TU1406

COST ACTION

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.*

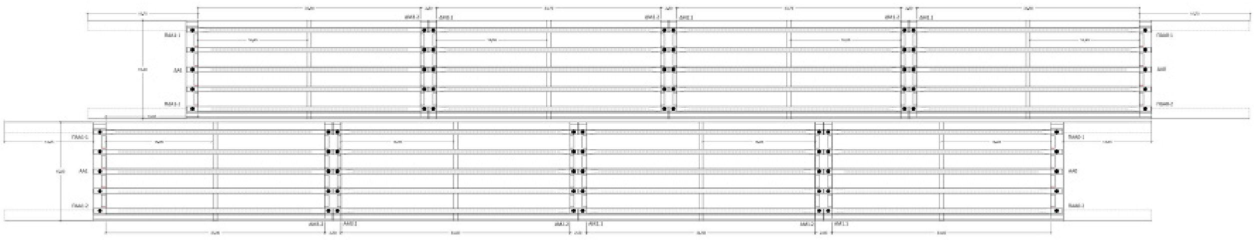


Figure 5. *General plan of the bridge.*

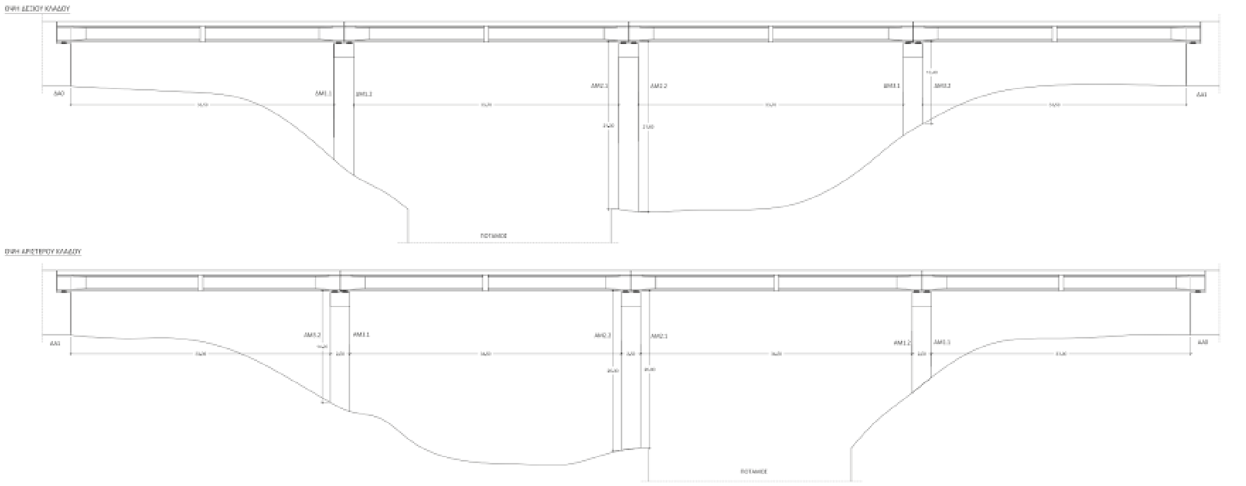


Figure 6. *Elevation of the bridge.*

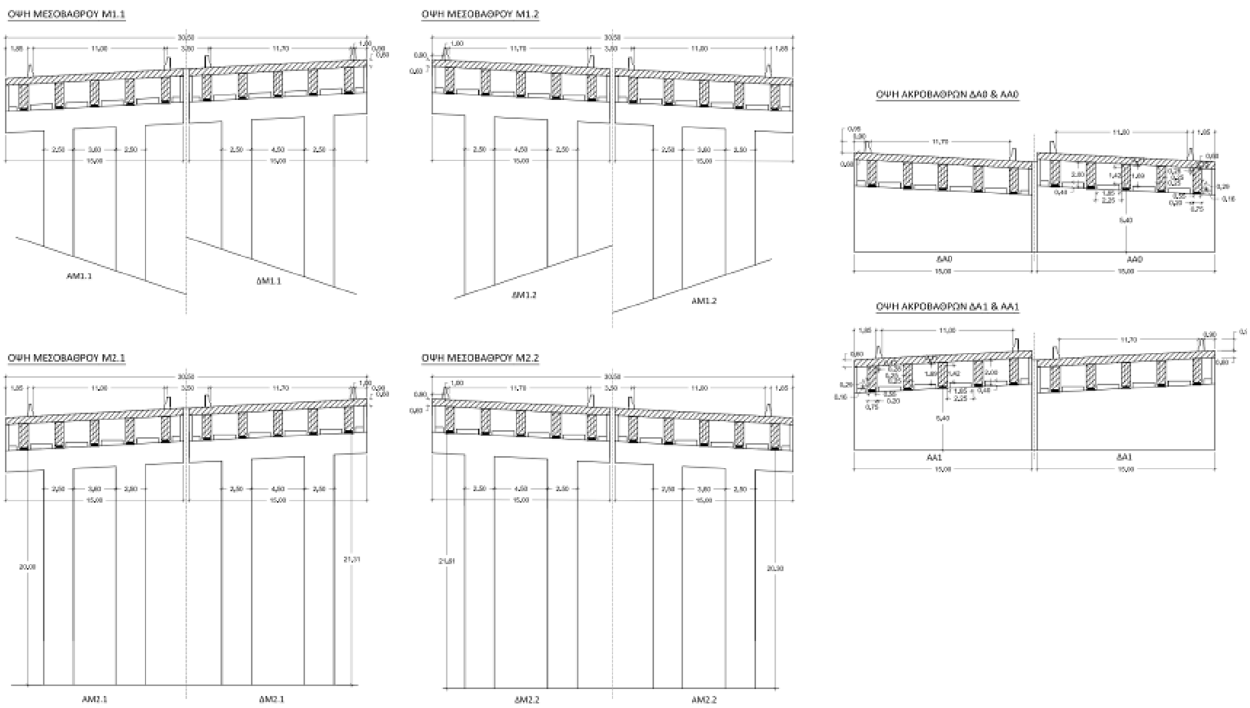


Figure 7. *Cross sections of the bridge.*

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%

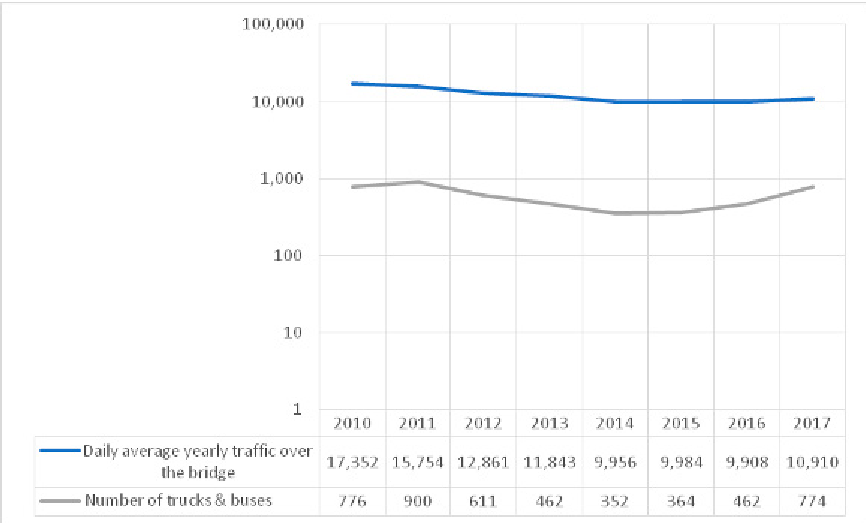


Figure 8. *Yearly frequencies of crossings of oversized/overweighed truck loads (logarithmic scale)*

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

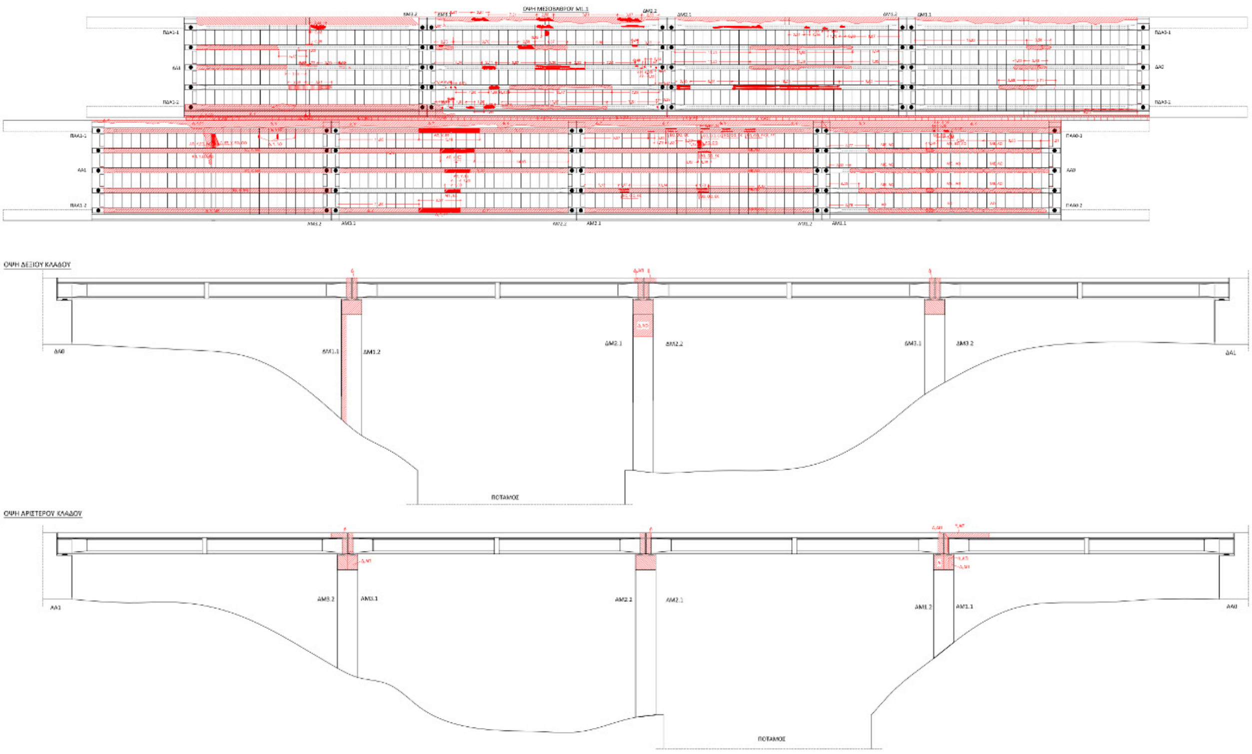
2.1. COLLECTION OF DEFECTS

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

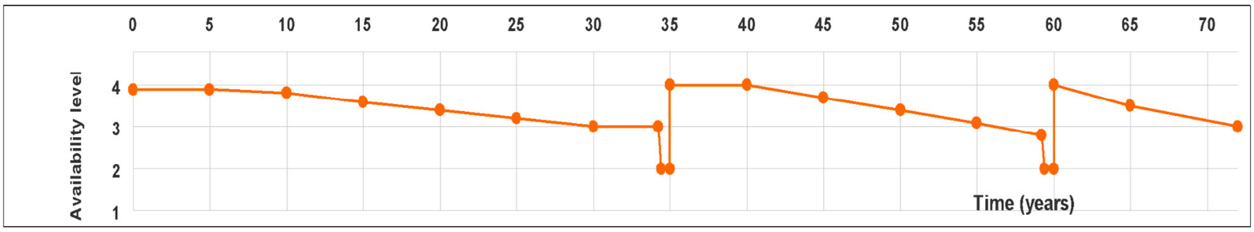
In 25 years:

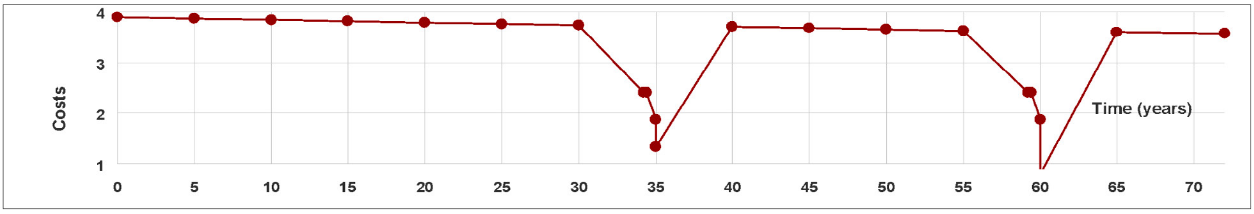
- safety barriers failure – due to spalling, cracking.
- pedestrian sidewalks – due to reinforcement corrosion, spalling.
- Severe structural loss of top beams of the piers – due to the on going corrosion of their revealed reinforcement bars
- Severe reduce of the bending strength of the precast beams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

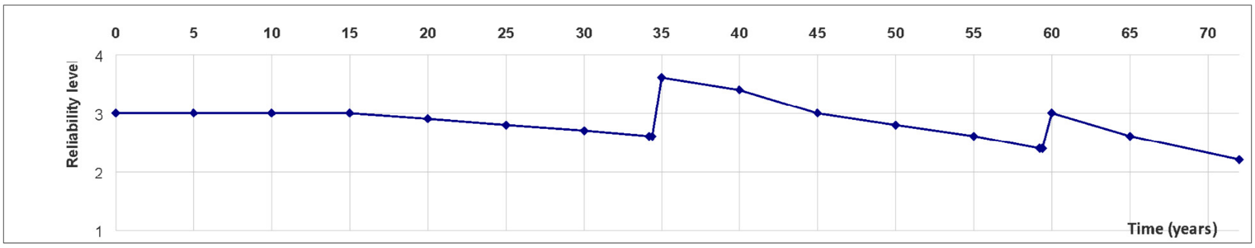
In 35 years:

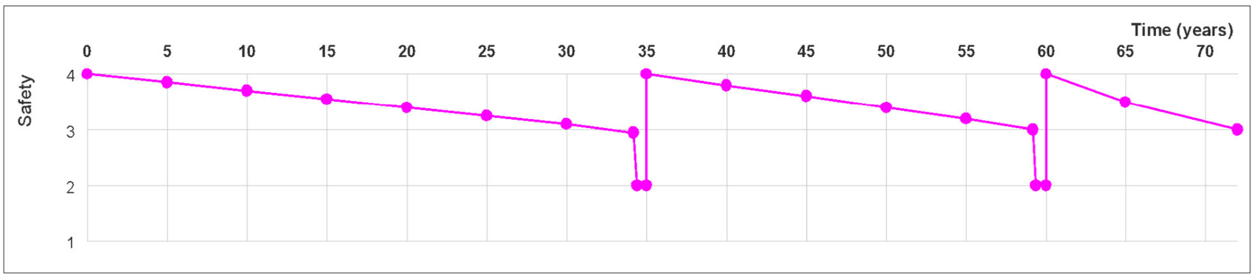
- Severe reduce of the bending strength of the precast beams - due to the corrosion of their pre-stressing strands
- Severe reduce of the shear strength of the precast beams - due to the corrosion of their pre-stressing strands

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:

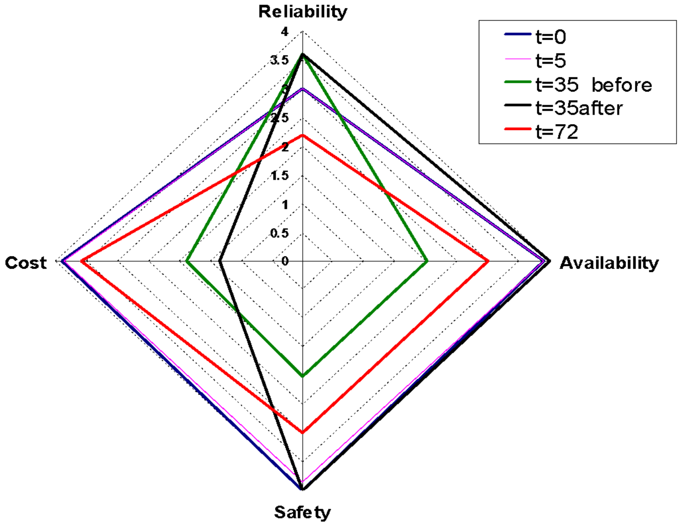








For various time points of the remaining bridge life, following the reference approach the performance of the bridge for the 4 selected KPIs is shown in the following spidergram.



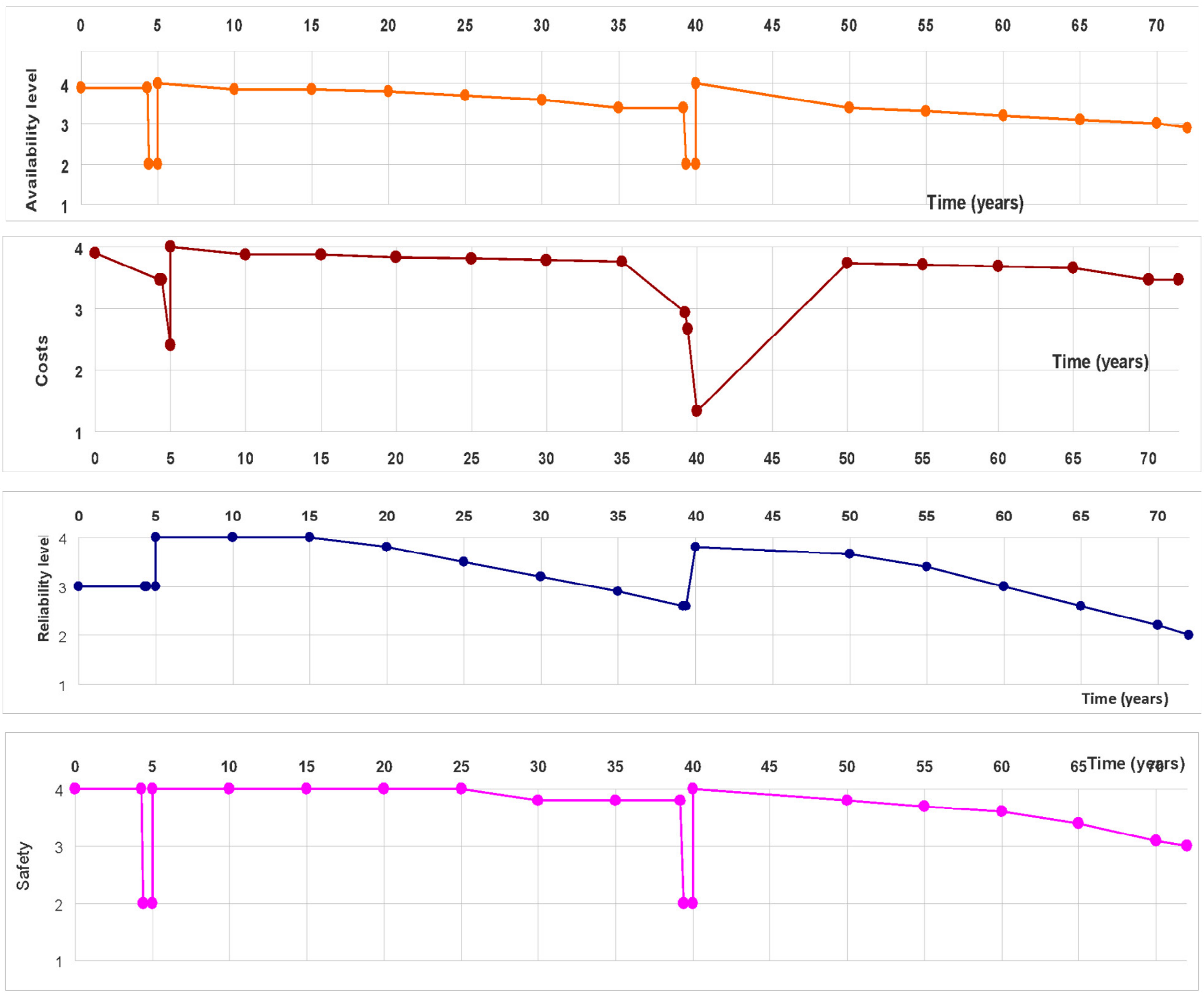
4.3. PREVENTIVE APPROACH

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.

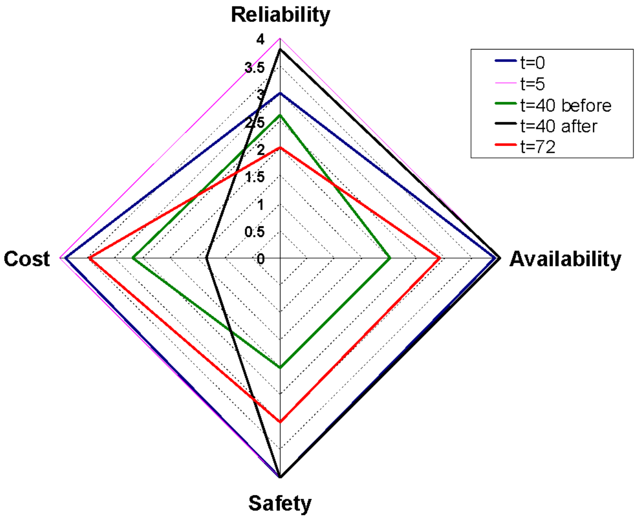
The planned interventions are as follows:

- 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:



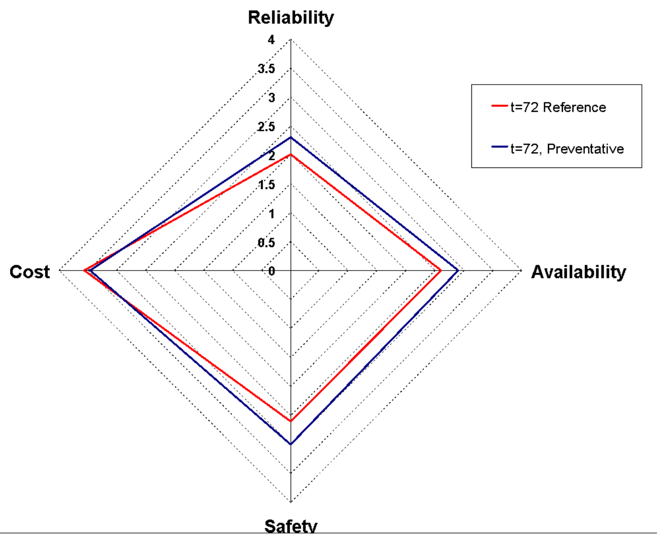
For various time points of the remaining bridge life, following the reference approach the performance of the bridge for the 4 selected KPIs is shown in the following spidergram.

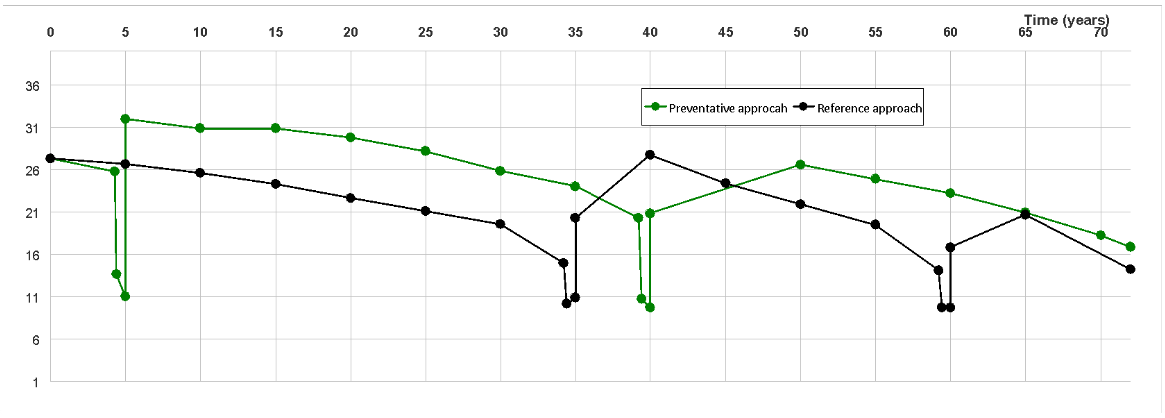


4.4. COMPARISON OF THE APPROACHES

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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