COST ACTION TU1406

QUALITY SPECIFICATIONS FOR ROADWAY BRIDGES,
STANDARDIZATION AT A EUROPEAN LEVEL

TU1406 WG4 Final report Appendix A10

Bridge Case study

Girder bridge Piiometsa in Estonia

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CONTENTS

1. INTRODUCTION

The Piiometsa bridge was chosen as a Case study bridge, because it is a common typology in all Baltic countries and it has been assessed based on 5 different methods: regular bridge inspections with historical information, damage assessment according to COST TU1406, material properties testing, load testing and carrying capacity calculations based on design documents. Compared to other case study bridges, the traffic intensity is low, but bridge is often used by timber trucks.

2. GENERAL DATA OF THE BRIDGE

Piiometsa (no. 235) is located on a secondary road and connects Paide town with smaller villages. The bridge was built in 1963 and edge beams and deck covering was renovated in 1998, it is a simply supported reinforced concrete beam structure, which is designed according to catalogue Типовые проекты сооружений на автомобильных дорогах. Выпуск 56 (1958). Extracts of the catalogue can be found in Figures 1-3.

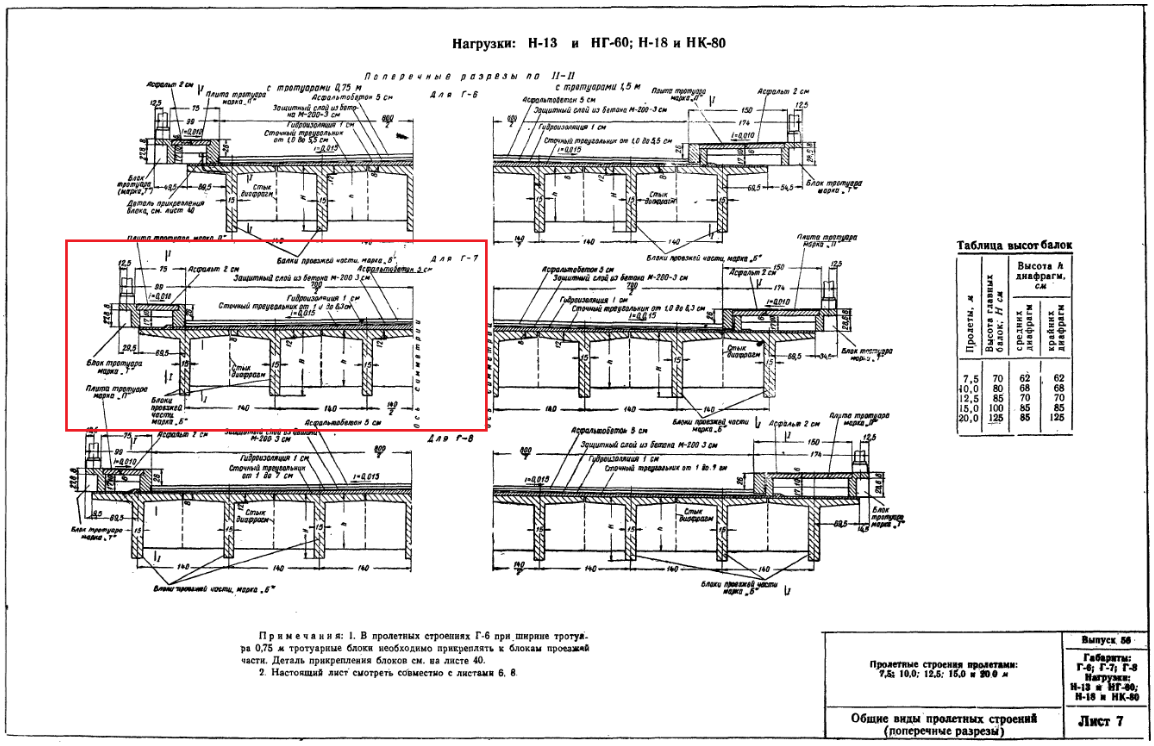


Figure 1. Cross-section of initial Piiometsa bridge.

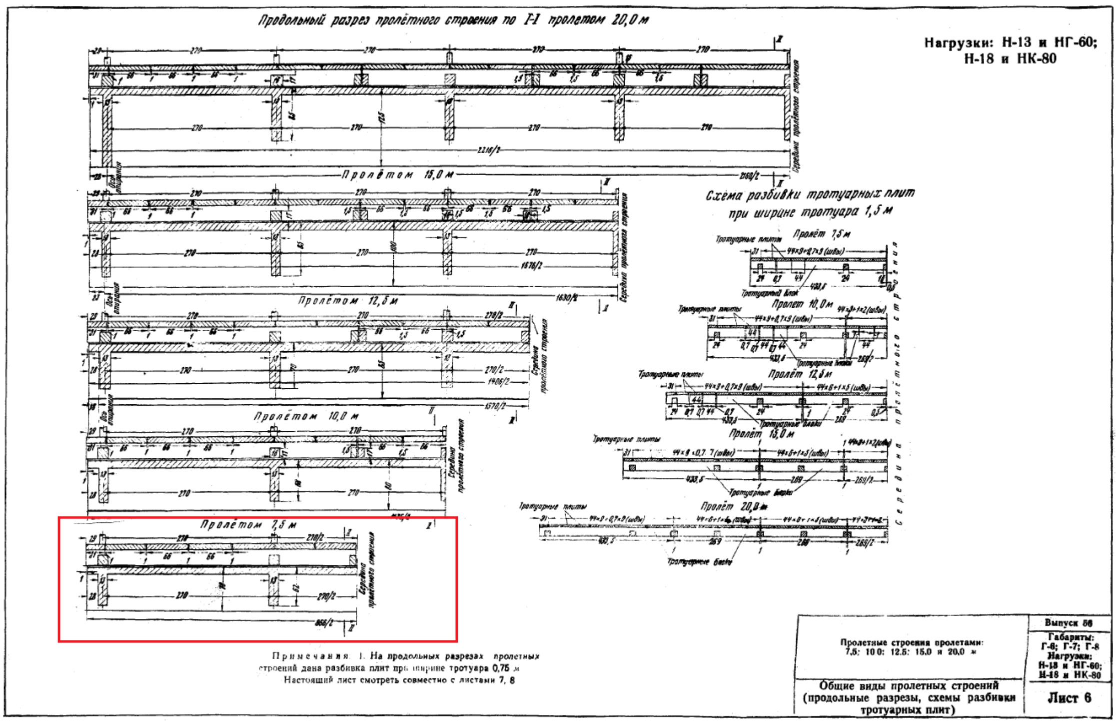


Figure 2. Longitudinal section of initial Piiometsa bridge

Нагрузка Н-13 и НГ-60

Пролет в свету, м	Расчетный пролет, м	Главная балка / усилия / постоянная нагрузка: момент в середине пролета, тм	постоянная нагрузка: поперечная сила на опоре, т	постоянная нагрузка: поперечная сила в середине пролета, т	толпа: момент в середине пролета, тм	толпа: поперечная сила на опоре, т	толпа: поперечная сила в середине пролета, т	Н-13: момент в середине пролета, тм	Н-13: поперечная сила на опоре, т	Н-13: поперечная сила в середине пролета, т	НГ-60: момент в середине пролета, тм	НГ-60: поперечная сила на опоре, т	НГ-60: поперечная сила в середине пролета, т	расчетные: момент в середине пролета, тм	расчетные: поперечная сила на опоре, т	расчетные: поперечная сила в середине пролета, т	высота главной балки, см	армирование	нормальные напряжения в середине пролета, кг/см²: в бетоне допускаемые	в бетоне полученные по расчету	в арматуре средние допускаемые	в арматуре средние полученные по расчету	в арматуре в крайнем ряду допускаемые	в арматуре в крайнем ряду полученные по расчету	скалывающие напряжения, кг/см²: на опоре допускаемые	на опоре полученные по расчету	в середине пролета допускаемые	в середине пролета полученные по расчету	принятая марка бетона
7,5	8,4	9 50	4,52	0	0,66	0,32	0.08	16,7	11,10	3,60	29,30	17,6	5,25	38,80*	22,12*	5,25*	70	4 № 32	157	62	2080	2000	2280	2090	28,6	24,50	28,6	5,78	250
10,0	11,1	17,10	6,15	0	1,14	0,42	0,10	24,8	12,77	4,90	42.50	19,4	6,90	59,60*	25,55*	6,90*	80	4 № 32 6 № 16	157	75,2	2080	2100	2280	2230	28,6	24,10	28,6	7,20	250
12.5	13.7	26,60	7,77	0	1,76	0,52	0,13	32,9	13,87	5,14	55,40	20,4	7,94	82,00*	22,20*	7,94*	85	6 № 32 2 № 16	157	90,5	2080	2110	2280	2260	22,0	19,65	28,6	7,13	250
15,0	16,3	40,25	9,85	0	2,45	0.61	0.15	40.5	14,75	4,35	68,30	21,3	6,82	108,55*	25,20	6,82*	100	8 № 32	157	93,3	2080	2060	2280	2220	22,0	18,50	28.6	5,58	250
20,0	21,6	76,40	14,15	0	4,40	0,81	0,20	62,0	16,40	4,36	94,50	22,2	7,60	142,80	31,36	7,60*	125	10 № 32 2 № 16	121	87,5	1600	1590	1750	1785	22,0	18,05	28,6	4,75	250

* Расчетные усилия получены от нагрузки НГ-60.

Нагрузка Н-18 и НК-80

Пролет в свету, м	Расчетный пролет, м	Главная балка / усилия / постоянная нагрузка: момент в середине пролета, тм	постоянная нагрузка: поперечная сила на опоре, т	постоянная нагрузка: поперечная сила в середине пролета, т	толпа: момент в середине пролета, тм	толпа: поперечная сила на опоре, т	толпа: поперечная сила в середине пролета, т	Н-18: момент в середине пролета, тм	Н-18: поперечная сила на опоре, т	Н-18: поперечная сила в середине пролета, т	НК-80: момент в середине пролета, тм	НК-80: поперечная сила на опоре, т	НК-80: поперечная сила в середине пролета, т	расчетные: момент в середине пролета, тм	расчетные: поперечная сила на опоре, т	расчетные: поперечная сила в середине пролета, т	высота главной балки, см	армирование	нормальные напряжения в середине пролета, кг/см²: в бетоне допускаемые	в бетоне полученные по расчету	в арматуре средние допускаемые	в арматуре средние полученные по расчету	в арматуре в крайнем ряду допускаемые	в арматуре в крайнем ряду полученные по расчету	скалывающие напряжения, кг/см²: на опоре допускаемые	на опоре полученные по расчету	в середине пролета допускаемые	в середине пролета полученные по расчету	принятая марка бетона
7,5	8,4	9,50	4,52	0	Расчетного значения не имеет						33,6	24,8	7,50	43,10*	29,32*	7,50*	70	4 № 32 2 № 16	178	67,0	2080	1920	2280	2020	31,5	31.7	31.5	8,07	300
10,0	11,1	17,10	6,15	0	То же						53,4	26,6	9,26	70,50*	32,75*	9,26*	80	6 № 32 2 № 16	178	85,5	2080	1990	2280	2150	31,5	31,0	31,5	9,1	300
12,5	13,7	26,60	7,77	0							72,6	27,8	10,70	99,20*	35,57*	10,70*	85	8 № 32 4 № 16	178	108,0	2080	2010	2280	2270	31,5	31,4	31,5	10,3	300
15,0	16,3	40,25	9,85	0	2,45	0,61	0,15	48,9	19,2	5,67	89,0	29.0	10,00	129,25*	38,85*	10,00*	100	8 № 32 6 № 16	178	109,8	2080	2070	2280	2300	31,5	28,6	31,5	8,1	300
20,0	21,6	76,40	14,15	0	4,40	0,81	0,20	67,2	21,0	6,00	122,9	29,9	10,70	199,30*	35,96*	10,70*	125	12 № 32	178	122,0	2080	1990	2280	2290	24,2	20,8	31,5	6,9	300

* Расчетные усилия получены от нагрузки НК-80.

Пролетные строения пролетами: 7,5; 10,0; 12,5; 15,0 и 20,0 м	Выпуск 56 Габариты: Г-6; Г-7; Г-8 Нагрузки Н-13 и НГ-60; Н-18 и НК-80
Расчетный лист	Лист 1

Figure 3. Measurements and properties of Piiometsa bridge typology.

The bridge consists of 2 spans with 6 beams connected with cross beams. The abutments and pier are all constructed on piles. In total, the bridge has a length of 17.4 m and width of 8.4 m. There are no bearings between super- and sub-structure. During the reconstruction in 1998, only the top of a bridge was changed: precast reinforced concrete pedestrian pathway segments were removed and bigger edge beam with safety barriers were installed. Figures 4-7 gives an impression of the overall structure.

Figure 4. Side view of the bridge (left side)

Figure 5. Side view of the bridge (right side)

Figure 6. View from underneath the bridge

Figure 7. View from the top on the bridge

The original design documentation does not exist, so the measurements of the existing bridge were taken in September 2018 and are presented on Figures 8-10.

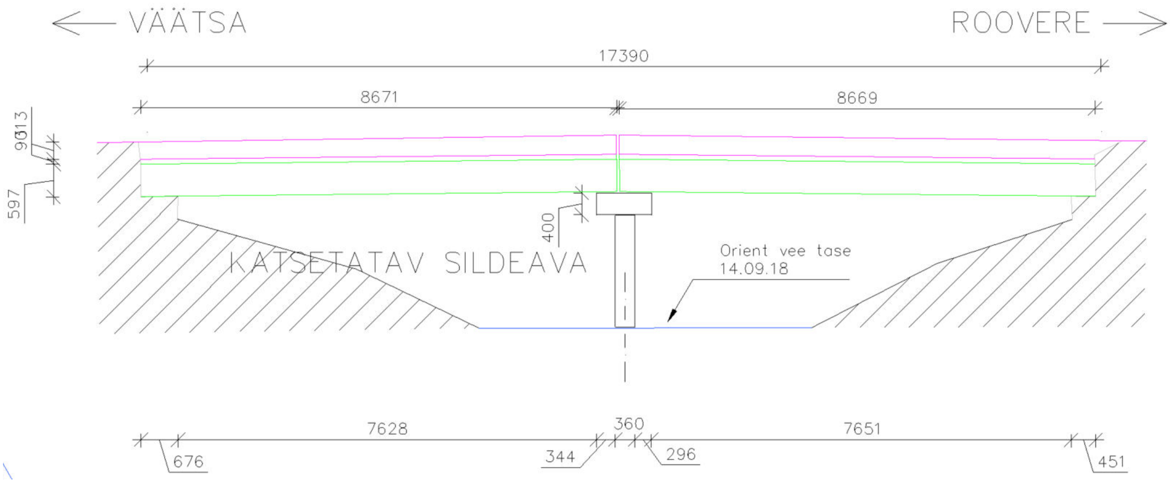


Figure 8. Side view of the Piiometsa bridge

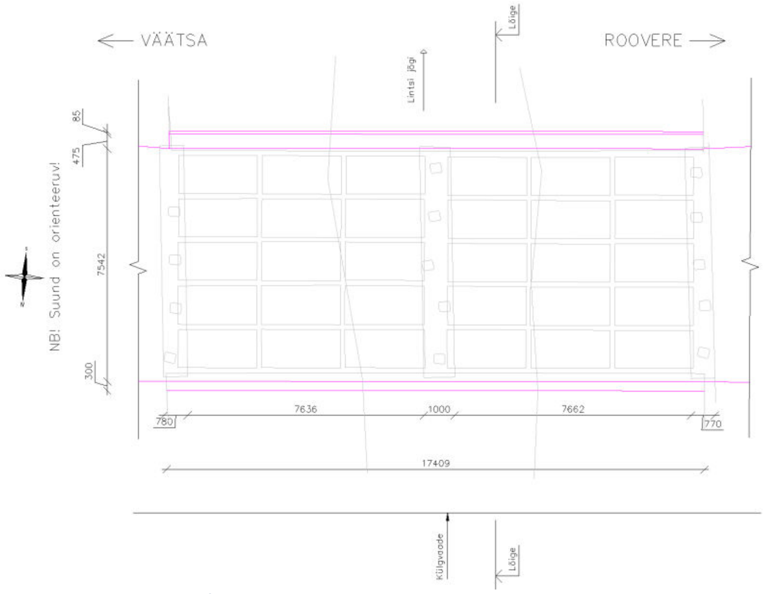


Figure 9. Top view of Piiometsa bridge

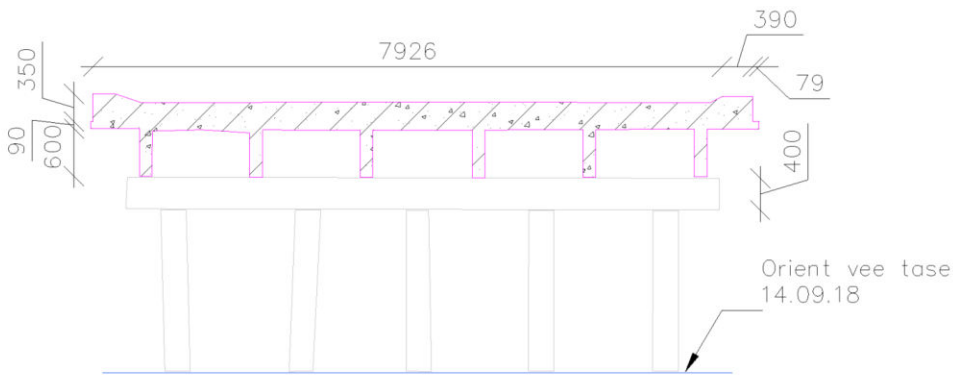


Figure 10. Cross-section of Piiometsa bridge

2.1. TRAFFIC INFORMATION

The last information about the traffic are calculated based on the last counting and analytical models for the year 2017.
Number of cars / 24h : 216
Percentage of the heavy vehicles from the total amount / 24h : 10%

2.2. SUBSTRUCTURE

Substructure is formed by the pile-abutments (label AB3 in WG3 Report) and pile piers (label P6 in WG3 Report) constructed from the reinforced concrete.

2.3. SUPERSTRUCTURE

The superstructure consists of precast reinforced concrete beams with cross-beams connected with welded steel plates.

2.4. ACCESSORIES

The cover of deck plate was initially gravel but has been covered with asphalt layer and the safety railings are made from galvanized steel. No special slope protection has been built, and embankments have erosion.

2.5. LOAD CAPACITY

The load carrying capacity of the bridge is designed for Soviet era traffic loads N-13 (Figure 11) and special vehicle NG-60 (Figure 12).

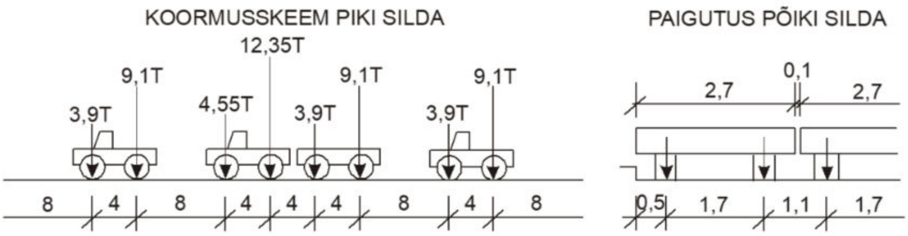


Figure 11. Load model to imitate motorcade N-13

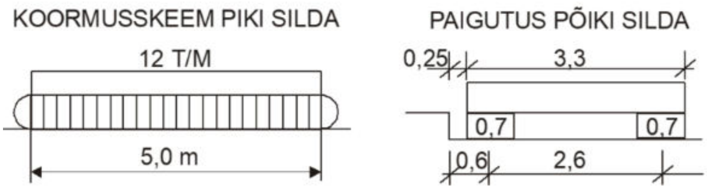


Figure 12. Load model to imitate special vehicle NG-60.

More detailed calculations of load carrying capacity of the existing bridge are provided in paragraph 4.5

2.6. GENERAL DATA ACCORDING TO WG3

In addition to overall description, the information is also presented as proposed in WG3 Final report (Table 1). The information can be expanded as needed.

Table 1.General data of Glattfelden bridge

Structure type	Other relevant data (can be expanded as needed)								
	Location	Environmental exposure	Seismicity	Building year	Length, m	Width, m	Traffic volume	Previous intervention	
								Date	Type
GA1	Estonia, 15129 Paide - Roovere - Kuimetsa, 10.592 km	rural	NA	1963	17.4	8.4	216 (2017)	1998	Reconstruction

2.7. IDENTIFICATION AND SEGMENTATION OF BRIDGE ELEMENTS

Since the primary knowledge about materials, deterioration processes and damages come from the ontology, then the elements are listed according to proposed taxonomy. As stated in previous chapter, it is a simply supported girder bridge built from reinforced concrete. Elements are grouped according to Estonian bridge management system, but explanations are taken from WG3 Final report.
Since Piiometsa is a simple bridge, then most of the functions are easily grouped, but since the original drawings aren't are also elements with non-available information (marked as NA). Elements are listed in Table 2.

Table 2. List of bridge elements and grouping

Element	Primary function	Typology	Material	Quantity	Unit
Deck slab	Load bearing	SA1	Reinforced concrete	134	m2
Main girder	Load bearing	GA1	Reinforced concrete	12	Pcs
Cross beam	Load bearing	NA	Reinforced concrete	40	Pcs
Abutments incl. Wing walls	Load bearing	AB3	Reinforced concrete	16	m
Pier	Load bearing	P2	Reinforced concrete	8	m
Foundations	Load bearing	FU1	Reinforced concrete	NA	NA
Piles	Load bearing	NA	Reinforced concrete	5	Pcs
Bearings	Articulation/load bearing	NA	NA	NA	NA
Expansion joints	Articulation	Buried	NA	16	m
Run-on slab	Comfort	NA	Asphalt concrete	60	m2
Waterproofing	Protection	NA	NA	134	m2
Pavement/Overlay	Protection and comfort	NA	Asphalt concrete	134	m2
Hand-rail	Protection and comfort	Hollow	Steel	35	m
Barrier	Protection and comfort	W-beam	Steel	35	m
Run-on barrier	Protection and comfort	W-beam	Steel	130	m
Signs	Protection and comfort	Reflectors	Steel	4	Pcs
Embankment	Protection	NA	Soil	91	m2
River bed	Protection	NA	Soil	8	m

3. IDENTIFICATION OF FAILURE MODES AND DEFINITION OF VULNERABLE ZONES

Since the Piiometsa bridge represent a common typology, that has been built in Baltics from 1958, then there are a lot of information collected for this typology. For this typology, most common failure area is the connection of cross beams, which tend to corrode and break. This can’t be directly formulated as a conceptual weakness, but in connection with poor build quality and lack of maintenance, there has been some partial collapses due to this section (Figure 13). In addition, the deformation joints tend to leak and will increase the deterioration of material.

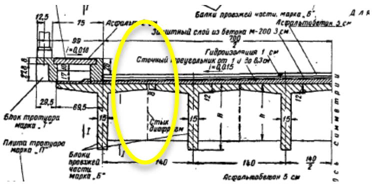


Figure 13. Conceptual weakness of Piiometsa bridge typology

Vulnerable zones related to superstructure are typical to simply supported beam, where bending failure can occur in the middle span and shear failure near the end of the beam (Figure 14).

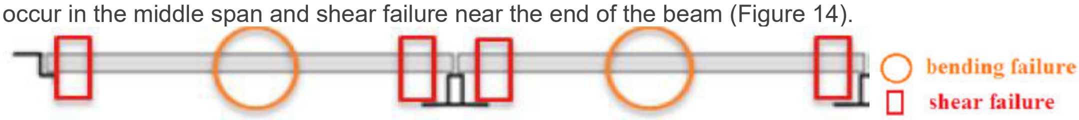


Figure 14. Visualization of vulnerable zones of Piiometsa bridge superstructure

For the substructure, which consist of piles, the failure can occur due to compression, accompanied with shear or bending, only shear and buckling. In addition, the connection of sub-and superstructure is a vulnerable zone (Figure 15).

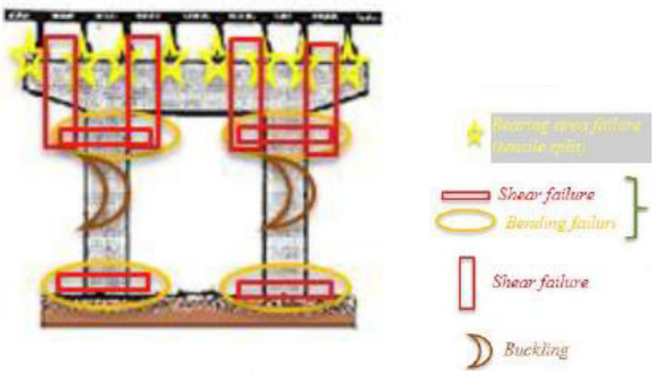


Figure 15. Visualization of vulnerable zones of Piiometsa bridge substructure

As mentioned in previous section, the damage caused by missing bearings can also be treated as a failure mode. All the possible visible failure modes are listed in Table 3.

Table 3. List of all possible failure modes

Element	Failure mode	Location of damage
Deck slab	Bending failure	Mid span
Deck slab	Shear failure	End of span
Main girder	Bending failure	Mid span
Main girder	Shear failure	End of span
Cross beam	Bending failure (transversal)	Side connection

Cross beam	Shear failure (lateral)	All
Abutments incl. Wing walls	Compression failure	Center
Abutments incl. Wing walls	Bending failure (due to compression)	Sides
Abutments incl. Wing walls	Shear failure (due to compression)	Sides
Abutments incl. Wing walls	Shear failure	Sides
Pier	Compression failure	Center
Pier	Bending failure (due to compression)	Sides
Pier	Shear failure (due to compression)	Sides
Pier	Shear failure	Sides
Foundations	Compression failure	All
Foundations	Scour	All
Piles	Bending failure (due to compression)	All
Piles	Shear failure (due to compression)	All
Piles	Buckling	Center
Bearings	Restricted movement	All
Expansion joints	Leakage	All
Run-on slab	Comfort failure	All
Waterproofing	Leakage	All
Pavement/Overlay	Comfort failure	All
Pavement/Overlay	Safety failure	All
Pavement/Overlay	Dead load	All
Hand-rail	Safety failure	All
Hand-rail	Comfort failure	All
Barrier	Safety failure	All
Barrier	Comfort failure	All
Run-on barrier	Safety failure	All
Run-on barrier	Comfort failure	All
Signs	Safety failure	All
Signs	Comfort failure	All
Embankment	Scour	All
Embankment	Settlement	All
River bed	Scour	All

4. TECHNICAL CONDITION

The technical condition has been assessed using 5 different methods: regular bridge management inspections with historical information starting from 2006, damage assessment according, material properties testing, load testing and carrying capacity calculations based on design documents.

4.1. REGULAR BRIDGE INSPECTIONS

During the regular inspection the information is collected according to Bridge Inspection manual, which is related to AASHTO Bridge Inspection Guide Manual (AASHTO,2010), assessing the element in a scale of 1 (very good) to 4 (critical). Summary of the results are normalized to scale of 0 to 100%, where 100% shows, that structure is in a perfect condition. Since the assessment is concentrated on damages quantities, then outcome is conditional rating, because this assessment method does not take account the reliability or safety of a structure.

Table 4. Previously collected conditional information from regular inspections

			Observations					
Date	Performance cluster	Performance indicator	Failure mode	Location/position - Vulnerable zone	Degradation process (Damages/symptoms)	Primary Key Performance Indicator	Assessment level	Performance value
22.05.2006	Rating	Condition rating	NA	NA	NA	Rating	System	80,7
12.08.2013	Rating	Condition rating	NA	NA	NA	Rating	System	79,3
16.10.2015	Rating	Condition rating	NA	NA	NA	Rating	System	68,2

The last overall condition rating is lower due to fact, that the inspection was carried out during rainy day, where leakages were visible. This rating marks condition, where intervention with repair works are needed, but due to low traffic intensity and lack of safety issues the bridge has ranked to 306th place from 1005 and will be repaired in 2028.

4.2. COLLECTION OF DEFECTS

Collection of defects were done on 17th of September 2018 by a group of specialists. The types of defects discovered on the analyzed bridge are listed below and sketch of defect locations are shown on Figure 13 and 14.

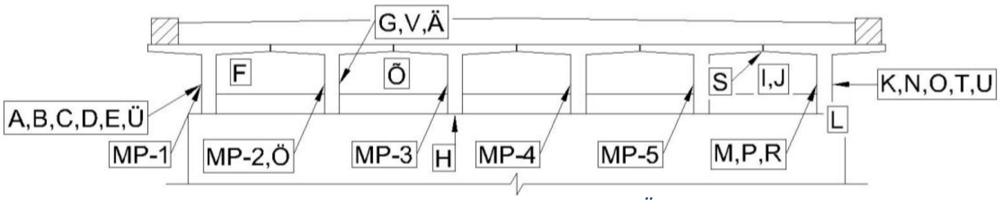


Figure 16. Locations of main defects (capital letters from A to Ü) and material properties testing (MP-1 to MP-5) in cross-section.

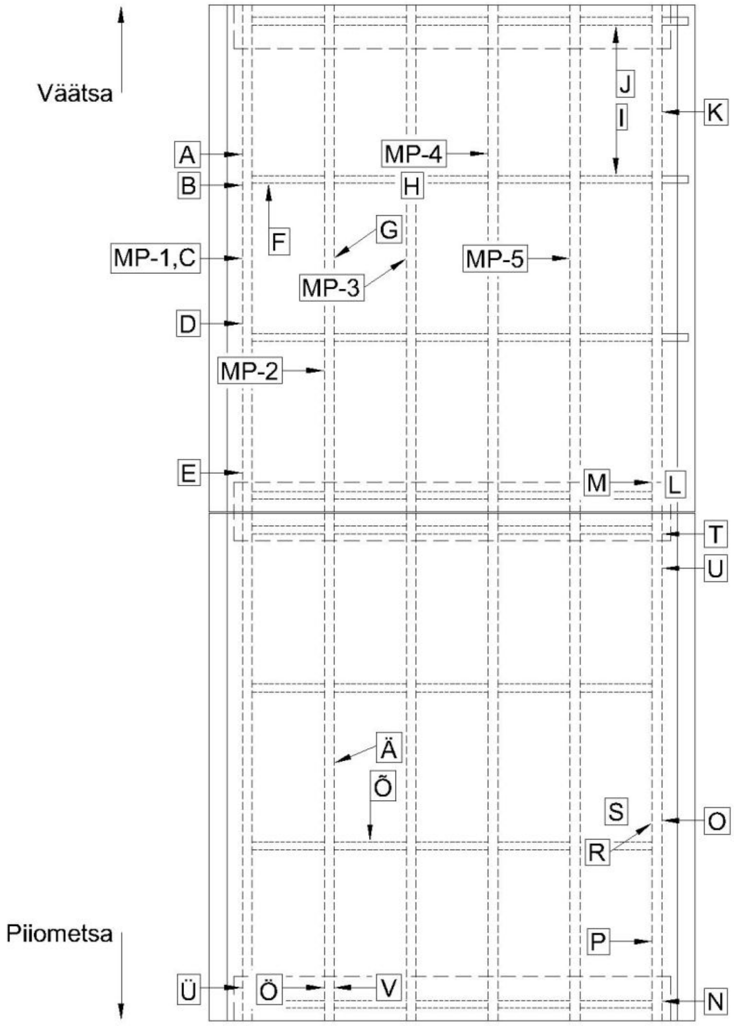


Figure 17. Locations of main defects (capital letters from A to Ö) and material properties testing (MP-1 to MP-5) from top view.

The main damages are presented and described in Table 5. The damages were examined by experienced inspector without the previous definition of vulnerable zones, failure modes and assessment of performance value.

Table 5. Pictures and short description of all collected damages of Piiometsa bridge

Figure	Location, Description
	A: spalling of concrete and corrosion of rebar on main girder
	B: spalling of concrete and corrosion of rebar on main girder
	C: spalling of concrete and corrosion of rebar on main girder

	D: spalling of concrete and corrosion of rebar on main girder
	E: spalling of concrete and corrosion of rebar on main girder
	D: spalling of concrete and corrosion of rebar on cross-beam
	G: thin protective layer of a rebar, exposure of a rebar and spalling of concrete

	H: thin protective layer of a rebar, exposure of a rebar and corrosion.
	I: spalling of concrete and corrosion of connection plate of cross-beams
	J: spalling of concrete and severe corrosion of connection of cross-beams
	K: thin protective layer of a rebar, exposure of a rebar and spalling of concrete

	L: spalling of concrete
	M: degradation of concrete, due to leaking expansion joints
	N: degradation of concrete, due to leaking expansion joints
	O: thin protective layer of a rebar, exposure of a rebar and spalling of concrete

	P: thin protective layer of a rebar, exposure of a rebar and spalling of concrete
	R: thin protective layer of a rebar, exposure of a rebar and spalling of concrete
	S: exposure and corrosion of rebars on the top flange of main girders
	T: peeling of concrete and corrosion of rebars due to leakage of deformation joints

	U: exposure and corrosion of rebars of main girders
	V: spalling and peeling of concrete and corrosion of rebars affecting load carrying capacity of main girder.
	Õ: patchwork and cracking of concrete and corrosion of connection plate of cross-beams
	Ä: thin protective layer of a rebar, exposure of a rebar and spalling of concrete

	Ö: spalling and peeling of concrete and corrosion of rebars affecting load carrying capacity of main girder.
	Ü: peeling of concrete and corrosion of rebars due to leakage of deformation joints

In conclusion to collection of defects, the observations are presented in same format as proposed by COST TU1406 in Table 4.
The defects were assessed afterwards based on the pictures and only key performance indicator of Reliability has been evaluated in the scale of 1-5. Only reliability were assessed, because observed damages only affect this performance area. The quantitative scale in Table 6 is added as an additional information and has not been used in the assessment.

Table 6. Scale of element level reliability assessment

Reliability scale	**Qualitative scale and urgency of intervention**	**(Quantitative scale (β))**
1	Elements with no resistance reduction.	>4.00
2	No or marginal resistance reduction compared to the virgin state (< 8%).	3.25-4-00
3	Some resistance reduction compared to the virgin state (8 – 17%). In depth reassessment should be considered	2.50-3.25
4	Elements with major resistance reduction compared to the virgin state (17 – 23%). In depth reassessment and possible intervention shall be performed shortly after inspection.	2.00-2.50
5	Severe resistance reduction. Immediate action is required.	<2.00

Only elements with damages were assessed and although two damages were noted, then both of them were related to one main girder, so based on the damage assessment of reliability performance, the Piiometsa bridge has some resistance reduction compared to virgin state in main girder which will result as a shear failure.

Table 7. Collection of defects for Piiometsa bridge

Structure type	Group of elements	Observations					Performance metric		Note	Observations					Performance value
		Element	Type of cross-section	Date	Performance cluster	Performance indicator	Primary	Secondary		Failure mode	Location/position - Vulnerable zone	Degradation process (Damages/symptoms)	Primary Key Performance Indicator	Assessment level	
GA1	Superstructure	Main girder	SC2/RC	20.09.2018	Defects	Spalling	0,5m2	Mid span	A	Bending moment failure	HMM/Bott	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Scaling	0,1 m2	Mid span	A	Bending moment failure	HMM/Bott	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Scaling	0,5 m2	Mid span	B	Bending moment failure	HMM/Bott	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Scaling	0,5 m2	Mid span	C	Bending moment failure	HMM/Bott	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Scaling	0,5 m2	End of beam	D	Bending moment failure	HMM/Bott	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Scaling	1 m2	End of beam	E	Bending moment failure	HMM/Bottom	Corrosion	Reliability	Element	1
		Cross beam	SC2/RC			Scaling	0,5 m2	Connection	F	Overloading of an element	HS/Connection	Corrosion	Reliability	Element	1
		Main girder	SC2/RC			Insufficient concrete cover	1 m2	Mid span	G			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	0,5 m2	Bottom	H			Corrosion	Reliability	Element	
		Cross beam	SC2/RC			Scaling	2 m2	Connection	I	Overloading of an element	HS/Connection	Corrosion	Reliability	Element	1
		Cross beam	SC2/RC			Scaling	0,5 m2	Connection	J	Overloading of an element	HS/Connection	Corrosion	Reliability	Element	2
		Main girder	SC2/RC			Insufficient concrete	1 m2	End of beam	K			Corrosion	Reliability	Element	
		Supporting beam	SC2/RC			Scaling	2 m2	End of beam	L	Bearing area failure	Pier	Freeze thaw	Reliability	Element	2
		Main girder	SC2/RC			Scaling	1m2	End of beam	M			Freeze thaw	Reliability	Element	
		Main girder	SC2/RC			Scaling	2 m2	End of beam	N			Freeze thaw	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	1 m2	Mid span	O			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	1m2	Shear zone	P			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	2 m2	Mid span	R			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	5 m2	Topflange	S			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Scaling	1 m2	End of beam	T			Erosion and corrosion	Reliability	Element	
		Main girder	SC2/RC			Scaling	1 m2	End of beam	U			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Scaling	2 m2	End of beam	V	Shear failure	HS	Corrosion	Reliability	Element	3
		Cross beam	SC2/RC			Scaling	2 m2	Connection	O			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	2 m2	Mid span	Ä			Corrosion	Reliability	Element	
		Main girder	SC2/RC			Insufficient concrete cover	1 m2	End of beam	Ö	Shear failure	HS	Corrosion	Reliability	Element	3
		Main girder	SC2/RC			Scaling	1m2	End of beam	[illegible]			Freeze thaw	Reliability	Element	
	Substructure	Supporting beam	SC2/RC			Scaling	2 m2	End of beam	L	Bearing area failure	Pier	Freeze thaw	Reliability	Element	2

4.3. MATERIAL TESTING USING NON-DESTRUCTIVE METHODS

Material testing was carried out in 5 different places, all marked on Figure 13 and 14 as MP. Used methods are most commonly used in Estonian practice: sclerometer/rebound hammer test, carbonization depth using phenolphthalein, rebar cover and diameter measurement, tension strength of steel, electrical resistivity of concrete. 3 out of 5 methods are suggested as good addition for regular bridge inspections (Kušar, M. et al. 2018). The photos of tested places are on Figures 15-19.

All the tested locations were on main girders and picked by random choice but considering that every location should be on different beam.

Figure 18. MP-1

Figure 19. MP-2


Figure 20. MP-3

Figure 21. MP-4

Figure 22. MP-5

Test results are presented in Table 8 and Table 9. The only assessed performance area was Reliability, because the material properties only affect this indicator. All the carried out test were done according to EN standard or some other international (RILEM) or national manual.

Table 8. Overall results of non-destructive testing

Location	Rebound hammer	Concrete cover depth [mm]	Carbonization depth [mm]	Resistivity of concrete	Tensile strength of rebars

	[N/mm2]			[kOHMcm]	[N/mm2]
MP-1	62	20…30	5	20	474
MP-2	49	10…40	10	312*	-
MP-3	40	20…50	10	103*	-
MP-4	54	20…50	5	100	-
MP-5	50	10…40	4	74	-

The test results are mostly above the desired threshold values, but some cover depth results in MP-2 and MP-5 are lower less than minimum environmental requirements suggest and carbonization depth in MP-2 and MP-3 are also above desired level.

In conclusion to material testing the results are presented according to format of COST TU1406 in Table 6.

Table 9. Overview of test results of Piiometsa bridge

Structure type	Group of elements	Observations: Element	Type of cross-section	Date	Performance cluster	Performance indicator	Performance metric: Primary [Result]	Performance metric: Secondary [unit]	Note	Location/ position - Vulnerabl	Degradation process (Damages/symptoms)	Primary Key Performa	Assessment level	Time to failure, years	Performance value
		Main girder	SC2/RC		Material properties	Concrete quality	62	N/mm2	MP-1	HMM	Change in properties	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Cover	20..30	mm	MP-1	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Carbonization	5	mm	MP-1	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Resistivity	20	kΩcm	MP-1	HMM	Corrosion	Reliability	Element	NA	2
		Main girder	SC2/RC		Material properties	Steel strength	474	N/mm2	MP-1	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Concrete quality	49	N/mm2	MP-2	HS	Change in properties	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Cover	10…40	mm	MP-2	HS	Corrosion	Reliability	Element	NA	2
		Main girder	SC2/RC		Material properties	Carbonization	10	mm	MP-2	HS	Corrosion	Reliability	Element	NA	2
		Main girder	SC2/RC		Material properties	Resistivity	312	kΩcm	MP-2	HS	Corrosion	Reliability	Element	NA	NA
		Main girder	SC2/RC		Material properties	Concrete quality	40	N/mm2	MP-3	HMM	Change in properties	Reliability	Element	NA	1
GA1	Superstructure	Main girder	SC2/RC	20.09.2018	Material properties	Cover	20…50	mm	MP-3	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Carbonization	10	mm	MP-3	HMM	Corrosion	Reliability	Element	NA	2
		Main girder	SC2/RC		Material properties	Resistivity	103	kΩcm	MP-3	HMM	Corrosion	Reliability	Element	NA	NA
		Main girder	SC2/RC		Material properties	Concrete quality	54		MP-4	HMM	Change in properties	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Cover	20…50	mm	MP-4	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Carbonization	5	mm	MP-4	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Resistivity	100	kΩcm	MP-4	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Concrete quality	50	N/mm2	MP-5	HMM	Change in properties	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Cover	10…40	mm	MP-5	HMM	Corrosion	Reliability	Element	NA	2
		Main girder	SC2/RC		Material properties	Carbonization	4	mm	MP-5	HMM	Corrosion	Reliability	Element	NA	1
		Main girder	SC2/RC		Material properties	Resistivity	74	kΩcm	MP-5	HMM	Corrosion	Reliability	Element	NA	1

4.4. LOAD TESTING OF A BRIDGE

Piiometsa bridge was load tested on 27th of September 2018. Before the testing the bridge load carrying capacity was calculated using values present in design documents and values based on the NDT results. The bridge was tested using 52- and 60-ton vehicles with different axle configurations (Figure 20). During the testing, deformations in the mid span was measured (Figure 21).

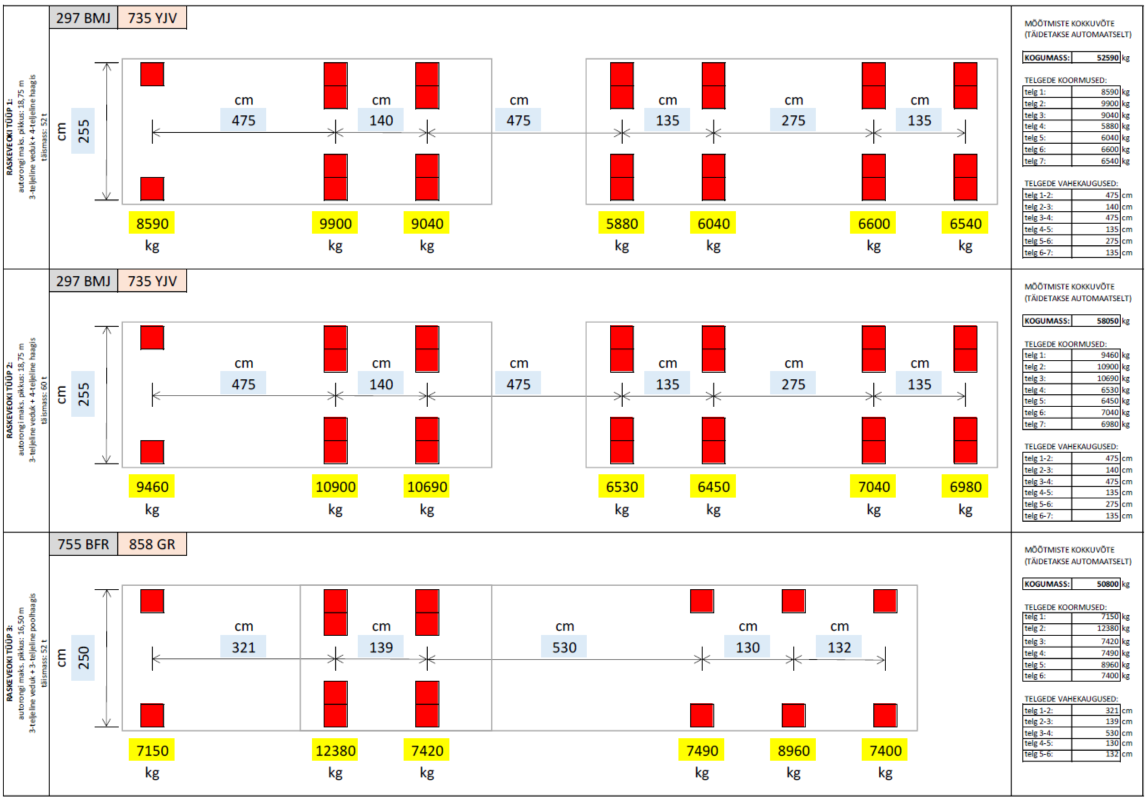
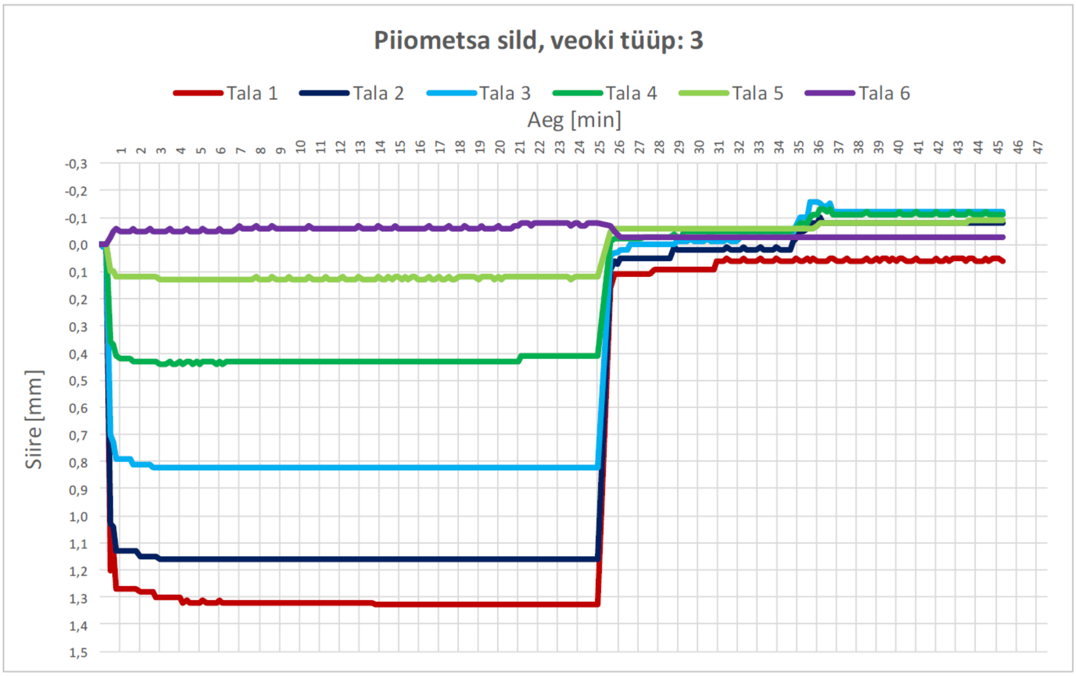


Figure 23. Measured axle configuration and masses.

Heaviest vehicle was 7 axle truck (58050 kg (type 2)) and heaviest axle load (12380 kg) was with 6 axle truck (type 3) with total load of 50800 kg. Due to the length of beams, the biggest deformations were caused by 6 axle truck.

Test results were smaller in comparison to calculated deformations, which show that the bridge is more rigid than based on the initial properties of design catalogue and there is a relative amount of stiffness hidden in other non-structural elements. Results of load testing was used for further analysis in comparison to calculate resistance models.



***Joonis 47.** Piiometsa silla mõõtetulemused (veoki tüüp: 3)*

Figure 24. Deformation measurements of 60-ton vehicle

Test results are presented in Table 10.

Table 10. Results of Piiometsa loading with 52-ton 6-axle truck

Structure type	Group of elements	Date	Observation										Performance value
			Performance cluster	Performance indicator	Performance metric Primary [Result]	Performance metric Secondary [unit]	Note	Location/ position - Vulnerabl	Degradation process	Primary Key Performa	Assessment level	Time to failure, years	
GA1	Superstructure	27.09.2018	Resistance of a structure	Deformation to 52 t 6-axle truck	1,33	mm	Beam 1	HMM		Reliability	Element	NA	1
			Resistance of a structure	Deformation to 52 t 6-axle	1,16	mm	Beam 2	HMM		Reliability	Element	NA	1
			Resistance of a structure	Deformation to 52 t 6-axle truck	0,82	mm	Beam 3	HMM		Reliability	Element	NA	1
			Resistance of a structure	Deformation to 52 t 6-axle truck	0,44	mm	Beam 4	HMM		Reliability	Element	NA	1
			Resistance of a structure	Deformation to 52 t 6-axle	0,13	mm	Beam 5	HMM		Reliability	Element	NA	1

4.5. COMPARISON OF LOADS AND RESISTANCE

In addition to load testing, the bridge superstructure bending and shear resistance to different load models in ultimate limit state were analyzed. The results show that bridge can resist most of the regular traffic, designed special vehicle НГ-60 is ensured with more than 90% confidence and the bridge can’t resist Eurocode load models 1 or 3.

Table 11.Piiometsa superstructure Ultimate Limit State calculations for different load models

Load model	Bending moment [kNm]			Shear [kN]			Reaction [kN]		
	M_{Ed}	M_{Rd}	Ratio	Q_{Ed}	Q_{Rd}	Ratio	V_{Ed}	VRd	Ratio
Dead load (G)	195		2,45	95		2,80	98		3,55
H-13(μ=1,27) + G	396		1,20	196		1,36	235		1,48
HΓ-60 + G	509		0,94	282		0,94	292		1,19
LM3 1200/200 + G	1094	477	0,44	617	266	0,43	600	348	0,58
LM3 2400/200 + G	817		0,58	450		0,59	446		0,78
LM1 (α_{Q1}=α_{q1}=0,8) + G	835		0,57	504		0,53	563		0,62
60 t timber truck + G	401		1,19	196		1,36	217		1,60
52 t timber truck + G	359		1,33	177		1,50	195		1,78

Reliability calculation is done for 60-ton vehicles considering the live load coefficient of variation 0,3, dead load characteristic value corresponds to coefficient of variation 0,1 with normal distribution and resistance has coefficient of variation 0,05 with lognormal distribution.
The mean values are taken directly from designed values.
For example, in bending moment calculation:

- Live load=206 kNm
- Dead load = 195 kNm
- Resistance = 477 kNm

Standard deviations of the variables are:

- Live load = 206*0,3= 61,8 kNm
- Dead load = 195*0,1=19,5 kNm
- Resistance = ln(477*0,05) 3,2 kNm

The reliability is calculated using equation 1:

$$\beta = \frac{\mu_R - \mu_Q - \mu_G}{\sqrt{\sigma_R^2 + \sigma_Q^2 + \sigma_G^2}} \qquad (1)$$

$$\beta = \frac{477 - 206 - 195}{\sqrt{3{,}2^2 + 61{,}8^2 + 19{,}5^2}} = 2{,}66$$

For the 60-ton trucks, the reliability index $\boldsymbol{\beta = 2,66}$, which means that according to the scale, the KPI Reliability to bending failure is 3. Shear reliability index is $\boldsymbol{\beta = 2,19}$, which mean that Reliability is 4 and bridge needs an intervention.

4.6. IDENTIFICATION OF DAMAGE PROCESSES

Based on the damage detection and assessment the main damage processes of the Piiometsa bridge are:

- Corrosion of reinforcement of main girders
- Corrosion of reinforcement of cross beam connections
- Freeze-thaw due to environmental conditions and leaking expansion joints

5. KEY PERFORMANCE INDICATORS

The case study approach is in accordance with COST TU1406, where key performance indicators are based on failure modes and agreed performance areas. These indicators are:

- Reliability
- Safety
- Availability

- Economy

5.1. KEY PERFORMANCE INDICATORS

It is agreed that originally there are 4 KPIs, but since Swiss has more indicators, then it is important to merge the lists and define KPIs clearly.

- Reliability (Table 6) - The reliability is related to structural safety and serviceability. Assessment of reliability is not the same as assessment of a condition indicator, since the reliability:
  - takes into account “virgin” reliability (in some countries it is assumed that “virgin” capacity is at least equal to the load effects based on the codes of practice at the time of construction; often spare capacity may be present in reality, as shear capacity was not well understood in older codes of practice)
  - focuses on failure modes, and
  - related vulnerable zones

Table 12. Scale related of reliability

Reliability scale	**Quantitative scale (β)**	**Qualitative scale and urgency of intervention**
1	>4.00	New bridges and old bridges with no resistance reduction.
2	3.25-4-00	Old bridges with no or marginal resistance reduction compared to the virgin state (< 8%).
3	2.50-3.25	Old bridges with some resistance reduction compared to the virgin state (8 – 17%). Reassessment should be performed before next inspection.
4	2.00-2.50	Old bridges with major resistance reduction compared to the virgin state (17 – 23%). Reassessment and possible intervention shall be performed shortly after inspection.
5	<2.00	Severe resistance reduction. Immediate action is required.

The above written scale is only valid when considering the governing failure mode (i.e. the most critical) in one of the vulnerable zones associated with the bridge type. Other failure modes and zones/areas are expected to have excessive capacity. The above written scale concerns only structural safety. However, similar definitions may apply for serviceability (e.g. reduction/loss of functionality).

- Safety - Safety issues are evaluated regarding user’s safety, and these relate to all structural and non-structural components i.e. equipment. It should be noted that spalling from the deck slab and cornices implies the risk of injuries due to chunks of concrete falling and potentially hitting trains under the bridge and protection roof roof over railway does not fulfil modern requirements

Table 13. Scale related to safety

Reliability scale	Quantitative scale (β)	Qualitative scale and urgency of intervention
1	Injury return period > 100 years	No danger. It is very unlikely that a person could get injured because of the current bridge performance.
2	Injury return period > around 75 years	It is unlikely that a person could get injured because of current bridge performance.
3	Injury return period around 50 years	It is likely that a person could get minor injuries because of the current bridge performance. Intervention shall be performed before next

		inspection.
4	Injury return period around 20 years	It is likely that a person could get injured because of current bridge performance. Intervention shall be performed shortly after inspection.
5	Injury return period around 10 years	Immediate danger. It is very likely that a person could get injured because of current bridge performance. Immediate action is required.

- Availability – availability and restrictions to traffic.

The quantitative scale related to availability has been given in Table 8.

Table 14. Scale of KPI availability

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

- Economy– costs of different rehabilitation works

5.2. PRESENT SITUATION

The different assessments described in chapter 4 can be concluded with spider diagram of present values, shown in Figure 25. All values are based on the highest ratings of previous assessments and the explanation for the evaluation of different aspects are below:

- Reliability – 4

Reliability is rated as 4 because of the reliability index calculation, which is lower than suggested by Eurocode. Resistance of the bridge has decreased due to the deterioration of concrete and corrosion of rebars at the end of main girder, described in damages V and Õ.

- Availability – 2

Bridge has no restrictions to traffic, although special vehicles will be allowed to cross after reassessment of the bridge and based on the reliability calculations, it is not allowed.

- Safety –.1

Bridge is safe for users thanks to modern restraint system.

- Cost – NA

At moment there haven’t been any bigger repairs planned, but due to the bad condition, the bridge needs intervention.

Figure 25. Snapshot of Piiometsa KPIs.

6. POSSIBLE MAINTENANCE SCENARIOS

Since in each way, the bridge should have an intervention within next 5 years and in order to decide upon the best plan for the bridge, it is defined two maintenance scenarios with same target reliability and safety. Since the bridge is constructed 55 years ago, then both scenarios will be compared on 60-year basis, so finally the bridge age would be 115 years. One scenario will follow the typical situation in Estonia, bridge will be reconstructed according to plans made in 2018, and second will follow the preventative scenario suggested by WG4 instructions to keep the reliability as high as possible. No discount rate is used in the comparison.

6.1. REFERENCED SCENARIO

For the referenced scenario the bridge will be reconstructed (new superstructure) in 2028 and after that superstructure will be repaired in year 2078, along with parapet and waterproofing layer replacement. Road surface repairs will be done in 2053 and 2078, along with barrier replacement and expansion joint filling. The regular maintenance covering cleaning of the bridge will be done annually.

The cost of the reconstruction work is 500 000 EUR, repair of superstructure costs 200 000 EUR and road surface repairs 30 000 EUR. The annual maintenance costs are 500 EUR/year.

The timelines of different KPIs are presented on Figures 26-29.

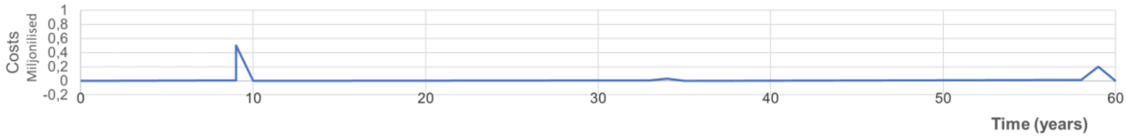


Figure 26. Cost KPI for referenced scenario

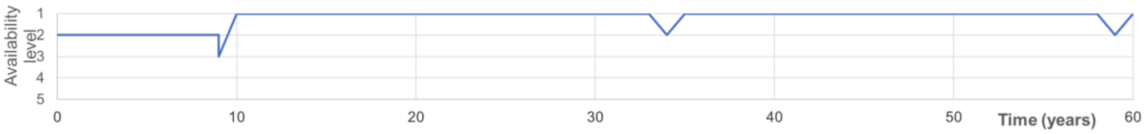


Figure 27. Availability KPI for referenced scenario

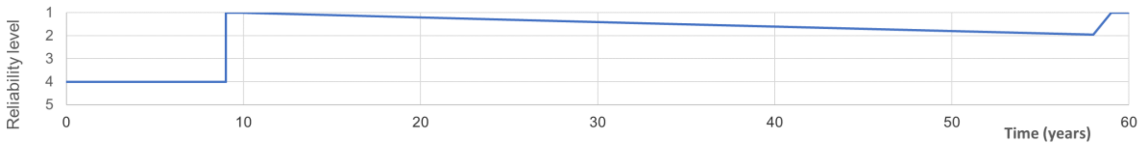


Figure 28. Reliability KPI for referenced scenario

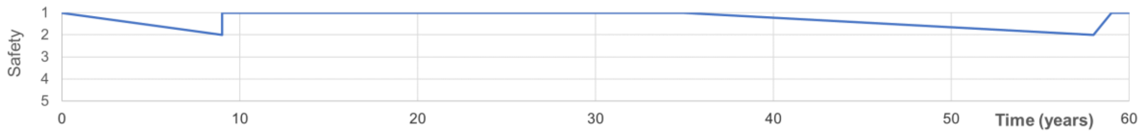


Figure 29. Safety KPI for referenced scenario

6.2. PREVENTATIVE SCENARIO

For the preventative scenario the bridge will be repaired (including waterproofing layer replacement) and strengthened with CFRP strips in 2021, after that the superstructure will be repaired in year 2046 and 2071. Road surface repairs will be done in addition 2041 and 2061, along with expansion joint filling. The regular maintenance covering cleaning of the bridge will be done annually.

The cost of the strengthening work is 300 000 EUR, repair of superstructure costs 200 000 EUR and smaller road surface repairs 20 000 EUR. The annual maintenance costs are 500 EUR/year.

The timelines of different KPIs are presented on Figures 30-33.

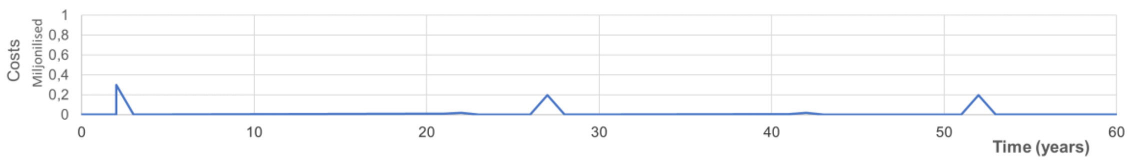


Figure 30. Cost KPI for preventative scenario

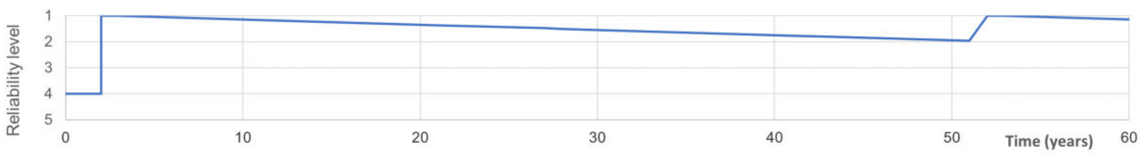


Figure 31. Reliability KPI for preventative scenario

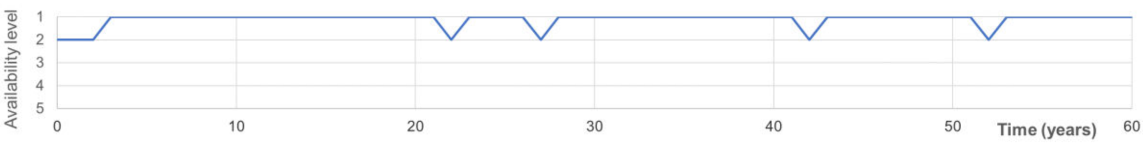


Figure 32. Availability KPI for preventative scenario

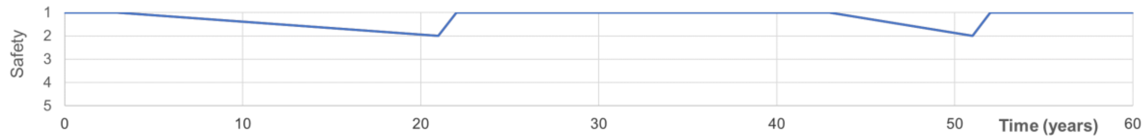
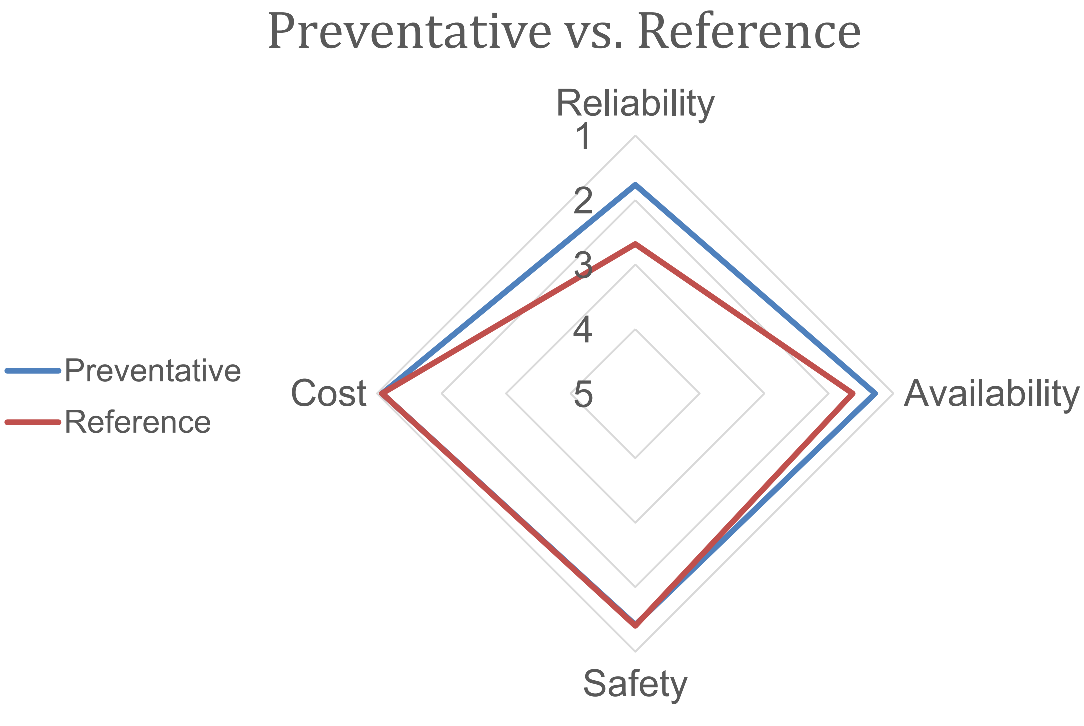


Figure 33. Safety KPI for preventative scenario

6.3. COMPARISON OF SCENARIOS

In the comparison of scenarios, it possible to see that overall costs are similar, but referenced approach is slightly cheaper. Similar is with safety of the bridge.
The main difference is with the reliability and availability, where preventative scenario should be preferred. The effect of reliability and availability reduction comes from earlier timing of intervention and keeping the bridge reliability as high as possible. This shows that comparing to traditional approach, making decisions based on the condition index, leads to a situation, where intervention of a bridge with reliability issues is postponed.



7. CONCLUSIONS

The Piiometsa bridge has been chosen as a case study bridge as one of the most common typologies in Baltic countries. The main advantages using the COST TU1406 approach comes from:

- Merging the different assessment results in one format
- Making the reliability calculation as a part of a quality procedure
- Comparing different maintenance scenarios from different perspectives

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