TU1406

COST ACTION

Quality specifications for roadway bridges, standardization at a European level

WG4

TECHNICAL REPORT
PREPARATION OF A CASE STUDY

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1. INTRODUCTION

COST Action TU1406 has brought together research and practicing community in order to accelerate the establishment of a guideline for implementing performance-based bridge assessment. There were eight main objectives (COST, 2014) in order to reach the final target which was to develop a guideline for the establishment of Quality Control plans (QCP) in roadway bridges. Following the work done by working group 1, 2 and 3, it was important to develop detailed examples for practicing engineers, which was one of the main objectives of Working Group 4 (WG4): 'Implementation in a Case Study'. The roadmap of WG4 is shown in figure 1.1.

Different methodologies for obtaining performance indicators, as well as threshold values, were used as the basis for the benchmarking. The basis was already finished during the first three steps of the action, which included establishing the use of performance indicators (PIs) by working group 1 (WG1), definition of standardized performance goals (PGs), definition of threshold types to specific key performance indicators (KPIs) by working group 2 (WG2) and the preparation of guideline for the establishment of QCP in roadway bridges working group 3 (WG3). In order to enable the preparation of the recommendations for practicing engineers (WG5. Drafting of guideline/recommendations), WG4 has used the developed guidelines with real bridge case studies and evaluated the suggested methodology (COST, 2014).

Several documents, which are all available at the website, were issued by WG4 in order to try and unify the work as much as possible, including the following:

- 'Guide for Documenting Bridge Data'
- 'Bridge ID Data tables (Excel)
- 'Case study bridge selection, data collection'
- Guidelines for Preparation of a Case study' (see appendix B)
- 'Spidertool' for comparing scenarios (Excel, prepared by Prof. Rade Hajdin, WG3)

Following the preparation of 17 case studies which are presented in appendices A1 to A17 of this document, the cases were compared and the main conclusions from the process of preparation are presented in this document.

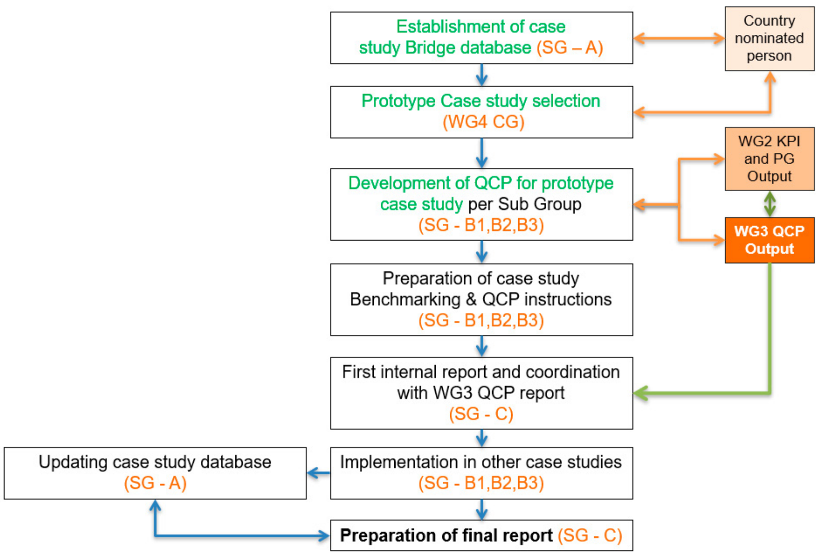

Figure 1.1. WG4 road map

2. COST TU1406 CASE STUDY DATA BASE

2.1. BRIDGE DATA BASE

The first task of WG4 was to identify a set of roadway bridges, located in different COST countries and to establish a bridge data base from which the most suitable case studies will be chosen. The case studies were selected carefully in order to represent correctly the most common topologies of highway bridges in use as suggested by COST TU1406 WG3 for implementation of the developed QCP methodology (WG3 Final report).

A bridge ID inventory was prepared. Each country's representative sent the ID form filled with data of three typical bridges, defined as candidates for the WG case studies. The three typical bridges were selected by each country according to predefined criteria (WG4 'Guideline for selecting case study bridges').

2.2. FIRST GROUP OF CASE STUDIES

After establishing the bridge data base, nine bridges were selected by the participants from WG4 for the preparation of the first case studies (figure 2.1). All Case studies were prepared according to the procedure described in paragraph 3 and added to the case study data base. The case studies were discussed with other work groups and some modified to better represent the process of preparing QCP as per the suggested methodology.

2.3. ADDITIONAL CASE STUDIES

After the preparation of the first group of case studies, a document named 'WG4 'Guidelines for preparation of a case study' was prepared (see appendix B) in order to unify as much as possible the process of preparing a case study. The document is available at TU1406 website together with the case study reports. The participating countries representatives were asked to contribute and prepare a case study according to these guidelines. To date, eight additional Case studies (figure 2.2) were prepared and added to the Data base. Additional case studies are expected in short time. The geographical location of the bridges is shown in figure 2.3.

We recommend the participation countries and specifically bridge owners, to use the list of selected bridges at their country (available at the website) in order to prepare their own first case study and check the applicability and the process suggested by COST TU1406 for the preparation of quality control plan for bridges.

No.	Country	Bridge Name	Bridge Type	Description	General Photo
A1	Czech Republic	**R4 most za obcí Dobříš**	Girder	One-span concrete pre-stressed girder structure. Carries the highway D4 across the local road III/10226. built in 1983	
A2	Greece	**Viotikos Kifisos bridge**	Girder	4 span bridge. Pre-stressed precast concrete beams, Location in Viotia prefecture, Central Greece. built in 1990	
A3	Greece	**Strimonas river bridge**	Girder	8 span bridge, pre-stressed concrete beams. Located in the north of Greece. built in 1987	
A4	Poland	**East Praski Canal Masovia (Warsaw)**	Girder	3 span bridge steel girders over Praski Canal, Warsaw, East side. built in 2001	
A5	Czech Republic	**Most přes řeku Skalice u obce Nerestce**	Arch	5 span concrete arch structure. Over the river Skalice. built in 1953	
A6	Bosnia and Herzegovina	**Carinski most, Mostar**	Arch	2 span Reinforced concrete Arch bridge situated in Mostar over Neretva river. built in 1916 reconstructed 1996	
A7	Portugal	**Guarda**	Arch	2 stone masonry arches, over the Ázere river, situated in a rural. built in 1953	
A8	Switzerland	**Unterführung SBB**	Frame	5 spans reinforced concrete structure. Rail Overpass connects Glattfelden with the major northern road to Zurich. built in 1941	
A9	Israel	**Joseph Vehicle Bridge -0002-089+0740-02/00**	Steel Frame	Single-span half-through steel truss bridge carries road no. 9779 across the Jordan river. built in 1956	

Figure 2.1. *First group of case studies*

No.	Country	Bridge Name	Bridge Type	Description	General Photo
A10	Estonia	**Piiometsa bridge**	Girder	2 span reinforced concrete bridge located on secondary road. built in 1963	
A11	Italy	**VE-TS-Rail overpass**	Steel Girder	A 4 and 5 span coupled steel girder highway bridge crossing VE-TS railway line in Veneto region, northeastern Italy. built in 1989	
A12	Netherlands	**A12 Vierlingbrug**	Girder	4 spans pre-stressed prefab-girders. Located in the A12 motorway nearby the city of Utrecht. Built in 1969	
A13	Spain	**C-58, PK 25+490**	Girder	3 span bridge prestressed girders. Located on the Viladecavalls stretch of the C58 road. Built in 1990	
A14	Turkey	**GÜNEY YAKLAŞIM VİYADÜĞÜ**	Girder	10 span bridge. Twin box orthotropic steel deck structure South Approach Viaduct located at south of Osmangazi. Built in 2016	
A15	Austria	**Brentenmais**	Girder	4 span prestressed concrete girder structure. Located on the motorway A1. built in 1963	
A16	Portugal	**Quintão bridge**	Arch	Masonry arch bridge with one single arch. Located on EN303 close to Viana do Castelo. built in 1900	
A17	Slovenia	**Motnišnica**	Girder	KA0040 bridge over Motnišnica river single span vehicle road bridge cast in-situ concrete girder bridge.	

Figure 2.2. *Additional group of case studies*

The case study bridges locations are presented on a map (figure 2.3) which will be continuously updated once additional bridges case studies received.

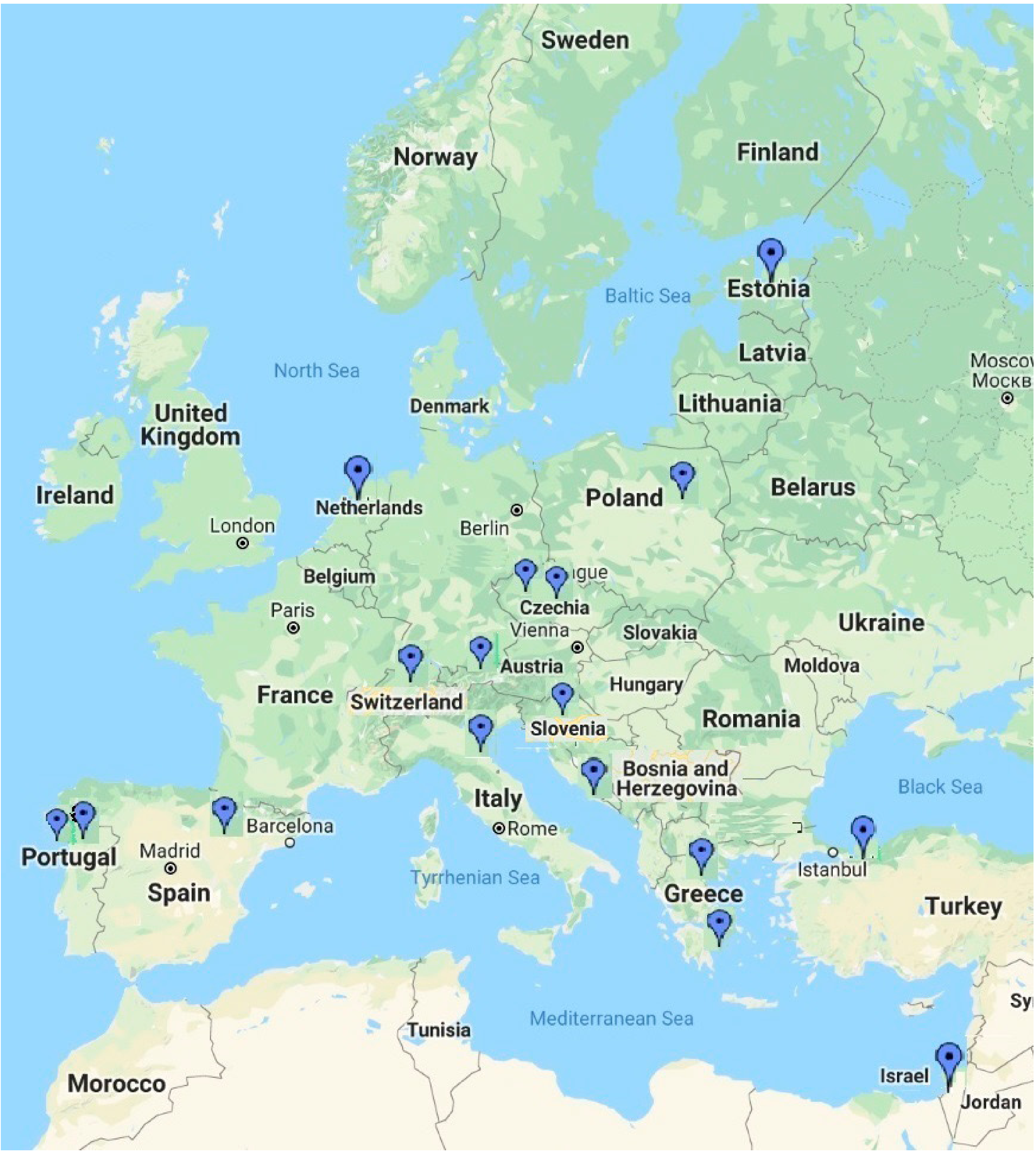

Figure 2.3. *Map of Case study locations*

3. PREPARING A CASE STUDY, PROCESS AND STAGES OVERVIEW

3.1. GENERAL DESCRIPTION OF THE PROCESS

The preparation process of the case studies was done according to the scheme described in figure 3.1. The content of each task was defined in the 'Guidelines for preparation of a case study' (Appendix B) and is briefly described in table 3.1.

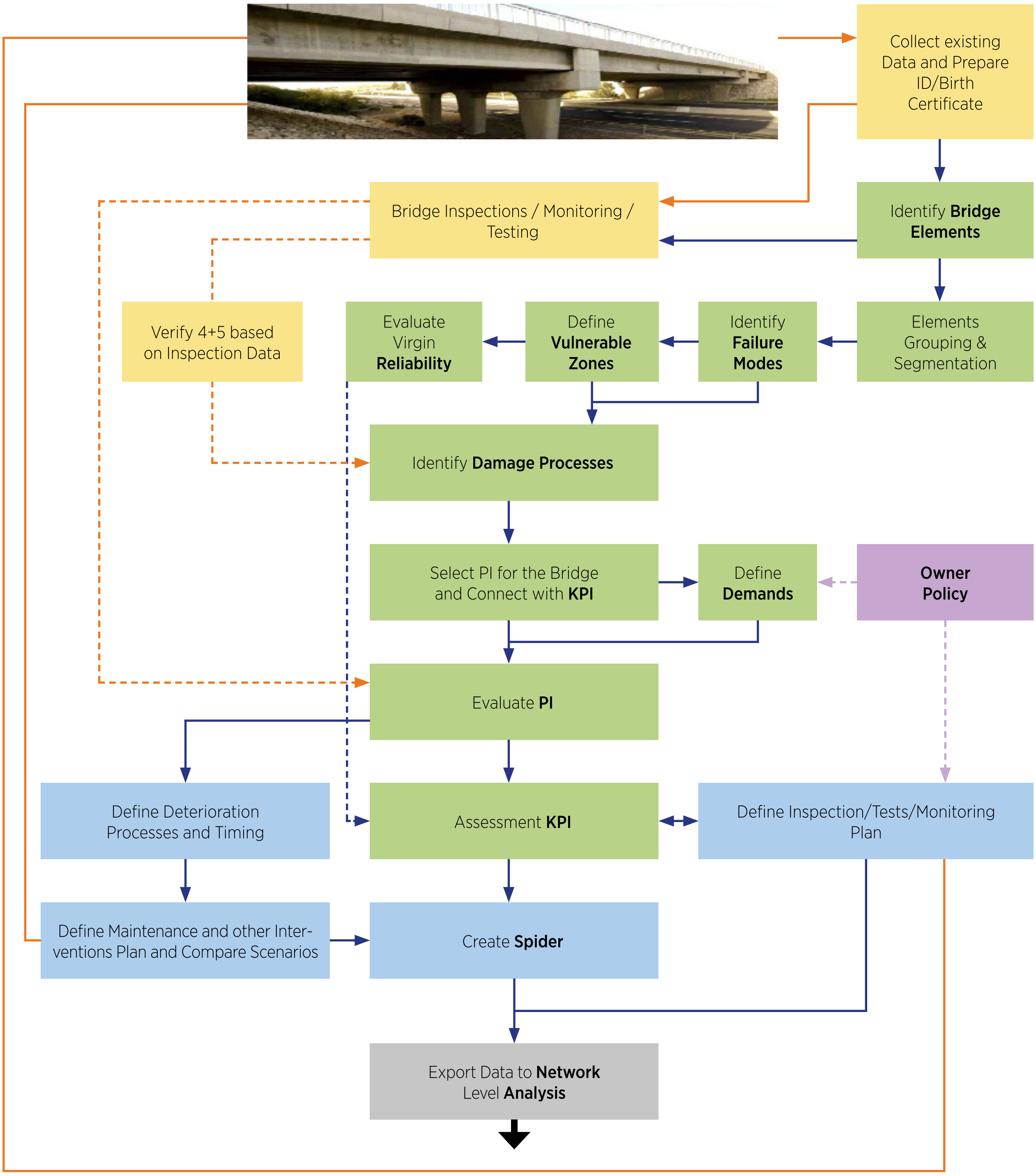

Figure 3.1. *Preparation process of Case study*

3.2. STEP BY STEP DESCRIPTION OF THE TASKS (BY STAGES)

According to the definitions set on WG3 report (WG3 report chapter 7.1) the listed tasks described in figure 3.1 were divided into two main groups: 'Static' and 'Dynamic'. Tasks no.1 to 10 are considered 'Static' represented by orange (site) or green (office) colours, while tasks 12 to 15 are 'Dynamic' represented by blue colour. In order to simplify the work of preparing the case studies examples, table 3.1 gives a short description of the work to be done and some useful references to previous work groups 1 to 3 reports and to additional explanations detailed in Appendix B.

No.	Task Name	Description of the work to be done	References*
1	Collect existing data and prepare ID/Birth Certificate	Prepare inspection by collecting existing data. Prepare/update a bridge ID/ birth certificate as per the format given in chapter 12 of this document. This information is relying on inventory data (If exist) and additional data acquired on site.	Chapters 2, 4 and 12. WG3 Report: Clause 12.1, Clause 8.5
2	Identify bridge elements	Identify all bridge elements and prepare a bridge element table using the defined taxonomy of TU1406. For each element document the dimensions and dimension units. Existing element list per country current practice can be transformed into the suggested format.	Chapter 4 and 12. WG3 Report: Girder & Frame Clause 8.1, Arch bridge: Clause 9.1 Example: Clause 8.5 Dimensioning: Clause 7.4 Case studies examples: Appendix A1 to A17
3	Elements grouping & segmentation	Arrange bridge elements by grouping together. Grouping can be according to different criteria such as geometry, functionality, materials, exposure etc.	
4	Identify failure modes	Use design documentation and define failure scenarios. For each scenario identify the possible failure modes, for example: rigid body movement (loss of stability), internal mechanism (shear, bending, ...), fatigue, functionality, comfort (to the user), visual appearance (to community), safety (falling parts) etc.	Chapter 5 WG3 Report: Clauses 8.3, 10.4.4, 10.4.5 Case studies examples: Appendix A1 to A17
5	Define vulnerable zones	Check for existence of conceptual weaknesses in the specific bridge type. Define and document the vulnerable zones on the bridge and correlate with the relevant failure mode. Documentation should include plan, elevations and sections as needed with marked positions of the zones and the relevant failure mode using WG3 defined labels.	Chapter 5 WG3 Report: Girder & Frame Clause 7.2 Arch bridge: Clause 7.3 Case studies examples: Appendix A1 to A17
6	Evaluate virgin reliability	If quantitively approach is selected, asses the "Virgin" reliability of the bridge using prototype and specific bridge, historical design data. Simplified or more precise models can be used.	Chapters 4, 8 WG3 Report: Clause 6.3, Clause 12.2,
7	Bridge Inspections	Perform on site visual bridge inspection with/without testing or monitoring. Inspection should be done taking into account the specific recommendations defined for the bridge prototype and the previously defined vulnerable zones and identified failure modes. Possible hidden defects/damages should also be investigated. Damages should be identified, compared with previous inspection results, documented and quantified by severity and extent. Documentation should follow WG3 report recommendations. The need for in depth investigation should be checked. Following the inspection, update the failure modes and vulnerable zones data from stages 4 and 5.	Chapters 6, 8. WG2 Report: Clause 3.1.4 WG3 Report: Clause 3.2, Clause 7.2.5, Clause 7.4 | Example: Clauses 8.5, 9.2 Chapter 11 Case studies examples: Appendix A1 to A17
8	Identify damage processes	Identify the damage processes on the bridge using the information collected during the bridge inspection and the predefined proposed damage processes as per WG3 report.	Chapters 7, 8. WG1 Report: Clause 4.2.1.1 WG3 Report: Chapter 4, Clause 5.2 Case studies examples: Appendix A1 to A17
9	Select PI for the bridge and connect with KPI	Select the appropriate PI and connect to relevant KPI considering the observations and connect with the damage processes (see WG3 report table 5.3).	WG3 Report: Chapter 5, Clause 5.2, table 5.3 Case studies examples: Appendix A1 to A17
10	Evaluate PI	Relevant PI should be selected for the bridge prototype (WG3) and for the specific bridge considering the specific scheme, materials and possible sudden events. The PI should be evaluated using predefined thresholds as per the owner demands (normally defined in the national professional guidelines).	WG3 Report: Clause 7.5, table 5.3, Clause 10.4 | Examples: Clause 8.5, 9.2 Case studies examples: Appendix A1 to A17
11	Assessment of KPI	Qualitative assess the resistance reduction based on the observed damages. Evaluate reliability and safety KPIs based on agreed methods ranging from simple "Engineering Judgment" to complex Bayesian Nets. Use suggested WG3 QCP protocol for performance evaluation and derivation of the KPIs from PIs. All KPIs should be normalized. Cost should be scaled based on the maximum yearly cost of all scenarios.	WG2 Report: Chapter 3 WG3 Report: Clause 7.5, table 5.3, Clause 10.4 | Examples: Clause 8.5, 9.2 Clause 12.2 (scale) Case study example: Appendix A7 clause 3.1

12	Define Deterioration processes and timing (time to failure)	Following the evaluation of the different PI and KPI assess the remaining service life i.e. the point in time at which Reliability or Safety will reach the defined threshold value (unacceptable return period for a failure) without any intervention. This includes assessment of the speed of the identified active deterioration processes and damage forecast. For each documented damage, indicate the relevant damage process and estimate the time to failure and document on the PI/KPI evaluation table. The assessment can use known existing models for deterioration in time or simple expert opinion.	Chapters 8, 9. WG3 Report: Clause 7.5, Clause 7.10, Clause 8.3 | Examples: Clause 8.5 Case studies examples: Appendix A1 to A17
13	Define Inspection/tests/ monitoring plan	For the reference scenario and for other preventive scenarios define the inspections type and intervals. For each inspection define the cost (as annual cost). Estimate the future type and timing for NDT/DT testing and monitoring with the related costs.	Chapter 10. WG3 Report: Clause 11.2, table 11.6, clause 12.1
14	Define maintenance and other	Define several maintenance scenarios with target reliability and safety over time. Define the time frame (for how many years). Estimate the cost of the different interventions per each scenario over time and combine with the costs estimated on stage 13. Define the function of decrease of Reliability and safety. For each scenario create graph per KPI (R, E, A, S) over time (excel file of WG3 can be used). All KPIs should be normalized (range 1 to 5).	Chapter 10. WG3 Report: Clause 7.5, Clause 7.6, | Examples: Clause 8.5, 9.2, tables 12.1 to 12.4 Case studies examples: Appendix A1 to A17
15	Create Spider	Create Spider diagrams of net present KPI for the scenarios and compare. This stage can be done for single point in time (spider) comparing the areas of the different scenarios spiders or as a continues process with preparation of 3D volume shape showing the change of the KPIs over time (few spiders). In such case the volume of the 3D shapes created for the different scenarios should be compared.	Chapter 10 and Appendix A1 to A17 WG3 Report: Clause 7.5 | Examples: Clause 8.5, 9.2,
16	Export data to Network level analysis	Part of the data should be used for "Network level analysis". The data format and the decision regarding the needed parameters rely on the network analysis method. A possible example using "Multi-objective optimization models" is given in WG2 Report.	WG2 Report: Chapter 5

*Note: references are coloured by WG 1-3 and Appendix B. (WG1 = Orange, WG2 = Blue, WG3 = Green, appendix B= Black)

Table 3.1. *Staged process of the preparation of a case study*

3.3. CASE STUDIES CONTENT (COMPARATIVE)

Table 3.2 is summarizing the content of each bridge case study. In some of the columns a three grade scale (1=comprehensive to 3=Brief) is used in order to evaluate the quality of the data presented in the case study report. In other columns it is just stated if the subject is covered/presented (yes/no). Where expert judgment is used, we mark E where model is used, we mark M.

Case study	General description of the Bridge	Traffic information	Foundations	Substructure	Superstructure	Accessories	Load capacity	Existing Condition rating	Vulnerable zones data	Technical condition (defects description)	Failure modes	NDT	Reliability assessment Method	PI	KPI	Number of Scenarios	Deterioration model	Comparing by Spider	Comparing by 3D
A1	2	yes	No	3	2	3	1	1	no	1	1	2	E+β	2	1	2	E	yes	yes
A2	1	yes	No	2	2	3	no	no	no	1	1	no	E	1	1	2	E	yes	no
A3	1	yes	1	1	1	1	1	1	3	1	1	1	E	1	1	2	E	yes	yes
A4	2	yes	3	3	2	1	1	2	1	1	1	no	E	1	1	2	E	yes	no
A5	2	yes	3	3	3	3	1	1	no	1	1	2	E	2	1	2	E	yes	yes
A6	1	yes	2	2	2	3	3	2	1	3	no	no	E	2	1	3	E	yes	no
A7	2	yes	3	3	3	3	no	no	1	1	1	no	E+β	2	2	3	M	yes	yes
A8	1	yes	No	3	2	3	1	no	3	1	1	1	E+α	1	1	2	E	yes	no
A9	1	yes	1	2	1	1	1	1	1	1	1	1	E	1	1	2	E	yes	no
A10	1	yes	3	2	2	3	1	1	1	1	1	1	E+β	1	1	2	E	yes	no
A11	2	yes	3	1	1	3	3	1	1	2	1	no	E	1	1	2	M	yes	no
A12	1	yes	1	1	1	1	no	1	1	2	no	no	E	1	1	2	E	yes	no
A13	1	yes	3	2	1	3	no	no	1	2	1	no	E	1	1	2	E	yes	no
A14	1	yes	1	1	1	1	2	1	1	2	3	no	E	3	3	2	E	yes	no
A15	2	yes	2	2	2	2	1	1	1	1	2	no	E	1	1	2	E	yes	no
A16	2	no	3	3	3	no	no	3	3	1	3	no	E	1	1	2	E	yes	no
A17	2	no	3	3	2	2	1	1	1	1	1	3	E	1	1	2	E	yes	no

Table 3.2. *Case studies content and ranking*

4. QUALITY AND QUANTITY OF THE DATA COLLECTED AND USED

Seventeen case studies were developed by 15 countries. In most of the examples very clear data regarding the general description of the bridges was given. The quality of technical description of the damages was of similar nature; however, the quantity of data was rather different. Failure modes and vulnerable zones were elaborated in different ways, from very scarce descriptions to detailed sketches. This is something that should be unified. Depending on the state of the bridge, different non-destructive and minor destructive tests were conducted for determination of the mechanical and physical characteristics of materials from which the bridge was constructed. On some bridges dynamic tests were done in order to determine the dynamic characteristic in order to determine the frequencies and Eigen modes, which could be connected to the existence or non-existence of structure damages and some kinds of deterioration. This data is rather variable in range and detail.

Some case studies included numerical models and some did not. How detailed should the case study be? If we are to include numerical analysis it is necessary to define what kind of numerical analysis (linear, non-linear) is recommended to be done, which will of course depend on the state and importance of the bridge in question. In this respect maybe a distinction should be made regarding the state and importance of the analysed structures. The sub-chapter “current state evaluation” as one of the major inputs in the report, is presented from poor to excellent quality. It is believed that this part of the report should be clearly defined and presented in the same manner for all the case studies. In this respect, the following general data is recommended:

- General identification data
- General classification data
- Service data
- Basic geometrical data
- Structural classification data
- Material classification data
- Loading classification data
- Bridge hydraulics data
- Existing Bridge performance indicators data
- Existing QC Plan data
- Bridge inspection data, representative pictures current and historical to identify deterioration in time (See appendix A3).

It is important to collect all available data from the design phase or from in-depth investigation. In case studies where this type of data was available it was easier to quantify the reliability KPI. Existing specific data should be collected including the following:

- Bridge drawings (originals or from other data source (e.g. from survey)).
- Inventory data items.
- Current and/or future loading on the structure
- Bridge static calculations (if available) or previous capacity assessments.
- Specific hazards data related to the bridge (Scour data, Seismic data, Geotechnical data, Special heavy load transportation data, etc.).
- Equipment properties and types (Bearings type and manufacturer, Expansion joints type, Safety barriers type etc.)

Environmental conditions are of physical, chemical or biological nature and can influence material properties (Rücker et al., 2006).

The minimal parameters to be investigated within the proposed COST TU1406 framework for load bearing elements are well defined in WG3 report and WG4 guidelines and should be closely followed. In addition to visual inspection it is suggested to use laboratory equipment in observations and investigation of damage processes (Rücker et al., 2006): Cross sectional and longitudinal geometry changes (defects) from overloading (cracks, ruptures etc) and from deterioration processes (corrosion, spalling, fatigue cracks etc). It is possible to detect these processes using laser, ultrasonic devices, slide gauge, electronic gauges, etc.; Structural integrity (e.g. for hidden damage or inhomogeneity) is possible to detect with impact echo testing; Material strength using tension and compression tests on samples, sclerometer method, pull-out tests, pull-off tests, etc.; Parameter, influencing the dead load and the superimposed dead load (e.g. material densities, permanent equipment); Duration influencing parameters of the structure (e.g. environmental conditions, carbonation and chloride content of concrete) using pH-test, phenolphthalein test, quantitative chloride analysis on samples, etc.; Serviceability matter (e.g. crack widths, surface conditions of roads). It is important that the results of the data acquisition should be of the same form, to be able to compare data from different methods and to be able to use data in future assessment procedures (Rücker et al., 2006). Elements related to equipment are related to nearly all bridge types. The lists are related to bearings, expansion joints, waterproofing, pavement/overlay, barriers and signs.

During the process of bridge inspection, it is important to record and measure in quantitive way the different observations (defects) and correlate them with the relevant PI or just define as symptoms. This is done in a satisfactory level in most of the cases, however, in some of the bridges the summary table of the PI and observation is not detailed enough.

The selection of the appropriate PI and connecting to relevant KPI considering the observations and connect with the damage processes was generally done in a properly way. For a network of bridges it is recommended that the relevant PI be selected in advances for each bridge prototype and for the specific bridge, considering the specific scheme, materials and possible sudden events. The PI should be evaluated using predefined thresholds as per the owner demands (normally defined in the national professional guidelines).

The main area where clear differences where found between case studies, is the estimating of virgin reliability and future change of reliability in time (by scenario). In most of the case studies an expert judgment was used. Expert judgment by itself can be accurate to a certain level depending on the experience of the engineer; however, it is very much dependent on the local practice and scale in use. In WG3 report, the use of β values is suggested; however, when looking at the actual work done, only in 4 out of 17 case studies a calculation of β was performed and by using different methods. It is clear that currently the use of β as the main parameter for evaluating the reliability is not well understood by practitioners and to some degree also by others. In order to use it, it is necessary to develop additional detailed examples.

5. DEVELOPING POSSIBLE MAINTENANCE SCENARIOS

Maintenance scenarios are defining the complex care of the bridge during its service life. The detailed approach for the creation of maintenance scenarios in a case study were defined in Appendix B. The maintenance scenarios are very specific for each country and bridge owner, some general recommendation used in some of the European countries can be found in the results of the SBRI+ project. The results of the project were used in appendix B for demonstrating how to establish the maintenance scenarios (Lemma et. al.).

General recommendations, based on the experience gained from the case studies, are listed here:

a. At least two, but preferably three (maximum in the case studies were 5 scenarios) maintenance scenarios with target reliability and safety over time should be specified.

b. Based on the specific data and age of the bridge, decide in advance regarding the preferred time frame, most common is the frame of 100 years, as it is the design life of the bridges in EU. Shorter time may also be used as in some old bridges the design life is nearly at its end or in bridges where changes to the road geometry is due in shorter time.

c. Preventive Scenario: A scenario with a 100-year service life, according to the normal service life of bridges, for which there will be enough money to undergo necessary inspections and maintenance/repair actions;

d. Referenced Scenario: A reference scenario must be defined based on minimum intervention approach (do nothing, or do only necessary repairs), it can be called as 'lack of money' scenario as well. The bridge will be critically deteriorated with significant reduction of its reliability and creating traffic restrictions. Inspection frequency will have to be increased in the last years for the knowledge of the actual bridge condition, and maintenance actions are introduced to extend the service life of some critically deteriorated elements;

e. The detailed content of the interventions should be defined. In most case studies it was based on engineering judgement of the engineer and the local experiences (owner policy, typical intervention types, etc.). It can also be used on the more detailed analysis; however, prediction of the deterioration is still a difficult task and was referred to in details only in few case studies.

f. For a preventive scenarios it is advisable to group together (by time) different tasks and create a periodical repeated intervention with the estimated cost and influence on availability. For example, the change of the waterproofing is usually connected with the change of the pavement, replacement of bearings can be grouped with expansion joint treatments, safety barrier and railing painting is usually combined with parapets rehabilitation etc.

g. The cost of the different interventions per each scenario over time should be based on the local country practice (usually, the cost index is defined in most of the European countries); however, basic recommendations are included in Appendix B. It is very important to specify, how the costs are defined, and to specify and explain the quantities in details. The cost for the traffic restrictions and cost for the individual reconstruction works must be calculated individually according to the experiences of the engineer. Usually, the standard prices are defined in many European countries. Also, it should be noted, that the repair of one element can results in the replacement of other elements. For example, replacement of the waterproofing means also to replace pavement and sometimes the parapets.

h. In both scenarios it is essential to specify the remaining service life of the bridge structural component. Also, the scenario and the actions must be carefully described, so that the assumptions are clear. This is essential, and in all case studies slightly different values were considered, as the environment and experiences in the countries varies. It is recommended not to use the service life directly from the past experiences (as it can be influenced by the local low-quality construction, such as communistic times in the East European countries), but to consider the real expectation based on the nowadays quality and standards (for example the existing cover of the reinforcement is significantly higher, than the one used in the past).

i. The costs of the maintenance, inspections and testing/monitoring should be also included. However, as those costs are usually smaller compared to the construction prices, they are not the most important in the comparison between the scenarios. In some case studies the reference scenario (do nothing) included a higher cost for inspection and laboratory tests as the bridge needs a closer monitoring when reliability goes down.

j. The KPI graphs (R, E, A, S) over time should be created. The value used should be normalized. The Spidertool excel file provided by WG3 is a basic good tool and can be developed more in needed. For Super KPI like the 3D spider option, no tool was provided, but AutoCAD can be used and the graph line can be easily converted to polyline and drawn in 3D. Alternatively it can be programmed in different software like 'Mathematica' or similar tools. It is important that the same direction of the scale in 3D (smaller number – better performance and smaller price – or vice versa) is used, so that the volume of the 3D spider can be compared between scenarios.

k. The current way of normalizing the Cost KPI (WG3 Spidertool Excel file) is somewhat biased as it gives a very small difference between scenario in most of the case study. It is recommended considering a change in the normalization methods of this KPI so the differences in Cost KPI will have more effect on the calculation of the Spider area which is the target function of the comparison. This should be carefully examined with WG3.

6. SELECTING THE OPTIMAL SCENARIO FOR A CASE STUDY

As mentioned in chapter 5, the correct definition of at least two alternative maintenance scenarios is required in order to derive the optimal maintenance scenario. For the definition of the 'referenced' maintenance scenario, the realistic prediction of the evolution of the triggered failure modes, versus the remaining life of the bridge, is needed, in order to determine until when the “do nothing” strategy is still acceptably safe for the structure and for the user. The degree of success in case of earlier interventions and the deterioration rate of the rehabilitated bridge are required for defining correctly the 'preventative' maintenance scenario.

The above are necessary for calculating the Reliability KPI. Availability and Safety KPIs can then be calculated sufficiently as they are well assigned to the reliability. In cases where the bridge doesn't exhibit apparent structural defects, such as to trigger one or more failure modes, it seems from the respective case studies that it is really difficult by the lack of adequate data to define the alternative scenarios which will be different and not practically similar. Also, the ratings of the performance of a non deteriorated bridge are prone to errors, subjectivity and conservatism. As a conclusion, the application of the method in those bridges is probably limited to the evaluation of serviceability and for the accessories.

The costs expected, for rehabilitating the bridge for different times and states of damage, shall be estimated based on adequate existing data. The life time costs shall be calculated and rated (Cost KPI) in a t way that will represent the real future needs. Otherwise, the superiority of the optimal scenario would be hidden, considering also the subsidising of the referenced life costs to be spent later, by calculating their net present values.

If the above are well addressed, then the scenarios can be compared as follows:

- ***a.*** Comparison of the bridge performance separately for each KPI, for a specific time point along the remaining life of the bridge.
- ***b.*** Comparison of the average bridge performance for each KPI, over the whole remaining life of the bridge.
- ***c.*** Comparison of the net present value of each KPI, over the whole remaining life of the bridge.
- ***d.*** Indirect combined comparison of the spidergram of the net present values of all four KPIs.
- ***e.*** Direct comparison of the volumes of the life time 3D spidergrams, as a Super KPI, as a clear direct method of comparing the two scenarios depending on their results to keep ratings of all four indicators simultaneously as higher as possible. It is to be noticed that in the case studies where this SKPI was calculated, it was easier to distinguish between the examined scenarios and choose the optimal one. The previous is not true when the indirect comparison using the spidergram of the net present values of d. was applied, where the optimal scenario's superiority was marginal.*

*The method of calculating the net present values of non economic KPI is not justified by engineering point of view. Spending later for maintenance is obviously preferable but keeping a structure reliable, safe and available it is desirable all the time and maybe more in the end of the bridge life time.

A general view of the scenarios compared in each case study is presented in table 6.1. In most case studies the preventative scenario was significantly better if we take into the account all KPI together (Availability, Safety, Reliability, Cost). In terms of the costs, in most of the cases the costs were almost the same or the referenced was slightly better. These results are not surprising as in most of the referenced scenarios (do nothing and repair) the reliability decrease dramatically in time while the total cost is somewhat similar or with small difference. In such case the spider area over time will be smaller (non favorable).

No.	Name	Country	No. of scenarios	Best scenario - globally	Best scenario – total costs
A1	R4 most za obcí Dobris	Czech	2	Preventative	Preventative
A2	Viotikos Kifisos bridge	Greece	2	Preventative	Reference
A3	Strimonas river bridge	Greece	2	Rehabilitated	Rehabilitated
A4	East Prazski canal	Poland	2	Preventative	Comparable
A5	Most pres reku Skalice u obce Nerestce	Czech	2	Preventative	Preventative
A6	Carinski most, Mostar	BiH	2	Preventative or referenced – two versions of the QCP	Comparable
A7	Guarda	Portugal	5	Corrective plus Preventative	Corrective plus Preventative
A8	Glattfelden SBB	Switzerland	2	Preventative	Reference
A9	Joseph Bridge	Israel	2	Preventative	Reference
A10	Pilometsa bridge	Estonia	2	Preventative	Reference
A11	VE-TS Rail overpass	Italy	2	Preventative	Reference
A12	Vierlingbrug	Netherlands	2	Preventative	Reference
A13	C58, PK25+490	Spain	2	Preventative	Comparable
A14	Guney Yaklasim Viyadugu	Turkey	2	Preventative	Reference
A15	Brentenmais	Austria	2	Preventative	Reference
A16	Quintao bridge	Portugal	2	Preventative	Comparable
A17	KA0040 Motnisnica	Slovenia		Corrective plus Preventative	Reference

Table 6.1. *Overview of the case studies scenarios*

7. SUMMARY AND FINAL CONCLUSIONS

Seventeen case studies from fifteen different COST countries were developed based on previous COST TU1406 WG1 to WG3 work and are presented in appendices A1 to A17 of this report. The case studies are demonstrating the applicability of the suggested QCP methodology using expert judgement, semi-quantitative approaches and advanced modelling. The number of works submitted (with some other on the way) shows clearly that the use of performance-based quality control, including obtaining of new performance indicators and comparing them with goals, is possible.

Some of the case studies were presented during the owners meeting held in BASt, Germany, with good acceptance. It is also evident, that the incorporation of new key performance indicators and changing of some of the current practice will requires additional effort from owners or operators in order to have a broader effect on the existing bridge network. The presented examples described how the QCP was implemented in different countries with different level of knowledge and amount of collected data.

The main advantages demonstrated from the case studies are:

a. Using vulnerable zones concept makes a difference between the current general inspection practice which is not focused on what can happen to the bridge and inspections that are zone oriented.

b. Defining failure modes and failure scenarios is helping to put the focus of the inspection work on real important deterioration processes which in combination with the vulnerable zones will give better estimation of the bridge ability to withstand the increasing loads over its design life.

c. Connecting PI in a direct way to KPI and estimating the remaining time to failure for specific component are elaborating the connection between the current 'defects' and their future possible influence.

d. Using at least four KPI's (Reliability, Availability, Safety and Cost) instead of the common single 'condition' index, which is the current practice in many countries; give a better understanding of the results of planed maintenance scenario over the bridge design life. This new approach will enable to create maintenance scenarios that will better use the already limited resources available to the bridge owner and to justify the need for maintenance and rehabilitation budget with focus on longer period.

e. The differences between the case studies are demonstrating that the suggested QCP methodology is flexible. Different level of adoption is possible for different bridge owners.

Additional efforts needed:

f. Based on the case studies examples, moving from estimating the reliability KPI by qualitative expert judgment to a more quantitative approach are not well understood by practitioners. Additional explanation and detailed examples together with training is needed in this field.

g. Calculating 'virgin reliability' for a bridge requires engineering knowledge. It is recommended that owners will initiate the process of calculating this parameter by using the services of bridge designers after defining the methodology. This can be treated as one time project which shall be executed for the entire network.

h. The quality and type of data needed for estimating the reliability should be collected and properly stored. Design data shall be stored and be available for the inspectors and designers involved.

8. REFERENCES

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9. APPENDICES

Appendix A - Case study reports (attached as separate file)

- Appendix A1 - Girder bridge in Dobříš, Czech Republic.
- Appendix A2 - Girder bridge, Viotikos Kifisos bridge, Greece.
- Appendix A3 - Girder bridge, Strymonas bridge, Greece.
- Appendix A4 - Girder bridge over the Channel of the Prague Port, Warsaw, Poland.
- Appendix A5 - Arch bridge in Nerestce, Czech Republic.
- Appendix A6 - Arch bridge, Carinski Bridge, Mostar, Bosnia and Herzegovina.
- Appendix A7 - Arch bridge in Guarda district, Portugal.
- Appendix A8 - Frame bridge SBB Glattfelden, Switzerland.
- Appendix A9 - Truss bridge, Joseph bridge over the Jordan River, Israel.
- Appendix A10 - Girder bridge, Pilometsa bridge, Estonia.
- Appendix A11 - Girder bridge, VE-TS-Rail overpass, Veneto region, Italy.
- Appendix A12 - Girder bridge, A12 Vierlingbrug, Netherlands.
- Appendix A13 - Girder bridge, C-58, PK 25+490, Viladecavalls Spain.
- Appendix A14 – Girder bridge, Guney Yaklasim Viyadugu orthotropic steel deck, Turkey.
- Appendix A15 - Girder bridge, Brentenmais, Austria.
- Appendix A16 - Arch bridge, Quintao bridge, Portugal.
- Appendix A17 - KA0040 bridge over Motnišnica river - Slovenia.

Appendix B – Guidelines for preparation of a case study (SPIDER excel file attached separately)

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TU1406 WG4 Final report Appendix A1

Bridge Case study: Girder Bridge in Dobříš - Czech Republic

WG4

TECHNICAL REPORT
PREPARATION OF A CASE STUDY

February 25, 2018 (updated 1-2019)
Impressum by **boutik**.pt

Editors / Authors
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1. GENERAL DATA ON THE BRDGE

The inspected bridge is a one-span concrete structure built in 1983. The bridge carries the highway D4 across the local road III/10226 close to Dobříš town. General views of the bridge are presented below.

Figure 1. *The view under the bridge*

Figure 2. *Side view of the bridge (right side)*

Figure 3. *A view along the road in the Prague direction*

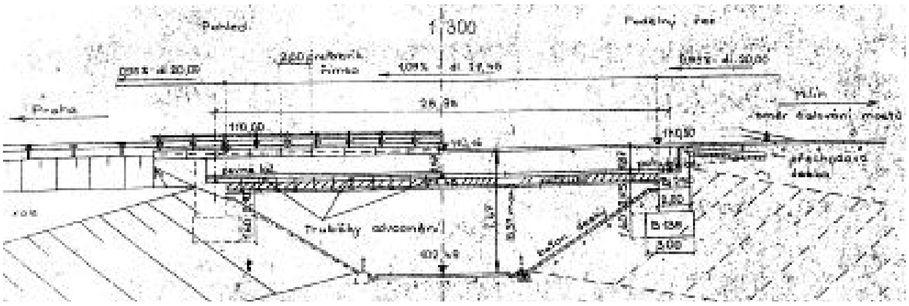


Figure 4. *Elevation of the bridge*

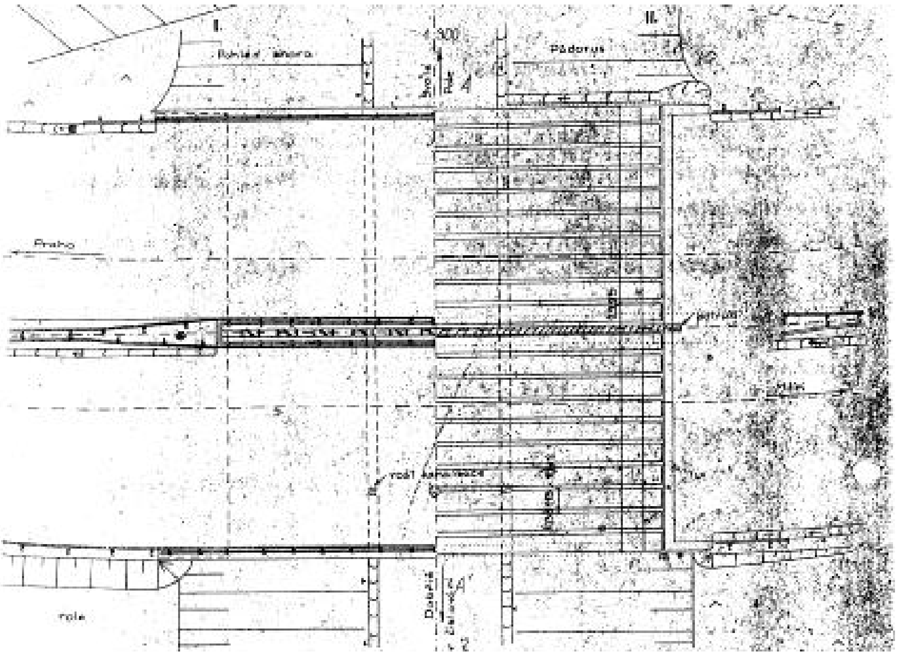


Figure 5. *The plan of the bridge*

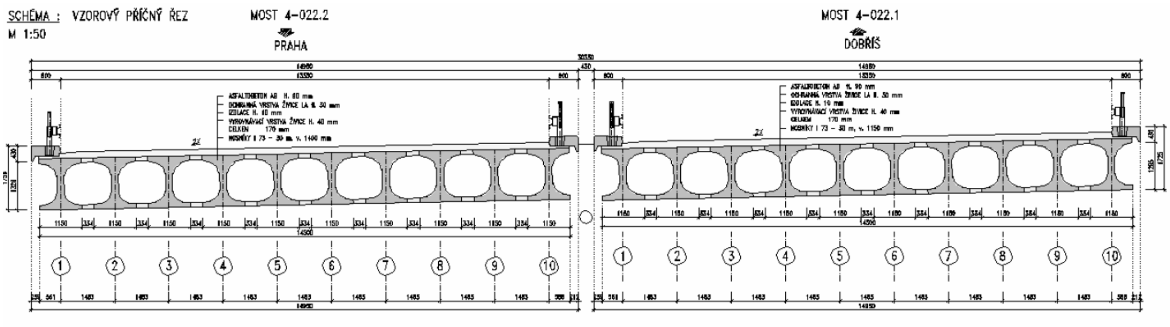


Figure 6. *General cross section of the bridge*

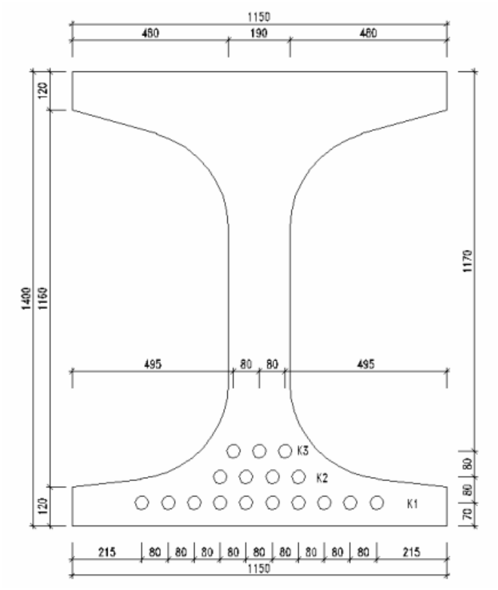


Figure 7. Cross section of the I73 girder

1.1. TRAFFIC INFORMATION

The last information about the traffic are from the last counting in 2010.
Number of cars / 24h: 20306
Number of heavy cars / 24h: 3868

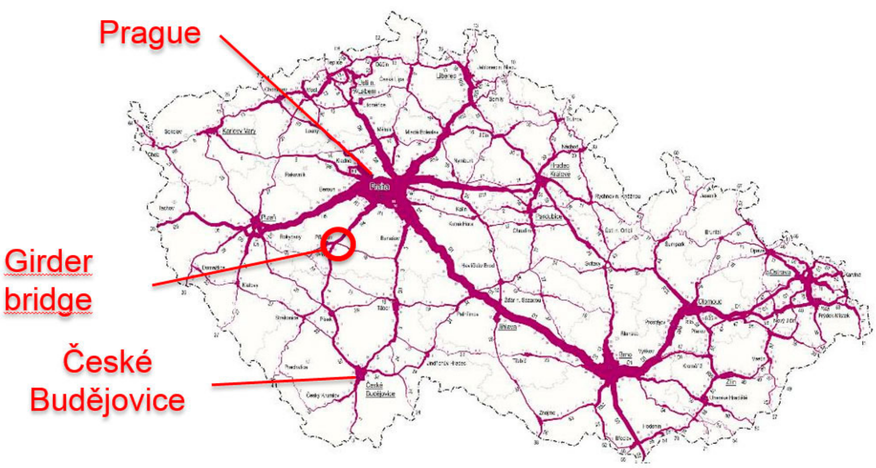


Figure 8. Location of the bridge on the map of traffic intensity

1.2. FOUNDATION

Foundations are inaccessible, and there are no existing precise drawings, showing them. According to to the sketches from BMS we expect there are pad foundations.

1.3. SUBSTRUCTURE

Substructure is formed by the abutments from the concrete.

1.4. SUPERSTRUCTURE

The superstructure is divided to two parts, there is a independent superstructure for each traffic direction. Each superstructure is formed by 10 precast and prestressed I73 girders. Each girder is supported on steel bearings, one fixed and one movable.

1.5. ACCESSORIES

There is asphalt pavement on the bridge. The walkway is made from concrete and equipped with steel crash barrier integrated with steel railing. The drainage is done on the bridge sides and water is drained out of the structure.

1.6. LOAD CAPACITY

The load capacity of the bridge is considered as:

- Normal capacity of the unlimited number of vehicles: Vn = 24 t
- The capacity of the one single vehicle on the bridge: Vr = 53 t
- Exceptional capacity for the heavy special transport: Ve =292 t
- Critical member is a side beam and its bending capacity.

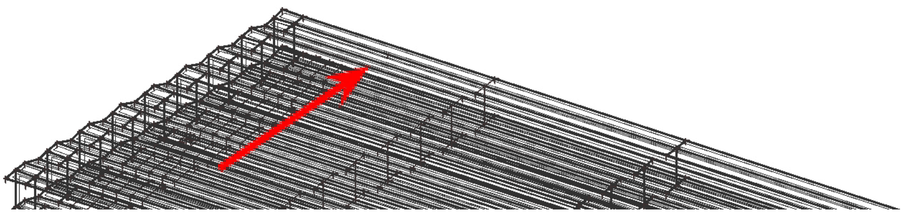


Figure 9. *The view on the numerical model for the load capacity calculation – shell model.*

1.7. RATING OF THE BRIDGE

According to the Czech rating system, the status is V (bad) for the superstructure and IV (satisfactory) for the substructure, on the scale between I (excellent) and VII (emergency). The availability is of the grade 2 (available with limitations) on the scale between 1 (available) and 5 (Unavailable).

2. TECHNICAL CONDITION

2.1. COLLECTION OF DEFECTS

The types of defects discovered on the analysed bridge are:

- Concrete deterioration and the reinforcement corrosion of both abutments
- Concrete deterioration and the reinforcement corrosion of main girders.
- Defects of expansion joints
- Waterproofing defects,
- Deterioration of the concrete parapets (ASR)
- Bearings damage

All the defects on the main members are presented on the sketches below.

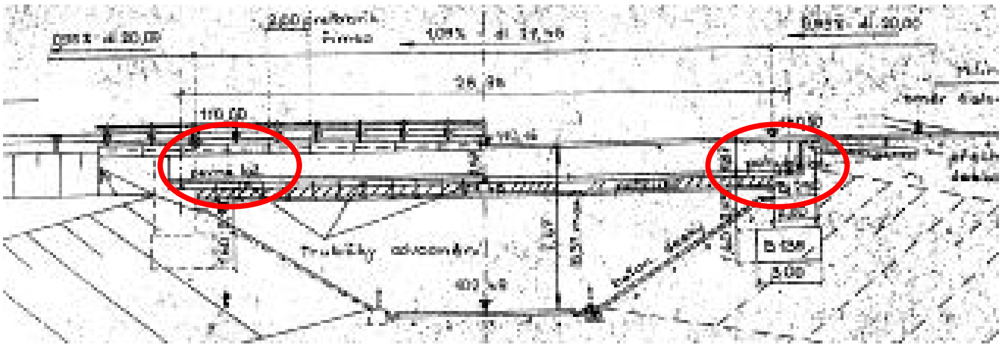


Figure 10. *The location of the most deteriorated parts, due to the water leaking*

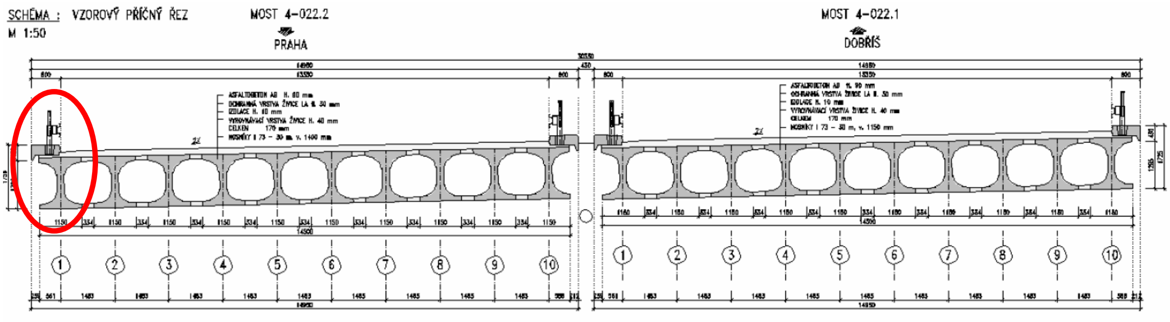


Figure 11. *The location of the most deteriorated beam*

2.2. DEFECTS OF THE MAIN STRUCTURAL ELEMENTS

2.2.1. CONCRETE DETERIORATION AND THE REINFORCEMENT CORROSION OF BOTH ABUTMENTS

Figure 12. *Deterioration of the abutment – concrete spalling, reinforcement corrosion*

Figure 13. *Deterioration of the abutment under the bearing, bearing corrosion*

2.2.2. CONCRETE DETERIORATION AND THE REINFORCEMENT CORROSION OF MAIN GIRDERS.

Figure 14. *The water leaking through the expansion joint, crack between precast and in-situ casted concrete of the main girder*

Figure 15. *The corrosion of the reinforcement and prestressing cables at the end of the side girder*

Figure 16. *The corrosion of the reinforcement and concrete deterioration of the side main girder.*

Figure 17. *The poor quality of the side main girder*

Figure 18. *The view inside the main girders*

2.2.3. DEFECTS OF EXPANSION JOINTS

Figure 19. *The defects in the expansion joints and pavement nearby*

2.2.4. WATERPROOFING DEFECTS

Figure 20. *The water leaking because of the expansion joint failure*

Figure 21. *The water leaking because of the waterproofing and expansion joint failure*

2.2.5. DETERIORATION OF THE CONCRETE PARAPETS (ASR)

Figure 22. *ASR reaction on the concrete parapets*

Figure 23. *Poor concrete of the parapets, safety barrier secured by timber*

2.2.6. BEARINGS DAMAGE AND CORROSION

Figure 24. *The significant corrosion of the bearings*

3. POTENTIAL FAILURE MODE OF THE BRIDGE

In accordance with current condition of the bridge following failures are considered:

- Failure of the edge girder, because of the concrete degradation and reinforcement corrosion, that will influence the prestressing cables and/or prestressing anchors, leading to the girder failure.
- This is the most probable scenario, as the leakage to the anchoring area can lead to the corrosion of the prestressing reinforcement close to the anchor.
- Failure of the bearings, because of the heavy corrosion – but this will take a long time, and the consequences are not critical.
- Loss of stability of the abutment under the edge bearing, the local pressure into the deteriorated concrete will lead to the local girder failure (slip of the girder, the failure will result in the large deformation, not to the global collapse).

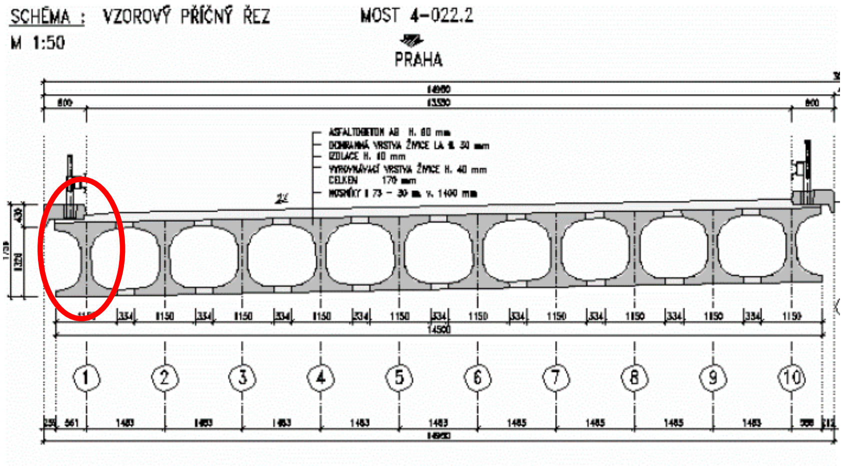


__Figure 25.__ The location of the most critical place – the edge girder

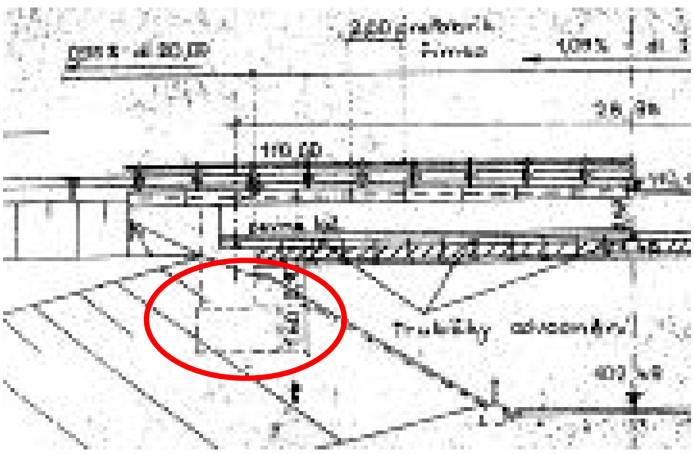


__Figure 26.__ The location of the abutment and possible failure scenario

4. MATERIAL TESTING

4.1. COMPRESSIVE CONCRETE STRENGTH TEST RESULTS

The received results of the tests are given below.

Specimen	Unit weight [kg.m-3]	Force [kN]	Compressive strength [MPa]
Main girder 1	**2377**	552	**77,6**
Main girder 2	**2384**	487	**69,0**
Main girder 3	**2386**	515	**75,0**
Main girder 4	**2374**	478	**70,0**
Abutment 1	**2280**	45,5	**36,7**
Abutment 2	**2277**	50,8	**40,8**
Abutment 3	**2276**	48,3	**38,7**
Abutment 4	**2278**	66,9	**53,6**

The concrete can be considered as a C60/75 for the girders, C30/37 for the abutments.

4.2. ALKALI – SILICA REACTION

The Rhodamin method was used to identify the existence of the silica gel. The ASR was found on the concrete of the parapets.

Figure 27. *Typical signs of the ASR*

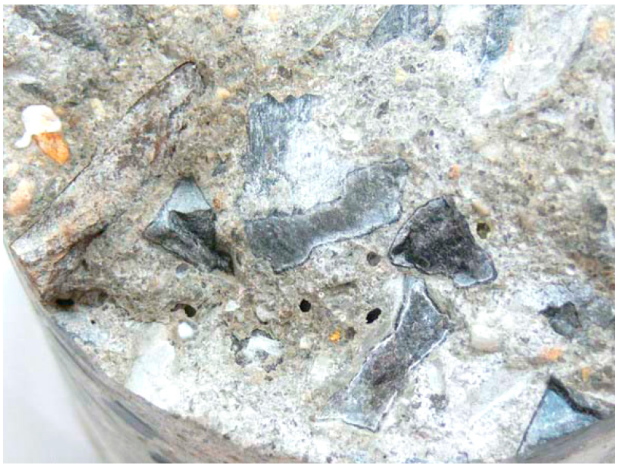


Figure 28. *Specimens with ASR signs*

4.3. CARBONATION

The next test was focused on the carbonation of the concrete. The depth is 8,6 mm on the main girders (5-11mm), and 26,7 mm on the abutment (11-46mm). The parapet shows 36,7mm (37-45mm).

4.4. FREEZING RESISTANCE

All samples were exposed to the 75 freezing cycles. The results show, that the concrete of the superstructure can resist to 75 cycles. The concrete of the abutment is much worse, it was fully damaged only after 25 cycles.

5. KEY PERFORMANCE INDICATORS

Key performance indicators are provided in accordance with best practice knowledge of the research team and experiences with bridge inspection in Czech Republic. The indicators are evaluated and failure modes of the bridge are estimated.

Furthermore, two life time cycle approaches are shown to evaluate the life time costs, reliability, availability and safety of considered arch bridge in following 100 years.

First Referenced approach consider a lack of any repairs of bridge except of very basic ones on the pavement and crash barrier. The bridge defects are developed till bridge failure and whole bridge is replaced with new structure.

Second Preventative approach consider set of repairs during life time cycle to prevent further defect development and overall damage to the structure.

The life time costs consider every year maintenance costs, pavement replacement costs every 20 years, bridge repair every 40 years and other costs described in following sections depending on considered approach.

5.1. CURRENT STATE EVALUATION

In accordance with current state of the described structure following KPIs are considered

Structure	Component	Material	Design and Con-strution	Failure Mode	Vurnerable Zone	Symptoms	KPI	Performance Indicator		Estimated Failure Time
Pre-stressed Girder Bridge	Edge Main Girder	Prestressed Concrete	1983	Global Failure	Bottom flange and Prestressing cables	Reinforcement cor-rosion deterioration	Reliability (structure safety)	3	3	20 years
	Edge Main Girder	Prestressed Concrete	1983	Global Failure	Anchors of Pre-stressing cables	Leakage, crack in the anchor zone		3		20 years
	Bearings	Steel	1983	Bearing Failure	Bearing	Corrosion		2		40 years
	Abutments	Reinforced Concrete	1983	Loss of stability under the edge bearing	Bearing block	Concrete deterio-ration		3		20 years
	Steel Para-pets	Steel	1983	Corrosion and Collapse	Bottom section of parapet	Reinforcement deterioration	Safety	3	3	10 years
	Pavement at EJ	Asphalt	1983	Serviceability and Failure	Expansion joint	Asphalt deteriora-tion, cracks		3		5 years
	Parapets	Reinforced Concrete	1983	Parapet degra-dation	Top surface	Crack & ASR		3		10 years

The estimated failure time is assumed according to research team experience with concrete structures in Czech Republic and estimated progress of the defects. It is however safe assumption under severe conditions.

5.2. RELIABILITY VERIFICATION

The load capacity of the bridge is considered as:

- Normal capacity of the unlimited number of vehicles: V_n = 24 t
- The capacity of the one single vehicle on the bridge: V_r = 53 t
- Exceptional capacity for the heavy special transport: V_e =292 t

Critical member is a side beam and its bending capacity. This load capacity was calculated as a heaviest vehicle, that can cross the bridge, based on following material and load safety factors:

γ_S = 1,15 – for prestressing steel
γ_G = 1,35 – Safety factor for dead load
γ_Q = 1,35 – Safety factor for live load

Those load factors are given in the Czech load capacity code for existing bridges.

For the prestressing steel, we do not have the exact data. However, we can writhe the material factor as:

$$\gamma_S = \frac{f_{yk}}{f_{yd}} = e^{\left(-k_n \cdot V_{fy}\right)} \Big/ e^{\left(-\alpha_R \cdot \beta \cdot V_{Rx}\right)}$$

Where the variability index can be written as:

$$V_{Ry} = \sqrt{V^2{}_{fy} + V^2{}_{geo} + V^2{}_{\zeta}}$$

We can assume, that variability for prestressing steel according to the literature is V_{fy}= 0,05, variability of the geometry is small, as it is precasted member, V_{geo} = 0,02, model uncertainty variability is V_{ζ} = 0,05. Then we can get:

$$V_{Ry} = \sqrt{V_{fy}^2 + V_{geo}^2 + V_{\zeta}^2} = \sqrt{0,05^2 + 0,02V^2 + 0,05^2} = 0,073$$

Then, following the above formula, we can write:

$\gamma_S = exp\ (-1,64 \cdot 0,05)\ /exp\ (-0,8 \cdot 3,8 \cdot 0,073) = 1,15$
Which is the same as the used value in the analysis, for β = 3,8.

For the dead load, as the geometry was not measured, we can assume for dominant load α_E= -0,7, and variability V_G = 0,1. Then, we take the factor of the model uncertainty as γ_{Sd}= 1,05. We can write:

$\gamma_G=(1 - \alpha_E \cdot \beta_t \cdot V_G) \cdot \gamma_{SD}= (1 + 0,7 \cdot 3,8 \cdot) \cdot 1,5 = 1,329$

Or to get 1,35
$\gamma_G=(1 - \alpha_E \cdot \beta_t \cdot V_G) \cdot \gamma_{SD}= (1 + 0,7 \cdot 4,1 \cdot 0,1) \cdot 1,05 = 1,35$
So for the dead load, β = 4,1.

If we assume the live load, and we assume the variability of the model uncertainties $V_{\theta E}$=0,1, we can get the β = 3,5.

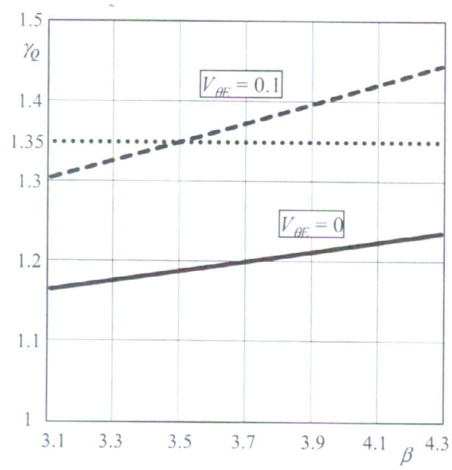


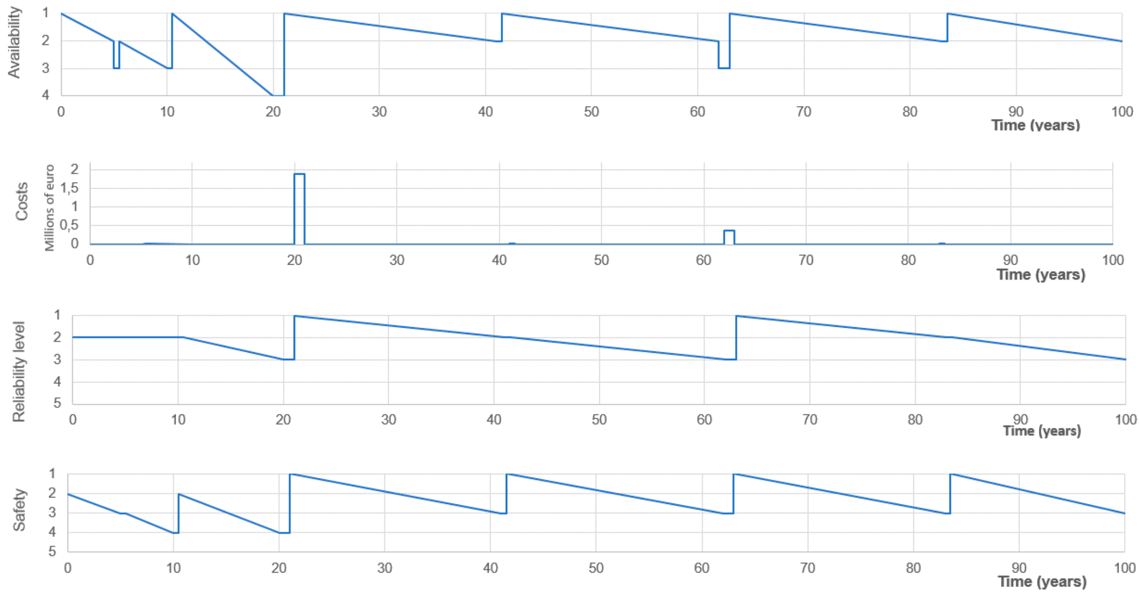
Figure 29. *The relation between β and γ_Q*

Based on previous calculations, and application of the standard safety factors, we can conclude that the smallest β was calculated for the live load impact. So we are on the safe side, if we take this β for the whole bridge. For more precise load capacity verification, the slightly smaller load factors for the dead load can be taken, if we take β = 3,5 and thus slightly increase the load capacity.

5.3. REFERENCED APPROACH

Lack of any major repairs of superstructure and accessories except of basic pavement repairs leads to the defects development up to the bridge failure. In accordance with previous section, the existing structure defects, development and estimated failure times are assumed as follows:

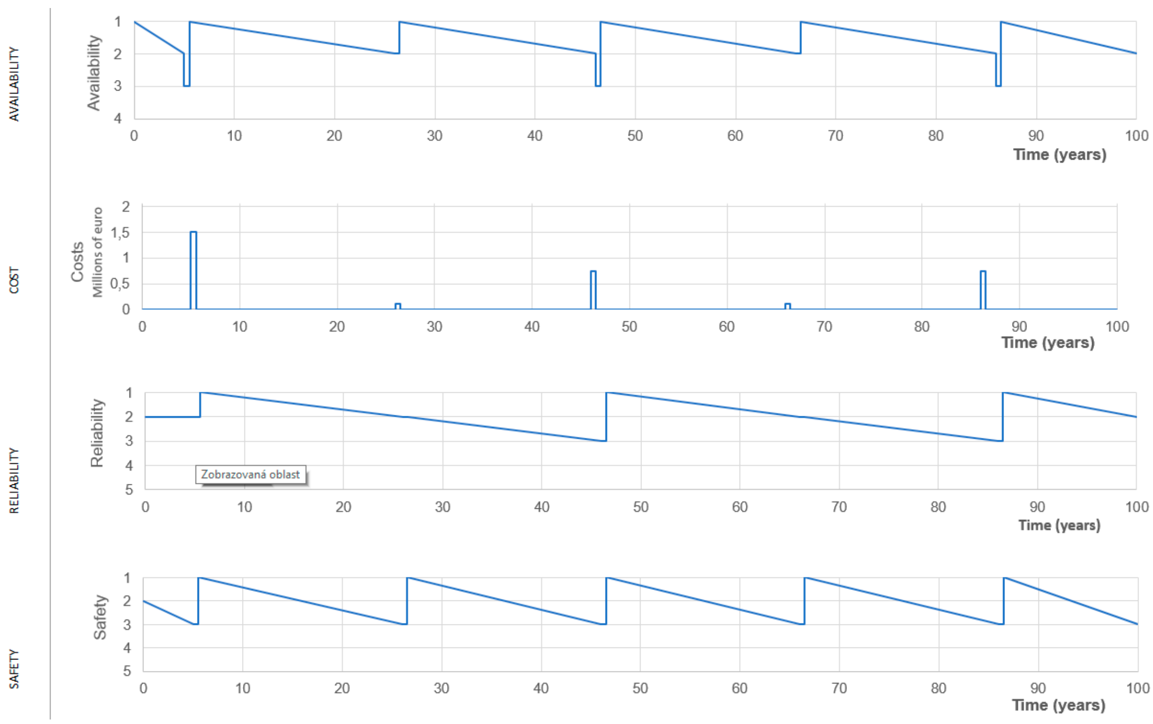
- Pavement failure in five years due to crack development at the EJ location, sweating and deformation in five years (as noted the pavement layer shall be repaired).
- Then the pavement will be repaired. But only the pavement, not the waterproofing. The cost is estimated as 40t Euro/bridge. It will temporarily decrease the availability.
- Concrete parapets collapse (meaning the unstable crash barrier, which is no more safe) in 10 years. At this time, the installation of the temporary concrete crash barrier is assumed. It means decrease of availability & safety, as the bridge is narrower. The cost is estimated as 50t Euro/bridge
- Loss of the stability of the abutment under the bearing, or more likely failure of the prestressing cables in 20 years (bridge failure and replacement with new structure).
- The drop of the availability, bridge will be closed. But the adjacent bridge will carry one traffic lane in each driving direction, so the traffic will be only slowed and traffic jams can be expected.
- The cost of the repair is 1 900 000 Euro.
- Preventative approach on the new bridge (pavement replacement every 20 years and bridge repair every 40 years).
- The repair will be done by halves of the bridge, so temporarily the availability is decreased. The cost of the pavement repair is 110 000 Euro, cost of the complex repair (pavement, crash barrier, railing, parapets) is 300 000 Euro.



5.4. PREVENTATIVE APPROACH

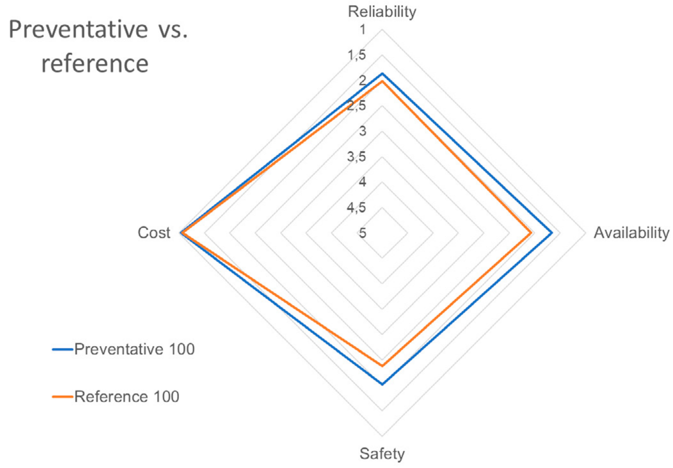
First the bridge repair shall be designed and done in 5 years. The whole bridge structure and accessories repair is considered. The life time cycle is considered as follows:

- pavement failure in five years due to crack development, sweating and deformation in five years (shall be repaired). The whole bridge and accessories repair is considered in the same time.
- The new concrete deck will be laid on the top of prestressed girders, side beam will be replaced by a new one. Cost is considered as 1 500 000 Euro.
- The drop of the availability, bridge will be closed. But the adjacent bridge will carry one traffic lane in each driving direction, so the traffic will be only slowed and traffic jams can be expected.
- In following years, the preventative approach on the repaired bridge is assumed (pavement replacement every 20 years and bridge repair every 40 years). Cost 40000 Euro or 750 000 Euro respectively (cost are increased, because of expected repair works on the renovated concrete).
- The repair will be done by halves of the bridge, so temporarily the availability is decreased.



5.5. COMPARISON OF THE APPROACHES

A comparison of the two considered approaches is shown in following "spider" diagram:



According to the carried-out analysis the preventative approach is more appropriate for the arch bridge - the indicators shows more favourable results for all aspects – safety, reliability, availability. Only the costs are almost comparable - the reason is the normalization of the costs based on the interest rate 2%.

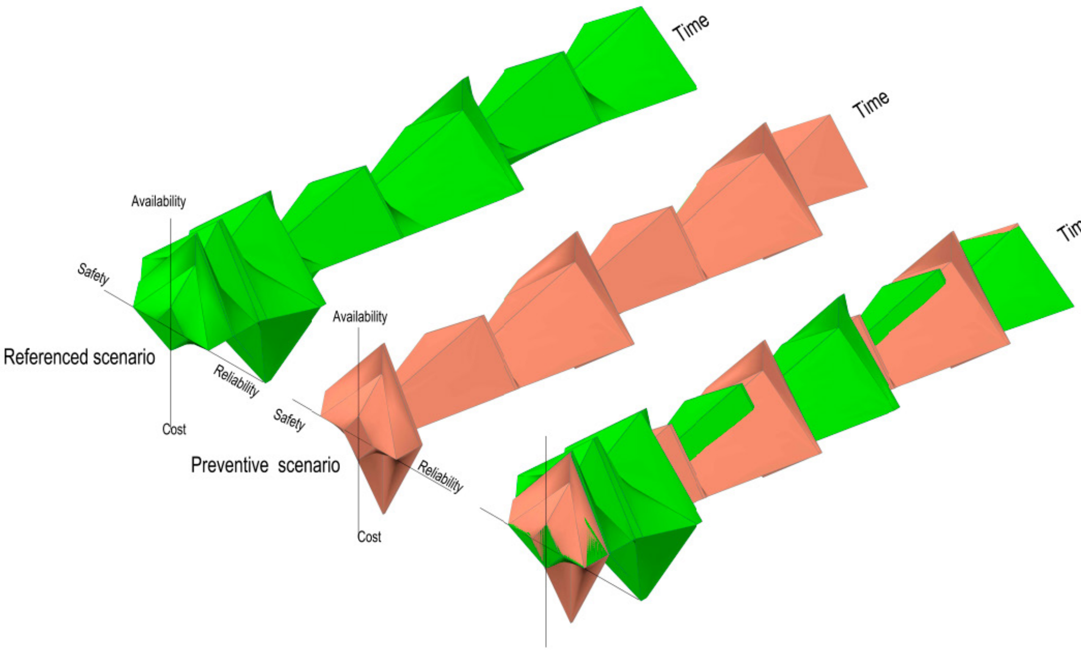


Figure 30. *The comparison of the safety, reliability, availability and cost in time and volume comparison*

Informatively, we can also compare the Referenced and Preventive scenario in the 3D spider graph, separately and in one image together for the whole period of 100 years. The comparison can be done on the comparison of the volume of the normalized graph (unitless), as an averaging tool. Then we have:

- Referenced scenario - 180
- Preventive scenario - 146

This means, that preventive scenario is generally closer to the best "1" grade, which means it is more appropriate here.

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TU1406 WG4 Final report Appendix A2

Bridge Case study - Girder bridge Viotikos Kifissos - Greece

WG4

TECHNICAL REPORT
PREPARATION OF A CASE STUDY

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1. GENERAL DATA OF THE BRIDGE

1.1. BASIC INFORMATION

Viotikos Kifisos bridge is a four (4) span bridge. The superstructure is built by 5 pre-stressed (post tensioned) precast concrete beams, that are transversely connected by a top deck slab (cast in situ on pre-slabs) and end diaphragms over their supports on piers and abutments. The superstructure is supported on piers and on abutments trough circular elastomeric bearings (anchored). Piers are composed by two square hollow concrete columns, frame at their top by a top rectangular concrete beam.

The total length of the bridge is 145,40m with main span length of 34,00m built by 5 precast pre-stressed concrete T beams. The pavement width - including the sidewalks is 15.00 m, providing two traffic lanes plus emergency lane. All spans are simply supported, through elastomeric bearings on the dual column bents. The age of the bridge is estimated some 30 years old.

The bridge is inspected and maintained in the frame of a 30 years private concession project, appointed to NEA ODOS AE from the Greek Public Works Ministeriat.

Technical and geometrical data for the bridge:

- Year of construction: 1990
- Superstructure: 5 post-tensioned concrete beams
- Bridge length: 145.40m
- Span no: 8 (~×16.55m long)
- Joint type: Elastomeric expansion joint (anchored) T120
- Bearing type: Elastomeric orthogonal, of type 4

Figure 1. *Side view of the Viotikos Kifisos bridge - uphil.*

Figure 2. *Side view of the Viotikos Kifisos bridge - downhill sides.*

Figure 3. *Location of the bridge 104km Northern from Athens, in Viotia prefecture, Central Greece.*

Figure 4. *A view along the bridge deck towards Thessaloniki direction.*

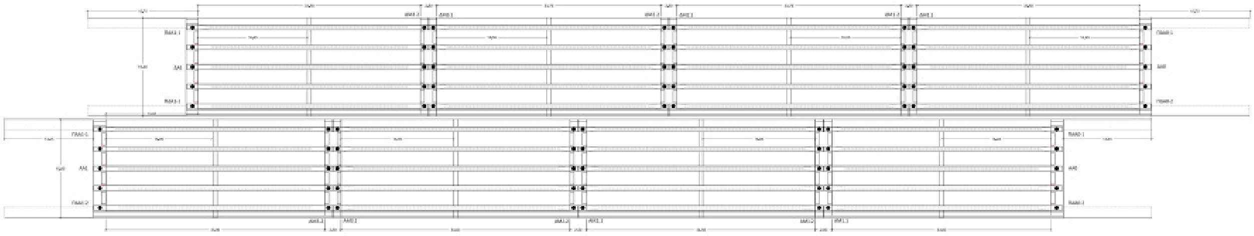


Figure 5. *General plan of the bridge.*

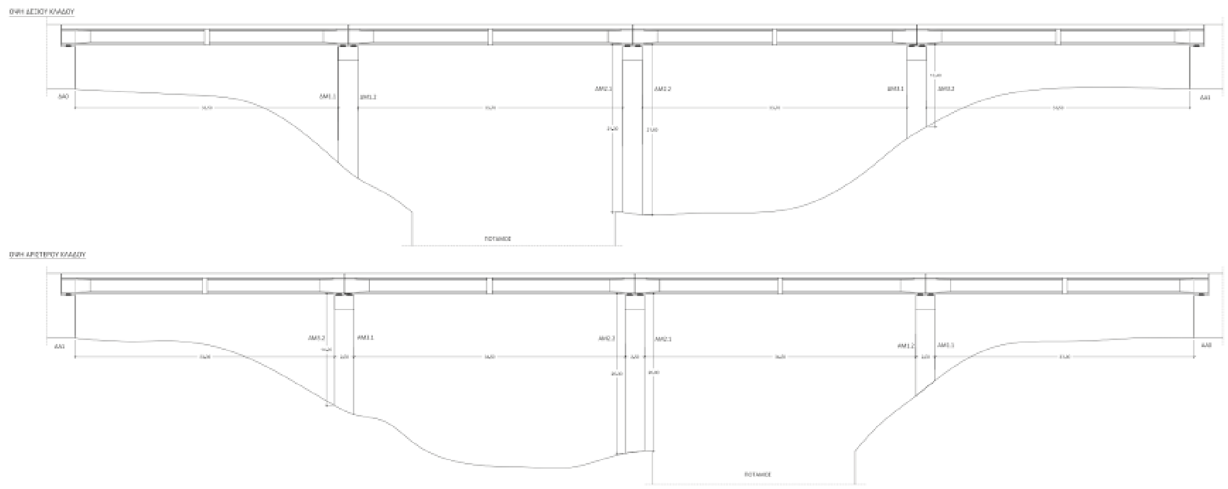


Figure 6. *Elevation of the bridge.*

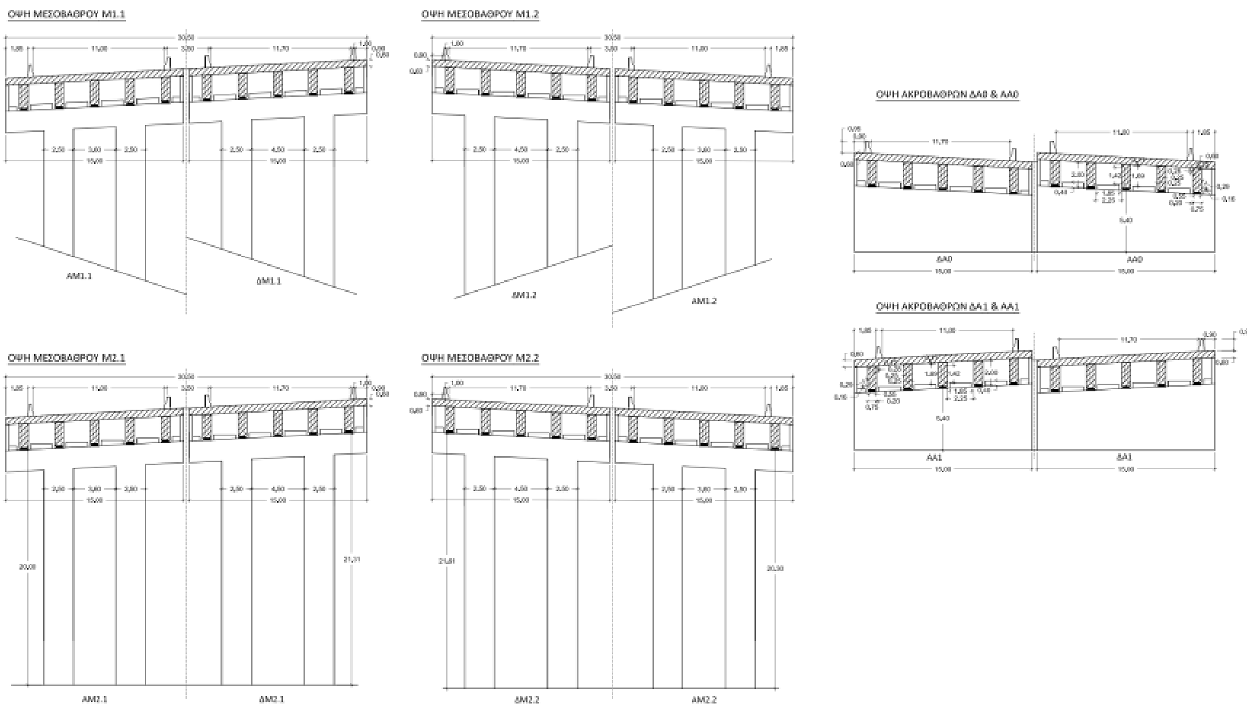


Figure 7. *Cross sections of the bridge.*

1.2. TRAFFIC INFORMATION

The last information about the traffic area, based on statistical analysis of both toll station vehicle crossing data base and overweighted vehicles (>80 tons) is as follows:

Year	Daily average yearly traffic over the bridge	Percentage of trucks & buses
2010	17.352	4.47%
2011	15.754	5.71%
2012	12.861	4.75%
2013	11.843	3.90%
2014	9.956	3.54%
2015	9.984	3.65%
2016	9.908	4.66%
2017	10.910	7.09%

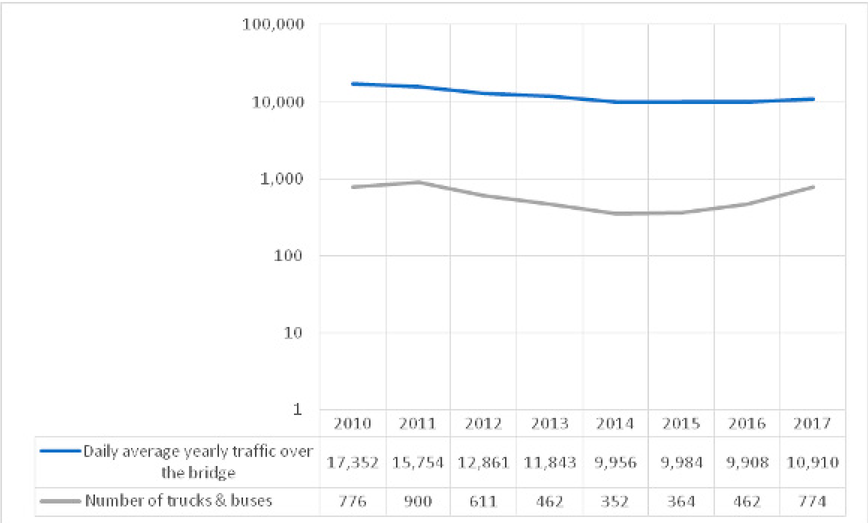


Figure 8. *Yearly frequencies of crossings of oversized/overweighed truck loads (logarithmic scale)*

1.3. FOUNDATION

Foundations are inaccessible, for inspection. No as built drawings have been found as well. Piers and abutments are supposed to be founded through rock sockets and on piles caps, respectively.

1.4. SUBSTRUCTURE

The substructure is formed by two (2) abutments and the three (3) reinforced concrete piers. Each pier has a hammerhead shape with body dimensions 2,50m x 2,50m and piercap dimensions 15,00m x 2,50m.

1.5. SUPERSTRUCTURE

The superstructure is composed by 5 post-tensioned concrete beams per span, 34,00m long, supported through elastomeric bearings on the piercaps. The five beams are connected on their top by an in-situ deck slab casted on precast pre-slabs (performing as formworks linking the transverse gaps between beams). The beams are also connected transversely through intermediate and end diaphragm beams, also post tensioned. Each superstructure span is simply supported on adjacent beams, separated by the adjacent spans by expansion joint gaps.

1.6. ACCESSORIES

There is asphalt pavement on the bridge, some 15cm thick. The sidewalks are by precast concrete elements. The safety barriers are for both central reserve and external sidewalks by concrete newjersey type

2. TECHNICAL CONDITION

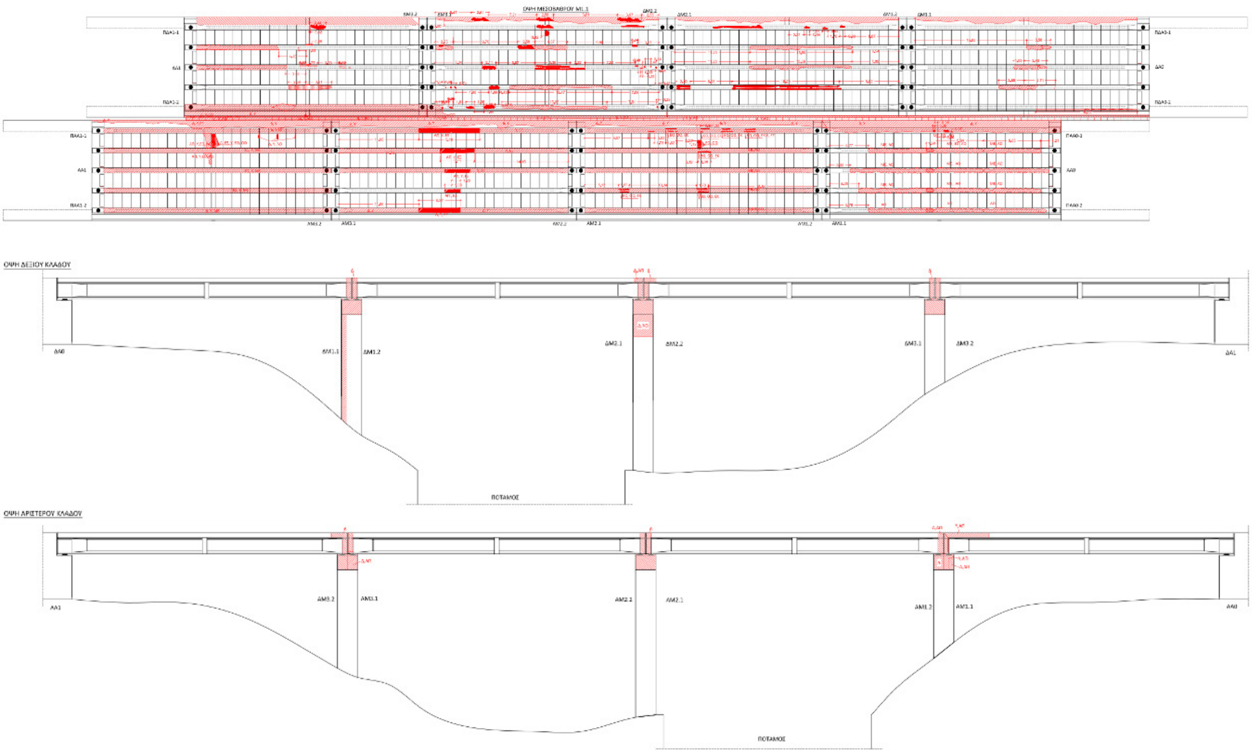
2.1. COLLECTION OF DEFECTS

The bridge is systematically inspected by Nea Odos S.A. skilled bridge engineering personnel, applying the Bridge Inspection & Evaluation Manual of Nea Odos S.A. Based on findings, measurements and site testing of various types, carried out in 2017, the condition of the bridge and its sufficiency was rated, applying the Nea Odos Bridge Inspection & Evaluation Manual. For rating the condition of elements, components and the bridge as a whole system a condition rating system very similar to FHWA sufficiency rating system was applied.

The following types of defects were identified during these inspections

1. Deep spalling of concrete in beams and in piers (pier caps), manifested by severe/deep loss of concrete section;
2. Exposition of reinforcement bars and corrosion in piers, pier caps and beams;
3. Delamination, swelling and cracking under advanced corrosion of reinforcement bars;
4. Water leakage of expansion joints;
5. Direct discharge of deck drainage wedges on the vertical surfaces of the superstructure and piers' concrete.

The most important defects on the main members are presented on the sketches below.



Efflorescence and spalling

Exposed/corroded bars

Figure 9. *Defects of the bridge.*

2.2. DEFECTS OF THE MAIN STRUCTURAL ELEMENTS

2.2.1. SUBSTRUCTURE DEFECTS AND ELEMENT CONDITION RATING

Figure 10 - 11. *Efflorescence/stains/damped areas on the surface of the abutments*

Figure 12 - 13. *Efflorescence/stains/damped areas on the surface of the abutments.*

Figure 14. *Pier M 1.1*

Figure 15. *Pier ΔM 1.2*

Figure 16. *Pier M 2.1*

Figure 17 - 18. *Pier M 2.2*

Figure 19 - 20. *Pier M 3.1*

Figure 21. *Pier M3.2*

2.2.2. SUPERSTRUCTURE DEFECTS AND ELEMENT CONDITION RATING

Figure 22 - 23. *Efflorescence and stains on the bottom of beams and preslabs under the water leaking central reserve*

Figure 23 - 24. *Efflorescence and exposed rebars (pre slabs and beams).*

Figure 25 - 26. *Delamination and exposed/corroded rebars (beams).*

Figure 27 - 28. *Delamination and exposed/corroded rebars (beams).*

3. VULNERABILITY ASSESSMENT

3.1. VULNERABLE ZONES

The vulnerable zones were presented in the previous paragraphs.

3.2. POTENTIAL FAILURE MODES OF THE BRIDGE

According to the current state of the bridge and to the identified types of defects, their extent and severity, the following failure modes are considered:

Potential Failure modes identified of the structure:

- Beam failure – fracture of post-tensioning tendons and brittle shear or bending failure under extreme live loads due to the future significant reduction of mild reinforcement and tendons cross-section caused by the predicted rate of their corrosion.
- Piers and abutment failure is far less probable under vertical loads (combination of traffic and permanent loads) due to their robust design type : framed rectangular hollow section columns designed to resist high seismic forces. The exception is the on going surface deterioration of the top beam that frames the two columns of each piers, exposed under the water leaking longitudinal joints that separate the two branches of the bridge.
- Deck failure – After advanced corrosion of the bottom reinforcement of the pre-slabs that bridge the gaps between adjacent beams, debris from extended spalling can disintegrate the deck soffit.
- Bearing failure – disintegration of the elastomeric bearings due to the advanced corrosion of their internal steel sheets cannot be excluded in the future.
- Expansion joints failure – Failure of the expansion joints, by priority of the right lane (heavy traffic) is predicted in the near future.
- Drainage inadequacy – Improper drainage system, discharging deck water directly on the concrete surfaces of piers' top beams and on the webs of the superstructure beams under the longitudinal gap of the internal central reserve.
- Waterproofing failure – loss of functioning of the waterproofing system due to perforations and discontinuities caused by incidental impact, execution defects or material aging.

Failure modes related with the safety of the structure:

- Disturbance to cyclists or drivers – due to the future deterioration of the pavement (pot holes, rutting etc), or due to the future anchoring failure or the disintegration of the expansion joint elements, or due to the settlement of the transmission embankments etc.
- Falling concrete chunks – due to spalled concrete cover items on the rural roads under the bridge, as a result of corrosion.

4. KEY PERFORMANCE INDICATORS

Key performance indicators are defined according to WG1, 2 and 3 guidelines and evaluated in accordance with best practice knowledge of the team and the experience with bridge inspection in Greece. The indicators are evaluated and the most prominent failure modes are identified and their impact on the bridge integrity and serviceability is estimated. Two life time cycle approaches are examined herein and their respective life time costs are calculated. The performance of the bridge in terms of reliability, availability, safety and the assigned costs are considered for the next 72 years, the remaining life of the bridge that has completed 28 years on service.

First referenced approach considers postponing of major interventions, of preventative nature, that would reverse the impact of the actual on going damage processes, already established on some of the bridge elements. Therefore according to this approach, the bridge damage process is considered to develop vers time with no control, due to the lack of protective measures and the critical time point when advanced structural loss is expected is predicted. Then at the time of the expected structural damage, reactive interventions of high cost are considered.

A second Preventative approach considers repair/anti corrosion protection measures on time in order to control/retard the corrosion rate of the bridge concrete reinforcement, which is expected to delay the future rehabilitation interventions.

4.1. CURRENT STATE EVALUATION

According to current condition of the described bridge structure following KPIs are considered:

Structure type	Group	Component	Material	Design & Construction	Failure mode	Location/ Position	Damage/ Observation	Damage process	KPI	PIE/ CL*	PV**		EFT
											R	S	
Girder beams	Structural elements	Precast post-tensioned beams	Pre-stressed concrete	1990	Beam bending failure mode	Bottom tensile flanges (HMS region)	Section loss of the bottom mild reinforcement	Corrosion stains	Reliability	3	3	3	25
								Efflorescence	Reliability	3			
								Swelling	Reliability	3			
					Beam bending failure mode	Bottom tensile flanges (HMS region)	Section loss of the bottom layer of pre-stressing strands	Corrosion stains	Reliability	4			35
								Efflorescence	Reliability	4			
								Efflorescence	Reliability	4			
		Precast post-tensioned beams	Pre-stressed concrete	1990	Beam shear failure mode	Beams' webs at supports	Section loss of the stirrups	Corrosion stains	Reliability	3	3		20
								Efflorescence	Reliability	3			
								Swelling	Reliability	3			
					Beam shear failure mode	Beams' webs at supports	Section loss of the bottom layer of pre-stressing strands	Corrosion stains	Reliability	4			35
								Efflorescence	Reliability	4			
								Swelling	Reliability	4			
		Piers top beams	Reinforced concrete	1990	Pier top beam failure	Pier top beam	Section loss of the reinforcement bars	Corrosion	Reliability	3	3		25
								Revealed bars	Reliability	3			
								spalling	Reliability	4			
		Abutment	Reinforced concrete	1990	Abutment failure mode	Abutment external side	Section loss of the reinforcement bars	efflorescence	Reliability	3	3		35
								Corrosion stains	Reliability	3			35
		Expansion joints	Elastometallic anchored	1990	Joint failure	Abutments/ Piers	Anchoring failure	Anchors' deterioration	Reliability	4	4		10
		Pedestrian sidewalk	Reinforced concrete	1990	Disintegration	Top /side faces	Spalling	Corrosion	Safety	4	4		20
		Transmission embankments	Soil	1990	Settlement	Abutments	Settlement	Water permeability/ heavy traffic	Safety	4	4		20
		Bearings	Elastomer	1990	Disintegration	Abutments/ Piers	Bulges/relative sliding of layers	Corrosion of internal sheets	Reliability	3	3		15
		Bearings	Elastomer	1990	Settlement due to disintegration	Abutments/ Piers	Bulges/relative sliding of layers	Corrosion of internal sheets	Safety	4	4		20
	Equipment	Safety barrier	Concrete	1990	Disintegration	Central reserve Side walks	Failure	Spalling/ cracking	Safety	4	4	4	25
		Road pavement	Asphalt	1990	Failure	Deck	Potholes/ruting/ cracks	Potholes/ ruting/cracks	Safety	4	4		10
		Drainage installation	Open verges	1990	Discharging without control	Deck	Flooding of deck lanes during heavy rain	Damped asphaltic layers	Safety	4	4		15
		Waterproofing	Asphaltic membranes	1990	Water leaking	Deck slab	Water leaking	Stains/damp/ efflorescence on the deck slab sofit	Reliability	4	4		10

* Performance Indicator Element / Component Level
** Performance Level
*** Estimated failure time [years]

The estimated failure time is assumed according to state of the bridge and the team experience with steel and concrete structures in Greece.

4.2. REFERENCED APPROACH

In the referenced approach to the maintenance of the bridge it is assumed that there is lack of any preventative repairs of the bridge structure and accessories in order to control and retard on time the on going deterioration process clearance, except inspection and routine maintenance. This approach leads to the defects escalation until the time that reactive major repair are necessary to reinstate the severe structural losses and damages expected to take place from the long term action of the actual on going deterioration process. The existing structure defects development and estimated failure times are assumed below.

In 10 years:

- expansion joints failure – due to corrosion and heavy traffic loading
- road pavement failure – due to traffic loading, aging
- waterproofing failure – due to blistering, aging

In 15 years:

- bearings disintegration failure – due to corrosion of internal sheets,
- pedestrian and road pavement failure – due to cracks and deformation,
- drainage failure – due to clogging of drainage outcharges

In 20 years:

- bearings settlement after their disintegration – due to corrosion of internal sheets,
- pedestrian sidewalks – due to cracks, spalling, corrosion induced
- transmission embankments' settlement – due to traffic loading
- Severe reduce of the shear strength of the precast beams - due to the icorrosion of their stirrups

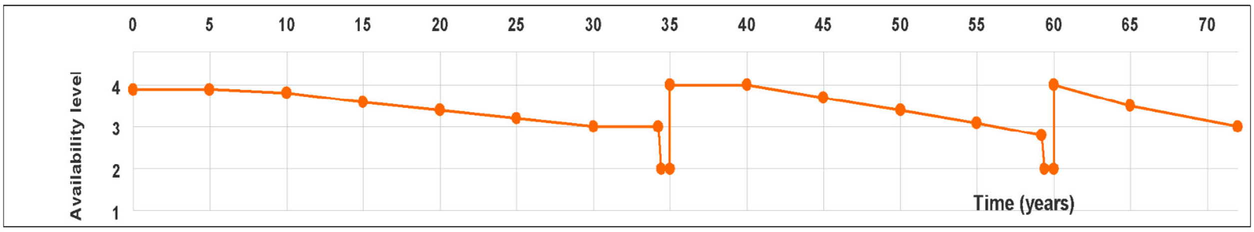
In 25 years:

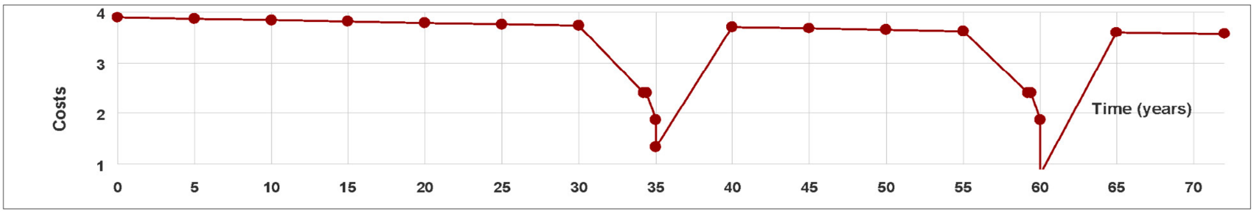
- safety barriers failure – due to spalling, cracking.
- pedestrian sidewalks – due to reinforcement corrosion, spalling.
- Severe structural loss of top beams of the piers – due to the on going corrosion of their revealed reinforcement bars
- Severe reduce of the bending strength of the precast beams - due to the corrosion of their mild bars

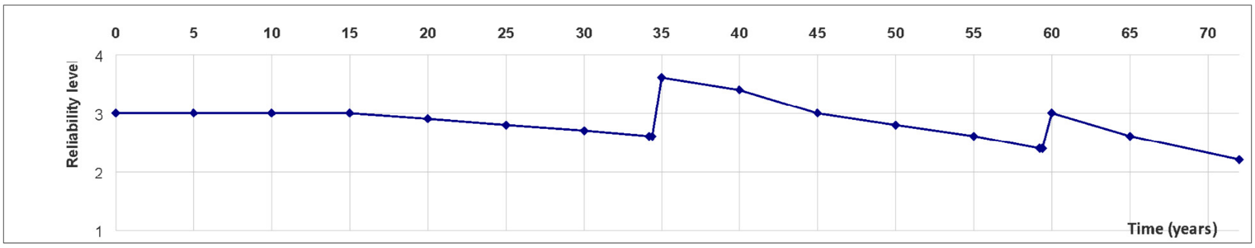
In 35 years:

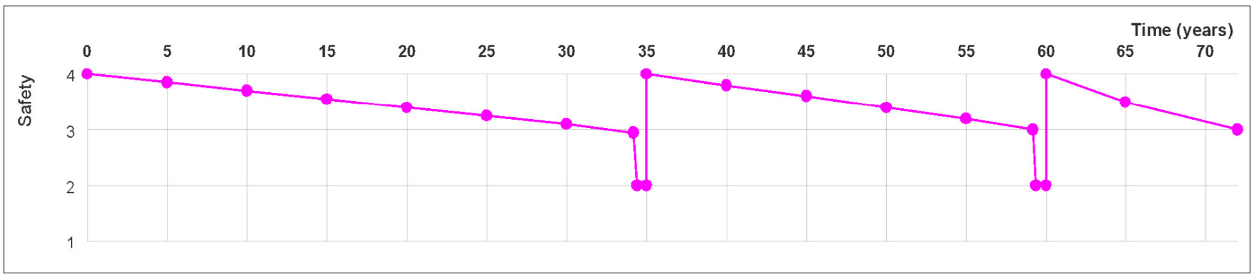
- Severe reduce of the bending strength of the precast beams - due to the corrosion of their pre-stressing strands
- Severe reduce of the shear strength of the precast beams - due to the corrosion of their pre-stressing strands

The predicted evolution of the four KPIs vers the remaining life of the bridge (72 years) are shown in the following diagrams, for this approach:

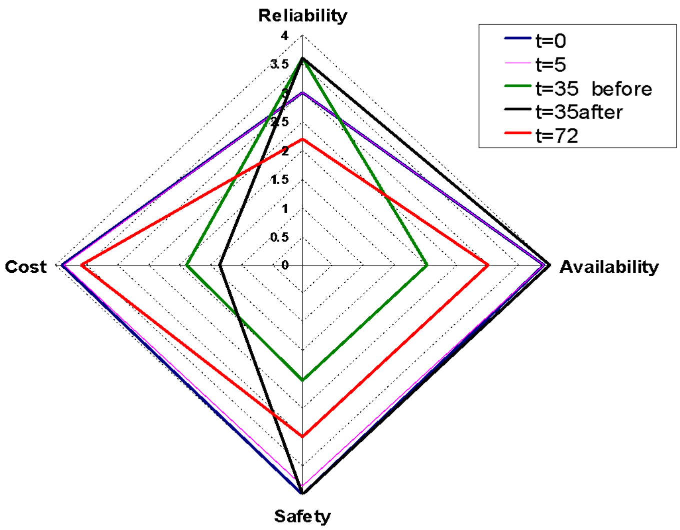








For various time points of the remaining bridge life, following the reference approach the performance of the bridge for the 4 selected KPIs is shown in the following spidergram.



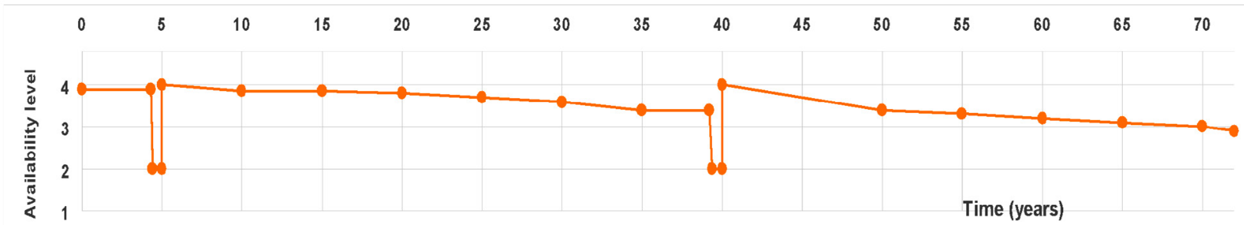
4.3. PREVENTIVE APPROACH

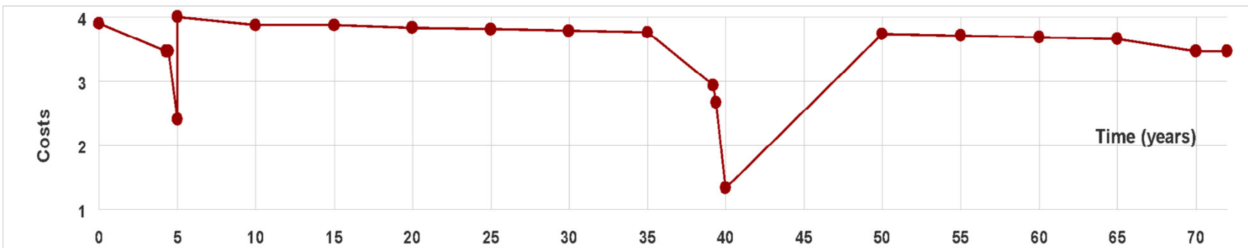
In the preventive approach to the maintenance of the bridge it is assumed that the bridge is protected on time (t=5years), by repairing the actual limited deteriorations and protecting the surface concrete of the affected beams and piers. By this preventative maintenance approach the bridge is considered to deteriorate in a more controlled manner and thus the second rehabilitation is expected after 40 years.

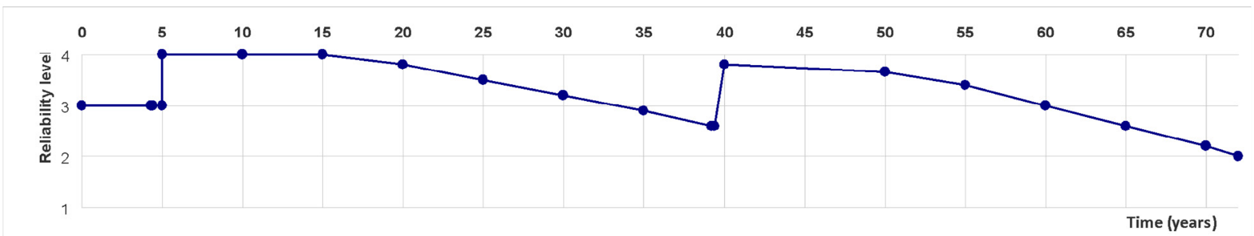
The planned interventions are as follows:

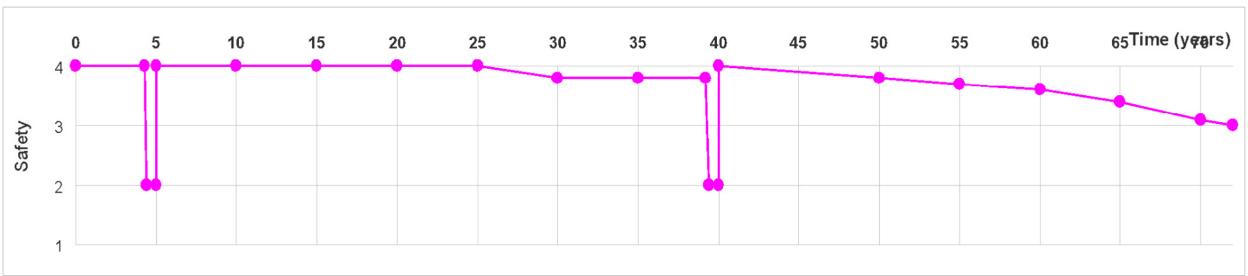
- After 5 and 40 years two rehabilitations of limited extent, comparing with that of the reference approach, where the bridge is left to deteriorate in along term uncontrolled manner.

The predicted evolution of the four KPIs vers the remaining life of the bridge (72 years) are shown in the following diagrams, for this preventative approach:

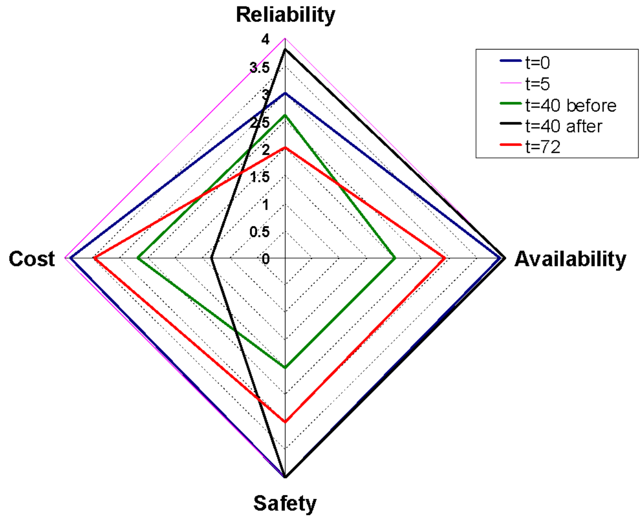








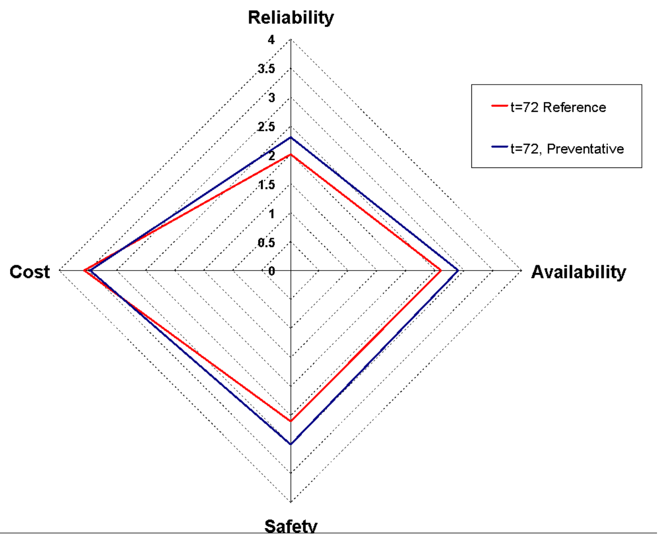
For various time points of the remaining bridge life, following the reference approach the performance of the bridge for the 4 selected KPIs is shown in the following spidergram.

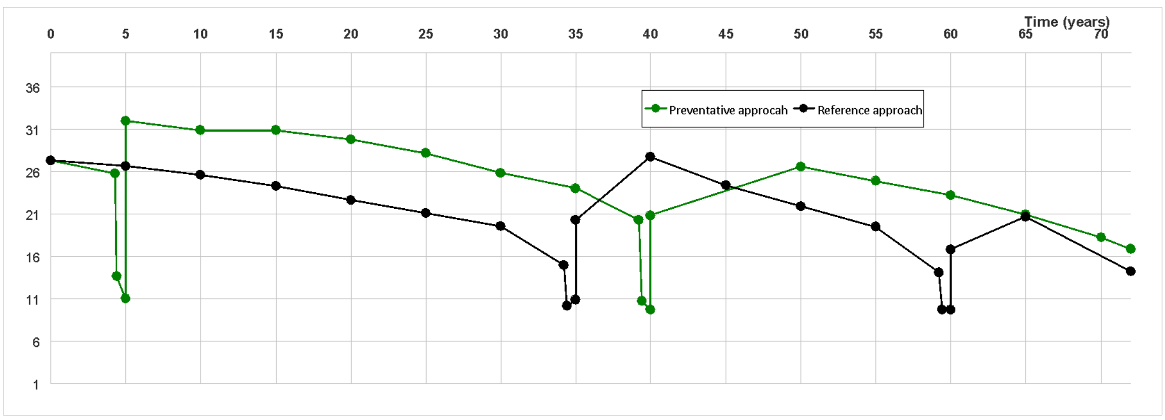


4.4. COMPARISON OF THE APPROACHES

A comparison of the two considered approaches is shown in "spider" diagram below.

Comparing two approaches, by drawing the spidergrams at the end of the remaining bridge life is shown in the following figure. Nonetheless the best way of comparing the two approaches is by considering the cumulative performance of each approach vers the whole remaining life, examined in this use case. This is proposed to be carried out by calculate the total spidergram volumes of the two approaches, as shown in the last chart.





The total volume of the spidergams vers the remaining life for the two approaches that their evolution vers time is calculated in the last chart, is 401 and 471, for the reference and the preventaive approcah, respectively. So the preformance of the brideg is being kept higher for all the KPIS along the remaining bridge life for the preventative approach, which is preferred.

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