not comprehensive, and in the most cases solely rely on visual inspections (*Section 10.4.1*). One of the aims of this COST action is to address this issue and suggest a practical solution, which is in accordance with the approach adopted within the COST for key performance indicators (KPIs).

More precisely, the solution must be based on a modest set of bridge performance indicators (PIs), observations from inspections and other relevant bridge data, that can jointly relate to the KPIs of: Reliability, Availability, Safety, Economy and Environment.

The idea of the presented approach is to give a minimum set of necessary information for evaluation of Reliability KPI, which is comprehensive and compatible with existing approaches and future BMS-ready qualitative/quantitative frameworks. The evaluation of the KPI of Safety, within the adopted COST approach, is not related to the event of a bridge failure in a flooding event, thus is not elaborated. The KPIs of Availability and Economy are assessed after an adequate measure for a bridge is chosen as previously discussed and presented in *Sections 3.4 and 8.5*, respectively. Due to the insufficient data/models and practical methodologies to account for the relevant impacts of a bridge failure to the community/ecology, the KPI of Environment is not treated in here.

The QC framework (*Figure 7.1*) is envisioned to be applied on the three types of roadway bridges: girder, frame and masonry arch bridges. It was necessary to perform a review of relevant literature on the BM practice related to the flooding impact on bridges (*Section 10.3*). Here, the accent was to look for relevant PIs and KPIs, as well as the information on related performance thresholds and required quality levels for inspections and maintenance actions. The aim was to structure the most relevant information from the survey, literature, practice and research (see *Section 10.4.2*), to be in accordance with the qualitative approach adopted within WG3. Here, it was of the utmost importance to address the resistance of diverse bridge types to local scour, which is not thoroughly considered in the current BM practice.

The vulnerability assessment, a risk-based approach, is suggested as a basis for the framework, as it entangles the most comprehensive information on the threat of a bridge failure in a flooding event i.e. the probability of a bridge failure and related consequences (*Section 10.4.3*). In the light of the chosen WG3 methodology, a semi-quantitative approach to assess the KPI of Reliability is presented (*Section 10.5*). This KPI is assessed by using the relevant data on a bridge exposure to local scour in a flooding event and a bridge resistance, accounting for a bridge type and failure modes (*Sections 10.4.4 and 10.4.5*).

In *Section 10.6*, the background for elaboration of adequate QC plans in respect to flooding hazard and related local scour is provided and the illustrative example is given in *Section 10.7*.

10.3 Literature review on the BM practice in the case of flooding events

Scour assessments broadly speaking are undertaken in a variety of manners, by various national authorities and infrastructure owners. The issue with scour assessment lies with the subjective nature of the damage caused by scour, which is highly dependent on river type, flow conditions, soil type and the nature and extent of the scour problem. Assessment procedures can include visual assessments based on rating systems, mechanical or electrical sensor-based equipment to monitor the evolution of scour on instrumented structures or more modern structural health monitoring approaches. The primary issue with the entire subject area lies with the question, how much scour is too much scour?

In reality, scour is a qualitative issue as the term “magnitude of a scour cavity” is a meaningless term and is completely dependent on each system to which it pertains. For example, a 1.0m deep scour cavity at a central pier of a multi-span concrete bridge may not be particularly detrimental, however the same on a single arch abutment structure could be fatal. For this reason, there has been significant problems in the industry attempting to develop a single, all-encompassing assessment procedure and related monitoring framework and to date, this has not been effectively achieved. Moreover, the advent of a modern technology coupled with some high-profile bridge failures in recent years has sparked a renewed interest in this area both on a research and practical level.

10.3.1 Girder and frame bridges

The triggers for maintenance/mitigation actions for the case of a flooding hazard are usually related to a qualitative condition score for bridge substructures. In general, this score is given based on a visual inspection (traces of erosion, settlements, rotations and cracking) and if possible, complemented with information from in-situ measurements, monitoring and indirect evaluation of scour depths. There is an absence of comprehensive guidelines on this matter in the European countries involved in the COST action (*Section 10.4.1*).

The flooding hazard impact on bridges has been extensively elaborated in the USA for the last 25 years. There are several approaches, qualitative and quantitative that are applied by departments of transportation in the U.S. In the *Table 10.1*, *Table 10.2* and*Table 10.3*, a brief review of the three representative methodologies is given, with the accent on the relevant data on hazard, bridge structure and possible consequences of a bridge failure.

The approach generally used in the USA is given by the Federal Highway Administration (FHWA) and rests on the qualitative scores of items from the National Bridge Inventory (NBI) database. Based on the inspection data, a bridge receives a score for a specific NBI item no. 113, which in the terminology of COST can be regarded as a PI. Based on this score, the decision is made on an adequate action – further evaluations & inspections, instalment of protective structures, traffic restriction or a bridge closure.

Table 10.1 FHWA methodology for bridges affected by scour

Data type	Performance Indicator: NBI Item 113 - Scour Critical Bridges
Hazard	Visual inspection (traces of scour on site), overtopping history, waterway adequacy
Structure	Indirect evaluation of scour depth; Countermeasures; Foundation type/depth;
Consequences	Stability endangered = traffic restriction / bridge closure

The approach distinguishes bridges with shallow and deep foundations. The procedures for the indirect evaluation of the scour depth and adequate countermeasures can be applied. However, the related QC plans are not explicit when it comes to different bridge types. The approach solely rests on the engineering judgement to assess the stability of a bridge. Broadly speaking, the addressed KPI in this approach are related to Reliability (safety and serviceability).

The most comprehensive qualitative methodology for assessment of scour at bridges has been given in (*NYSDOT, 2003*). It accounts for relevant data to assess bridge vulnerability to failure (Table 10.2). The manual suggests that the following factors should be considered: redundancy of the superstructure, simple span/continuous spans, bridge type, span length, support conditions, abutment/piers type & geometry. Besides the general discussion on bridge types and their classification in the two risk categories, the specific details on how the latter information affects the possible failures are not given.

Table 10.2 NYSDOT methodology for bridges affected by scour

Data type	Performance Indicator: Hydraulic vulnerability score
Hazard	Channel hydraulic properties; Foundation alignment; Historical scour
Structure	Redundant bridge types; Foundation type and soil type
Consequences	Three types of failure; Traffic volume

Based on the vulnerability score, an adequate QC plan for a bridge is selected. There are five available programs: safety program watch, safety program alert, capital program action, inspection program action and no action. Corrective/mitigation action, appropriate safety action (i.e. a flood-watch programme) or enhanced inspection are the possibilities which are related to these programs.

The HYRISK methodology (*Pearson, et al., 2002*) is used in the approach for management of bridges with unknown foundations (*Stein, et al., 2006*). Here, the estimation of relative annual risks of a bridge damage or failure due to scour, is performed based on the FHWA guidelines. The risk is estimated as product of an annual probability (in this case it is the rate) of scour failures and the associated economic consequences, by using pertinent items from the NBI database.

The main advantage of the methodology is its risk-based approach. It takes into account, up to a point, the bridge and foundation type, but its main drawback is that it „looks back" to the rate of previous failures in order to assess the probability of a bridge failure. Recently, a novel approach was presented in (*Khelifa, et al., 2013*), which is an add-on to the HYRISK methodology. It takes into account an important parameter for scour evaluation - the soil type.

Table 10.3 Methodology for bridges with unknown foundations affected by scour

Data	Performance Indicator: Risk of failure
Hazard related	NBI Items
Structure related	Span and foundation type; Foundation soil type
Consequences	Road category; Traffic volume and composition

In this quantitative approach, the adequate QC plans are based on the minimum prescribed performance levels i.e. annual probabilities of failure, governed by qualitative NBI data, frequency of overtopping and observed failures. If these are not met for a bridge, the immediate foundation survey is warranted. Otherwise, the choice comes to automated scour monitoring / instalment of protective structures, but first there is a need to evaluate monetized direct and indirect consequences of a failure.

10.3.2 Masonry arch bridges

Masonry arch bridges can be considered more threatened with flooding events than RC bridges, due to the way their foundation system is constructed. The *Figure 10.2* illustrates the foundation construction technique that was used in the past centuries to build bridges. The knowledge of the construction process of foundational systems for masonry arch bridges is a key aspect when dealing with the analysis of flooding impact. Usually, construction practice was based on the use of wooden cofferdams comprising a curtain of timber piles which were driven in the rived-bed and used as a soil containment system. Once one or two lines of cofferdams were positioned, the soil was removed from the bounded area and then a cementitious mixture was poured with tuff and large stones to create the base of a foundational nut and the basement of a pier. One of the main drawbacks of this technique was the reduced depth of the overall foundational system, thus resulting in an increased vulnerability to flooding events and related local scour depths reaching below the depth of the installation of the wooden piles.

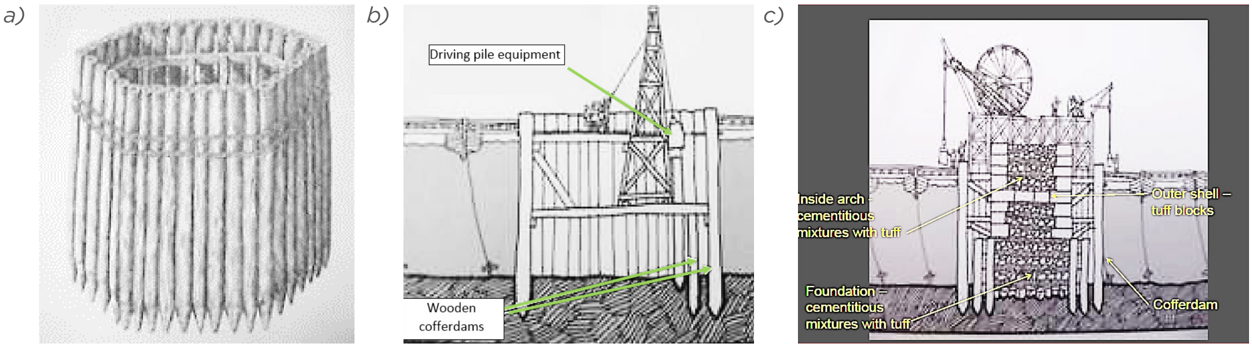


Figure 10.2 The main masonry arch bridge components and foundation construction process used in the past, (Madrid Engineering Group, INC., 2008) | a) A wooden cofferdam b) Positioning of the cofferdams; c) Foundation construction

In some cases, wooden piles were also used to increase mechanical properties of the soil. At these sites, there were observations of pier settlements after a flooding event, due to washing out the filling material between the piles (*Figure 10.3*).

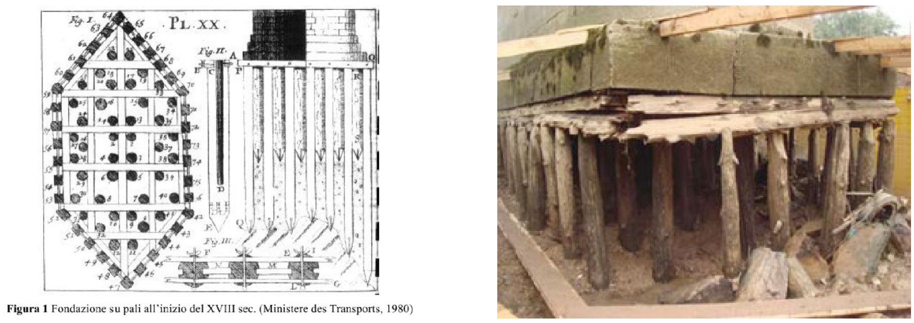
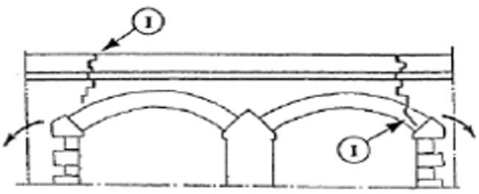
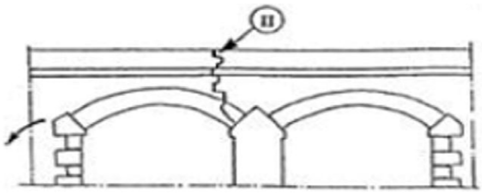


Figure 10.3 Wooden piles for increasing bearing capacity of soil under foundational systems, (Brencich, 2005)

Infraestruturas de Portugal S.A. (*Amado, 2013*), gives an extensive guideline for the definition of the procedures to be followed by technicians, in the planning of inspections for bridges with foundations placed in river beds, and at the same time, the identification of set of damages that can be caused by settlements induced by scour phenomena. *Figure 10.4* shows, as example, one possible scour-induced damage detectable regarding masonry bridges.



Crack pattern I:
diagonal cracking between the abutments and the arches due to settlement-induced rotations of the abutments.

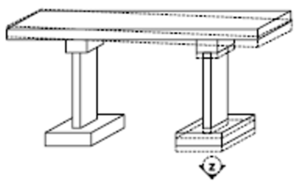


Crack pattern II:
diagonal cracking developed in correspondence of intermediate piers and involving spandrel walls, backfill and arches due to settlement-induced rotations of the abutments.

Figure 10.4 Types of damage detectable on masonry arch bridges caused by scour actions, (SÉTRA, 1996)

In Italy, Rete Ferroviaria Italiana S.p.A., the owner of the national railway infrastructural network, has developed a guideline for visual inspections of bridges, also considering damages related to foundation systems. In *Figure 10.5*, a part of the guidelines related to assessment of the damages (differential settlement and erosion) of the foundation system is presented.

Attention is directed in the evaluation of relative movements between adjacent piers, base rotations, differential settlements, and evidence of scour. Each damage type should be assessed quantitatively, by estimating its intensity and extension (see K2 and K3 in *Figure 10.5*). The guideline provides a brief description on how to assess these and which type of equipment an inspector has to use.

Damages of the substructures			
S3	**Differential settlement**		
Definition: vertical differential settlement between the two foundational systems supporting the same deck. Relative displacement measurement. **How to detect**: visual inspection and manual measurement. Since the damage sometimes is difficult to be detected, it is also possible to check soil conditions close to foundations, and the joints of the related substructures in the deck.			
Description: differential settlement between elements supporting the same deck. For each column, it is necessary to refer to both the adjacent columns for assessing the largest value.			
B	**3**	**How to quantify**: it has to be detected by measuring and checking relative displacements. A photo of the damage has to be taken.	
K_2	**Intensity criterion**: Measurement of the differential settlement	K_2 = 0.5	< 2 cm
		K_2 = 1.0	from 2 to 5 cm
		K_2 = 1.5	from 5 to 10 cm
		K_2 = 2.0	> 10 cm
K_3	**Extension**: Location of the damage	K_3 = 1.0	It is not necessary to assess the extension of the defect. K_3 = 1 if the damage is present.

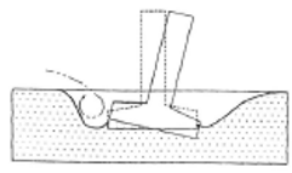
Damages of the substructures			
S5	**Foundation scour**		
Definition: foundation scour **How to detect**: visual inspection of the soil close to foundations.			
Description: soil settlement of the soil at the foundation. The substructures are an obstacle to flowing water and the damage occurs due to the related soil erosion in a flooding event.			
B	**4**	**How to quantify**: the evaluation of the scour area can be performed with a visual inspection but it is necessary to have information of foundation geometry. The damage is not to be reported when foundational systems are by design uncovered by soil (see original projects). The extension of this damage can be measured with a meter.	
K_2	**Intensity criterion**: Measurement of the differential settlement	K2 = 0.6	< 2 cm
		K2 = 1.0	from 2 to 5 cm
		K2 = 1.6	from 5 to 10 cm
		K2 = 2.0	> 10 cm
K_3	**Extension**: Location of the damage	K3 = 2.0	It is not necessary to assess the extension of the damage. K3 = 1 if the damage is present.

Figure 10.5 Assessment of defects/damages of a foundation system - differential settlements and erosion, adapted from (Rete Ferroviaria Italiana S.p.A. , 2014)

10.4 Influence of observations and PIs on the KPIs

10.4.1 Review of the PIs collected within COST TU1406 survey

The relevant information on PIs, their thresholds and related performance goals have been collected in the WG1 survey (*Strauss & Mandić Ivanković, 2016*). Although hazards were not the main topic, almost every country provided information on the appraisal of a flooding impact, namely scour & erosion at bridge substructures and embankments. However, it is a fact that when it comes to application of procedures & actions for timely mitigation of flooding related consequences, not much concrete information can be found in the relevant national documents - bridge inspection manuals & guidelines (*Figure 10.6*).

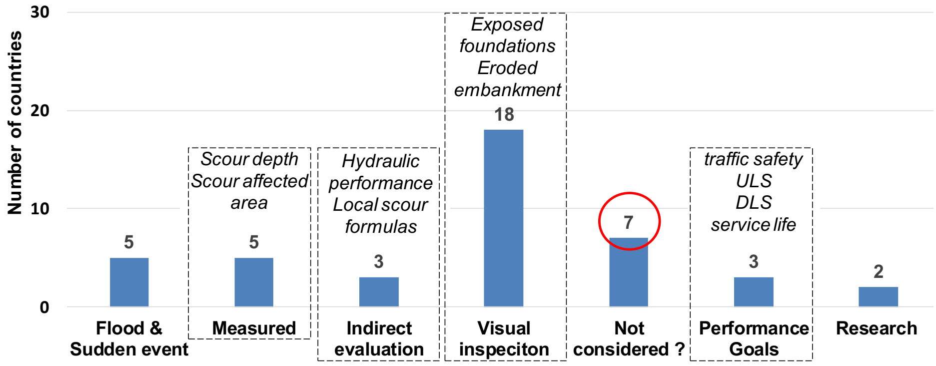


Figure 10.6 Flood & Scour in European BM guidelines (Tanasic & Hajdin, 2017a)

Most of the countries reported that they rely on visual inspections, some additionally perform measurements of scour depth/area, while a few accounts for hydraulic calculations at bridge sites. Only one country reported the use of a local scour evaluation formula, while seven countries have not reported that flood/scour is specifically considered in their national documents.

It can be concluded that there is no comprehensive (and in some cases no systematic) evaluation of a threat to a bridge from a flooding hazard. The decision making and related performance goals mainly rely on information from visual inspections on a local scour at bridge substructures, but it does not consider a bridge resistance nor grade possible cases of inadequate bridge performance i.e. resulting consequences.

10.4.2 The most relevant PIs, observations and other data for flooding/scour hazard

The results of the review of the WG1 survey are structured in lists and presented in *Figure 10.7*. Here, all the terms reported (at least by one country) are in grey colour, while those not reported, and the additional relevant data/ parameters from the practice and research, are given in white. This additional data in fact represent missing links between practice applied PIs and the KPIs. In the following, the most relevant information for a comprehensive assessment of a threat from a bridge failure due to scour is discussed.

		Performance indicators			
Structure	**Elements**	**Observation**	**Other relevant data**	**Damage process**	**KPI**
All bridge types and materials	Foundations	Scour depth	Bridge geometry & dead load	Flood/Scour	Reliability
	Embankments	Scour affected area	Foundation type	Erosion	Availability
	Protective structures	Exposed foundation	River bed properties		Safety
	Substructure	Eroded embankment	Foundation soil properties		Economy
	Bearings/Joints	Hydraulic performance	Flood magnitude		Environment
	Superstructure	Damage location and severity	Debris/ice potential		
		Condition state	Traffic data		

Figure 10.7 Review of the terms reported in WG1 survey & Other relevant data for flooding hazard and scour

All bridge types regardless of age, static system or materials may be affected by a flooding hazard, where foundations of bridge substructures and bridge embankments may be exposed to a process of scouring. There is removal of supporting soil at foundations, while the soil and the bridge structure jointly resist this adverse action until the bridge fails under its own dead load (*Tanasic, 2015*). This resistance of a bridge is not adequately accounted in the current BM practice, which is resulting in an overestimation of the factual threat of a failure.

The reported observations in *Figure 10.7*: exposed foundations, eroded embankment and scour affected area are not by themselves an effective PI. They are assessed in a visual inspection, and only have significant value if noticed timely, as they can relate to a potential future threat i.e. a failure scenario. The scour depth is an observation/indicator that may be directly measured, monitored or indirectly evaluated by empirical formulas. The choice of the right approach is based on available resources (e.g. budget, available data and equipment, etc.) and engineering judgement. Commonly in practice, visual inspection is the trigger either for monitoring/measuring of scour depth or instalment of protective structures, but this cannot be regarded as a reliable method since scour can be a sudden process. Also, indirect evaluations of scour depth are applied to complement a decision-making process, but this require additional effort and resources. It should be noted that these empirically estimated scour depths tend overestimate observed depths on site, which may be misleading in the quality control procedure, thus should be used with caution.

Although the information on the scour depth entangles the data on a flooding magnitude, soil properties and substructure geometry, it must be coupled with relevant data on bridge type/properties (e.g. condition of specific elements) and demand (e.g. traffic volume) for a reliable decision making. Indirect observations, which can point out a problem with local scour, are substructure settlement/rotation and resulting localized damage (e.g. cracks) at joints, bearings and other bridge elements. These indicate that a failure has already occurred, which requires immediate attention i.e. adequate repairs. For BM practice, these types of observations provide useful information for inspections of similar, yet undamaged bridges, but are not within the scope of WG3. However, any type of damage at structural elements, which is not a result of a foundation displacement/rotation, are of interest as it may decrease bridge resistance to an oncoming flooding event. Here, the importance of two parameters: damage location and its severity (e.g. area/depth affected) must be recognized for diverse bridge types and their elements to conduct a comprehensive analysis on possible bridge failure modes (*Section 10.4.4*).

The most relevant data, which are necessary for the evaluation of a threat of a bridge failure due to local scour, may be structured in three groups:

- Exposure to a local scour process at substructures
    - Flood magnitude & duration (i.e. a hydrograph)
    - Water channel geometry & properties
    - Debris/ice potential
    - Soil erodibility
    - Piers & abutments type/geometry, position and alignment in respect to a water flow
- Resistance to failure modes induced by local scour at substructures
    - Soil cover
    - Properties of a soil at foundations (geotechnics)
    - Type/detailing of substructures and superstructure
    - Location & severity of damage on bridge elements in vulnerable zones
- Consequences related to failure modes
    - Loss of life & limb
    - Costs of repairs or replacement
    - Down time
    - Network & traffic data to include indirect costs of failure: additional vehicle operating costs, accident costs and loss of travel time

It is evident that a risk-based methodology is a feasible solution to adequately consider a flooding hazard in BM. However, a major drawback for a practice-ready approach are diverse types & levels of uncertainties related to this data. Most of these data are not usually stored in bridge databases but exist up to an extent in a bridge design documentation and other databases (i.e. hazard and traffic information) or can be inferred based on experience/one-time inspection. Thus, the emphasis is on a simple, yet comprehensive approach, which relies on existing information and inexpensive, but reliable inspection procedures. Since the chosen approach in the COST action does not account directly for risk nor vulnerability, connections should be established between the latter relevant data groups and the KPIs.

10.4.3 The concept of vulnerability and the relationship to the KPIs

The application of robust hydrometeorology models and big data for a precipitation of rainfalls and estimating the magnitudes of oncoming flooding hazards that are likely to affect a bridge, is overwhelming for BM practice. It is far more convenient to assume a flooding magnitude (e.g. a 100-year flood), a related scenario, e.g. local scour at pier, and apply a vulnerability approach (*Tanasic & Hajdin, 2017*). From a BM perspective, the vulnerability may be defined as the expected total consequences of inadequate bridge performance caused by a hazard (e.g. flooding) of a certain magnitude (*Birdsall, 2009*). Here, the formula which defines this relationship is given as:

$$V_n^s = P_n^s \cdot \left(DC_n + IC_n \right) \tag{13}$$

where: V_n^s = the vulnerability of a bridge with respect to a flooding event of a specific magnitude s and a chosen failure mode n; P_n^s the conditional probability of a bridge failure in the chosen failure mode n, with the respect to a flooding event of a specific magnitude s; DC_n = direct consequences with respect to the chosen bridge failure mode n; and IC_n = indirect, mostly traffic related consequences with respect to the chosen bridge failure mode n.

The vulnerability is a comprehensive indicator for decision-making, in respect to the threat of failure due to a hazard, that points out and ranks the bridges that need to be investigated in more detail. However, it is not adopted within the COST action as a KPI. Nevertheless, the failure mode analysis is required for evaluation of the probability of a bridge failure, which is in a direct relationship with the KPI of Reliability. By definitions within WG3, the KPI of Safety does not relate to the direct consequences of a bridge failure. The KPIs of Availability and Economy are not directly related to the hazard, but to preventative maintenance scenarios. The indirect consequences of a failure are not accounted for in the evaluation of the KPIs within the COST.

10.4.4 Girder and Frame bridges - bridge failure modes and vulnerable zones

The failure modes (FMs), which are the essential ingredient for the assessment of Reliability, imply any traffic interruption i.e. accounts both for a partial damage or a collapse. Here, the influence of specific bridge elements on the bridge resistance to FMs needs to be addressed. The FMs are usually discussed in the light of observed damage, e.g. (*May, et al., 2002*), (*Briaud, et al., 2010*), (*Ettema, et al., 2011*), but the differentiation between diverse bridge types and specific damages, and an explanation how the related failure occurs, have not been provided in detail. There is a need for a simple, yet comprehensive FM analysis, and here a theory of plasticity and analysis of kinematic mechanisms can be a convenient approach.

The resistance of a bridge to a flooding event is primarily dependent on analysed scenarios (e.g. a local scour at a middle pier). For each scenario, the possible FMs are governed by the combined resistance of a soil-bridge system. Here, the height of soil cover at an affected substructure and soil geotechnical properties have the leading role, due to large safety factors in the design. Secondary, but not unimportant is the engaged superstructure resistance governed by the type/properties of bearings/joints and main girder (*Tanasic, 2015*). The main FM types and the key elements of girder/frame bridges, which reflect on how much of bridge resistance can be engaged, are listed in *Table 10.4*, while the related examples are given in *Figure 10.8*.

Evidently, a sizable portion of the bridge resistance may stem from the height of the soil cover at an affected substructure, which in most cases is the parameter with the highest level of uncertainty. As far as a structure is concerned, for the suggested FM analysis, of the outmost importance are the plastic strength of elements i.e. bearings & joints which govern static indeterminacy. Here, the conceptual weaknesses of the analysed bridge should be accounted for (see *Section 7.2.1*).

Table 10.4 Key elements for bridge resistance to local scour and related types of failure modes (FMs)

Bridge element	Attention	Resistance	FM type
Affected substructure foundation	Inadequate detailing/condition	Structure governed	1
Bearing/joint at the top of the affected substructure	Inadequate detailing/condition	Foundation soil & none/low superstructure resistance	2
Bearings/joints at the top of other substructures	Inadequate detailing/condition; Horizontal displacement in plane: free or restrained	Combined soil-bridge resistance	2 & 3
Main girder	Detailing (e.g. splicing locations of reinforcement)	Combined soil-bridge resistance	3
		Failure safe	4

The failure mode type 1 (FM1) is the most dangerous, since it may cause progressive collapse (*Figure 10.8a*), provided that the design of the main girder is not failure/collapse safe to a damage to one of the supports (e.g. FM4). The FM2 may occur if the top of the pier of an affected foundation is not restrained to a movement in the horizontal plane, by design or due to a poor condition state of bearings.

This FM is common for multiple-single span bridges (*Section 8.4*), e.g. as presented in *Figure 10.8b*. The FM3 is “the most desired” case, since the requirement for failure is that the foundation soil and the structure need to deplete their joint resistance to a partial loss of the support at the affected substructure foundation.

a)

b)

c)

d)

Figure 10.8 Examples of failure mode (FM) types related to Table 10.4
a) FM1 - Damaged pier (left), the related failure (right), Serbia, 2014; b) FM2 – Failures of multiple-single span bridges, Canada 2009 (left), Japan 1998 (right), (Fukui & Nishitani, 2002); c) FM3 - Local scour at a pier, two bridge failures in Serbia 2016; d) FM4 – Pier washed away, no collapse! (left), Abutment local scour, no failure of the frame bridge (right), Serbia 2016

Regarding Table 10.4, clear and concise guidelines are necessary for inspection of specific bridge types and their key elements. Vulnerable zones of a bridge comprise those bridge elements or their segments, which have a crucial role in resistance to a failure due to local scour at substructure foundation/s. Any type and severity of damage at these zones are of interest (e.g. Figure 10.9), as it may decrease the overall bridge resistance to an oncoming flooding event i.e. increase the probability of failure, thus decrease reliability.

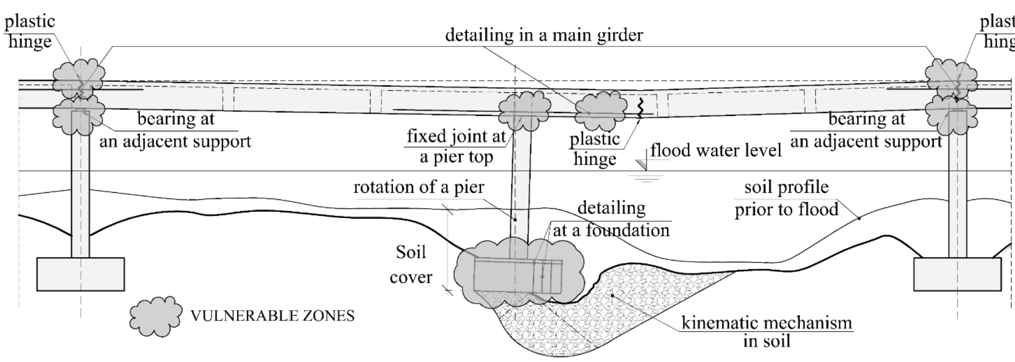


Figure 10.9 An example of a failure mode due to local scour at a pier - multiple-span RC girder bridge and its vulnerable zones - adapted from (Tanasic & Hajdin, 2017b)

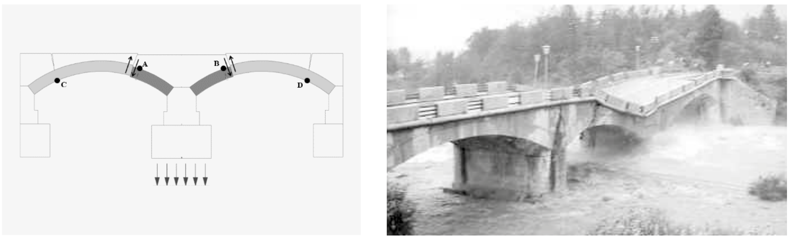
Besides local scour at substructures, the traffic interruption may be caused by washing away of access roads or overtopping, where a bridge structure remains intact. Here, the attention of inspectors should be directed in an examination of embankment material, bridge alignment in the flow, geometry of the obstacle (i.e. river channel), possible extreme flood level etc. These cases are not in the scope of this report and will be investigated in the future research.

10.4.5 Masonry arch bridges - failure modes and vulnerable zones

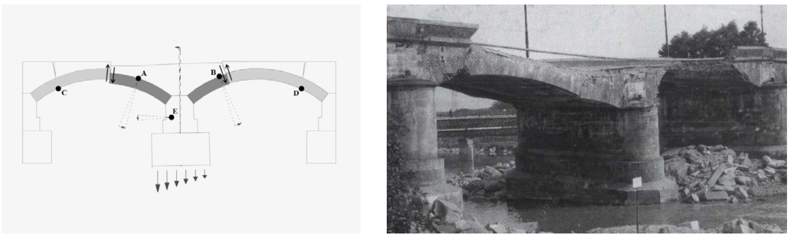
The observed damage of masonry arch bridges in the past flooding events can aid in classification of various failure modes here denoted as a FM_a (*Figure 10.10*). It can be noted that the inclination of the water flow can be considered as a key aspect in defining the shape/magnitude of a local scour cavity at the foundational system, thus reflecting in a specific failure mode. The main weakness of the masonry bridges is the lack of resistance in tension of the materials, which causes fragmentation of the structure in subparts and differential settlements. This response to local scour action is significantly different from the response of RC piers which usually imply rigid rotations of the entire element.

a) FM_a 1

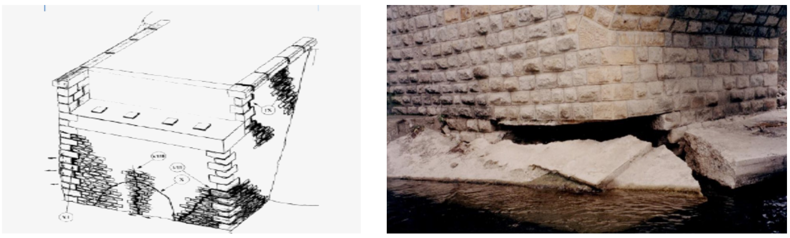
b) FM_a 2



c) FM_a 3



d) FM_a 4



e) FM_a 5

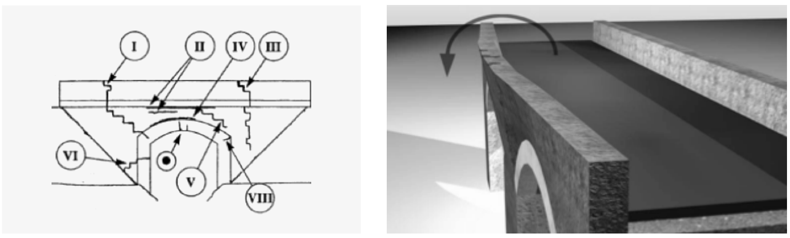


Figure 10.10 Failure modes of masonry arc bridges
a) Fragmentation of a pier, (FHWA, 2011); b) Symmetrical, in-plane failure mode, (Zampieri, et al., 2017); c) Non-Symmetrical, in-plane failure mode (Zampieri, et al., 2017); d) Failure mode related to fragmentation of an abutment, (SÉTRA, 1996) e) Out-of-the -plane failure and spandrel wall displacement, (SÉTRA, 1996);

Fragmentation of piers (FM_a1) can be related to transversal piers slenderness. It can be observed how for higher slenderness values, the failure mode tends to be associated to out-of-plane monolithic rotations (FM_a5), whereas in case of squat piers, the fragmentation of the structure is the key failure mode. FM_a1 is usually observed in bridges subject to water flows acting orthogonally with respect to the longitudinal axis of the bridge and when no beaks are installed to reduce the impact of water. On the contrary, when cutwaters or other diverters are present, water flow profiles are broken reducing their impact on piers and consequent frontal scour. In this case, the erosion is mainly associated to tangential actions along the piers' sides parallel to the water flow direction, resulting in symmetrical scour cavity shapes and consequent failure mode FM_a2. The FM_a2 is characterized by a quite uniform vertical settlement due to simultaneous loss of soil bearing capacity and decrease of the compression in the adjacent arches. The structure turns into a rigid-block mechanism.

When water flow is skewed with respect to the longitudinal bridge axis and piers plan orientation, flooding leads to a not-symmetric scour cavity i.e. the larger scour depths are along the pier side directly exposed to water flows. In this case, FM_a3 can be observed: the asymmetric scour cavity induces rotations at the base of the pier, with a differential settlement field, and the part of the structure turn into a rigid-block mechanism (*Zampieri, et al., 2017*). The main difference between FM_a2 and FM_a3 is that in the latter case, the crack pattern is not symmetric with respect to the pier axis, and in addition, horizontal cracks can appear in it at the mid-height level due to bending actions. The FM_a2 and FM_a3 can be reasonably considered as in-plane FMs, when bridges are characterized by squat piers in the transversal direction. However, in case of slender piers the response of the structure can show out-of-plane mechanisms (FM_a1).

For the abutments, soil erosion can produce loss of contact (in particular, in the wall corners) and consequent settlements/rotations that can develop in the FM_a4 with cracking also at the arch level (see hinges C and D for the FM_a3). One of the most vulnerable bridge elements in a case of out-of-plane rotation, is the spandrel wall. It is related to FM_a5, facilitated by the self-weight of the back fill, thus significantly compromising bridge functionality.

10.5 Assessment of KPI Reliability

In the following paragraphs, the semi-quantitative approach is presented, where the reliability class (REL) is assessed via two complex PIs exposure class (EC_L) and resistance class (RC_L) (*Figure 10.14*). The exposure class aims to grade the threat of a certain flooding scenario and related local scour at bridge substructures. The resistance class aims to account for diverse bridge types, their characteristic FMs and a temporal aspect i.e. update on the condition of elements in vulnerable zones. For the scenario were several substructures may be affected by local scour, the assessments of exposure and resistance classes are performed for each substructure separately, to estimate which is the case that has the worst reliability class.

10.5.1 Assessment of the exposure class (EC_L)

There are four EC_L on the vertical axis of the matrix in *Figure 10.14*, where one denotes the lowest, and four the highest EC_L of a bridge substructure. The main data for this assessment comprise an extreme flood hydrograph, river channel properties/geometry and information on foundation soil type/erodibility. The first class denotes bridges with a substructure that is not in a floodplain for the adopted extreme flood (e.g. return period of 100 years), whereas the second class implies that a substructure is going to be in a contact with water. Still, this does not necessarily involve any local scour, which can be confirmed by a visual inspection or measurement of soil cover on-site. Alternatively, the soil erodibility can be assessed by direct/indirect testing (on-site/laboratory) or empirically. The latter is further elaborated since it can practically provide additional information on the scour threat by utilizing the available data on river channel and foundation soil properties.

The two parameters are commonly used as a threshold value for initiation of scour i.e. the rate of soil erosion of 0.1mm/hr. These are the critical shear stress at the soil interface (τ_c) and the critical velocity for initiation of the erosion (Vc). Their application depends on the methodology for empirical evaluation of local scour, and here the two approaches, given in (*FDOT, 2010*) and (*Briaud, et al., 2009*), are suggested to use for coarse-grained and fine-grained soil, respectively. Any other approach may be used, but preferably it should account for temporal aspect of local scour i.e. evolution of its magnitude over flood duration, to provide a more realistic prediction of scour depths at substructures.

The transition from the second to the third EC_L implies that the mean approach velocity of flow upstream (V), for the adopted extreme flood (Q_{ext} in *Figure 10.11*), can induce soil erosion. To evaluate V, the assumption of steady one-dimensional flow is adopted. The necessary information comprises river channel geometry (cross section and slope) and the Manning coefficient. In the absence of reliable data for an investigated location, these may be verified easily in one-time inspection. The Vc may be either evaluated based on the median soil particle diameter (D50>0.01mm) and critical soil particle shear velocity (*Melville & Coleman, 2000*) or conservatively accounted for the recognized soil type by using erosion function charts (*Briaud, et al., 2011*), that can be also used for estimation of the τ_c.

If a bridge has EC_L*=3* and have a history of flooding events (e.g. overtopping), it should be classified in the fourth class. This class update is also for a case where there are conditions on site, which may exacerbate local scour (e.g. debris/ice, flow constriction, etc.), but cannot be readily accounted in the empirical evaluation of local scour. The schematics for the assessment of the EC_L is given in *Figure 10.11*.

To sum up, the evaluation of EC_L may be done directly i.e. in a visual inspection (e.g. measurement on-site), or indirectly by an assessment of the soil erodibility. The right choice of the method is up to the decision-maker. Although, the first method is straightforward, its results do not necessarily clear all uncertainties regarding the exposure of a bridge in future. If the scour cannot be easily confirmed (e.g. due to an infill or unavailability of substructures for inspection), or the substructures have not been exposed to extreme flooding since the bridge was built, the second method should be enforced in a QC plan of these bridges.

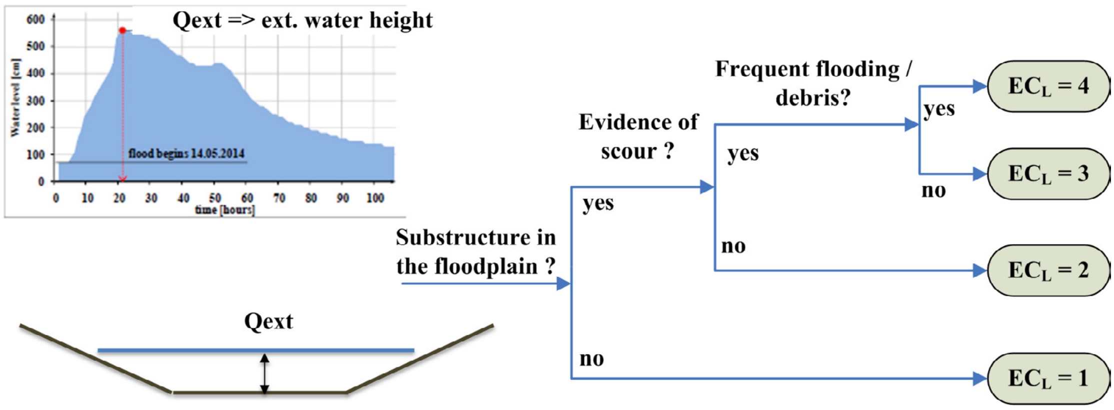


Figure 10.11 Schematics for assessment of the exposure class (EC_L)

10.5.2 Assessment of the resistance class (RC_L)

The resistance of a bridge to local scour is graded in five classes (horizontal axis of matrix in *Figure 10.14*), where one denotes the highest and five the lowest RC_L. The main data for this assessment comprise knowledge on possible FMs and condition of bridge elements in vulnerable zones, including the height of a soil cover at an affected substructure. In *Figure 10.12*, the schematics for assessment of the initial RC_L for girder and frame bridges is given, which is further elaborated.

The bridges which have a conceptual weakness of a substructure/foundation system are to be categorized in the fifth class (FM1 in *Figure 10.8*). Further, the bridge may be set in the first class if there are undamaged protective structures at substructures, otherwise the protection is ignored. The bridge can also be in the first class if it is failure safe (FM4 in *Figure 10.8*). This however can be debated as in the case that a pier is washed away, technically there would be an operational failure as the traffic would be closed. For all other cases, a bridge is initially categorized based on a relevant FM, i.e. by accounting for the foundation type, type of the joint at the substructure top and the type of connections of the main girder with adjacent substructures (*Table 10.4*).

For the girder and frame bridges with shallow foundations, the ultimate extent of scour cavity beneath the foundation level govern the probability of failure (*Tanasic, 2015*). When exposed to a same scour scenario, the least resistance to local scour is displayed by the bridges where there is no restraint to the horizontal displacement of the top of the affected substructure. Where this is not the case, the role of the main girder and adjacent substructures can be observed.

By design, the bridges with deep foundations are clearly more resistant to a failure, but here FMs which involve pile buckling should be considered (*Ramey & Brown, Sept. 2004*). Within WG3, the resistance of bridges with deep foundations is considered to be of a higher order than shallow foundations. This does not necessarily have to be a rule, e.g. especially for bridges where it is known that the detailing is not adequate.

Transition to a higher class (i.e. a lower resistance) is possible in a case where a condition of a segment/element, which is in a vulnerable zone (see *Figure 10.9*), becomes worse and that may adversely affect the possible FMs. Also, when the EC equals three or four, an update of the RC_L is possible when the soil cover at a substructure reaches a predefined threshold. For this assessment, a potential local scour depth at an affected substructure may be assessed via well-established scour evaluation formulas (*Arneson, et al., 2012*).

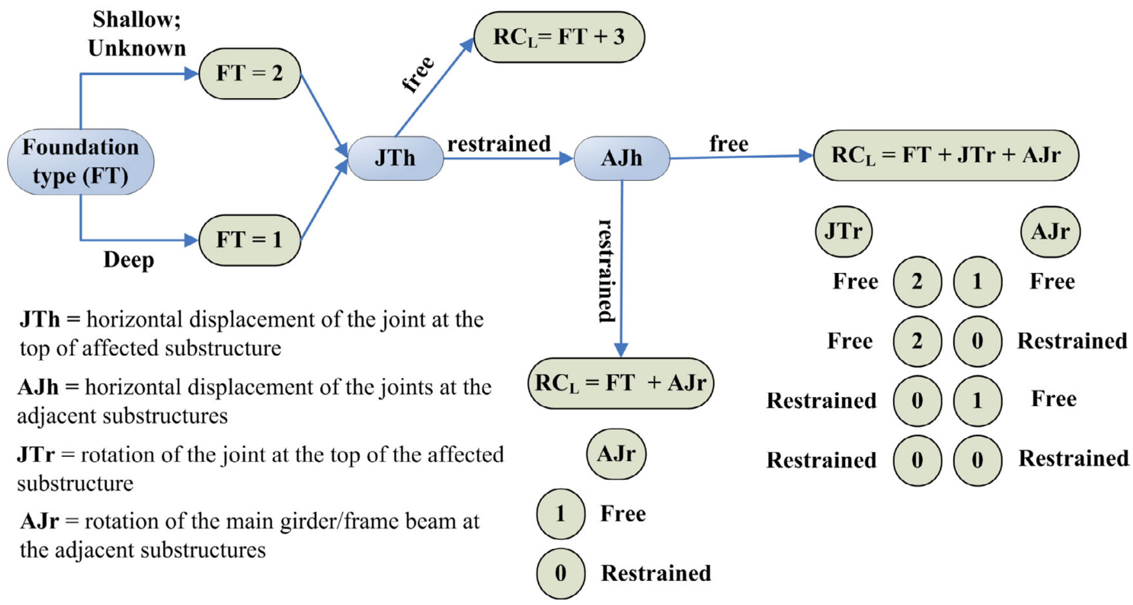


Figure 10.12 The schematics for assessment of the initial RC_L for girder and frame bridges

This update of RC_L is considered conservative, as the related scour evaluation formulas tend to overestimate local scour depths observed on-site. Nevertheless, it is regarded as sufficiently accurate and comprehensive for BM practice, provided that the FMs are adequately considered. An example of thresholds for the height of soil cover is given in *Figure 10.14c*, which is based on the adopted extreme event magnitude.

The common situation in practice is that the data on foundation type/geometry is unknown and it cannot be reliably obtained from the available project documentation, date of construction, design code, etc. The bridges with unknown foundations are to be treated as bridges with shallow foundations. Another issue are foundation depth and the actual soil cover at the affected foundation, since they can be also unknown. If the EC_L for these substructures is equal or higher than three, their initial RC_L should be increased by one (five is the max. value). The justification for this update may be obtained by indirect evaluation of local scour as previously explained.

Besides the magnitude of a scour cavity, its location in respect to the affected substructure may have a determining effect on the structure resistance. This is the factor of the substructure & foundation geometry as well as the type of the soil and a flow property, which are the parameters accounted in the state-of-the art local scour evaluation formulas. In the adopted scenario of exposure, this should be considered as the most unfavourable location for the structure resistance e.g. for an in-plane failure mode (e.g. *Figure 10.8a,c*) or out of the plane failure mode (*Figure 10.8b* and *Figure 10.14a,d*). The latter is especially important for masonry arch bridges, which have two main weaknesses when exposed to local scour, i.e. the design of their foundations and building material (*Section 10.3.2*).

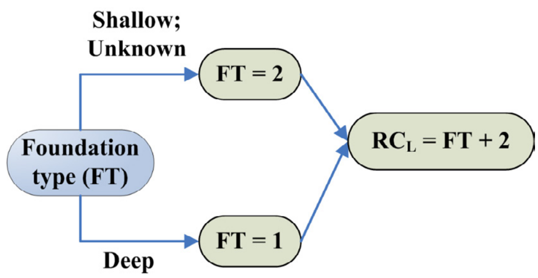


Figure 10.13 The schematics for assessment of the initial RC_L for arch bridges

Thus, when exposed to a certain scour scenario, it can be adopted that the total resistance of arch bridges stems only from the soil cover, disregarding the structural resistance owing to their type, geometry and detailing. This is conservative but acceptable due to low resistance of bridge elements to tension. In contrast to the ***RC*** bridges, the scour magnitudes just below the shallow foundation base can cause localized damages (e.g. *Figure 10.10d*), thus trigger an immediate maintenance action. Thus, when compared with the girder and frame bridges, in this framework, their initial RC_L should equal three for deep and four for shallow/unknown foundations (*Figure 10.13*), with a possibility for an update based on local scour measurement/evaluation (*Figure 10.14c*).

10.5.3 The reliability class (REL) matrix

To sum up, the two complex PIs i.e. *exposure class* and *resistance class*, are used to assess the KPI of Reliability. The presented approach accounts for:

- Soil erodibility
- Indirect local scour assessment via well-established empirical formulas
- The resistance of diverse types/static system of bridges
- Possibility for an upgrade to a full quantitative approach

Herein, the indirect local scour assessment is suggested, which has been applied in bridge management practice for over two decades predominately in the U.S. (*Arneson, et al., 2012*) and New Zealand (*Melville & Coleman, 2000*). The resistance of different bridge types to local scour have not been included in the decision-making process until now.

The suggested approach for assessment of Reliability gives a practical approach for rating bridges exposed to flooding events, which has a potential for a full quantitative approach that would provide a more detailed differentiation between the given reliability class thresholds.

a) **Reliability class**

	1	**high**
	2	**medium**
	3	**low**
	4	**very low**
	5	**critical**

b) **Transition to a higher resistance class**

3	Ls < 0.5 S_c	three and more consecutive flooding events
4	0.5 S_c > Ls > S_c	two consecutive flooding events
5	$L_s > S_c$	one flooding event

*Ls = Indirectly evaluated local scour depth
*Sc = Soil cover is measured from the foundation level; for deep foundations it is the maximum height of soil cover at pile for a pile buckling failure mode

c)

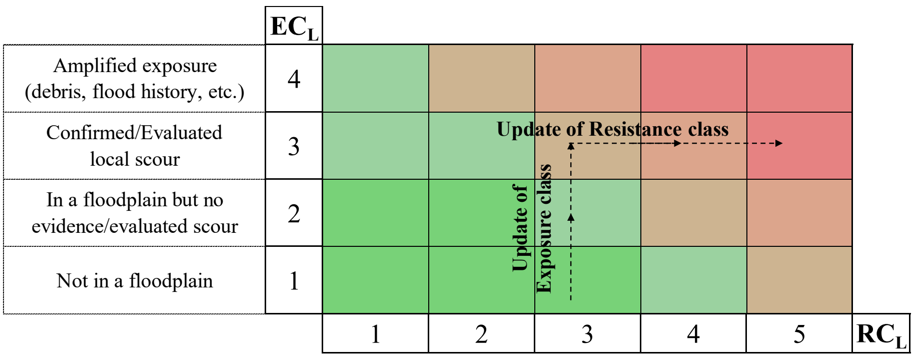


Figure 10.14 An example of the reliability class (REL) matrix
a) REL scale; b) A transition between RCL_S, based on indirect local scour evaluation; c) REL matrix

10.6 Guidelines for structuring of QC plans - background information and adequate actions

Based on the previous sections, the QC plans with respect to sudden events should be tailored for similar types of bridges, based on the adequate PIs. The time schedule and analysis of surveyed data should be defined along with the thresholds for initiating adequate actions. The scope & frequencies of the inspections and data updates to provide the adequate background information are discussed in (*Table 10.5*).

Table 10.5 The background data for QC plans – schedule of activities/inspections, adapted from (Tanasic & Hajdin, 2017b)

PI group	Specific information	Activity	Attention	Schedule
Exposure	Extreme hydrographs	Data update	Climate change	After every flood
	River channel properties	Special inspection/ survey	Properties vary in time	After a major flood
	Substructure geometry & location	Data review/site inspection	Water level in floodplains	Once
Resistance	Soil cover	Special inspection/ monitoring	Infill of scour cavities	After a major flood/ continuously
	Soil geotechnical properties	Data review/ Special inspection	Natural variability of properties	Once
	Bridge structure	Regular inspection	Deterioration at vulnerable zones	Periodically*
Consequences	Damage catalogue	Definition of failure modes	Bridge types	Once
	Network & traffic data**	Data update	Major detours & road works	Minimum once a year

*Governed by a specific bridge element or groups of elements; ** Within the project, indirect consequences of failure are not accounted but should be part of a QC plan in future BMS

The following preventative interventions may be considered to reduce the probability of a failure in a specific hazard scenario, i.e. increase Reliability to hazards, but affect the Availability as well (see in the brackets):

- Decrease the exposure to a flooding scenario
    - Soil works at the bridge site e.g. armouring of the river bed/embankment (low or no traffic interruption)
    - Protective structures at substructures (low or no traffic interruption)
- Preservation or increase of a structure resistance
    - Foundation repair/retrofit (one-time interruption/no traffic interruption)
    - Bearings/joint repair or retrofit (one-time/periodic traffic interruption)
    - Repair/retrofit of a main girder (one-time traffic interruption)
- Monitoring of scour depth at substructures (one-time or no traffic interruption)

It must be noted that the interventions which are related to the preservation or increase of structure resistance may also benefit the overall bridge performance to other sudden or slow (deterioration) processes, thus should be considered adequately in a long-term cost analysis. However, the increased resistance of certain elements or instalment of protective structures may decrease the bridge resistance to other hazards, e.g. earthquakes. Monitoring of scour depth may be the most adequate action for bridges in non-mountainous terrains where they can be frequently exposed to the scour as a slow process, leaving the ample time to react. The feasibility of this action should be waged against the actions needed to reliably estimate the soil erodibility at site, which is up to the decision maker.

10.7 Illustrative example

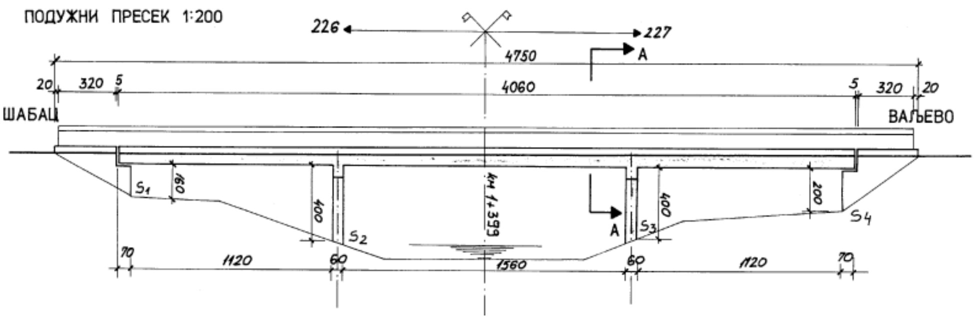
In the example, the reliability class (REL) is evaluated for a multiple-span girder RC frame bridge from the Serbian road network. Although, the bridge collapsed in 2014 during a flooding event (*VRS, 2014*), the example gives valuable information on the failure mode, vulnerable zones and the necessity for survey of specific data on the bridge site. The inventory data of the bridge is reviewed from the Serbian bridge database (*Putevi Srbije, 1998*), data on the flooding event at the site was retrieved from Republic Hydrometeorological Service of Serbia (*RHMZ, 2014*), while the soil profile was extracted from the project of the bridge reconstruction (*Institut za Puteve, 2015*). The original project documentation of the bridge was not at disposal.

Bridge inventory

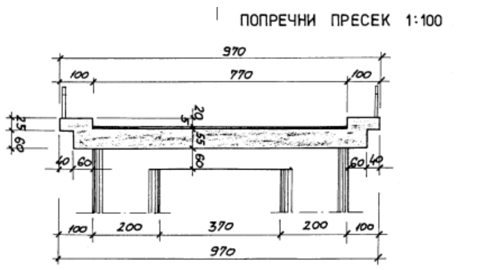
The inventory data gives specific information on: the type & static system, spans, bridge cross section and the channel cross section at the time of construction (*Figure 10.15*). However, the information on the foundations is not available, which is a frequent problem in bridge databases. Based on foundations of similar bridges in the network, it is assumed that the foundations were shallow. There are two possibilities, the Case A and B in *Figure 10.15c*, which should be duly considered if the indirect evaluation of local scour is to be performed.

a)

b)



c)



d)

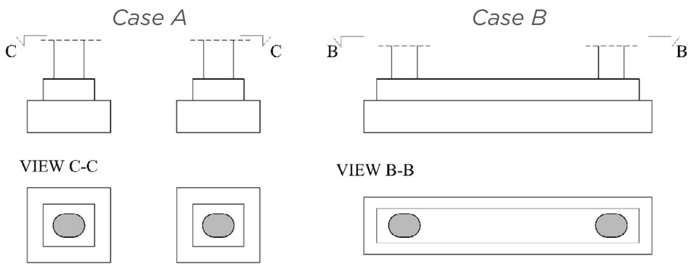


Figure 10.15 Bridge inventory data (Putevi Srbije, 1998)
a) The bridge – sideway view b) Longitudinal section; c) Cross section A-A d) Possible foundation types

The foundation depth and the soil cover are unknown for this bridge. However, based on the data from the two soil boring samples, which were probed after the demolition, the terrain and soil profile have been reconstructed. The border between alluvial deposits sandy clay (CL) and gravelly sand (SW) (*Figure 10.16d*), can be a valid assumption for the max. foundation depth.

The minimum foundation depth is adopted to be 1.0m below the river bottom (according to information on similar bridges and the design code/recommendations from the time of construction). There are no protective structures and there is no data on Larsen sheets left after the foundation construction.

The failure mode forensics

Based on the pictures from the site, the failure mode has been analysed (*Figure 10.16a,b*). The part of the bridge structure had become a mechanism with two plastic hinges (*Figure 10.16d*), coinciding with a soil failure beneath the scour affected pier foundation, due to undermining.

There was an excessive deformation of the longitudinal reinforcement at the Hinge I, and the unstable part of the bridge translated backwards in the longitudinal direction, coinciding with the end span sliding from the abutment and "closing" of the Hinge II (*Figure 10.16b,c*). The unstable part of the bridge than tilted transversely, in the direction of the water flow, and assumed final position.

Figure 10.16 Bridge failure forensics
a) Bridge failure – during the flood b) The ultimate position of the collapsed bridge section; c) Damage at the plastic hinges d) The sketch of the failure mode and the soil profile, adapted from (Institut za Puteve, 2015)

Vulnerable zones

The failure revealed for this load case a conceptual weakness of the main girder regarding splicing of reinforcement, which somewhat limits its structural resistance. Although, the exact mechanism of soil failure is unknown, the locations of the hinges are compatible to a rotational kinematic mechanism in soil. Alternatively, a sinking of the pier (i.e. vertical displacement) can be regarded as possible mechanism. The elaboration of the difference in the mechanisms of soil failure and related local scour magnitudes is beyond the adopted framework in WG3.

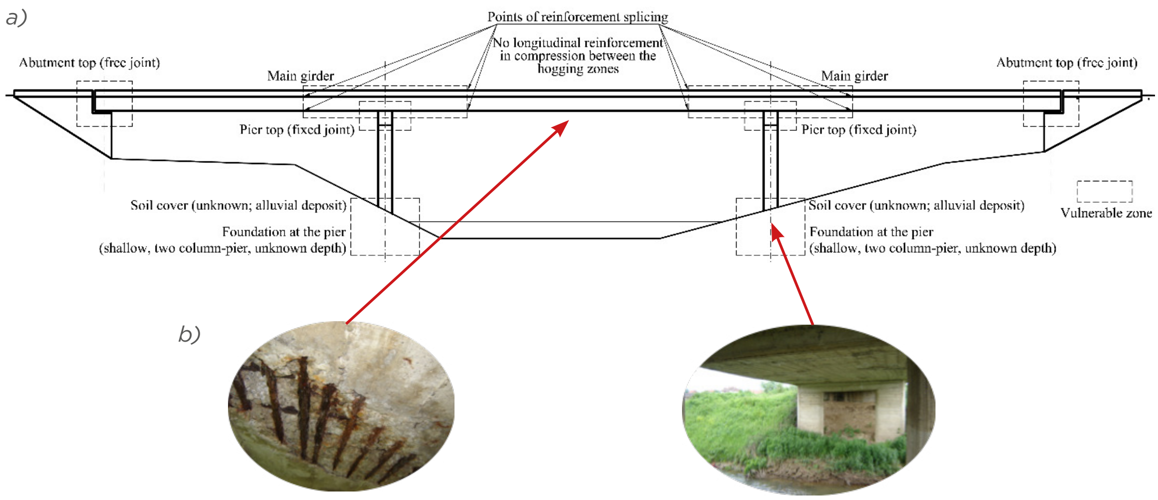


Figure 10.17 Vulnerable zones & observed damages for the investigated bridge
a) Vulnerable zones; b) The observed damage on the bottom of the main girder (left) and the observed soil cover at the pier (right), (Putevi Srbije, 1998)

In *Figure 10.17*, the bridge vulnerable zones are presented. Here considered is a failure scenario where one of the piers is affected, while the failure modes involving the abutments are disregarded as being less plausible according to the terrain profile. However, in the absence of data on soil type/cover, an inspector should bear in mind all possible failure modes. The two observations from inspections are discussed. The corrosion of the reinforcement in bottom zone of the main girder in the mid-span, is of no importance for the action of local scour. However, the soil cover at a pier is for this case of pivotal importance. The only data on this was found in the bridge inspection records, where it is stated that there is "a lot of vegetation and alluvial deposits in the river channel, at the pier". This record is underestimated, which was eventually demonstrated as the primary weakness of the bridge in respect to local scour.

Assessment of the RC_L

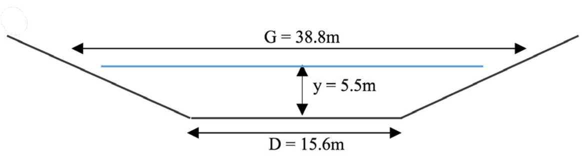
The initial RC_L is evaluated according to the *Figure 10.12*. The foundation type is unknown thus, it is treated as shallow (*FT=2*). The connection of the affected pier and the main girder is rigid (*JTh = restrained*). The horizontal displacements of the adjacent joints are restricted as there is a rigid connection at an adjacent pier (*AJh = restrained*). The fact that there is a conceptual weakness in the main girder, determines how it will probably fail in the event of local scour. This means that for an adopted scenario, there is negligible aid of the adjacent span in the resistance to a failure. Thus, *AJr = 1* i.e. both adjacent supports (at the pier and the abutment) do not restrict rotations. Finally, the initial $RC_L = FT + AJr = 3$. A rotational mechanism in soil may govern the failure mode, with a low resistance of the structure. The possibility to update the initial RC_L can be only based on the information about the soil cover but for this the EC_L needs to be assessed first.

Assessment of the EC_L

For the given flood, the hydrograph was available (*Figure 10.18b*). In this case, the highest magnitude of flood corresponds to the maximum clearance below the bridge, which agrees with pictures following the failure (*Figure 10.16a*). In the absence of a flood hydrograph, the latter assumption can be a starting point for the evaluation of ECL and local scour depth.

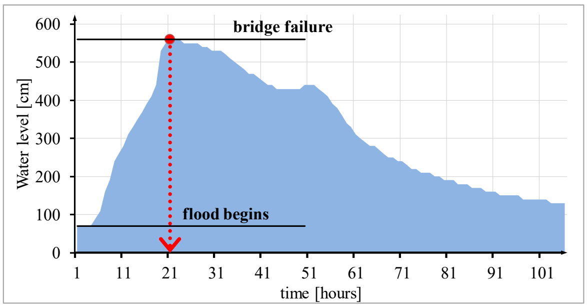
The both piers can come into the contact with water (i.e. ECL = 2), and the question stands can the water level, corresponding to the extreme flood magnitude, induce any local scour. There are several possibilities to make this assessment (see *Section 10.5.1*). It can be done directly, by a visual inspection on-site, i.e. an inspector sees the traces of scour form previous floods. According to the pictures from the previous inspection, this is not the case. In fact, there is an alluvial deposit at the piers and no conditions on site which could exacerbate potential scour. Based on the approximated channel geometry (*Figure 10.18a*) and the available hydrograph, it was estimated by using FDOT methodology for local scour evaluation, that there is going to be local scour for the adopted magnitude of flood i.e. the velocity of the water in the channel is greater than 40% of the critical velocity (*Figure 10.18c*), thus ECL = 3.

a)



A = 136.2m² (Water channel area)
Q = 274.2m³ (Water flow)

b)



c)

d_{50}=	10	mm	Median particle diameter
y=	5.5	m	Water height in a channel
a=	0.80	m	effective pier width
S=	0.001		Channel slope
m=	0.035		Manning roughness coef.
V_c=	**1.92**	**m/s**	**Critical particle velocity**
40% V_c=	**0.77**	m/s	40% of Critical particle velocity
V=	**2.12**	m/s	**Velocity of water in channel**

Figure 10.18 Assessment of possibility for local scour at a pier
a) Geometry of the channel, based on data from (Institut za Puteve, 2015); b) Flood hydrograph at the bridge site, based on data from (RHMZ, 2014); c) Critical particle velocity calculation according to methodology in (FDOT, 2010)

Assessment of the Reliability class and urgency of intervention

Based on the EC_L and initial RC_L, it is obtained that the bridge has REL=3 (see *Figure 10.14*). However, the inspector must acknowledge that a fine graded alluvial deposit at piers cannot be regarded as a "valid" part of soil cover, since it can be swiftly washed away as a flood starts. Thus, due to this uncertainty, the RC_L may be updated to four, and thus the REL=4. Alternatively, the latter judgement may be confirmed/denied by indirect evaluation of local scour depths (see *Figure 10.14b*), which is up to the decision maker. Here, the foundation type from *Figure 10.15c*, which gives larger scour depths should be accounted for.

For this bridge case and with regard to *Table 10.5*, an example of a Reliability qualitative scale with adequate actions is given in *Figure 10.19*.

Reliability		**Urgency of intervention**
1		Regular visual inspection of vulnerable zones & inspection after a major flood (e.g. min 50 year return period)
2		**1** + gather information on soil type to assess erodibility or possibility for local scour to occur
3		**2** + Perform soil cover inspection (after every flood) & evaluate local scour potential
4		**3** & Monitoring for early warning (specific water level) or go to **5**
5		Protective structures at substructures or Increase of a structural resistance

Figure 10.19 A qualitative scale for KPI of Reliability and urgency for intervention – hazard related

For REL=4 it is suggested to consider monitoring as an adequate option. For the given case, the bridge collapsed 16 hours after the flood have started, so the monitoring would only help to close the bridge in time to prevent casualties. The conceptual weakness of this bridge cannot be resolved, thus the viable option in this case would be installment of protective structures (e.g. gabion piles), despite the unknown height of soil cover.

11. Human resources and equipment

The bridge inspection process is critical to ensuring the safety of bridges, identifying repair and maintenance needs and determining appropriate allocation of funds. It is the foundation of most, if not all, bridge management systems. Data accuracy has to be consistent throughout the period when inspections are conducted. Only then, it is possible to get the best possible information describing the bridge current state, performance, deterioration rate and similar, over the years of use. The primary objective of bridge inspection is to register type, extent and intensity for every damage recorded on each element of the bridge. During field investigations, every element is being examined separately and at the bridge component level (there are no complex analyses to be performed on this level), one of the most important goals to be reached is objective damage assessment. There are four main approaches in damage detection and assessment (*Strauss & Mandić Ivanković, 2016*):

- visual inspection,
- non-destructive testing,
- probing and
- structural health monitoring.

Structural health monitoring (SHM) is generally performed on the bridges of utmost importance for the road network. Equipment acquisition, its maintenance, data collection and analysis require financial assets that are not affordable for large scale use. Therefore, SHM is in most cases used for bridges with large spans only. Probing provides the most reliable results regarding the state of the bridge and its individual components. Its biggest weakness is the fact that its implementation causes a certain damage to the construction. In most cases, it is performed when remediation or reconstruction of a specific bridge is already envisaged, however more accurate information on the state of the bridge components is still needed.

The use of SHM and probing is therefore not suitable for large-scale periodical damage detection and assessment. Although somewhat less reliable, for long-term data acquisition regarding the bridge state and its changes over time, two types of data collection techniques remain available: visual inspection and non-destructive testing (NDT). Both approaches have advantages and disadvantages from the viewpoint of data acquisition, reliability, work pace, required equipment etc. Most importantly, visual inspection disadvantages can be to a large extent eliminated, with the implementation of a suitable inspection protocol and complementary use of NDT. Defining the most appropriate use of resources available (i.e. human and equipment) for bridge inspection practice is therefore necessary. A survey was conducted to determine inspectors' minimum requirements and best practice to review their work. To determine the most suitable use of equipment during inspections and equipment minimum requirements a smaller group of engineers with experience in NDT was formed.

11.1 Inspectors requirements and guidelines

The majority of the existing bridge maintenance systems are based primarily on information obtained through visual inspections. Although visual inspections have some limitations they will remain the main aid for collecting data. This point of view was accepted as an undisputed fact in the WG3 meetings of COST Action TU1406 in March 2016 (Belgrade, Serbia) and October 2016 (Delft, Netherlands). Bridge inspectors' qualifications are one of key elements to ensure quality data acquisition. Their qualifications as well as review of their work therefore needs to be defined. As inspectors are the main source of information appropriate formal education and trainings are vital, however, reliability of their inspection also increases with years of experience. To determine inspectors' minimum requirements, necessity of refresh trainings, benefits of rotating inspectors and best practices to review their work a survey was conducted (*Table 10.1*) where 29 engineers with bridge inspection experience from 14 countries participated. The most important questions related to visual bridge inspection were discussed and as a result, guidelines were produced.

The frequency of bridge inspections should depend on bridge state and bridge importance to the network. Therefore, bridges with low reliability as well as the most important bridges (if they do not have SHM or similar special inspection program implemented) should be inspected more frequently as the majority of bridges in the network. On the other hand, new(er) bridges with little or no damage could be inspected less frequently. It is suggested that the bridge inspection frequency should vary between 2 and 4 years depending on the bridge state. In national inspection regulations of most countries, a single value needs to be determined for inspection frequency, which should according to experts participating in the survey be set at 3 years. Inspectors' minimum formal education is prescribed in most European countries, but surprisingly in some countries the type and the level of education is not defined and only short initial training course is needed to become a licenced bridge inspector. More than three quarters of the experts participating in the survey agree Bachelor of Science is the minimum formal education an inspector should have. The remaining experts evenly chose either higher either lower formal education than suggested. The inspectors' experience is the second extremely important aspect. The question is, when can the inspector be considered experienced? According to the survey, it is after 5 years of work on field.

Experts were further asked should bridges that are more important be examined by inspectors with even higher level of education (i.e. M.Sc.). 55% agree with the statement, while 45% believe that is not necessary. The decision regarding the subject should be left to the infrastructure owner/manager. The same question was given regarding inspector experience. 19 out of 29 (65%) experts agree most important bridges should be examined by experienced inspectors only as: 1) they are less likely to overlook damages or defects that are harder to record, 2) their data is more reliable. In order to improve data reliability and uniformity among inspectors, initial inspectors training should be mandatory. The survey respondents almost uniformly agree with the stated. A 5-day course should be implemented for all new inspectors. Newly trained engineers should also conduct first inspections jointly with experienced ones. At least 10 bridges should be inspected jointly, preferably more (up to one year according to the results of the survey). In addition, a 1 to 2-day refresh training should be conducted for practicing inspectors every second year. Training should be theoretical as well as practical, i.e. conducted on field.

When conducting each bridge inspection, a report of a previous inspection describing the state of the bridge at the time is needed. However, not all data can be recorded on paper, as inspectors have to decide to what detail the state of the bridge should be described. Having the same inspector inspect the same bridge every year would eliminate the mentioned shortcoming as the inspector can more precisely assess the damages over time. On the other hand, certain damage or deficiency might be overlooked each year. The participating experts were asked whether it is better that inspections are conducted by the same inspector or should the rotations of inspectors on bridges be mandatory. Given the diversity of responses, it was concluded no guidelines can be given regarding this subject. The method of conducting the field inspection is also important. Values of variety of indicators have to be determined during inspection. Some of the indicators are more difficult to assess accurately and they should be given more time and emphasis during visual inspection in order to detect and evaluate them, thereby increasing data reliability (e.g. scour). Data can further be improved by use of non-destructive testing equipment (the topic is discussed in the next section). Method of producing the reports was also examined. In some countries the report is completed on field immediately after the inspection, but the majority of interviewed experts believe it is better to complete it in office.

During or at the end of each inspection season reviews have to be carried out to control data quality. According to the survey at least 10% of all inspection reports should be reviewed in office, but preferably, the percentage should be much higher. Almost half of the experts estimate at least 50% of reports should be reviewed in office. The scope of the on-field review should be considerably smaller with 3% to 10% of inspections to be reviewed. Since all inspection reports cannot be scrutinized a suitable manner of sampling them must be adopted. Sampling in a way to ensure every inspector is reviewed was determined to be the most appropriate. The results of the survey are shown in *Table 11.1*. For each question the most frequently selected answer is displayed in bold, except in cases where there is not a predominant difference between the answers. For questions where answers for not pre-specified the most common answer is given (e.g. After how many wears of work is inspector considered experienced?)

Table 11.1 Guidelines for inspectors and inspection processes

INSPECTION FREQUENCY

What is (in general) the best suitable frequency for bridge inspection?	3 years	-
Should it be dependent on the bridge state (shorter for the most damaged bridges)?	**yes**	24
	no	5
Should it be dependent on bridge importance to the network (shorter for the most important bridges)?	**yes**	21
	no	7

INSPECTORS EDUCATION AND EXPERIENCE

What should be inspectors' minimum formal education? [H.S., BSc, MSc, PhD]	H.S.	4
	B.Sc.	22
	M.Sc.	3
After how many years of work on field is inspector considered experienced?	5 years	
Do important bridges need to be inspected by inspectors with even higher degree than stated in the previous answer?	yes	16
	no	13
Do important bridges need to be inspected by experienced inspectors only?	**yes**	19
	no	10

Should there be a mandatory initial inspectors training prescribed (up to 5 days) to improve uniformity among inspectors?	**yes**	28
	no	1
After the initial training, should inexperienced inspectors conduct inspections jointly with experienced inspectors?	not necessary	0
	yes, first 10 bridges	13
	yes, first month	4
	yes, first year	12

REFRESH TRAINING

Should refresh trainings (1 to 2 days) be conducted for practicing inspectors every second year?	**yes**	25
	no	4
If so, should refresh trainings be purely theoretical (in class only, but with pictures) or conducted on field as well?	in class	9
	on field as well	20

INSPECTORS ROTATION

It is better that same inspectors inspect the same bridges every year as they can better assess damages with time.	yes	8
	no	13
	not important	8
Inspector rotation should be mandatory as the same inspector might overlook something every year.	yes	16
	no	12

ON FIELD

Do some performance indicators (damages, defects, etc.) require more of your time to be detected and assessed than others?	**yes**	25
	no	4
Do you support the use of non-destructive testing (limited in scope) during visual inspection?	**yes**	25
	no	4
Should the whole report regarding inspection be completed on field?	yes	1
	no	18
	not important	10

REVIEW

What percentage of bridge reports should be reviewed in office (to check all data was recorded)?	5% to 10%	2
	10% to 30%	8
	30% to 50 %	7
	51% or more	12
What percentage of bridge inspections should be reviewed on field (to check data quality)?	1 to 3 %	7
	3 to 10%	12
	10 to 30 %	4
	31 % or more	6
How do we select bridges to be reviewed on field?	Random sampling among the whole bridge stock.	7
	Random sampling among critical bridges only.	4
	Sampling in a way to ensure every inspector is reviewed.	18

11.2 Requirements on equipment

11.2.1 Equipment suitability

The primary inspection task is damage, material properties, defects and other irregularities detection and evaluation. There are three possibilities of their detection: correct detection, false detection, no detection. Some performance indicators are more likely to be undetected or falsely detected via visual inspection than others. The shortcoming can to a certain extent be eliminated by complementary use of NDTs. As their regular use is impracticable due to inspection time and financial resources available per bridge, they should only be employed when detecting performance indicators, which have inadequate reliability when using eyesight alone.

The bridge network age and damage state, predominant bridge design and materials used strongly influence the selection of the most suitable NDT to be used. Asset manager and inspectors must first define, which PIs are being addressed during inspections and then analyse them. Some PIs and their assessment may be specific for individual networks. For majority of PIs visual inspection alone is enough to accurately determine their values, however, in some cases (delamination, concrete cover, concrete strength, etc.) it cannot adequately assess the extent or intensity of damage. The aim of the outlined analysis is to determine PIs that are more difficult to detect and assess (*Table 11.2*) and should therefore be given more emphasis during the inspection by employing NDTs.

Table 11.2 Visual inspection reliability assessment and need for NDTs

PI	Visual detection	Visual assessment	NDT employed
defects			
concrete cover (insufficient)	semi-reliable	demanding	concrete cover-meter
crack form/pattern	reliable	reliable	-
…			
related to material properties			
carbonation depth	unreliable	impossible	phenolphthalein test
cathodic protection deficiency	semi-reliable	demanding	-
chloride content	semi-reliable	demanding	quantab test
concrete strength deficiency	semi-reliable	impossible	rebound hammer
…			
related to equip. & protection			
absence of equipment component	reliable	reliable	-
approach slab settlement	reliable	reliable	-
asphalt pavement cracking	reliable	reliable	-
…			

The second step of the analysis is to determine the NDTs suitable to be employed into regular bridge inspection practice. The selection of the criteria to assess the NDTs is based on the assessment of WG3 – Task group 1 and a review of similar research assignments (*Gucunski, et al., 2013*; *Hesse, et al., 2015*; *Omar & Nehdi, 2016*) performed in the past.

Depending on the method of determining the values, the criteria are classified as descriptive or measurable. For measurable criteria, a specific value can be determined (e.g. test duration time is 10 minutes, test is standardized). The values of descriptive criteria can be partially subjective as they cannot be precisely defined. However, based on expert judgement, they can be reliably classified into classes.

Description of criteria selected is given in *Table 11.3* and although some of the criteria selected seem to be related, they are independent. For example, standardization does not guarantee high reliability of results: pull off test is standardized (EN 1542) and its results are reliable, however rebound hammer test is also standardized (EN 12504-2), but the reliability of its results is questionable (*Alwash, et al., 2017*). On site test duration and time required for the interpretation of the results are also independent: the duration of the pull-off test duration is relatively long as the adhesive between the disc and the substrate has to harden before the test can be executed, however test results are instant, as no interpretation is needed. Usability and cost criteria are unrelated.

Table 11.3 Criteria used for NDTs assessment

Criterion	Description
Results' reliability	Descriptive criterion: It defines reliability or accuracy of the results/measurements. It deals with the technological perfection of the investigation (accuracy) and the sensitivity of the method to various external factors.
Standardization	Measurable criterion: If there is a standard prescribed for the NDT under consideration, the results should be more reliable.
Usability	Measurable criterion: It defines the number of parameters that can be measured with the NDT under consideration. Ability to investigate two or more materials, different types of damages or defects and similar.
Test duration	Measurable criterion: It defines the speed of the NDT execution and the speed of data acquisition. The criterion is predominantly related to the time spent by the inspector but can also be related to possible traffic disruption (bridge or individual traffic lane closure) due to the investigation.
Results' interpretation complexity	Descriptive criterion: it relates to the obtained raw measurements and the need for long and demanding analysis to obtain final results (computer equipment and experienced engineers needed).
Cost	Measurable criterion: It defines the cost of equipment acquisition, the cost of test execution and the cost of data analysis.

To determine the relative importance of the criteria selected a survey using Analytic hierarchy process – AHP (*Saaty, 1980*) was performed. The weight (relative importance) of each criterion was determined as an average value of all ratings attributed to the individual criterion by the experts participating. Only engineers with experience in bridge inspection and NDT use participated, thereby minimizing the judgement subjectivity. The consistency of judgements was further increased using the SCB Associates Ltd software tool (SCB, 2017) that alerts the respondents in case of inconsistent judgements. The results are shown in *Table 11.4*.

Table 11.4 Criteria weights in order of importance

Criterion	Weight
Results' reliability	0.280
Test duration	0.233
Results' interpretation complexity	0.170
Cost	0.134
Usability	0.108
Standardization	0.075

In addition to the selection of criteria and assigning their weights, the criteria threshold values had to be determined (*Table 11.5*). Results' reliability is defined as the most important criterion. It is evaluated on the basis of the sensitivity of the test to external factors (e.g. humidity, temperature, test micro location) and on the basis of reliability of equipment or technology used. An example of NDT with high reliability is the phenolphthalein test as it provides reliable data regarding the carbonation depth at the micro-location. The rebound hammer, on the other hand, according to most researchers, is unreliable. To ensure higher reliability of rebound hammer test results, this method needs to be combined with other investigations or, alternatively, a large number of tests has to be performed (*Alwash, et al., 2017*).

Test duration is defined as the on-site time required for the test execution, while the time in the office is addressed by the result interpretation complexity criterion. The NDT is defined as a quick test if it can be carried out without substantially increasing the duration of the visual inspection of the bridge. Test duration is considered moderate if the time of the entire inspection is prolonged by up to 50% as a result of an NDT execution. If time consumption is greater, the usefulness of such NDT as a part of regular bridge inspection is questionable and should only be implemented when demonstrating exceptional performance in other criteria selected. For the test duration criterion, classification of the selected NDTs is to a certain extent subjective. Time needed to perform some tests is short, but these tests only provide local results and need to be performed numerous times to provide comprehensive results (e.g. hammer taping), while others require more time to be performed, but determine the state of construction as a whole for the parameter measured (e.g. infrared thermography). Additionally, for NDTs with wide usability, test duration may vary greatly for different measurements.

In addition to time spent for conducting field investigation, some NDTs require additional time for data interpretation that is usually performed in the office. Results interpretation complexity criterion considers NDT undemanding when its results are immediate (e.g. phenolphthalein test), satisfactory when short office or on sight analysis is required (e.g. cover meter requires undemanding data processing with computer software) and demanding when prolonged analysis with highly qualified personal is required (e.g. ground penetrating radar).

The cost criterion deals with equipment acquisition, maintenance, software cost and possible additional equipment needed for testing, while the value of inspectors' time is indirectly taken into account in the test duration and result interpretation criterion. The cost of acquisition is highly dependent on the technical characteristics of the equipment, therefore, the assessment based on this criterion is to some extent subjective.

NDT diverse usability is favourable for several reasons, such as less equipment needed on site and possibility of implementing unplanned types of measurements. The criterion is defined as less important as the inspectors are rarely surprised with the condition state of the bridge and consequently all tests are pre-determined. The standardization criterion was identified as the least important of the criteria selected. The NDT is given the highest score if it is supported by EN standard; if a national standard (domestic or foreign) is available a point is deducted and the lowest score is obtained if no relevant standard exists for the test under consideration.

Table 11.5 Criteria evaluation

Criteria	**Scoring**		
	3	2	1
Results' reliability	High, external conditions do not affect the results	Moderate, various factors can affect the results	Low, complementary investigations needed to confirm the results
Test duration	Short, total bridge inspection time is not noticeably increased	Moderate, total bridge inspection time is prolonged	Long, total bridge inspection time is doubled
Results interpretation complexity	Immediate results	Short analysis required	Prolonged analysis and high professional qualification necessary
Cost	Low	Moderate	High
Usability	Investigation of various materials and their parameters possible	Investigation of one material and two of its parameters possible	Limited usability, only one parameter is investigated
Standardization	EN standard	National standard available	No relevant standard

The NDTs selected were scored according to each criterion described above. Scoring was based on literature dealing with various aspects of NDTs (*SustainableBridges, 2007*; *Orbán & Gutermann, 2009*; *Van der Wielen, et al., 2010*; *Solla, et al., 2012*; *Gucunski, et al., 2013*; *Lee, et al., 2014*; *Hesse, et al., 2015*; *Hoła, et al., 2015*; *Lee & Kalos, 2015*; *Omar & Nehdi, 2016*) (*Rehman, et al., 2016*; *Alwash, et al., 2017*; *Omar, et al., 2017*) and the COST Action TU1406 WG3 members' experience. In some cases, the available data was contradictory, for example GPR is, according to (*SustainableBridges, 2007*), costly and time consuming when larger areas are under investigation, while according to (*Omar, et al., 2017*), the method is cost effective with an ability to rapidly scan large areas.

Additionally, within the engineering field, there is a common belief that GPR raw data interpretation is very demanding; while (*Lee & Kalos, 2015*) evaluated the method by the survey in which the participants assessed it as moderately difficult. In the literature review, several other contradictory views were recorded, making the evaluation difficult and open to different scoring.

The evaluation of an individual NDT was performed based on the utility function (Ui):

$$U_i = \sum_{c=1}^{m} V_{c,i} \cdot w_c \tag{14}$$

where i = NDT considered, c = criteria, $V_{c,i}$ = value of criteria c for NDT under consideration, w_c = criteria weight, U_i = {1, 2 or 3}. NDTs are classified depending on their application into three categories: material properties, damage and defects and corrosion. Within the category, their suitability for regular bridge inspection is determined on the basis of the utility function U_i (*Table 11.6*).

Table 11.6 Suitability of individual NDTs for regular bridge inspection practice

Application	U_i	**NDT**
Material properties	2,71	Cover measurement
	2,55	Phenolphthalein test
	2,43	Probe penetration test
	2,42	Pull-off test
	2,22	Rebound hammer
Damage and defects	2,22	Impact echo
	1,86	Thermography
	1,83	Acoustic emission
	1,80	Ground penetrating radar
	1,63	Ultrasonic pulse echo
Corrosion	1,89	Half-cell potential
	1,82	Galvanostatic pulse
	1,82	Electrical resistivity
	1,65	Linear polarization resistance

The results show that all NDTs measuring material properties have high utility rating. Despite some of the methods being semi-destructive (phenolphthalein, probe penetration and pull off test) and rebound hammer having poor reliability, these tests are fast, inexpensive and undemanding to perform, making them suitable for use during regular bridge inspection. Additionally, given the fact that some PI dealing with material properties are difficult to detect and assess by visual observation only (e.g. *Table 10.2*) the discussed NDTs complementary use is recommended.

For damage and defects assessment, the NDTs considered are less suitable for use during regular bridge inspections. Their biggest weakness is high complexity of the results' interpretation, followed by the on-site test duration. As large number of bridges need to be inspected daily, time consumption is of utmost importance giving these tests lower utility. Impact echo investigation is rated higher than other methods measuring damage and defects and is at least conditionally suitable for use during regular bridge inspections.

Based on literature review, all non-destructive methods dealing with corrosion detection and assessment exhibit similar characteristics (*Table 11.6*), consequently they all have similar utility function value. Their use in regular bridge inspection should be limited to bridges exhibiting high degree of corrosion only where reinforcement bars are already exposed, as most methods require direct contact with the reinforcement bars. Presented and additional types of NDTs with descriptions, principles of application, and general characteristic are discussed in (*SustainableBridges, 2007*).

11.2.2 Equipment use and maintenance

To ensure proper use of equipment, maximum possible accuracy and reliability of results, initial training of inspectors and regular calibration of equipment is required. The training should include:

- Theoretical background of the equipment used as this knowledge improves the interpretation of the data gathered.
- Display of all equipment capabilities to maximize its utilization.
- Use of equipment in practice (it should always be used in the same manner to obtain comparable results).
- Critical evaluation of the data gathered in order to obtain relevant and reliable information.

Quality education of inspectors is one of two key elements when assessing the state of a bridge with NDT. The other is assuring the quality of equipment used. The equipment needs to be regularly calibrated by an accredited organization. The type and the frequency of calibration depends on the type of equipment. The most common are one-year calibration periods.

11.3 Inspection Protocol

Bridge inspection methods diversity is a consequence of diversity of bridge designs, age, materials used, damages and deficiencies. To evaluate all the characteristics of a bridge, different types of inspections are required. For bridges in a good condition, visual inspections are sufficient to determine the values of their KPIs, while for other cases additional investigations are necessary to reliably determine the KPIs. As a part of QC plan, an inspection protocol is developed to ensure that the inspection data and the decision processes are at or above the desired level of reliability (e.g. performance goal) (*Figure 11.1*).

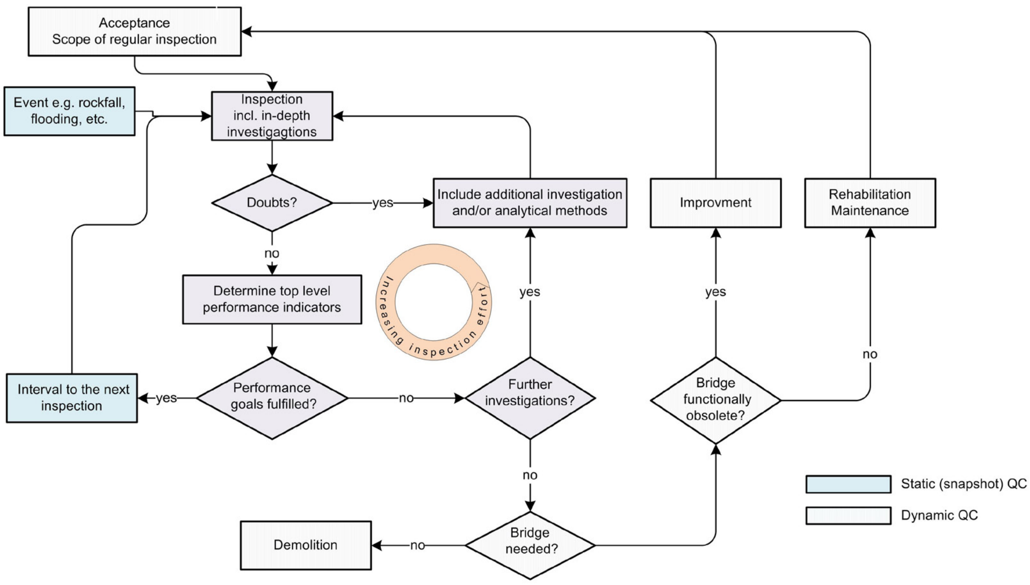


Figure 11.1 Example of an inspection protocol (VSS, 2018)

Visual inspections need to be performed in predefined intervals (see *Section 11.1*). In case of no doubts regarding the state of the bridge performance indicators values are determined and fulfilment of performance goals is checked. If they are fulfilled no further action is necessary until the next scheduled visual inspection. In case performance goals are not fulfilled further investigation might be necessary. The decision is taken jointly by the bridge inspector and the bridge owner as additional financial resources are needed for further investigations and/or analytical methods to be performed. If they are not undertaken several decisions regarding the bridge can be made by the owner. The bridge can be demolished, rehabilitated, improved or even replaced by a new one.

When the decision is made for further investigations to be conducted, in current practice a detailed inspection of the entire bridge is most commonly performed. In the case described we only have a two-stage process: visual inspection and detailed inspection. The implementation of the latter is usually very costly. The developed protocol proposes more inspection stages as only the performance indicators with poor data reliability or those affecting the performance goals not being fulfilled need to be addressed. This way substantial financial resources can be saved. After completing the additional investigation possible doubts and performance goals are assessed again. If the bridge condition is considered satisfactory the next inspection is performed as scheduled, if not an increased inspection effort is necessary and the displayed decision cycle is repeated (*Figure 11.1*).

It is not possible to develop an inspection plan that addresses requirements, procedures, practices and trainings to an extent where the data gathered and the decisions taken will have an impeccable precision. An element of interpretation in the inspection results will always be present and will therefore allow certain variations in the decision making, but this can greatly be reduced using the protocol presented. Complete, consistent and reliable observations from inspections are of utmost importance for determining needed maintenance and repairs, for prioritizing rehabilitations and replacements, for allocating resources, and for evaluating and improving design of new bridges.

12. Appendix

12.1 Other relevant data for performance evaluation

Ideally, a bridge inventory data should be available in the Bridge Management System (BMS) together with relevant parameters/data that are not strictly related to the structural type but are needed for establishment of QCPs (the list is not definite):

- Data on previous inspections/interventions
- Bridge age i.e. construction year (insight to historically used codes of practice)
- Clearance
- Environmental conditions
    - Location: costal, industrial, urban, rural
    - Microclimate
    - Hazard zone/exposure (flood, earthquake, landslide, etc.)
- Traffic
    - ADT (Annual Daily Traffic)
    - ADTT (Annual Daily Truck Traffic)
    - Possibility of detour
    - Load posting
- Inspection and maintenance aspects (incl. costs)

Data gathering may be performed using the guidelines (*LTBP, 2016*). The *Figure 12.1* presents an example to include the abovementioned data as an addition to the QCP protocol in *Table 8.3*.

Structure type	Other relevant data (can be expanded as needed)										
	Location	Environmental exposure	Natural hazard exposure	Underpass obstacles	Structural age/ building year	Overpass Annual Average Daily Traffic (AADT)	Set of informations related to the bridge original design and previous interventions performed on the bridge				
							Original design/documents and drawings	Previous inspection	Previous intervention		
								Date	Date	Type	Costs (kEUR)
FA	Urban	Atmospheric	None	*local traffic AADT 1000*	1968	9,554	link to any documents related top the original design and execution including drawings	(month/year)			

Figure 12.1 An example of other relevant data to be included in a QC protocol for performance evaluation

12.2 Key Performance Indicator scales

In this section, the qualitative scales related to KPIs of Reliability, Safety and Availability are given. There has been no attempt to align the three scales, e.g. transforming all scales into a monetary unit. The qualitative scales for Reliability also provide statements on urgency of intervention. For some observations, this cannot be evaluated from inspection of the current state (i.e. snapshot in time) but has to be paired with performance prediction models or future observations, as urgency of intervention is dictated by a specific damage process.

The scale for KPI of Reliability

The reliability is related to structural safety and serviceability. Assessment of reliability is not the same as assessment of a condition indicator, since the reliability:

- takes into account the “virgin” reliability (in some countries it is based on the load effects from the codes of practice at the time of construction - often spare capacity may be present in reality, remark: shear capacity was not well understood in older codes of practice)

- focuses on failure modes, and
- related vulnerable zones

For structures of a similar: span, structural type and cross section type, with respect to a similar/same dominant failure mode, reliability curves can be elaborated (*Figure 12.2*). Here, a qualitative Reliability KPI scale (e.g. *Table 11.1*) can be related to the quantitative results. Probabilistic modelling of the bridge resistance (R), bridge dead load (DL), and load effect (P), is based on recommendations from literature, e.g. (*JCSS, 2013*).

The significant input for modelling are percentages of live and dead load in respect to total loading. Here, the ratio of these loads is denoted as the parameter r. In *Figure 12.2*, two examples are given: r_1 = 25%/75% and r_2 = 5%/95%, for which the reliability indexes (blue lines) and probabilities of failure (red lines) are evaluated for different values of the resistance reduction. The first ratio would be appropriate for girder and frame bridges, while the second can be adopted for masonry arch bridges. For presentation purposes it is adopted that β=5.2 for the 100% of capacity in both cases (denoted as 1.0 on the horizontal axis). The additional capacity for some failure modes and vulnerable zones may extrapolate the diagram to the right, i.e. probability of failure can be even lower than illustrated.

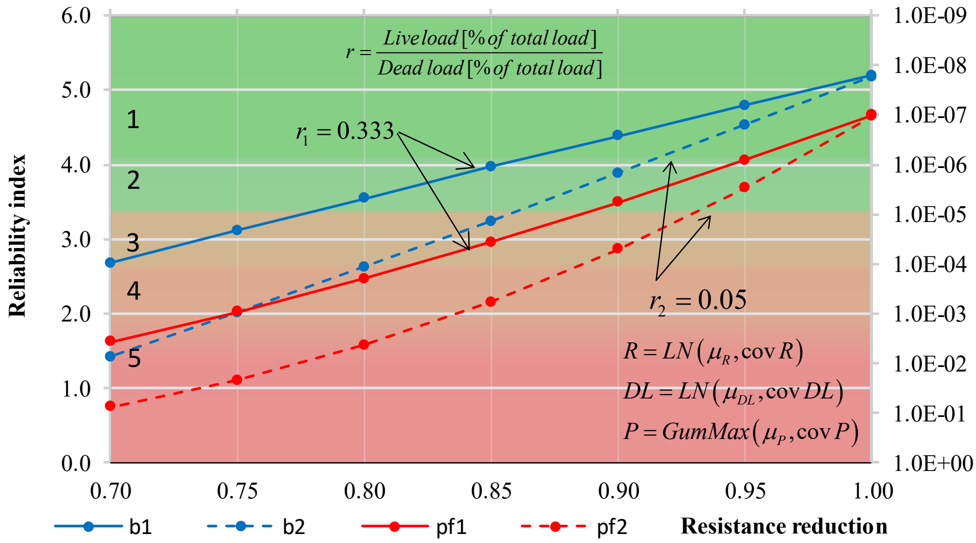


Figure 12.2 Influence of a resistance reduction on the reliability index and probability of failure

When estimating the virgin reliability, it is of the outmost importance to account for the bygone codes with limited/no knowledge on the adequate shear design. The known conceptual weaknesses and detailing issues for certain systems (e.g. poor splicing of reinforcement) should be duly considered as well.

An example of the correlation between the quantitative and qualitative performance indicator scale related to reliability is proposed in *Table 12.1*.

Table 12.1 Scale for KPI Reliability (structural safety) and urgency of intervention

Reliability scale	**Quantitative scale (β)**	**Urgency of intervention**
1	> 4.00	Regular inspection
2	3.25-4.00	Reassessment should be performed to update the period between inspections
3	2.50-3.25	Reassessment should be performed to plan an optimal time of an intervention
4	2.00-2.50	Reassessment and possible intervention shall be performed shortly after an inspection
5	< 2.00	Immediate action/intervention is required

The above written scales are valid when considering the governing failure mode (i.e. the most critical) and concern only structural safety. For serviceability (e.g. reduction/loss of functionality), similar definitions may be elaborated e.g. as in *Table 12.2*.

Table 12.2 Scale for KPI Reliability (serviceability) and urgency of intervention

Reliability Scale	**Quantitative Scale (β)**	**Urgency of intervention**
1	> 2.50	Regular inspection
2	2.00-2.50	Reassessment should be performed to update the period between inspections
3	1.50-2.00	Reassessment should be performed to plan an optimal time of an intervention
4	1.00-1.50	Reassessment and possible intervention shall be performed shortly after an inspection
5	< 1.00	Immediate action/intervention is required

The scale for KPI of Safety
An example of a qualitative scale, related to Safety, and related correlation between qualitative and quantitative values is proposed in *Table 12.3*.

Table 12.3 Scale for KPI Safety

Safety Scale	**Quantitative Scale**	**Qualitative scale**
1	Injury return period > 100 years	No danger. It is very unlikely that a person could get injured because of the current bridge performance.
2	Injury return period ~ 75 years	It is very unlikely that a person could get injured because of current bridge performance.
3	Injury return period ~ 50 years	It is unlikely that a person could get injured because of current bridge performance. Intervention shall be performed before next inspection
4	Injury return period ~ 20 years	It is likely that a person could get injured because of current bridge performance. Intervention shall be performed shortly after inspection.
5	Injury return period < 10 years	Immediate danger. It is very likely that a person could get injured because of current bridge performance. Immediate action is required.

The scale for KPI of Availability
An example of a qualitative scale related to Availability has been given in *Table 12.4*.

Table 12.4 The scale for KPI Availability

Availability Scale	**Qualitative Scale**
1	No restrictions to traffic
2	Weight, speed and lane restrictions for heavy trucks
3	Closure except for cars and regular lorries. Possible lane restrictions for regular lorries.
4	Closure except for cars. Possible lane restrictions for cars.
5	Complete closure

Duration of intervention could be included in the above scale. However, no attempt in this respect has been made. If Availability is monetised, as in many European countries, duration of intervention and its impact on user costs (discounted) is naturally considered.

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