TU1406
COST ACTION

QUALITY SPECIFICATIONS FOR ROADWAY BRIDGES,
STANDARDIZATION AT A EUROPEAN LEVEL

TU1406 WG4 Final report
Appendix A11

Bridge Case Study

Orthotropic steel girder bridge in Veneto district, Italy

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CONTENTS

1. GENERAL DATA OF THE BRIDGE

In this section, a detailed description of the selected case study is provided.

1.1. GENERAL DESCRIPTION

The selected bridge is a coupled multi-span continuous orthotropic steel girder highway bridge located in Dese municipality, built in 1989 and crossing both two-way local roadway line and railway line connecting Venice to Trieste. Figures 1-4 show general view, elevation, areal map and under the deck view of the selected bridge case study.

Figure 1: General view of the bridge.

Figure 2: Elevation of the bridge.

Figure 3: Areal map of the bridge.

Figure 4: Under the deck view of the bridge.

1.2. SUPERSTRUCTURE

Continuous multi-span (4 and 5 spans in the two-way lines, respectively) orthotropic steel girders, realized with two main I-coupled steel beams each, constitute the bridge decks. Span lengths are between 19 m and 32 m. Figures 5 and 6 show span IDs and main deck beams and transverse deck beams IDs used for identifying defects during visual surveys, whereas Figures 7 and 8 illustrate original design drawings of the deck.

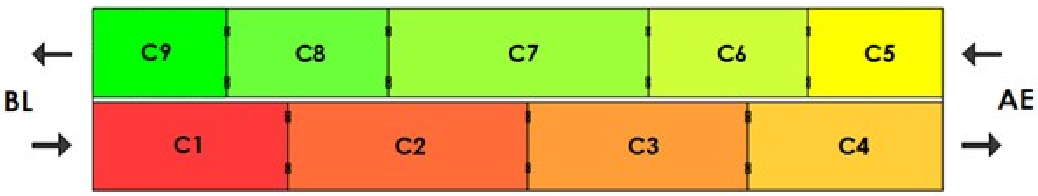


Figure 5: Spans IDs for the analyzed bridge case study.

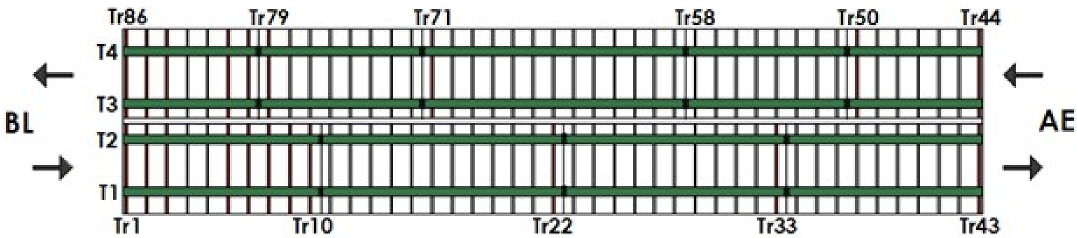


Figure 6: Longitudinal and transverse deck beams IDs for the analyzed bridge case study.

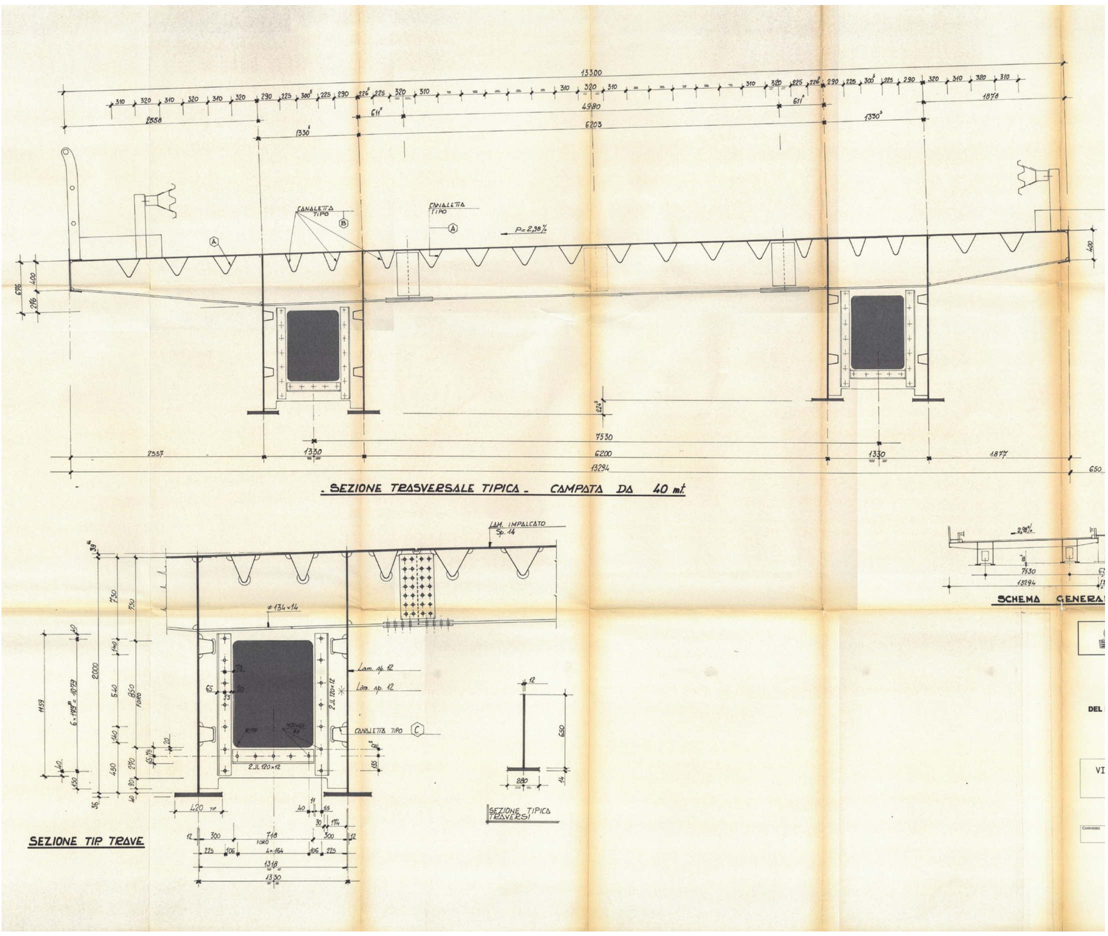


Figure 7: Deck section – original drawings -1/2.

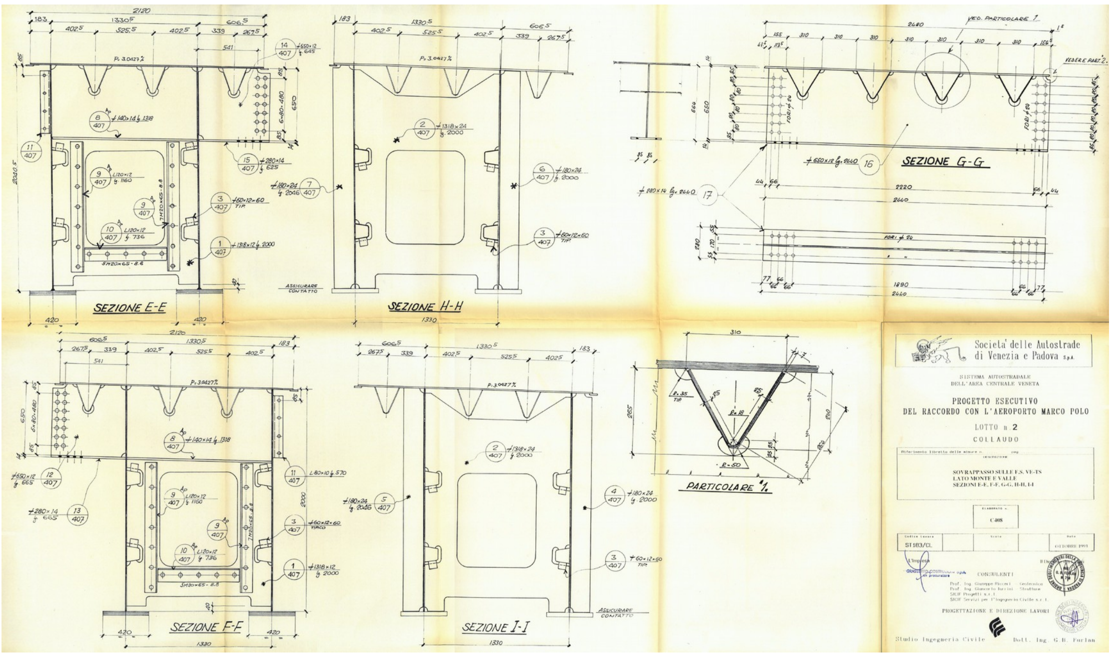


Figure 8: Deck section – original drawings -2/2.

1.3. SUBSTRUCTURE

Steel decks are supported by coupled multiple reinforced concrete piers, with maximum pier height equal to 9 m. Abutments are made in reinforced concrete and belongs to the stub abutment with cantilever retaining wall façade type. Figure 9 shows main piers and abutment ID schemes used during bridge inspections whereas Figures 10 and 11 illustrates some details of the original drawings for piers and abutments, respectively.

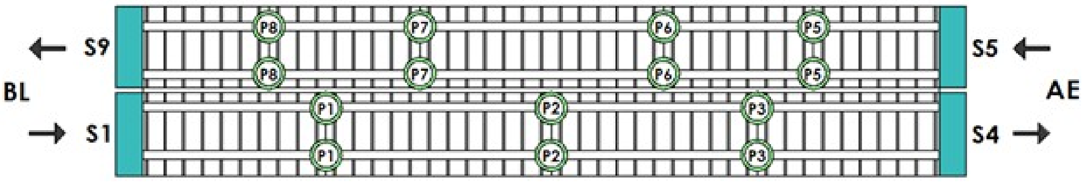


Figure 9: Piers and abutments IDs for the analyzed bridge case study.

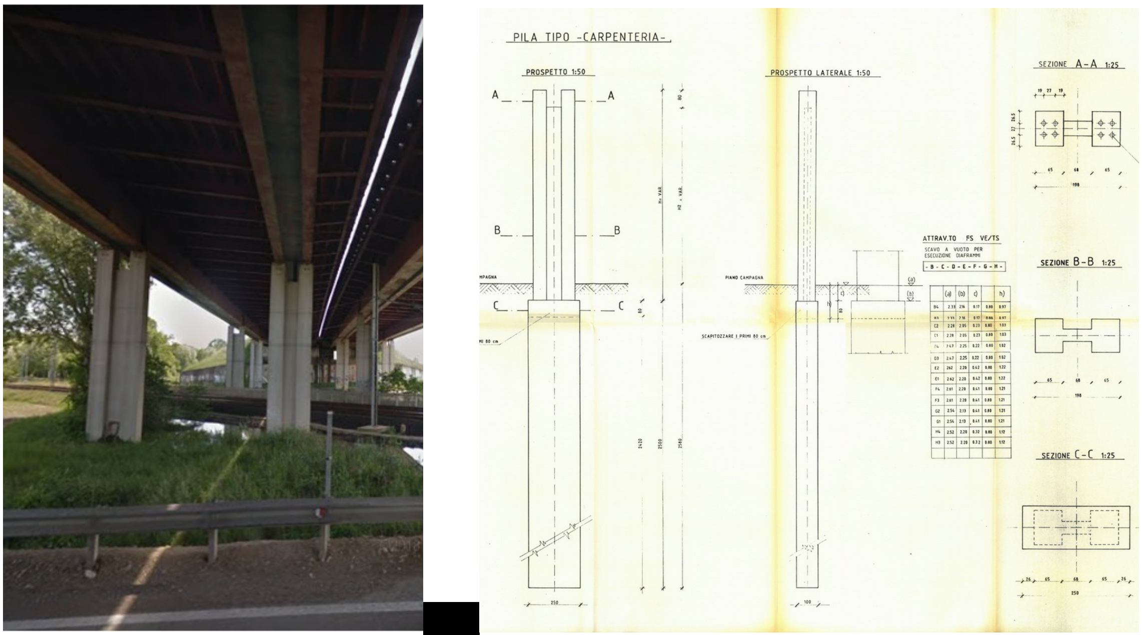


Figure 10: Piers – original drawings.

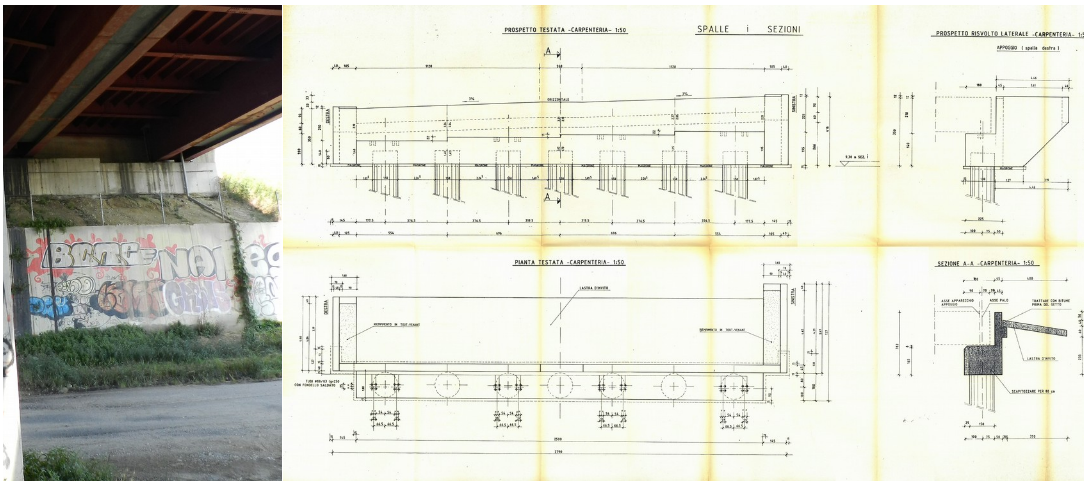


Figure 11: Abutments – original drawings.

1.4. FOUNDATIONS

Foundations are realized with grouted rectangular reinforced concrete diaphragms of 25-m depth for piers and 7 grouted circular piles of 1.5 m diameter for each abutments.

1.5. ACCESSORIES

Deck is orthotropic steel type, so over the steel corrugated web there is an asphalt layer of about 15 cm thickness. The drainage water from the deck is realized with vertical steel tubes to the ground.

1.6. LOAD CARRYING CAPACITY AND NATURAL HAZARDS

The bridge was designed according to the Italian Design Codes “*Criteri generali e prescrizioni tecniche per la progettazione, esecuzione e collaudo di ponti stradali*” issued in 1980. Actually, load carrying capacity is gonna be re-analyzed accordingly to the current Italian Design Codes “*Norme Tecniche per le Costruzioni*” issued in 2018. The bridge is not subject to hydraulic hazards, and is located in a low-to-moderate seismic hazard area.

1.7. TRAFFIC INFORMATION

The bridge is located on a highway branch linking Venice to its airport. Information about annual average daily traffic (AADT) of cars and trucks is not available, whereas a detour path exists in case of bridge closure and it implies a detour length of about 2 km. Figure 12 shows detour path in case of bridge closure.

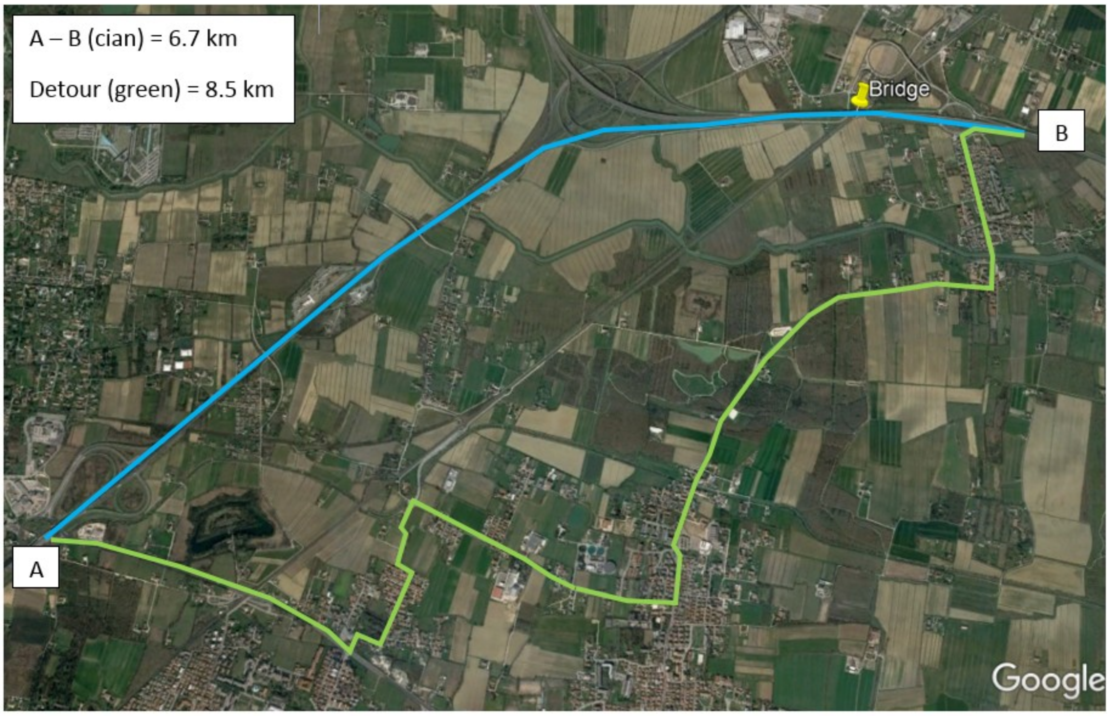


Figure 12: Detour estimation in case of bridge closure.

1.8. CONDITION RATING OF THE BRIDGE

Bridge Condition Performance Indicator has been rated using Total Sufficiency Rating (TSR) index. TSR assessment is done with the procedure detailed in A17. Zanini M.A., Faleschini F., Pellegrino C. (2017) Bridge residual service-life prediction through Bayesian visual inspection and data updating. Structure and Infrastructure Engineering, 13(7): 906-917. ISSN: 1573-2479, DOI: 10.1080/15732479.2016.1225311. TSR ranges between 0 (worst condition) and 100 best condition). The analyzed orthotropic steel girder bridge has a TSR equal to 62.

1.9. VULNERABLE ZONES

The vulnerable zones are marked on the following Figures:

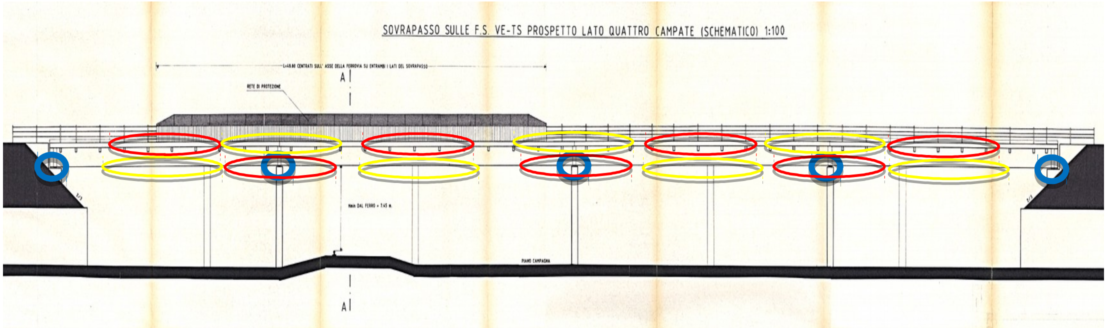


Fig. 13 Vulnerable zones – main truss (**Red**=high compression zones, **Yellow**=high tension zone, Blue=Bearing area.

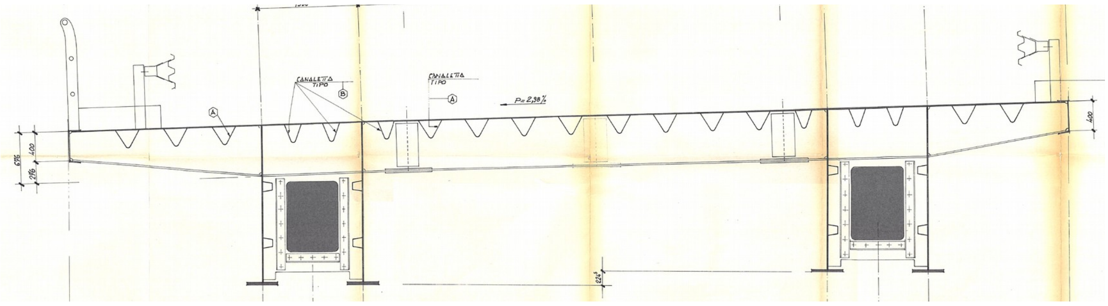


Fig. 14 Vulnerable zones –deck cross section at mid-span (**Red**=high compression zones, **Yellow**=high tension zone.

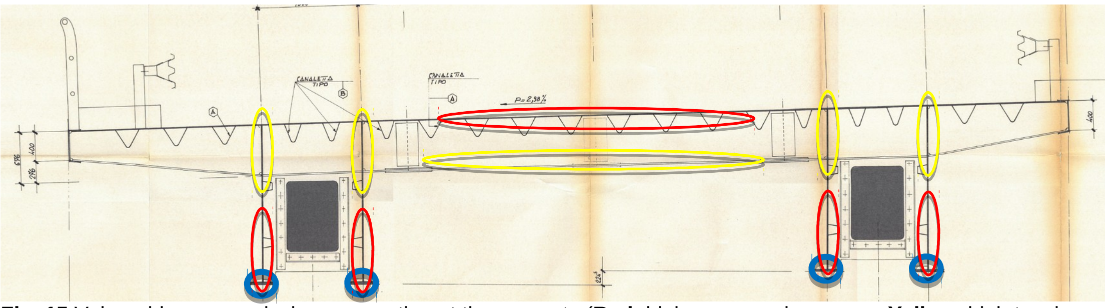


Fig. 15 Vulnerable zones –deck cross section at the supports (**Red**=high compression zones, **Yellow**=high tension zone, Blue=Bearing area.

2. TECHNICAL CONDITION

2.1. COLLECTION OF DEFECTS

The main types of defects discovered on the bridge inspection are:

- Steel painting delamination due to development of light corrosion along main beams;
- Steel painting delamination due to development of light corrosion along transverse beams;
- Steel painting delamination due to development of light corrosion over steel corrugated web;
- Pitting corrosion due to chlorides (the bridge is close few km from seaside);
- Concrete deterioration mainly in abutments than in piers;
- Vegetation on abutments;
- Drainage pipes broken or damaged.

2.2. DEFECTS OF THE MAIN STRUCTURAL ELEMENTS

Figure 16: Steel painting delamination and corrosion in transverse beams.

Figure 17: Steel painting delamination and corrosion in main longitudinal beams.

Figure 18: Concrete deterioration in RC abutments and vegetation.


Figure 19: Water drainage broken or damaged pipes.

3. POTENTIAL FAILURE MODES OF THE BRIDGE

In accordance with current condition of the bridge, following failure modes are considered:

ULS:
- **Deck failure** – Yielding (tension) or bucking (compression) of some relevant longitudinal beams in case of heavy load transportation vehicle due to reduced steel profile sections with respect to design ones due to pitting corrosion.
- **Deck failure** – Local failure of V-ribs due to cyclic loading (fatigue).
- **Failure due to Seismic Loading** – Even if the bridge is located in a low seismic hazard area, horizontal actions can be critical due to the substantial gravity design approach used in designing the bridge.

SLS:
- **Bearing Failure** – Loss of functioning of bearings due to corrosion and/or debris accumulation.
- **Joints Failure** – Loss of functioning of joints due to corrosion and/or debris accumulation.
- **Asphalt Pavement Failure** – Due to nonfunctioning joints or drainage.
- **Main Safety Barrier Failure** – Due to accidental load from heavy transportation vehicle.
- **Water Drainage Pipes Failure** – Broken or nonfunctioning drainage pipes that can accelerate deterioration of closer structural/non-structural elements.

4. DETERIORATION MODELS

A linear relationship has been preliminary assumed between TSR and reliability index (RI), ranging between TSR=100 – Reliability Index=6 and TSR=0 – Reliability Index = 1. According to COST TU1406 WG3 report, 5 Reliability Levels have been considered from the best (RL1) to the worst (RL5) in order to match quantitative assessment with qualitative classes to be linked with inspection outcomes. Figure 20 shows correlation between RI, TSR and RL. Further analyses are required to improve the representativeness of these relationships.

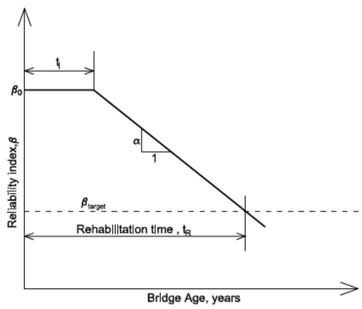
Reliability Index	6	5	4	3	2	1
TSR	100	80	60	40	20	0
Reliability Level	1	2	3	4	5	

Figure 20: Assumed correlation between RI, TSR and RL.

Herein, a bi-linear deterioration model for reliability index was used:

$\beta(t)=\beta_o \qquad 0 \leq t \leq t_i$

$\beta(t)=\beta_o-(t-t_i)\alpha \qquad t \geq t_i$



where $\beta(t)$ is the time-dependent reliability index, t_i is the time of initiation of deterioration of the reliability index herein assumed equal to 5 years, α is the deterioration rate of reliability (assumed equal to 0.07 year) and t is the age of the bridge (years).

The bridge was built in 1989, so assuming an initial RI = 6 for the new bridge (i.e. TSR=100), the value is constant up to 1994, and then decrease linearly up to the time of the inspection (2018) with loss in reliability index equal to 24 years * 0.07 = 1.68. This results in an estimated RI(2018) = 6 – 1.68 = 4.32, corresponding to a TSR value of about 66 (in good agreement with TSR coming from visual inspection, equal to 62).

In case of preventive actions, t_i is equal to 3 years is representative of the effect in delaying further reduction of RI due to progression of deterioration.

5. KEY PERFORMANCE INDICATORS AND QC PLAN

Key performance indicators are provided in accordance with best practice knowledge of the team and the experience with bridge inspection in Israel. The indicators are evaluated and failure modes of the bridge are estimated. Two life time cycle approaches are shown to evaluate the life time costs, reliability, availability and safety of considered truss bridge in following 100 years.

First referenced approach considers a lack of any repairs of the bridge except of very basic ones on the pavement. The bridge defects are developed until component or system failure and a comprehensive intervention is performed for the relevant component or system only while others are continuing to deteriorate.

A second Preventative approach consider first major rehabilitation of the bridge and a later periodical set of timely interventions during the life time cycle to prevent further defect development and overall damage to the structure.

1.1. CURRENT STATE EVALUATION

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

Component	Material	Design & Construction	Failure mode	Location / Position	Damage / Observation	Damage process	KPI	PI Component level
Main Beams	Steel	1989	Deck failure	Upper/lower chord bucking (compression)	Corroded plates	Corrosion	R	2
					Corroded welds	Corrosion		
				Upper/lower chord yielding (tension)	Corroded plates	Corrosion	R	1
					Corroded welds	Corrosion		
			Deck failure	Local failure of V-ribs	Corroded plates	Fatigue	R	2
					Corroded welds	Fatigue		
Transverse Beams	Steel	1989	Deck failure	Upper/lower chord bucking (compression)	Corroded plates	Corrosion	R	2
					Corroded welds	Fatigue		
			Deck failure	Upper/lower chord yielding (tension)	Corroded plates	Corrosion	R	1
					Corroded welds	Corrosion		
			Deck failure	Local failure of V-ribs	Corroded plates	Corrosion	R	3
					Corroded welds	Leaching		
Bearings	Steel	1989	Bearing Failure	Abutments	Bearing restrained no movement due to corrosion and debris	Corrosion	R	2
Bearings	Steel	1989	Bearing Failure	Piers	Bearing restrained no movement due to corrosion and debris	Corrosion	R	2
Joints	Steel	1989	Joints Failure	Abutments	Closing of joints	Deck movement	R	2
Wing walls	Reinforced concrete	1989	-	Wing walls on abutments	Spalling, vegetation, cracks	Corrosion	R	3
Pavement	Asphalt	2009	Skid resistance	Top surface deck	Cracks	Deformation	S	2
Safety barriers	Steel	2006	Falling of the deck	Safety barriers	Broken, missing parts	Impact	S	2
Water drainage	Steel	1999	Loss of functioning	Piers and abutments	Induced corrosion to washed reinforced concrete members	Corrosion	S	4

The estimated failure time is assumed according to the team experience with steel and concrete structures in Italy and estimated progress of the defects.
Further work should be done regarding this part.

5.1. REFERENCED APPROACH

The reference approach is lacking of any major repairs of the bridge component, leaving the bridge running to failure. For this scenario, availability, costs and safety were evaluated only qualitatively. Availability is decreasing rapidly during the transition of the reliability from level 4 to level 5. It has highest value during the replacement of the bridge. Since no maintenance in this scenario was considered, costs were included only due to the bridge replacement. Decrease of the user safety was considered to be faster than the decrease in structural reliability. Supposing a unit replacement cost of 1000€/m^2 deck surface, the deck surface area being equal to 1670m2 for each carriageway, in total a 3’340’000€ for full replacement of bridge is estimated. Figure 21 shows results in terms of reliability level, availability, costs and safety for the referenced approach.

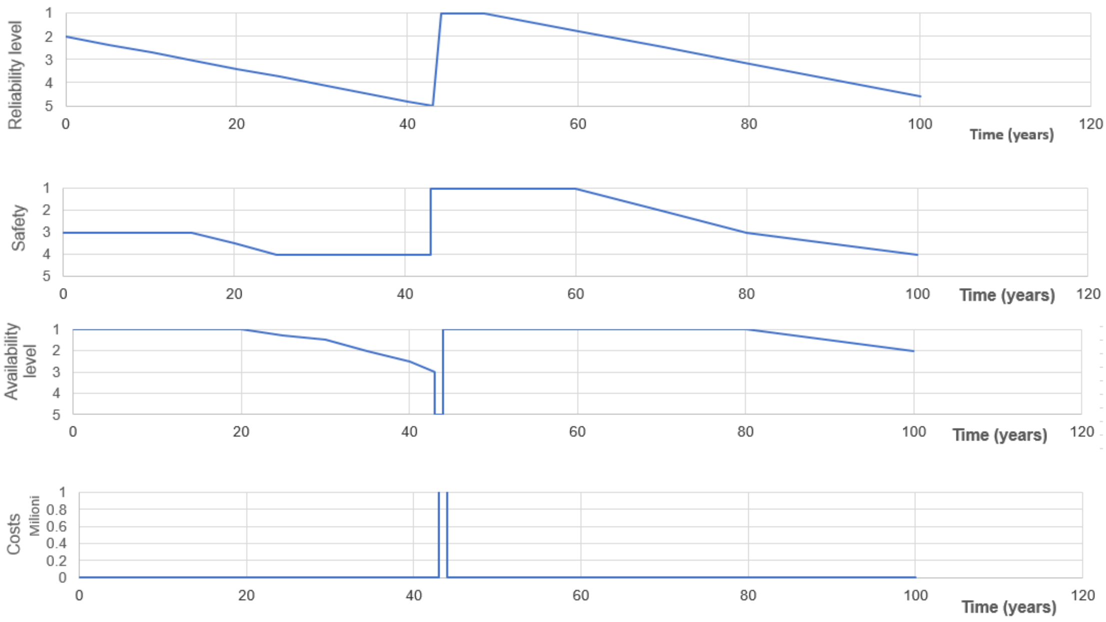


Figure 21: Qualitative performance indicators for the “Referenced” scenario.

5.2. PREVENTIVE APPROACH

The concept of preventive bridge maintenance suggests that repairs and activities are performed to keep the bridge in good condition and thereby avoid large expanses in major rehabilitation or replacement. In the present example it has been considered to implement repairs when bridge reach a reliability level of 3, instead of running to failure (i.e. to RL = 5) as in the referenced approach described above. This reflects into a reduced impact in terms of availability, with partial closure of the bridge, and lower costs for minor repair works, estimated in 40% of the replacement cost of the bridge. Figure 22 shows results in terms of reliability level, availability, costs and safety for the preventive approach.

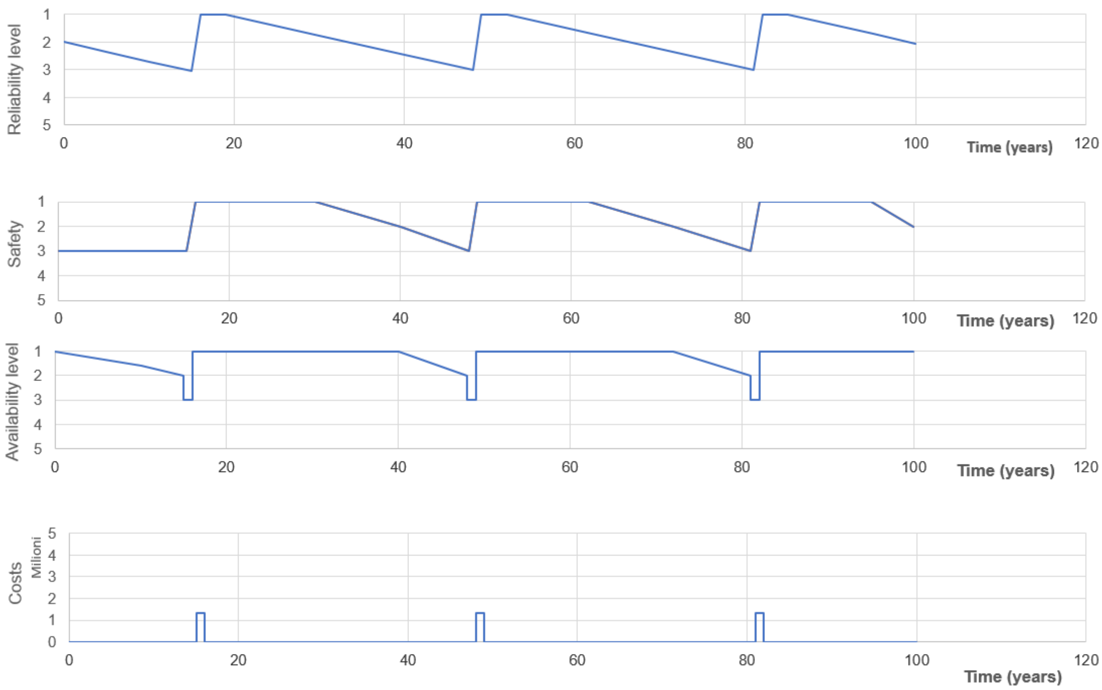


Figure 22: Qualitative performance indicators for the “Preventive” scenario.

5.3. COMPARISON OF THE APPROACHES

Lastly, Figure 23 shows a comparison of the two considered approaches in terms of “spider” diagram. According to the analysis the preventive approach is clearly more appropriate for this bridge: it can be noted in fact how the area under the spider is higher for preventive scenario with respect to referenced one. So, even if the cost is little more, all other indicators shows more favorable results in terms of reliability, availability and safety. In addition, preventive scenario allows keeping the reliability and safety in higher levels all over the period, with respect to values considered in referenced scenario.

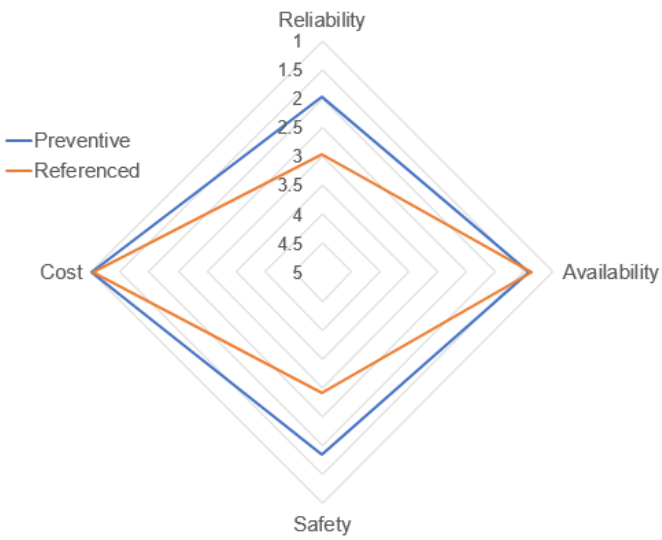


Figure 23: Comparison between considered preventive and referenced maintenance scenarios.

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