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Value of additional traffic data in the context of bridge service-life management

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ABSTRACT

The assessment of existing road bridges as parts of infrastructure networks is required in consideration of their deterioration and age. Advanced monitoring and management tools are mainly used for landmark bridges while the decision making process for small to medium bridges, which constitute the majority of the bridge network, mainly relies on condition assessment based on experience and conservative analysis related to design codes. In the analysis of the load-carrying capacity of theses bridges, loads imposed by the passing traffic are predominant due to their variable nature and level of uncertainties. The research presented in this paper outlines the value of additional traffic data, collected with both traffic counters and Weigh-in-Motion (WIM) method in the scope of assessment procedures for these bridges. Adequate processing of collected traffic data is crucial for subsequent extrapolation of maximum load effects on a particular bridge over a certain period in time. By taking into account all related costs, the purpose of this paper is to prove the benefits of employing traffic load monitoring data in structural assessment and subsequent decision-making process in service life management of bridges.

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Assessment; decision-making; existing bridges; service-life management; Value of information; weigh-in-motion

1. Introduction

Vast majority of civil infrastructure in the USA and Western Europe has been constructed in the 1960s and 1970s and is currently at the risk of ageing and in dire need of assessment and rehabilitation. The deterioration and ageing process is especially evident on the existing road bridges, which represent a critical part of global transportation networks, as the consequences of their potential failure would be severe, from both social and economic aspects. Therefore, the safety assessment of these bridges is required for the evaluation of their reliability levels, as they have been designed and constructed according to old codes, which were not as strict as current standards, in terms of loading and resistance modelling (Skokandić, 2020).

One of key steps in the design or assessment process for new or existing bridges is the determination of total load effects at critical sections of the bridge. Due to their variable nature, most significant effects are induced by the traffic passing over the bridge itself (O'Connor & O'Brien, 2005). Practical application of traffic load models from current design codes for new bridges (Eurocode, 2005) in the assessment procedure for existing ones may provide conservative results suggesting that majority of these bridges need to be strengthened or even replaced. On the other hand, more recent research has proven that the application of site-specific traffic load models, derived from Structural Health Monitoring (SHM) data, results in increased reliability levels and, consequently, in an unrestricted use of the bridge over a much longer remaining service life (Skokandić, 2020). In addition, extreme traffic loads can be quantified from collected data so as to numerically evaluate bridge response under extreme load scenarios and compare them with alarm levels established by bridge designers (Sousa, Costa, Henriques, Bento, & Figueiras, 2015). These load models are developed from the collected real-life traffic data obtained using the Weight-in-Motion (WIM) technology, a measurement procedure for the collection of traffic data as a part of SHM tools. WIM devices installed outside the bridge length are particularly interesting from the network-level perspective since, if well designed, they allow characterisation of traffic load patterns for a set of bridges within a roadway network (Mandić Ivanković, Strauss, & Sousa, 2020).

In addition to these studies, a number of research projects, both in Europe and worldwide, have focused over the last two decades on the topics of bridge assessment, inspection, and Structural Health Monitoring (SHM) in the context of the bridge management process. One of these projects is the recently concluded COST Action TU1402 "Quantifying the Value of Structural Health Monitoring". The project was initiated to address the challenges of validation and quantification of the SHM data from the perspective of stakeholders and infrastructure owners (Thöns et al., 2017) and it resulted in the decision-supporting guidelines for operators, practicing engineers and scientists (Diamantidis, Sykora, & Sousa, 2019; Helder Sousa, Wenzel, & Thöns, 2019; Thöns, 2019). The theoretical framework developed within the TU1402 action is based on implementation of the Value of Information (VoI) analysis and decision tree method in the decision-making process regarding the utilization of SHM data. In the current state of practice,

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SHM is more used for research purposes than for real structures. This is mainly due to the lack of understanding of the value of additional information that is gained using SHM tools, as even in the cases when SHM is implemented, its information is often disregarded by bridge owners and engineers in charge, and the decisions are based on experience, frequently with conservative assumptions (Zonta, Glisic, & Adriaenssens, 2014). Unfortunately, such practices can result in unnecessarily high maintenance and rehabilitation costs for bridges and viaducts, which are therefore deemed critical elements of the transport infrastructure networks. Furthermore, every partial or complete closure of these bridges leads to both direct losses incurred by bridge owners and indirect losses to bridge users, not to mention socio-economic costs for the local community. These indirect costs can be significant and, for important bridges, they can even be higher than the direct ones (Thoft-Christensen, 2009, Thoft-Christensen, 2012).

Nonetheless, if bridge monitoring could be designed and implemented as a complement to visual inspection, to enhance its effectiveness and improve on its shortcomings, bridge owners could decide to recognise its advantage (Mandić Ivanković et al., 2020). In order to address these issues in the context of bridge management process, a detailed algorithm for validation of additional SHM data in bridge assessment procedures has been developed in (Skokandić, 2020), based on theoretical framework defined in the COST TU1402 Action.

The work presented in this paper aims to quantify the value of incorporating bridge assessment results based on traffic load monitoring data into the decision-making process for bridge maintenance and management, using the VoI methodology developed in (Skokandić, 2020). Although traffic measurements discussed in the paper are recorded regularly, they are currently only used for traffic analysis and selection of overloaded vehicles. In this research, they are implemented in the procedure for bridge assessment. The benefits of the assessment results from the owner's point of view in terms of reduced overall maintenance costs are also investigated. By doing so, results of posterior VoI (based on available data from traffic counters in the country and WIM measurements on a road leading to a certain bridge) could convince the operator to invest in more traffic load analysis and WIM measurements (at different locations) and to use the existing and subsequently collected traffic load and WIM data in bridge management, and not only for traffic counting and weight limitations as it has been done so far.

In the first part of the paper, the emphasis is placed on the WIM technology, development of traffic load models, and reliability analysis of the Case Study bridge. Three distinct assessment levels (strategies) will be considered: at the initial level, the assessment is performed without any additional traffic information, using the codified Load model 1. At the second level, the assessment is conducted based on Load model 1 adjusted in respect to heaviest traffic measurements in the country and, at the third level, the assessment is based on specific traffic load related to continuous WIM measurements on a road leading to a bridge. Development of the posterior VoI analysis algorithm and estimation of all related costs and benefits are provided in the second part of the paper for the three assessment strategies (S0 related to level 1 assessment, S1 related to level 2 assessment, and S2 related to level 3 assessment). The analysis of VoI results, and recommendations for future research, are given in the concluding section that presents benefits of employing traffic load monitoring data in structural assessment and subsequent decision-making process within the service life management of bridges.

2. Importance of traffic load modelling in assessment of existing bridges

2.1. Overview

Reliability analysis for both new and existing structures is a procedure in which structural resistance is evaluated in relation to the total effect of the applied loads, in order to quantify the safety level or reliability of the structure. For bridges, dominant loads are described as permanent loads, consisting of the structure self-weight and additional dead loads (road surfacing, railings, etc.), and live loads induced by the passing traffic. Regardless of the reliability analysis method (deterministic, semi-probabilistic, probabilistic), traffic loads are associated with the highest level of uncertainties, due to their variable and unpredictable nature. Additionally, for existing bridges, which have reduced reliability levels when compared to new bridges, permanent loads can be accurately calculated based on the on-site geometry measurements and material testing, thus further reducing their uncertainty levels. On the other hand, loads induced by the passing traffic can be either estimated using codified load models for the design of new bridges, or developed using the recorded traffic data. Some countries, such as Netherlands, Denmark, Switzerland, and Slovenia, have developed specific bridge assessment codes based on the reduced traffic load models or site-specific models (Skokandić, 2020; Wiśniewski, Casas, & Ghosn, 2012).

Principles of weighing vehicles in motion using bridges, which are valid to this day, were first established by Moses in the USA (Moses, 1979). In the late 1990s, research interest in B-WIM intensified as two research projects supported by the European Commission were initiated based on the B-WIM work from Slovenia and studies from Ireland: COST Action 323 – Weigh in Motion of Road vehicles (Jacob, 2002) and FP4 project WAVE – Weighing of Axles and Vehicles (Jacob, 2002) in Europe. More recent improvements in B-WIM technology were achieved as a part of two FP7 research projects, TRIMM (Ralbovsky et al., 2014) and BRIDGEMON (Corbaly, Žnidarič, Leahy, Hajializadeh, & Zupan, 2014; Favai et al., 2014).

In Croatia, there are still no official codes or guidelines for modelling traffic loads in the assessment process for the existing road bridges, and this modelling is also not included in official EU standards Eurocodes. On the other hand, traffic data measurements have been conducted regularly on Croatian state roads for over two decades, using both WIM and B-WIM technology (Skokandić, Mandić Ivanković, Žnidarič, & Srbić, 2019). Research focusing on the use of recorded traffic data in the assessment procedures for road bridges in Croatia has been conducted at the University of Zagreb over the last decade, as reported in several papers (Mandić Ivanković, Skokandić, Žnidarič, & Kreslin, 2019; Mandić, Radić, & Šavor, 2009; Skokandić, Mandić Ivanković, & Džeba, 2016) and PhD thesis (Mandić Ivanković, 2008; Skokandić, 2020). This paper focuses on the quantification of measured traffic data from the perspective of bridge owners and bridge users.

In general, traffic load on road bridges can be divided into congested traffic, basically a traffic jam situation, and free-flow traffic, which is a steady traffic flow of 60-100 km/h. Furthermore, from the engineering point of view, the traffic load is divided into the static and dynamic components. Most of the current design codes have the dynamic part already integrated with the specified load models but, in older codes, dynamic factor was calculated manually depending on bridge characteristics (Bruls, Croce, & Sanpaolesi, 1996; Bruls, Mathieu, Calgaro, & Prat, 1996; Dawe, 2003; Eurocode, 2005; Skokandić et al., 2019). More detailed historical review of traffic load models developed over the years can be found in the book by Dawe (2003). In Croatia, a majority of existing state road bridges have been designed according to older codes, mainly PTP-5 (valid until 1973) and the codes based on the German DIN 1072 (valid until 2002). The Case Study bridge analysed in this paper was built in the 1960s according to PTP-5 code. Significant increase in the average annual daily traffic (AADT) over the last two decades of the past century caused the revision of design codes and acceptance of European standards in the 2000s (Mandić & Radić, 2004; Skokandić et al., 2019).

The basic approach to the development of traffic loads, both site-specific and modern codified ones, is to collect a certain amount of traffic data, including axle loads and spacings, and to apply one of statistical methods to extrapolate the collected data and estimate the maximum expected load effects. There is a number of traffic data collection methods available, but most widely accepted ones are based on the WIM and B-WIM methods (Žnidarič, Kreslin, Lavrič, & Kalin, 2012). Codified traffic load models have been developed for the design of new bridges and, therefore, they may provide conservative results in the assessment procedure for existing bridges. The application of localised, adjusted or site-specific traffic load models in the assessment of existing bridges is crucial for making optimum management decisions.

2.2. Current traffic load models for the design of new bridges

The European code EN 1991-2:2003 (Eurocode, 2005) defines imposed loads, both models and representative values, associated with road traffic, which includes dynamic effects, centrifugal, braking, and acceleration actions to be used for the design of new bridges. These load models were developed based on traffic data collected with WIM technology on a motorway in France in the 1980s. The data were used for calculating load effects using influence lines and areas, and extrapolations were made to evaluate reference values of representative traffic loads. A more detailed review on the background and development of EN 1991-2 codes can be found in (Bruls, Croce, et al., 1996; Bruls, Mathieu, et al., 1996).

The Load Model 1 (LM1), defined as a general traffic model that already takes into account dynamic amplification due to vehicle-bridge interaction, is used in the majority of bridge designs for every road and bridge type and is therefore implemented in the assessment procedure for the Case Study bridge in this paper. It is comprised of two tandem systems (TS) representing concentrated axle loads and uniformly distributed load (UDL) across the entire width of the carriageway. Graphical representation of LM1 for state road bridges (with the total width w under 9.0 m) is given in Figure 1 (Skokandić et al., 2019).

Adjustment factors (Figure 1) $\alpha_{Q,i}$, $\alpha_{q,i}$ and $\alpha_{q,r}$ are used for the adjustment of total traffic loads depending on the road category and expected traffic density and weight. Values of these factors are defined in National Annex for each country, or if not specifically indicated, they can be taken equal to 1.0 for all new bridges, as it is the case in the majority of EU countries. Nevertheless, some countries, such as France, Germany, and Netherlands apply increased values of adjustment factors to take into account predicted increase in traffic growth. A detailed list with values of these specific adjustment factors for selected EU countries can be found in (Skokandić et al., 2019).

Adjustment factors from Figure 1 can also be used for the reduction of total traffic load effects in the assessment procedure for existing road bridges, by reducing their initial value of 1.0 based on the measured traffic data. For example, Switzerland defined national assessment codes for existing bridges and implemented reduced adjustment factors based on bridge type and span length (SIA, 2011). The procedure for calibration of adjustment factors based on WIM data, defined by O'Brien et al. (2012), can be used for a single bridge or the local transport network. In Croatia, the traffic load effects calibration based on measured traffic data was conducted by Mandić Ivanković (2008, 2009), through analysis of national traffic records. As a result, reduced values of adjustment factors are calibrated and will be used in this paper as one of assessment strategies in the analysis of the Case Study bridge. Reduced factor values, depending on bridge type and span length, are given in Table 1.

2.3. WIM and B-WIM as a part of structural health monitoring

Traffic data collected using both WIM and B-WIM systems constitute an unbiased traffic sample as the measurement is conducted in uncontrolled conditions and without the need for vehicle to slow down or stop. The data set obtained for each vehicle passing over the measurement site includes its gross weight (GVW), axle load, number and spacing, vehicle

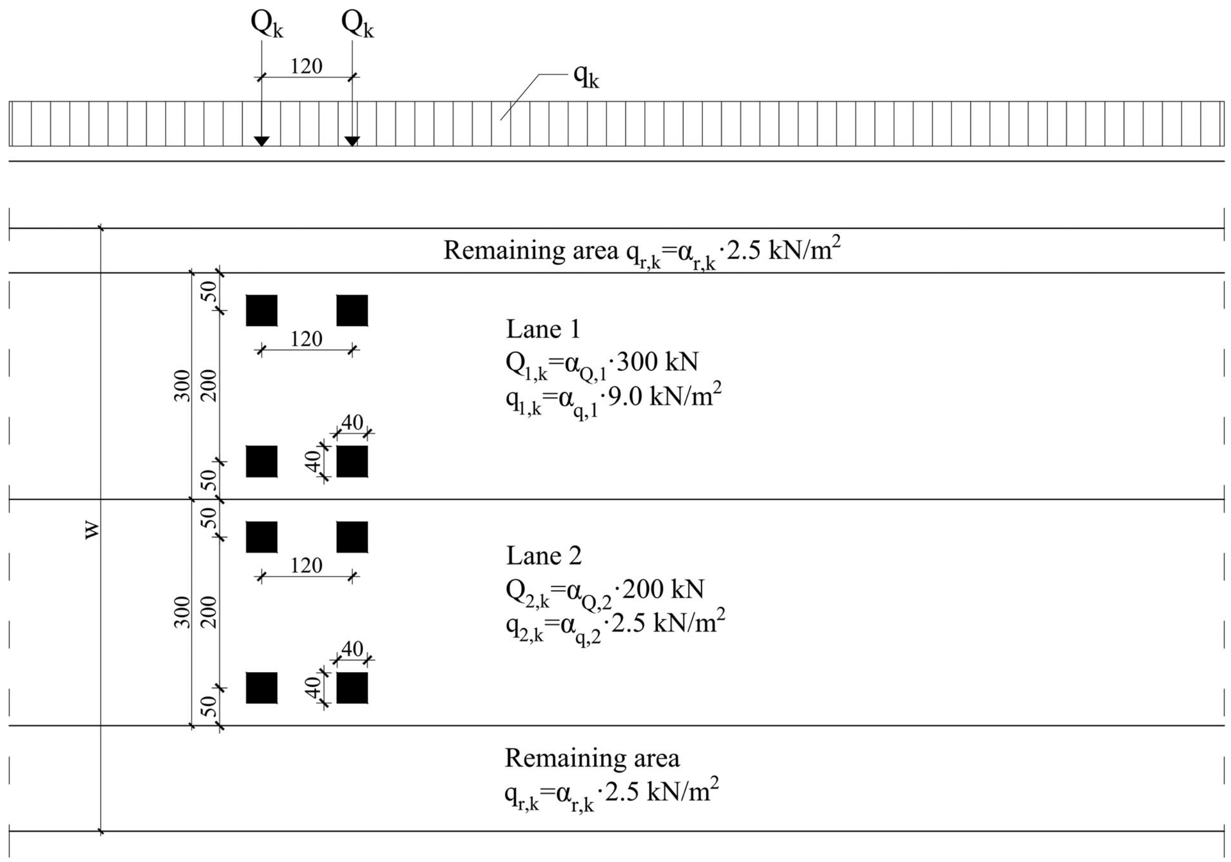

Figure 1. Example of LM1 on a two-lane state road bridge.

Table 1. Reduced adjustment factors for assessment of state road bridges in Croatia - research-based proposal (Mandić et al., 2009).

Span [m]		≤ 10	10 – 20	20 – 30	30 – 40	40 – 50
Simply supported bridge	$\alpha_{Q,1}$	0,80	0,80	0,80	0,80	0,80
$\alpha_{q,2} = \alpha_{q,r} = 1{,}0$	$\alpha_{Q,2}$; $\alpha_{Q,3}$; $\alpha_{q,1}$	0,30	0,38	0,51	0,58	0,62
Continuous bridge	$\alpha_{Q,1}$	0,80	0,80	0,80	0,80	0,80
$\alpha_{q,2} = \alpha_{q,r} = 1{,}0$	$\alpha_{Q,2}$; $\alpha_{Q,3}$; $\alpha_{q,1}$	0,48	0,72	0,78	0,81	0,82

speed, and timestamp of the passage. Post-processing of collected traffic data is required for their extrapolation and subsequent estimation of maximal load effects on the selected bridge over a certain time period, as it was done based on collected WIM data for the Case Study bridge in this paper. Additionally, B-WIM systems also provide supplemental structural data on bridge response to the effect of the passing traffic, such as measured influence lines, load distribution, and dynamic factors. This additional information can be used as a key input in assessment procedures for existing road bridges, applied for calibration of numerical models, as presented in (Mandić Ivanković, Skokandić, et al., 2019; Žnidarič, Kalin, & Kreslin, 2018). For the presented Case Study bridge, continuous traffic data measurements were conducted using the pavement WIM system on the road leading to the bridge. Therefore, the VoI analysis presented in the second part of the paper will be conducted in order to quantify benefits resulting from incorporation of recorded traffic data in the assessment of existing road bridges.

In general, there are two main approaches for the post-processing of collected traffic data, either using statistical methods, i.e. extrapolating the data by fitting it to a certain distribution, or using a very large number of long-run simulations like the Monte Carlo method. For example, in the development process for current design load models from EN 1991-2, the post-processing was conducted using three distinct methods, two based on statistical approach (fitting the upper data tail to a half-normal and a Gumbel distribution) and Monte Carlo (MC) simulation for the validation of obtained results (Bruls, Croce, et al., 1996). Other commonly used methods include Block Maxima, Peaks over Threshold (POT), Box-Cox approach (O'Brien et al., 2015), and convolution method (Žnidarič, 2017).

While statistical methods are subject to a certain level of subjectivity and can, therefore, have a considerable margin of error, MC simulations are not practical for general use, as they require a certain level of knowledge and high computational power. Further details on the most widely used post-processing methods for the extrapolation of traffic data can be found in the review paper by O'Brien et al. (2015). Along with the selection of a statistical extrapolation method, the selection of a reference time period is essential in the post-processing of traffic data and subsequent calculation of maximum expected traffic load effects. For example, characteristic values of LM1 (Figure 1) were extrapolated for a 50-year reference period during the development of EN

1991-2 (Bruls, Mathieu, et al., 1996). In order to apply the LM1 for shorter time periods, two approaches can be used. The first one is to shift the Gumbel distribution to lower reference periods in order to obtain a lower mean value for traffic load effects. Additionally, EN 1990 (Eurocode, 2002) provides simplification for reduction of characteristic values of LM1 to a one-year period by simply reducing initial values (Figure 1) by 20%. The first approach is used for the reliability analysis of the Case Study Bridge as the simplification provided by EN 1990 is relatively conservative. Chosen approach utilizes the property of the Gumbel distribution that the standard deviation is independent of the considered reference period and that the mean value depends on the period T in the following way (Faber, 2012):

$$\mu_{50} = \mu_1 + 0,78 \cdot \sigma_1 \cdot \ln(T_{50}) \tag{1}$$

where:

μ_{50}, μ_1 – are the traffic load effects mean values for 50- and 1- year reference periods;

σ_1 – is the traffic load standard deviation for 1- year reference period ($\sigma_1 = \sigma_{50}$);

T – is the chosen reference period.

The extrapolation method chosen for this research, called convolution method, has proven to provide similar results like long-run simulations and, at the same time, it is computationally less complex and more suitable for practical application. It was first proposed by Moses and Verma (1987), and has been used and constantly improved in Slovenia (Žnidarič et al., 2012) for over two decades with data recorded from SiWIM® B-WIM system (Žnidarič, 2017). The convolution method is based on assumptions that the traffic in two adjacent lanes on the bridge is independent and that the highest load effects are achieved when two vehicles in each lane meet side by side at a critical section of the selected bridge. The described method was developed around the fact that, due to the typical length of heavy vehicles, critical loading scenarios for short to medium size bridges occur in the free flow traffic (while the traffic jam situations typically represent critical loading scenarios for long bridges). Such an approach is justified on a majority of simply supported continuous bridges whose influence-line lengths between supports are up to 30 meters, and has therefore been chosen for the Case Study bridge analysed in this paper.

The convolution method applies the influence line theory for the calculation of traffic load effect of each vehicle, followed by generation of load effects histograms for each independent lane, the convolution of these histograms to simulate the presence of vehicles in both tracks simultaneously, and subsequent extrapolation of maximum values to certain time periods. For further details, this can be found elsewhere (Skokandić, Žnidarič, Mandić Ivanković, & Kreslin, 2017; Žnidarič, 2017).

3. Case study bridge

3.1. Overview

The Case Study bridge used in this research was built in 1961 as a continuously reinforced concrete (RC) slab bridge over three spans. It is located on a Croatian state road, near the town of Posedarje, and features a total deck width of 8.50 meters, and two traffic lanes, one for each direction of travel. The bridge is continuous across three spans, $9.0 + 15.0 + 9.0$ m, divided with RC piers and abutments, and supported on RC foundations and wooden piles. The bridge setup involving a larger central span has been selected due to heavy rainfall, which caused the collapse of the old concrete arch bridge that had been built on the same location in the 1960s. The original documentation and design plans, along with the built-in reinforcement, are available from the archives (Šram, 2002). The longitudinal and transverse sections of the bridge are presented in Figures 2 and 3.

The numerical FE model of the bridge was used for calculation of total load effects for the load-carrying capacity assessment. It was developed using Sofistik software (Sofistik & Sofistik, 2014) for structural analysis, using 2D quad elements, with finite element size of 0.2×0.2 m, presented in Figure 4. Material characteristics and additional permanent load values (road surfacing, railings etc.) were obtained from the original documentation, as presented in Tables 2 and 3. In addition to self-weight and additional permanent load, only traffic load effects were taken into account in structural analysis, as dominant variable loads on road bridges. Based on the preliminary visual inspection, documentation review and linear analysis, the critical failure mode for the selected bridge was defined as a flexural failure due to bending moment in the middle of the central span (resistance to load ratio 0,836 – section 2-2 in Figure 4), and failure at internal bridge supports due to hogging moment (resistance to load ratio 0,742 – section 1-1 in Figure 4). The limit state equation (LSE) for the cross-sectional bending capacity was defined for both critical sections based on the geometry, material characteristics, and built-in reinforcement.

3.2. Assessment strategies

The multi-level assessment of the Case Study bridge was conducted in order to quantify the value of additional traffic data obtained using the previously described WIM measurements, with each level representing one of the defined assessment strategies. Multi-level assessment procedures are suitable for existing bridges as the complexity and accuracy increase consecutively throughout the levels. At the initial level, the assessment procedure is performed without any additional information, using the codified procedure and Load model 1 for the design of new bridges from EN 1991-2. The results obtained at this level are considered as a reference value, which will be used for comparison and quantification of additional traffic data implemented at subsequent levels. The re-assessment of the Case Study bridge is performed at the second level of the defined procedure, using reduced values of adjustment factors for codified LM1, as based on traffic measurements conducted on the heaviest loaded road in Croatia and presented in Table 1 (Mandić et al., 2009). Traffic load effects are developed in the final step of the assessment procedure using the convolution

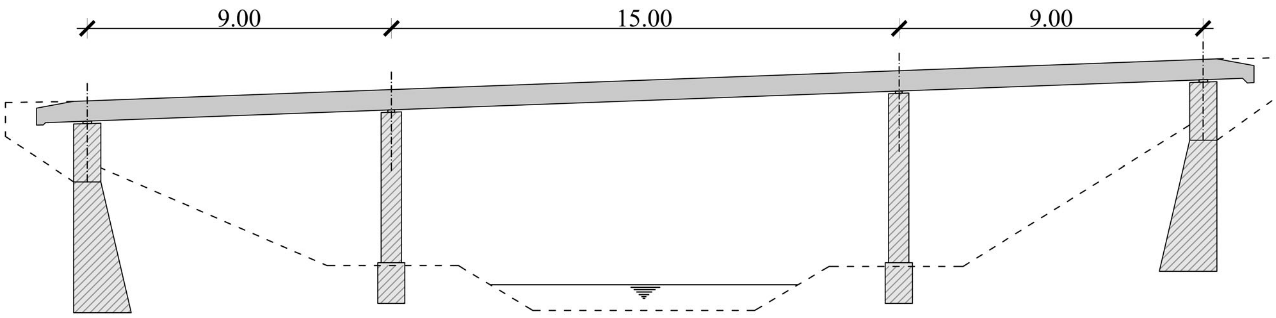

Figure 2. Longitudinal section of Case Study bridge (units in m).

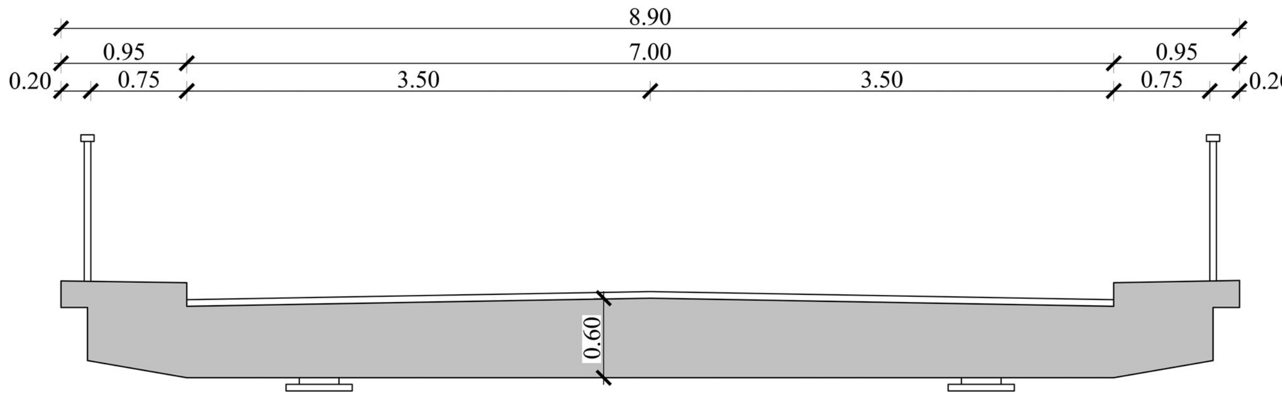

Figure 3. Cross-section of Case Study bridge (units in m).

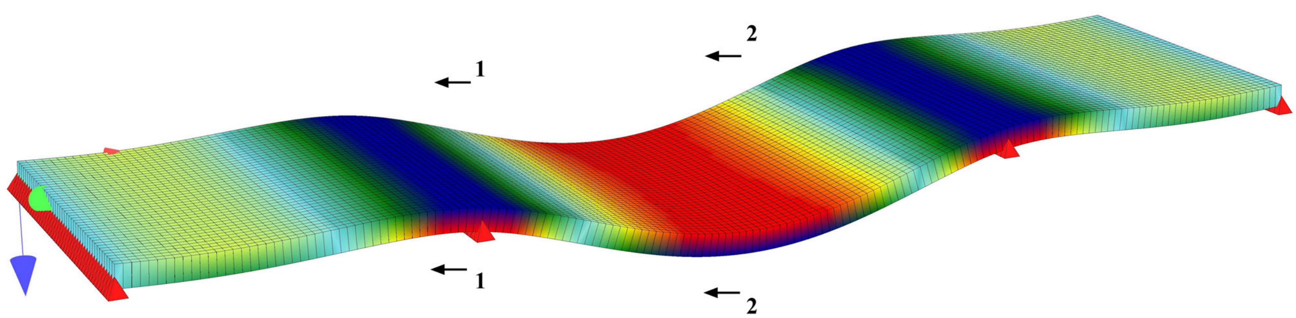

Figure 4. FE numerical model of Case Study bridge – deformation under permanent load (developed in the *Sofistik* software for structural analysis).

Table 2. Parameters for modelling cross-sectional resistance – statistical characterisation.

Variable	Symbol [Units]	Distribution	Nominal Value	Mean Value (μ)	St.Dev. (σ)	Source
Effective depth of bars	d [m]	Normal	0.56	0.56	0.10 μ	(JCSS, 2001b)
Number of bars per slab section	n_b	Deterministic	14	14	/	
Yield strength of reinforcing steel	f_y [kN/cm^2]	Normal	22.0	24.46	0.05 μ	
Area of rebar	A_s [cm^2]	Normal	3.14	3.14	0.02 μ	
Resistance uncertainty	θ_R	Lognormal	/	1.00	0.06 μ	(Fib, 2016)

method based on continuous WIM measurements on the road leading to the Case Study bridge.

The reliability analysis of the Case Study bridge is conducted for each level using a fully probabilistic approach, as recommended in the Probabilistic Model Code (JCSS, 2002), and the results are presented in terms of calculated probabilities of failure p_f and the corresponding reliability indices β. The basic limit state equation for reliability analysis is developed based on the defined critical failure mode and JCSS recommendations (JCSS, 2001b). Design codes for new bridges (Eurocode, 2002) propose a semi-probabilistic procedure based on the partial safety factors method (PSFM), but it has been proven that probabilistic approach provides improved assessment results in terms of load-carrying capacity (Lauridsen, Jensen, & Enevoldsen, 2007). The flow chart of the multi-level assessment procedure defined for the Case Study bridge, as based on the one developed in (Skokandić, 2020), is given in Figure 5.

3.3. Assessment procedure and results

The basic limit state equation (LSE) for the reliability analysis of the Case Study bridge is defined as:

$$Z = \theta_R \cdot R - \theta_E \cdot E \tag{2}$$

where:

Table 3. Parameters for modelling total load effects for a reference period of one year – statistical characterisation.

Variable	Symbol [Units]	Distribution	Nominal Value	Mean Value (μ)	St.Dev. (σ)	Source
Self-weight load effect	M_G [kNm/m]	Normal	/	253.60	0.04 μ	(JCSS, 2001a)
Additional dead load effect	$M_{\Delta G}$ [kNm/m]	Normal	/	51.70	0.05 μ	
Traffic load effect – level 1	$M_{Q,1}$ [kNm/m]	Gumbel	/	216.10	0.14 μ	(Eurocode, 2005)
Traffic load effect – level 2	$M_{Q,2}$ [kNm/m]	Gumbel	/	163.97	0.14 μ	
Traffic load effect – level 3	$M_{Q,3}$ [kNm/m]	GEV	/	67.10	0.13 μ	(ARCHES D10,10, 2009)
Dynamic amplification factor	DAF	Gumbel	/	1.25	0.10 μ	
Dead load uncertainty	$\theta_{E,G}$	Normal	/	1.00	0.05 μ	(JCSS, 2001b)
Traffic load uncertainty	$\theta_{E,Q}$	Normal	/	1.0	0.10 μ	

R – is the cross-sectional resistance to selected load effect (bending moment, shear force, etc.);
E – is the value of the corresponding load effect at critical cross-section;
$\theta_R; \theta_E$ – are additional model uncertainty distributions accounting for deviations between the model and reality (JCSS, 2002).

Further derivation of Equation (2) is conducted based on the selected critical failure mode for which the assessment procedure is performed. For the Case Study bridge, based on the preliminary condition assessment the cross-sectional flexural failure due to negative bending moment on both inner supports is defined as the critical failure mode. Furthermore, as the Case Study bridge is a continuous system, flexural failure in the middle of the central span is also considered in the assessment. Therefore, Equation (2) can be re-written as:

$$Z = \theta_R \cdot M_R - \theta_E \cdot M_E \tag{3}$$

where:
M_R – is the cross-sectional bending moment resistance;
M_E – is the total bending moment load effect at a critical cross-section;

The cross-sectional bending resistance of the Case Study bridge M_R can be calculated based on original documentation and built-in reinforcement, as follows:

$$\theta_R \cdot M_R = \theta_R \cdot 0.9 \cdot d \cdot n_b \cdot A_s \cdot f_y \tag{4}$$

where:
d – is the effective depth of reinforcing bars;
n_b – is the total number of reinforcing bars in the critical cross-section;
A_s – is the cross-sectional area of a single reinforcing bar;
f_y – is the yield strength of reinforcing steel, available from original documentation.

Bending moment values as the total load effect in critical cross-sections M_E can be defined as:

$$\theta_E \cdot M_E = \theta_{E,G} \cdot (M_G + M_{\Delta G}) + \theta_{E,Q} \cdot M_Q \tag{5}$$

where:
M_G – is the portion of total bending moment induced by self-weight of the bridge;
$M_{\Delta G}$ – is the portion of total bending moment induced by additional dead load (e.g., road surfacing, railings, etc.);
M_Q– is the portion of the total bending moment induced by traffic load;
$\theta_{E,G}$ – is the permanent load model uncertainty function;
$\theta_{E,Q}$ – is the traffic load model uncertainty function.

Finally, the fully derived LSE for the Case Study bridge can be defined as:

$$Z = \theta_R \cdot 0.9 \cdot d \cdot n_b \cdot A_s \cdot f_y - \theta_{E,G} \cdot (M_G + M_{\Delta G}) - \theta_{E,Q} \cdot M_Q \tag{6}$$

All parameters in Equations (2)–(6) are modelled as stochastic variables (or random variables – i.e., parameters whose values depend on certain uncertainty or an outcome in their quantifications) with the corresponding statistical parameters and distribution types, as presented in Tables 2 and 3. Values of statistical parameters and recommended distribution types are taken from the Probabilistic Model Code (JCSS, 2002) and *fib* guidelines (Fib, 2016), while nominal values for resistance variables are obtained from original documentation (Šram, 2002) and from the numerical model analysis for load effect variables. Both load effects and cross-sectional resistance are calculated in the middle of the middle span (section 2-2, Figure 4) and at inner supports (section 1-1, Figure 4). Based on structural analysis of the Case Study bridge numerical model (Figure 4), the critical failure mode is defined as flexural failure due to the negative bending moment on the supports above the piers. The corresponding values are shown in Tables 2 and 3.

The total traffic load effect M_Q is calculated separately for each of the three assessment strategies as explained in the flowchart shown in Figure 5. For the first two assessment levels, it is derived from the numerical model for a reduced LM1 compatible with the one-year reference period based on Equation (1). As for the final level, M_Q is derived directly from WIM measurements for various time periods using the convolution method. Mean values for WIM traffic load effects are defined as median values of cumulative distribution functions (CDFs) for various time periods, as presented in Figure 6. The initial CDF of traffic load effects $f(x)$ (Figure 6a) is developed with the convolution method from actual WIM measurements recorded for a period of over two months, in both summer and winter seasons. CDFs for other time periods are extrapolated using the extreme value theory (Ang & Tang, 1975) by exponentiating the initial distribution $f(x)$ to a certain power. Žnidarič (2017) proposes the variable N for exponentiation, whose value is based on three parameters: number of days taken into consideration (e.g. number of working days per year), selected time periods for extrapolation, and the number of multiple presence events on the bridge expected in a chosen time period. The last of these parameters presents the most influencing parameter for the value of N and is explained in more detail in (Žnidarič, 2017).

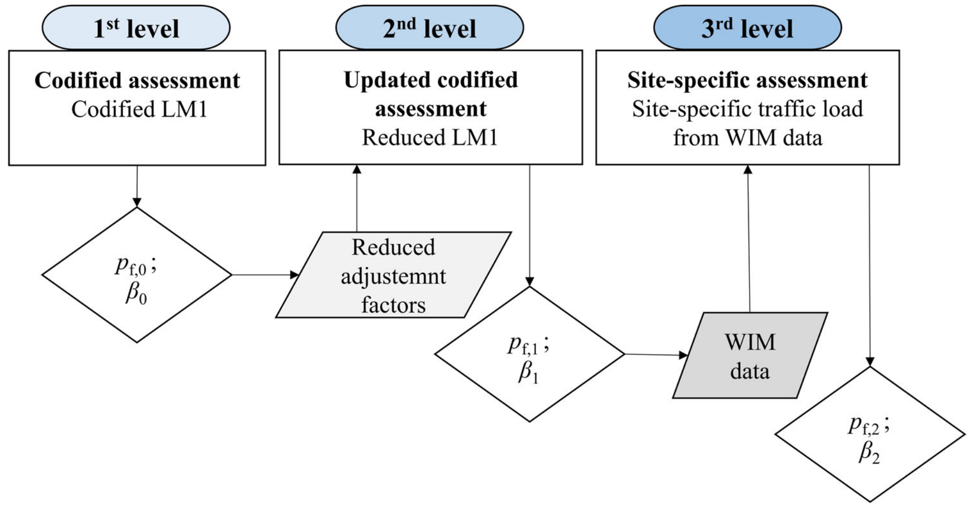

Figure 5. Multi-level assessment strategy supported by full-probabilistic analysis.

For the reliability analysis of the Case Study bridge, the maximum expected traffic load effects from WIM measurements are extrapolated for a reference period of one year only, to be compatible with the target reliability index. The value of N is calculated directly from the obtained data and the CDF is presented in Figure 6b.

It is clear from Figure 6 that the CDF for the time period of 1 year has shifted to the right compared to the initial one, resulting in increased mean and characteristic values, but is also steeper, meaning that the variability is decreasing. For longer time periods, the variability decrease even more, as the parameter N increases exponentially and the CDFs will be more and more steeper, as described in (Žnidarič et al., 2012).

Expected bending moment values shown in Figure 6 are derived from WIM measurements for the total width of the bridge cross-section and are therefore expressed in kNm. The absolute value from Figure 6 must be modified as the LSE in Equation (5) is defined for the reliability analysis of the critical bridge-deck section, with the total width of 100 cm, based on recommendations given in design codes (Eurocode, 2004). Before their implementation in Equation (6) their absolute value is multiplied with the factor (LDF), which defines the proportion of total load transferred to the critical slab section. For the Case Study bridge, the LDF factor is derived directly from the numerical model, as a ratio of absolute value of total load effects [kNm] to their proportion transferred on the critical section [kNm/m] is equal to 0.167. Furthermore, to take into account the dynamic proportion of load effects due to bridge-vehicle interaction, the values from Figure 6 need to be multiplied with the dynamic amplification factor (DAF). In the absence of additional traffic data, the value from design codes for new bridges is used for the DAF (Bruls, Mathieu, et al., 1996; Eurocode, 2005). Its value depends on the bridge type, span and selected load effect, and for the Case Study Bridge, it is equal to 1.25. The traffic load effects presented in the Table 3 are calculated for one-year reference period, using Equation (1) for levels 1 and 2, while n level three they are derived directly from WIM measurements (Figure 6).

The reliability analysis for the Case Study bridge is conducted for each of the three defined assessment strategies using Monte Carlo simulation with 10^8 runs, based on the defined LSE (6) and the values from Tables 2 and 3. The number of simulations is calculated from the recommendations given by Nowak and Collins (2007) based on the expected probability of failure (10^{-5}) and the coefficient of variance (0.05). The probability of failure is selected approximately based on recommendations for new structures while the coefficient of variance is approximated to take statistical error into account. Results in terms of calculated probabilities of failure, and the corresponding reliability indices, are presented in Table 4.

Results presented in Table 4, on one hand, clearly point to the benefits of additional data in both second and third assessment levels, validating the additional data with the reduction of probability of failure and increase in the corresponding reliability index. However, on the other hand, obtained reliability indices are too low compared to the minimum required value of 3.3, based on the consequence class and relative cost of safety measures (JCSS, 2002). Therefore, the conclusion can be made that the Case Study bridge is not suitable for traffic loads prescribed in the current design codes for new bridges (Eurocode, 2005) at level 1, neither for the reduced values based on traffic analysis at level 2. Even at the third level, where assessment results are based solely on traffic data from WIM measurements, the bridge cannot be assessed as safe to use as its reliability index is below the required minimum of 3.3. The authors are aware that the assessment results from Table 4 are not as realistic as they should be, taking into the fact that the Case Study bridge is in everyday use. Improved and more accurate results can be obtained using non-linear analysis, which would provide much higher values of the yield strength of the used plain-round reinforcement (GA220/360), as proved in (Srbić, Ivanković, & Brozović, 2019). Such analysis is not implemented in this paper as its focus is on the VoI analysis of additional SHM data, and the reliability analysis results are only used as input data.

Furthermore, in the third level of assessment, it is important to note that research has proved that DAF value

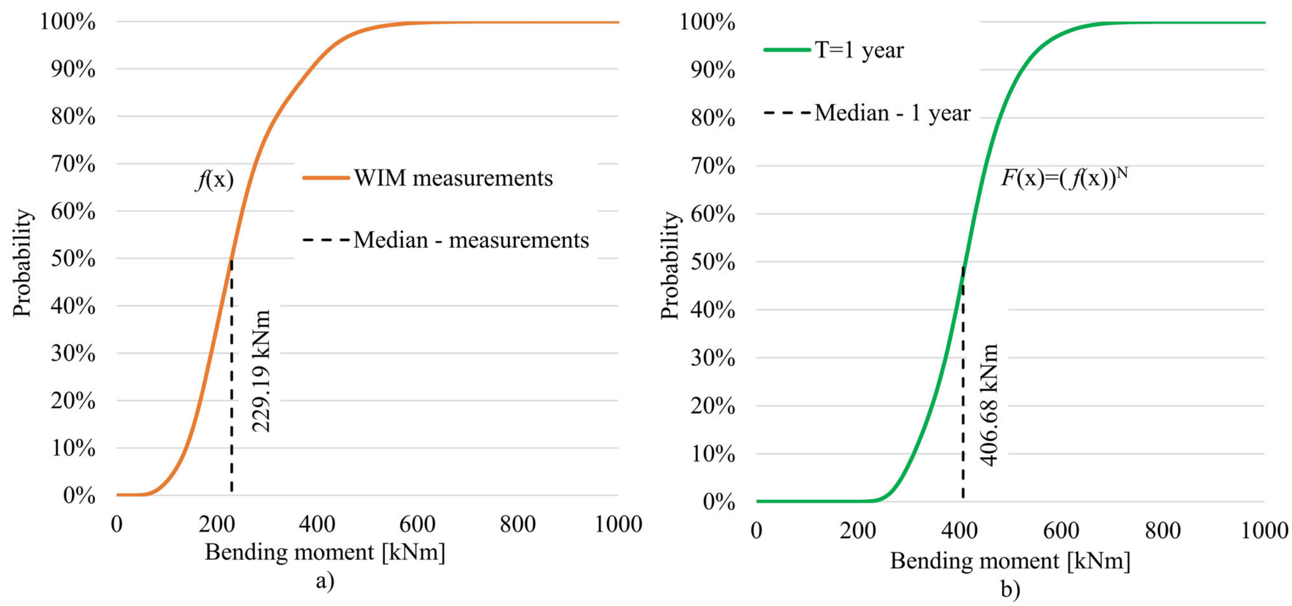

Figure 6. CDFs of maximum estimated traffic load effects on Case Study bridge for a) measurement period, b) 1-year reference period.

Table 4. Reliability analysis results for one year reference period – Case Study bridge.

Assessment strategy (see Figure 5)	Reliability index β	Probability of failure p_f
No additional data – Level 1	0.74	$2.30 \cdot 10^{-1}$
Reduced adjustment factors based on heaviest measured traffic in the country– Level 2	1.41	$7.91 \cdot 10^{-2}$
Site-specific traffic load model based on measured WIM data on the road leading to the bridge – Level 3	2.65	$4.01 \cdot 10^{-3}$

converges to approximately 1.00 with the increase of the vehicle weight (Mandić Ivanković, Skokandić, et al., 2019; Žnidarič, 2017). Therefore, it is obvious that the reliability index from Table 4 on level 3 would increase even further if the additional traffic data is obtained and used for DAF calculation.

4. Value of additional WIM information in the assessment procedure

The results from Table 4 justify the initial investment in SHM procedures and tools from the technical point of view. Nevertheless, as stated in the introduction, SHM tools are used predominantly for landmark bridges, whereas for smaller less important ones, the decision making process is based mostly on experience and conservative assumptions from design codes (Zonta et al., 2014). The VoI analysis algorithm is developed in this section in order to quantify the economic feasibility of additional traffic data for both levels two and three, from the bridge owner and user perspective.

4.1. Theoretical framework for VoI analysis of SHM data

The basic assumption for the quantification of additional SHM data from the perspective of infrastructure owners is defined as the difference between the expected (prior) operational cost of the selected infrastructure object C (estimated when the SHM is not implemented) and the posterior cost C^* which arises upon implementation of the SHM in the decision-making process (Thöns et al., 2017; Zonta et al., 2014). The value of additional SHM information analysis can in general be conducted prior to its implementation (pre-posterior analysis) and after its implementation when the effects are expressed in terms of additional costs and benefits (posterior analysis – analysis of conditional information). The pre-posterior VoI analysis is commonly described as the analysis of an unknown information and it answers the question of whether the acquirement of the additional information will be cost-effective from the perspective of the structure owner. On the other hand, the posterior VoI analysis is conducted after the additional information has already been acquired and is used for the comparison of the prior (before implementation) and posterior (after implementation) results in terms of cost and benefits. More commonly, it is described as the process which gives an answer to the question of whether the money spent for additional information is cost-effective in terms of its utilization from the perspective of the structure owner (Faber, 2012; Thöns, 2018).

The theoretical framework developed within the COST Action TU1402 is focused around implementation of the pre-posterior VoI analysis in the decision process (Thöns et al., 2017). Nevertheless, the posterior VoI analysis is used for the Case Study bridge as WIM measurements used in the assessment procedure in the first part of this paper have been conducted continuously for several years in Croatia (Skokandić, 2020). This approach is justified by the fact that the SHM measurements have already been performed and additional information has been obtained by the infrastructure owner. Thus, the purpose of this research can be summarized as evaluation of all related costs in the assessment procedure, and definition of potential savings from both the perspective of bridge owner and user, which arises from implementation of the selected SHM method. This is particularly important as, for the time being, bridge owners are not using the available traffic data for bridge assessment. In order to do so, the decision tree graphical tool for engineering decision processes with multiple alternative outcomes (Faber, 2012), defined in (Skokandić, 2020), has been modified for implementation in this paper and is presented in Figure 7. In general, the decision tree consists of main branches, each representing an alternative decision scenario of choice nodes (squares) and chance nodes (circles) for probabilistic models, such as decision outcomes (Thöns, 2017).

The decision tree shown in Figure 7 has three main branches, each representing one of the assessment levels from Table 4, denoted as $S_0$for reference strategy where no additional traffic data is used (level 1), $S_1$for implementation of the reduced load model based on traffic measurements (level 2), and $S_2$for a strategy where site-specific traffic load model based on WIM measurements is implemented (level 3). The choice nodes on each main branch are defined as "Action nodes" and they represent the bridge repair and strengthening decision, where the choice for no repair is denoted as a_0 while the choice for full-bridge repair is denoted as a_1. Based on the choice made in the action nodes, each sub-branch can result in one of two system states in the final choice node, which stand for analysis outcomes and are denoted as system states. The outcomes are simply divided into state X_1 when the structure is safe and X_2 when it is unsafe, modelled with the corresponding probabilities of occurrence which in sum are equal to 100%. For the Case Study bridge, these outcomes are calculated directly from the calculated probabilities of failure (Table 4), when the choice of no bridge repair is selected (a_0). On the other hand, when the full bridge repair is chosen (a_1), the probability for the state X_2 is defined as the maximum allowed probability of failure for new structures from the current design codes (Eurocode, 2002), and is equal to $5 \cdot 10^{-5}$. Finally, each system state is correlated with benefits (consequences) that arise in case of its occurrence, presented in Figure 7 as $B_{1,2}(S_i;\, a_j)$, where S_i and a_j represent choices on assessment strategy and bridge repair, respectively, which lead to the selected system state.

Benefits arising from the outcome in each of the branches of the defined decision tree (Figure 7) can be calculated as defined in (Skokandić, 2020):

$$B(S_i; a_j) = p(X_1) \cdot B_1(S_i; a_j) + p(X_2) \cdot B_2(S_i; a_j) \tag{7}$$

The next step is defined as selection of an optimum action branch (a_0 or a_1 in Figure 7), which is defined as the one with maximum benefits:

$$\mathrm{B}(S_i) = \max[B(S_i; a_j)] = \max\begin{cases} B(S_i; a_0) \\ \\ B(S_i; a_1) \end{cases} \tag{8}$$

In the final step, an optimum assessment strategy S (S_0, S_1 or S_2 in Figure 7) is chosen using the same approach as in Equation (8):

$$B(S) = \max[B_i(S_i)] = \max\begin{cases} B_0(S_0) \\ \\ B_1(S_1) \\ \\ B_2(S_2) \end{cases} \tag{9}$$

The absolute value of assessment strategies with additional SHM information can be expressed as the increased benefits due to their implementation in the assessment:

$$V_{S1}\ [\text{EUR}] = B_1(S_1) - B_0(S_0) \tag{10}$$

$$V_{S2}\ [\text{EUR}] = B_2(S_2) - B_0(S_0) \tag{11}$$

where $B_0(S_0)$ denotes the benefits of reference (prior) strategy when no additional information is implemented in the assessment procedure.

Finally, relative value of additional SHM information is defined as:

$$V_{S1,\,relative}\ [\%] = \frac{B_1(S_1) - B_0(S_0)}{|B_0(S_0)|} \tag{12}$$

$$V_{S2,\,relative}[\%] = \frac{B_2(S_2) - B_0(S_0)}{|B_0(S_0)|} \tag{13}$$

The presented procedure can be applied in the VoI analysis of additional traffic information for the assessment of Case Study bridges if the maximum expected benefits in Equations (7) to (13) are modelled as negative expected costs for each outcome. A similar procedure is recommended within the COST Action TU1402 framework (Thöns, 2017). In order to do so, all related costs need to be estimated based on the basic bridge characteristic, expected consequences of potential bridge closure and/or failure, total repair costs, and the SHM measurement costs.

4.2. Definition of costs implemented in VoI analysis

4.2.1. Introduction and global cost function

Total costs included in the construction and operation of new infrastructure facilities can be estimated relatively accurately based on previous projects and basic costs of materials, construction work, etc., which are defined in the majority of national codes and guidelines. On the other hand, this process is more complex for existing structures, as it includes detailed prior inspection of the structure itself, review of all available documentation, etc. In order to do so for the existing bridges, a detailed literature review is normally conducted prior to the definition of a global cost function, e.g. (De Brito, Branco, Thoft-Christensen, & Sørensen, 1997; De Brito & Branco, 1998; Frangopol & Liu, 2007; Imhof, 2004; Stewart, Rosowsky, & Val, 2001; Thoft-Christensen, 2012).

For the purpose of this research, the global cost function defined by Skokandić (2020) and based on the one proposed by de Brito and Branco (1997), will be used as follows:

$$C_{TOT,\,assessment} = C_{REP} + C_{FAIL} + C_{SHM} + C_{N/A} \tag{14}$$

where:

$C_{TOT,\,assessment}$ – are total costs for each outcome on a defined decision tree (Figure 7);

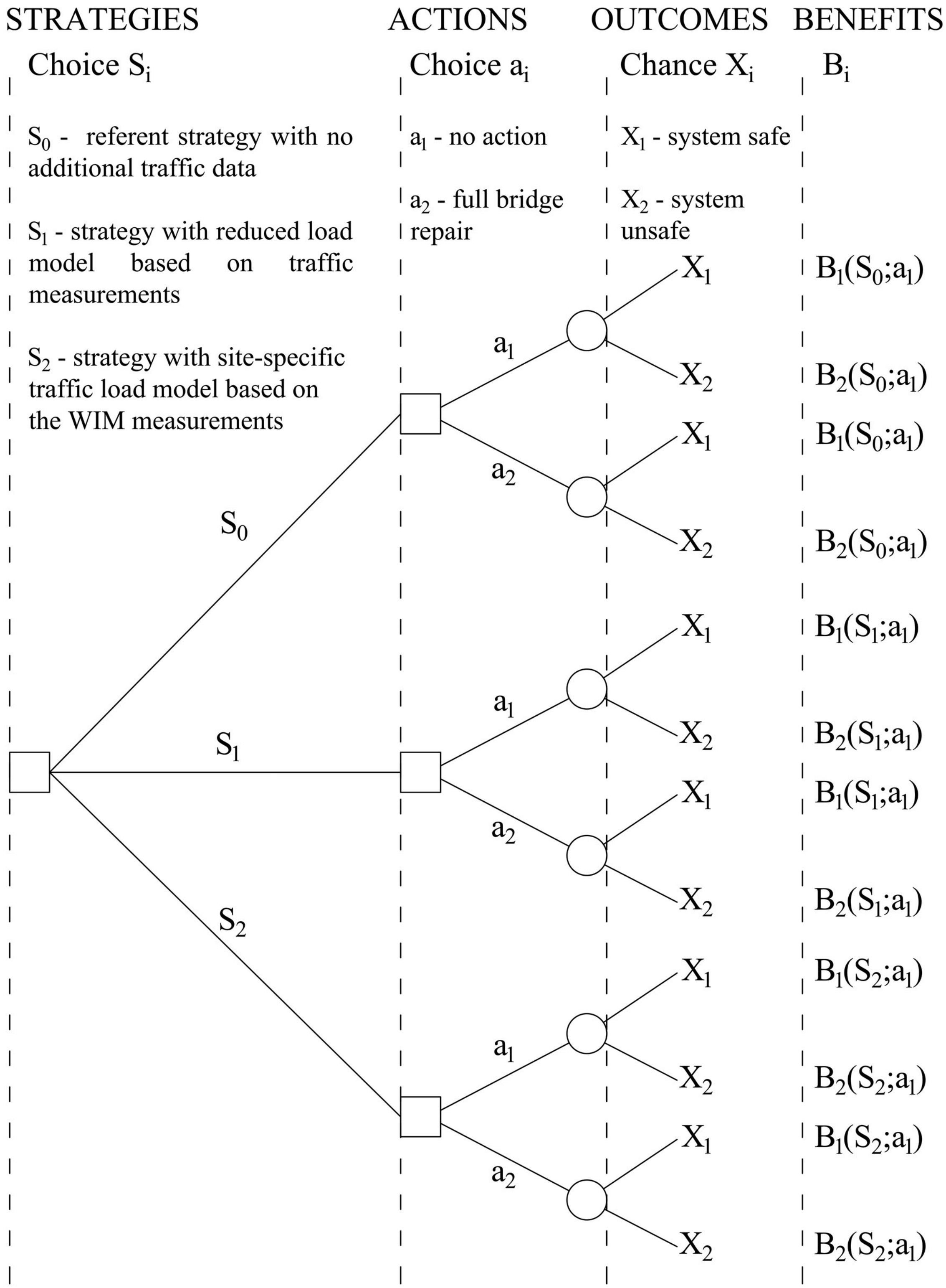

Figure 7. Decision tree for posterior VoI analysis of additional traffic information (reproduced from (Skokandić, 2020)).

C_{REP} – are bridge repair costs;
C_{FAIL} – are overall costs in case of structural failure of the bridge;
C_{SHM} – are costs of bridge monitoring, defined in the scope of this research as WIM measurements;
$C_{N/A}$ – are users' costs that arise from the unavailability of the selected bridge.

All costs from Equation (14) will be estimated based on general bridge characteristics (dimensions, span length, road category, etc.), probabilities of failure (Table 4), and the SHM monitoring costs based on its previous applications. Each parameter of the total cost function will be monetized but will also be presented as the percentage of the total bridge value, denoted as C_{BV}, defined by Skokandić (2020) as:

$$C_{BV} = f_B \cdot C_0 \tag{15}$$

where:
C_0 – are structural costs (construction costs) – defined as the initial bridge value per square meter of the deck;

f_B – is the factor for multiplication of bridge value due to its importance in the network based on the procedure defined in (Mandić Ivanković, Kušter Marić, Skokandić, Njirić, & Šiljeg, 2019).

Basic structural costs C_0 are estimated as new-bridge construction costs, based on an average bridge value applicable in the EU. Although these costs vary based on the bridge type, size, road category, obstacles, etc., two approaches have been selected based on literature review. The first one is from the SERON research project (SeRON, 2012) where costs are estimated as 1200 EUR per square meter of the bridge deck, multiplied with an additional factor which takes into account specific bridge construction conditions (e.g. height of piers, foundation soil quality etc.). The second one is proposed in the PhD thesis by Imhof (2004), where prices vary from 1220 to 1490 EUR/m^2 based on bridge span, type etc. For the purposes of this research, and due to the fact that the foundation soil under the Case Study bridge is of insufficient quality (wooden piles are used, (Šram, 2002)), structural costs C_0 are estimated to 1300 EUR/m^2 based on SERON recommendations.

The importance of the Case Study bridge in the regional transport network is taken into account with factor f_B from Equation (15), using the approach developed by Mandić Ivanković, Kušter Marić, et al. (2019). According to this approach, each bridge is valued based on five basic parameters:

- Road category (G_{RC});
- Average annual daily traffic (G_{AADT});
- Detour distance (G_{DD});
- Largest span (G_{LS});
- Total length of the bridge (G_{TL}).

Each of the listed parameters is associated with grade (1 to 5) and weighting factor (0.25 or 0.125). Finally, f_B is calculated with Equation (16), where the first three parameters (G_{RC}; G_{AADT}; G_{DD}) are taken to be equally important and mutually independent and are associated with the weighting factor of 0.25. On the other hand, the parameters dealing with bridge dimensions (G_{LS}; G_{TL}) are both describing the complexity of bridge construction and the initial investment, and each of them is therefore associated with the weighing factor of 0.125:

$$f_B = 1 + \frac{1}{5} \cdot [0,25 \cdot (G_{RC} + G_{AADT} + G_{DD}) + 0,125 \cdot (G_{LS} + G_{TL})] \tag{16}$$

4.2.2. Bridge failure costs - C_{FAIL}

Complete or partial bridge failure while in service occurs only in rare situations, but the consequences of such failure can be very severe, and so it needs to be taken into account in the global cost function. These consequences include direct costs (costs of bridge replacement and new construction) but even more indirect costs (property damage, traffic closure, traffic jams on alternate routes, loss of reputation, etc.). Furthermore, bridge failure often causes fatalities, like in the case of a recent bridge collapse in Genoa (Calvi et al., 2019). All these costs and indirect consequences are mutually correlated, and their explicit estimation is not possible for multiple bridges. Therefore, authors often estimate them as a percentage of the total bridge value, such as in the SERON project where the bridge demolition and new construction is defined as over 140% of the bridge value but, when indirect consequences are taken into account, total costs rise to 700% of the bridge value (SeRON, 2012).

Thöns and Stewart (2019) estimate that the total costs of an iconic bridge failure would amount to between 5 and 25 times the bridge value (modelled with triangular distribution), while the costs are lower in the case of bridge closure prior to collapse, between 1 and 5 times the bridge total value (modelled with uniform distribution). The SERON project approach will be used in this paper, as the potential failure of the Case Study bridge is estimated without prior traffic closure. Additionally, as the users' costs due to prolonged travel time in case of the Case Study bridge failure are modelled separately as $C_{N/A}$, direct costs (including damage to property and possible fatalities) will be estimated to 400% of the Case Study bridge total value. The probability of its failure will be calculated in the VoI analysis according to probabilities of failure obtained from the reliability analysis (Table 4).

4.2.3. Bridge repair costs - C_{REP}

A number of authors have focused their attention on the development of a general procedure for estimating bridge repair, maintenance and operational costs in the context of bridge service life management. For example, de Brito and Branco (1997) estimate that repair costs during bridge lifetime can be approximated as an average annual value of 5% of the initial bridge cost. Estes and Frangopol (1999) use reliability approach for the optimization of different bridge repair strategies, based on real-life costs, while Thoft-Christensen (2012) have developed a time-variant procedure based on probability of failure and majority of bridge characteristics.

However, this procedure is too complex for application in the VoI analysis for the Case Study bridge. A more practical approach for general use was developed within the SERON project, which roughly estimates repair costs as a percentage of bridge value for different damage levels (SeRON, 2012). This approach has been modified by Skokandić (2020) so that it can be used for rough repair costs estimations based on the obtained reliability levels; this modification involves the definition of the factor f_{REP}, which represents repair costs as a percentage of the total value of the bridge C_{BV}.

In the proposed approach, damage levels (described in Table 5) are related to the corresponding range of reliability indices, based on the damage description and influence on the bridge structure. The values of f_{REP} in Table 5 are presented with max. and min. values for the selected reliability index, based on the one-year reference period given in (JCSS, 2002). The maximum value for f_{REP} is 2.00 (200%), which is obtained for bridges in critical condition, and it takes into account costs for the demolition of the existing bridge, its removal, and replacement with the new one.

The correlation between the reliability index β and the corresponding value of the repair factor f_{REP}, given in Table 5 can be described analytically, using the parametric equation defined by Skokandic (2020):

$$f_{REP} = 0.3613\beta^2 - 2.8572\beta + 5.622;\ \max(f_{REP}) = 2.00\ (200\%) \tag{17}$$

The estimation of repair costs for the Case Study bridge is conducted in the paper using Equation (17) and the calculated reliability indices (Table 4) in Section 4.3.

4.2.4. Indirect user costs due to bridge unavailability – $C_{N/a}$

Indirect costs that arise from partial or complete bridge closure, resulting in an increase in bridge user travel time, are often neglected in global cost-benefit analyses performed by infrastructure operators. It is only in the last two decades that several authors have studied these costs in terms of daily money loss due to prolonged commuting time, based on fundamentals defined by Daniels, Ellis, and Stockton (1999). Total user costs include vehicle operating costs, costs of accidents that occur due to traffic jams on alternate routes, and hourly wage not collected due to loss of time, with the last two being the most significant ones. Although these costs are not explicitly modelled in the study on infrastructure deterioration (Koch, Brongers, Thompson, Virmani, & Payer, 2002), the authors estimate that they can be very high, up to 10 times of bridge repair and maintenance costs during its service life. Furthermore, in his studies that compare direct and indirect costs only, Thoft-Christensen (2009; 2012) has come to similar conclusions. Skokandić (2020) has developed an approach based on the fundamentals by Daniels et al. (1999), based on the Eurostat data for basic hourly wage and number of passengers per vehicle in the EU (Eurostat, 2018). Total user costs $C_{\mathrm{N/A}}$ is calculated as:

$$C_{N/A} = N \cdot C_{N/A,\,vehicle} \cdot t_{N/A} \tag{18}$$

where:

N – is the average annual daily traffic on the bridge (AADT);

$C_{N/A,\ vehicle}$ – are the costs of unavailability per vehicle per day;

$t_{N/A}$ – is the unavailability period (duration of partial or complete bridge closure).

The average number N of daily traffic (AADT) from Equation (18) is a basic information regarding the bridge and can easily be obtained from bridge operator, while costs of unavailability per vehicle per day can be calculated based on the estimated prolonged travel time. In the VoI analysis for the Case Study bridge, the average hourly wage is taken as 20.3 EUR and the average number of passengers per vehicle as 1.5, based on Eurostat data. On the other hand, the estimation of the unavailability period $t_{\mathrm{N/A}}$, is rather complex, as a large number of parameters must be taken into account, such as the urgency of intervention, bridge size and type, priority in transport network, etc. Therefore, the method used by Skokandić (2020) on the basis of the SERON project is implemented in this paper. In this method, the unavailability period is estimated for each damage level given in Table 5 based on the reconstruction procedure in Germany (SeRON, 2012).

For example, total user costs are calculated for the unavailability period of one month based on the prolonged travel time of only one minute per vehicle. This is presented in Figure 8 as the ratio of user costs $C_{\mathrm{N/A}}$ to the total bridge value C_{BV}. The ratios given in Figure 8 clearly show that the user costs for smaller bridges can have a significant impact on total costs, which confirms conclusions made by Thoft-Christensen (2009, 2012) and Koch et al. (2002).

4.2.5. Cost of SHM measurements - C_{SHM}

Initial investments in SHM measurements are justified if they can match prices of in-depth visual inspection of the bridge and at the same time provide improved results, based on the book by Wenzel (2009). Bajwa, Coleri, Rajagopal, Varaiya, and Flores (2017) have set the average cost of the pavement-based WIM system (including the initial price, installation, and road closure) at around 22,000.00 USD per lane. Similar prices are listed for bending plate WIMs in the NCHRP report (Hallenbeck & Weinblatt, 2004), and they will be used for the assessment strategy at level 3 from Table 4 as C_{SHM}, but in EUR as approximately 20,000.00 per lane.

Reduced traffic load models used in the assessment strategy at level 2 are based on the analysis of traffic data acquired at multiple locations in Croatia (Mandić Ivanković, 2008). These mainly included the data recorded by the static weighing plates located at the border crossing with high truck transit, and are published annually in Croatia (Croatian Bureau of Statistics, 2018). Therefore, additional data for this assessment strategy were not obtained explicitly from the operator, but were derived using statistical analysis from the existing records, making it complex to monetize its value as an initial investment. For the purposes of this research, only the post-processing of the data in order to develop codified reduced traffic load models will be denoted as C_{SHM} for assessment strategy at level 2, as a 10 000 EUR, based on the average hourly wage of structural engineers in the EU (Eurostat - statistical office of the European Union, 2018). The estimation of SHM costs for each strategy in the assessment procedure for the Case Study bridge is summarized in Table 6.

4.3. Voi analysis for the case study bridge

4.3.1. Input data for VoI analysis

All costs listed in the previous section will be presented both in their total value [EUR] and as a proportion of the

Table 5. Values for repair factor f_{REP} based on (Skokandić, 2020).

Damage level (SeRON, 2012)	1 – Minor damage	2 – Slight damage	3 – Medium damage	4 – High damage	5 – Demolition imminent
Damage description	No influence on the stability, durability or traffic safety	Safety in tolerable range, no impact on traffic	Safety in tolerable range, medium impact on traffic, traffic obstruction	Safety under min. requirements, durability and traffic severely affected	Component failure
Damage impact	Very low, negligible	Low	Medium	High	Very high
Repair urgency	Regular maintenance	Scheduled construction maintenance	In medium term	Short term repair imminent	Immediate reconstruction required
Reliability index β	$\beta > 3.82$	$3.82 < \beta \leq 3.3$	$3.3 < \beta \leq 3.0$	$3.0 < \beta \leq 2.3$	$\beta < 2.3$
f_{REP} [%]	$\leq$ 1.00	1.00-16.00	16.00-34.00	34.00-100.00	$\geq$100.00

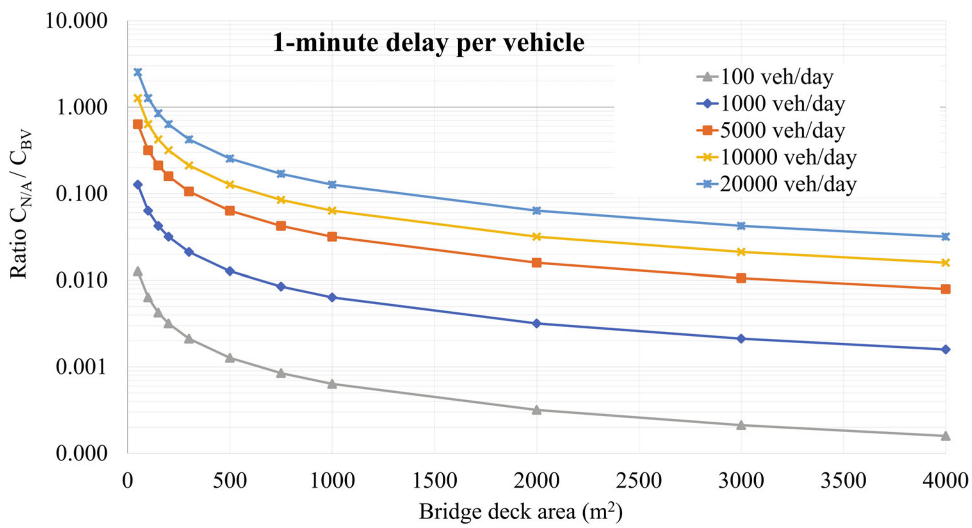

Figure 8. Estimated user costs $C_{N/A}$ as a proportion of total bridge value C_{BV} for the unavailability period of one month.

bridge total value C_{BV}, calculated using Equation (15) and Equation (16) based on the data provided by bridge owner:

$$f_B = 1 + \frac{1}{5} \cdot [0.25 \cdot (G_{RC} + G_{AADT} + G_{DD}) + 0.125 \cdot (G_{LS} + G_{TL})] = 1 + \frac{1}{5} \cdot [0.25 \cdot (4 + 3 + 3) + 0,125 \cdot (2 + 2)] = 1 + 0.60 = 1.60$$

$$C_{BV} = f_B \cdot C_0 = 1.60 \cdot 1.300,00 = 2.080,00 \ \text{EUR/m}^2$$

$$C_{BV(total)} = C_{BV} \cdot A_{bridge} = 2.080,00 \ \text{EUR/m}^2 \cdot 33 \ m \cdot 8.50 \ m = 583.440,00 \ EUR$$

The SHM cost C_{SHM} is taken from the data presented in Table 6 for each assessment strategy, while bridge repair costs C_{REP} are calculated for each branch using the factor f_{REP} from Table 5 and Equation (17) for an adequate damage level of the Case study bridge corresponding to the reliability indices obtained from the three distinct assessment levels (Table 4). In cases when bridge repair is conducted, partial bridge closure is predicted, prolonging the travel time for approximately 1.5 minutes, with the assumption that the repair works are conducted separately by lane. The reconstruction work period of around 3 months is estimated based on SeRON (2012) recommendations. The same approach is used in case of possible bridge failure, where reconstruction period (including demolition and construction of a new bridge) is 12 months, while prolonged travel time is around 5 minutes, based on available alternate routes. The same parameters are used for the bridge repair period when costs of bridge repair are over 200% of the bridge value (Equation (17)).

Costs in the event of total bridge failure C_{FAIL} are taken as 400% of the bridge total value C_{BV} (section 4.2.2.). Costs of unavailability in cases when bridge repair or failure occurs are calculated using Equation (18) and Eurostat data. The average cost per weekend days is estimated at 50% of the workday cost, while the total cost per vehicle per month for every minute of prolonged time is calculated as 12.75 EUR. With an average AADT of 9500 (taking into account both summer and winter seasons) obtained by bridge owner, costs are calculated as follows:

$$C_{\frac{N}{A}}(REP) = N \cdot C_{\frac{N}{A}, vehicle} \cdot t_{\frac{N}{A}} = 9500 \cdot 12,75 \cdot 1.5\text{min} \cdot 3\text{months} = 545.062,00 \ \text{EUR}$$

Table 6. Costs of SHM measurements for Case Study bridge.

Assessment strategy	C_{SHM} per lane	C_{SHM} – total
No additional data – Level 1	0	0
Reduced adjustment factors based on heaviest measured traffic in the country– Level 2	/	10.000 EUR
Site-specific traffic load model based on measured WIM data on the road leading to the bridge – Level 3	20.000 EUR	40.000 EUR

$$
\begin{aligned}
C_{\frac{N}{A}}(FAIL) &= N \cdot C_{\frac{N}{A}, vehicle} \cdot t_{\frac{N}{A}} \\
&= 9500 \cdot 12,75 \cdot 5.0\text{min} \cdot 12\text{months} \\
&= 7.267.500,00 \ \text{EUR}
\end{aligned}
$$

VoI analysis is conducted using the algorithm developed in the Excel spreadsheet software for the assessment results given in Table 4. Summarized input data for VoI analysis for 1-year reference period are presented in Table 7, with benefits for each outcome Bi calculated using Equation (7) as negative total costs $C_{\text{TOT,assessment}}$ using Equation (14).

The VoI analysis is performed by means of data from Table 8 and the decision tree concept presented in Figure 7, using numerical model developed in the Excel spreadsheet software. The results are presented in Figures 9 and 10. The optimum assessment strategy branch is presented with a thick dashed line, as the one resulting in maximized benefits $\boldsymbol{B}_{\mathbf{i}}$ (negative costs $C_{\text{TOT,assessment}}$ which are presented as percentage of the total bridge value C_{BV}).

4.3.2. Voi analysis including both bridge owner and user's costs

Results of VoI analysis given in Figure 9 show that the assessment strategy S2 at level 3 (dark shaded cell on Figure 9), using site-specific traffic load model developed from WIM data, is an optimum strategy for assessment of the Case Study bridge, in terms of costs and benefits for both bridge owner and user. Furthermore, strategy S_1, based on reduced codified traffic load model using the recorded traffic data, is also feasible, compared to the initial strategy S_0 in which no additional data is used in the assessment. The relative value of additional information in both strategies using additional data, S_1 and S_2 is calculated with Equation (12) and (13), and total benefits for each strategy (light shaded cells on Figure 9):

$$
V_{S1,relative} = \frac{B_1(S_1) - B_0(S_0)}{|B_0(S_0)|} = \frac{-1.3188 - (-3.7849)}{|-3.7849|} = 0.6515 = 65.15\ \%
$$

$$
V_{S2,relative} = \frac{B_2(S_2) - B_0(S_0)}{|B_0(S_0)|} = \frac{-0.1340 - (-3.7849)}{|-3.7849|} = 0.9646 = 96.46\ \%
$$

The difference between strategies S1 and S2 is not large as it is between S1 and the prior strategy S0, which is mainly due to the difference in the mean value and standard deviation of the bending moments related to traffic action at level 2 and level 3, respectively. But the difference could get larger for bridges with lower specific traffic loads. Results of the Strategy S1 with the adjusted Load model 1 are also very important as these adjustment factors are based on traffic count in the country in general, although a more localised traffic load could be of greater importance for a specific bridge.

Furthermore, as it is clear from results given in Figure 9 and separate C_{REP} cost values from Table 7, the results of each branch show that the repair of the Case Study bridge is not optimum maintenance approach due to high costs of bridge repair as the reliability indices are very low (choice a_0 – do nothing values are closer to 0 than a_i – repair bridge values). However, it is also visible from Figure 9 that the VoI results strongly depend on calculated user costs arising from bridge unavailability ($C_{N/A}$ specified in Table 7) in case of repair works or its failure. This proves the assumptions made in previous research (Koch et al., 2002; Skokandić, 2020; Thoft-Christensen, 2009) that user costs quickly become dominant in the global cost function even for smaller bridges when the unavailability period is prolonged.

These results could be of key interest for decision makers at the government level. Nevertheless, as these costs are commonly not taken into account by some road and bridge owners in the scope of bridge management systems, VoI from Figure 9 is re-performed by taking into account only direct costs incurred by bridge owner. These results, aimed particularly at bridge owners, in which all unavailability costs are equal to zero, are presented in the form of a decision tree in Figure 10.

4.3.3. Voi analysis including only direct costs by bridge owner

The results of re-performed VoI analysis given in Figure 10 present a similar trend as the ones in Figure 9, as the assessment strategy S_2 is still the most optimal one, followed by the strategy S_1.

$$
V_{S1,relative} = \frac{B_1(S_1) - B_0(S_0)}{|B_0(S_0)|} = \frac{-0.3335 - (-0.9200)}{|-0.9200|} = 0.6375 = 63.75\ \%
$$

$$
V_{S2,relative} = \frac{B_2(S_2) - B_0(S_0)}{|B_0(S_0)|} = \frac{-0.0840 - (-0.9200)}{|-0.9200|} = 0.9086 = 90.86\ \%
$$

4.3.4. Case study bridge VoI results – discussion

Results presented in this case study clearly emphasize the benefits of incorporating bridge assessment results based on

Table 7. Input data for VoI analysis of the Case Study Bridge.

Choice S_i	Choice a_i	Total costs C_{TOT}	C_{REP}	C_{SHM}	$C_{N/A}$	C_{FAIL}
S_0 – reference strategy with no additional traffic data	a_0 – do nothing	$0.000 \cdot C_{BV}$	0.000	0.000	0.000	0.000
		$16.456 \cdot C_{BV}$	0.000	0.000	12.456	4.000
	a_1 – bridge repair	$14.456 \cdot C_{BV}$	2.000	0.000	12.456	0.000
		$18.456 \cdot C_{BV}$	2.000	0.000	12.456	4.000
S_1 – strategy with reduced load model based on traffic measurements	a_0 – do nothing	$0.0171 \cdot C_{BV}$	0.000	0.0171	0.000	0.000
		$16.473 \cdot C_{BV}$	0.000	0.0171	12.456	4.000
	a_1 – bridge repair	$14.473 \cdot C_{BV}$	2.000	0.0171	12.456	0.000
		$18.473 \cdot C_{BV}$	2.000	0.0171	12.456	4.000
S_2 – strategy with site-specific traffic load model based on WIM measurements	a_0 – do nothing	$0.068 \cdot C_{BV}$	0.000	0.068	0.000	0.000
		$16.524 \cdot C_{BV}$	0.000	0.068	12.456	4.000
	a_1 – bridge repair	$1.589 \cdot C_{BV}$	0.587	0.068	0.934	0.000
		$17.111 \cdot C_{BV}$	0.587	0.068	12.456	4.000

Table 8. Summarized results – VoI analysis of additional data – relative value [%].

	The relative value of additional SHM information [%]	
Assessment strategy	Including both bridge owner and user's costs	Including only direct costs by bridge owner
Strategy S1: Reduced adjustment factors based on heaviest measured traffic in the country – Level 2	65.15	63.75
Strategy S2: Site-specific traffic load model based on measured WIM data on the road leading to the bridge – Level 3	96.46	90.86

traffic load monitoring data in the decision-making process for bridge maintenance and management. The implementation of country specific traffic load measurements (strategy S1) should be included in the assessment of existing bridges as they reduce direct costs of the bridge owner (Figure 10 and result with 63.75% relative benefit). Additionally, although the Case Study bridge is not iconic and is relatively small, the investment in site specific WIM measurements (with the strategy S2) would benefit the bridge owner even more (90.86%).

When direct cost for the owner and indirect user costs due to bridge unavailability are considered, both traffic load collection methods (country specific traffic load measurements as a part of strategy S1, and site-specific WIM measurements at the road leading to the certain bridge as a part of strategy S2) result in even higher benefits for society in general (65.15% and 96.46% respectively). The difference of the benefits for two strategies would become even larger for bridges with lower specific traffic loads. Additionally, in order to present the dominant effect of indirect user costs in total cost reduction (in EUR), the absolute values of additional SHM information are given in Table 9, as total savings for both bridge users and its owner (calculated using Equations (10) and (11)).

5. Conclusions

The assessment procedure of the Case Study bridge described in this paper is based around the implementation of additional traffic information, obtained with vehicle weighing process and WIM technology. The purpose was to prove that traffic data, regularly collected by most road directorates worldwide mainly for traffic analyses and selection of overloaded vehicles, can additionally be used as a basis for site-specific assessment of existing road bridges, which will consequently lead to a more efficient bridge management.

The benefit for both the bridge owner and bridge user is presented, and both the relative and absolute value of additional information for each assessment strategy are summarized in Tables 8 and 9. It is important to note that the calculated relative values of additional SHM information are very dependent on the input parameters (probabilities of failure and corresponding reliability indices from Table 4). In cases when bridges have higher reliability levels, the difference between relative values when only direct costs are taken into account and when user costs are added is much larger, as presented on two newer bridges in (Skokandić, 2020).

The added value of the presented research can be summarized as follows:

1. the global cost function is developed with detailed modelling of each cost parameter as a percentage of the total bridge value
2. the trade-off between the bridge owner perspective (smaller total benefits) and society perspective (higher total benefits) for both strategies S1 and S2
3. the critical influence that indirect costs have on the outcomes of the VoI analysis is identified – the dominance of users' costs in global cost function – difference between the total costs reduction with and without users' costs in Table 9

VoI based case studies, as the one presented in this paper, can convince bridge operators, and consequently

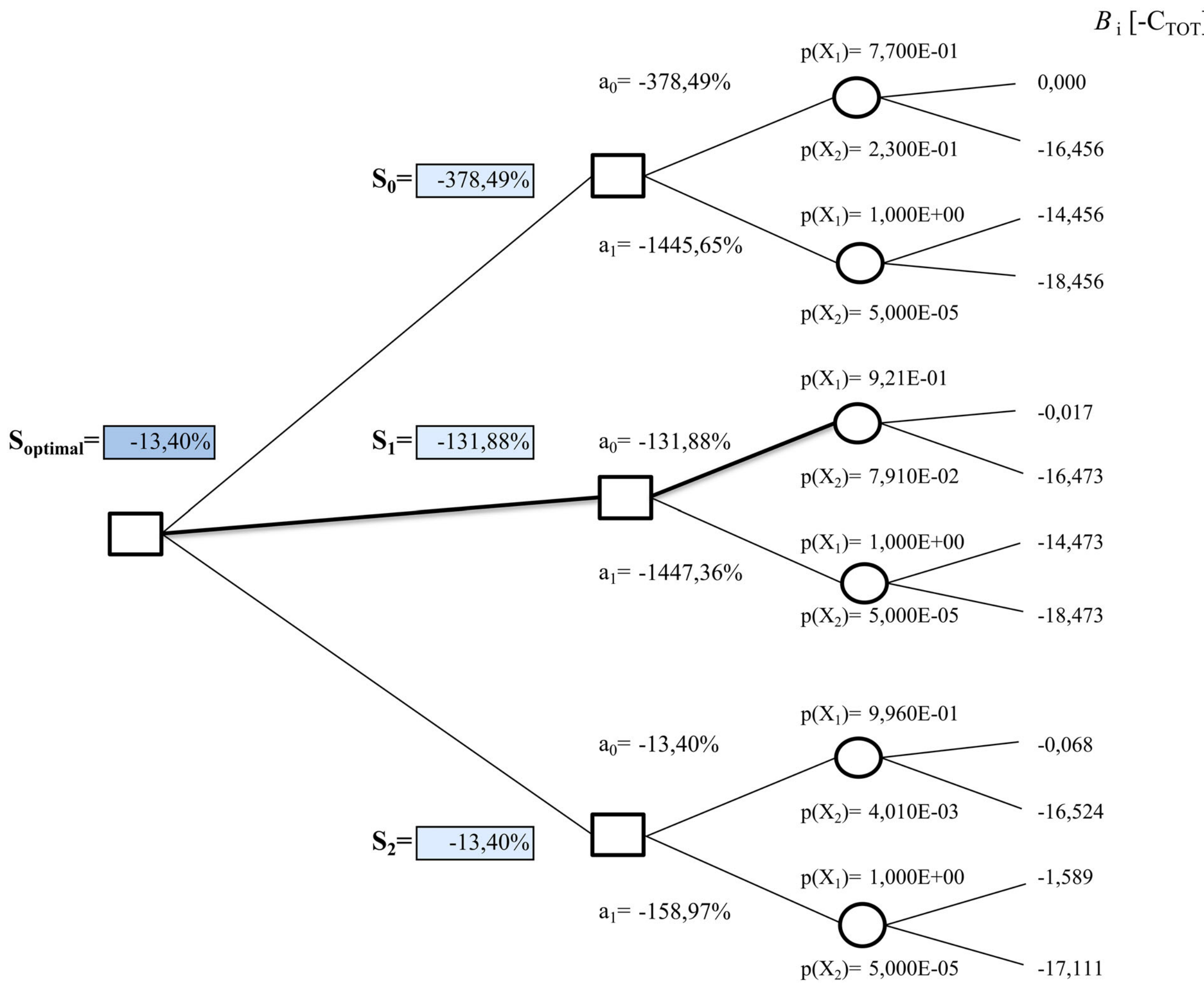

Figure 9. VoI analysis of additional traffic data in assessment of the Case Study bridge, including both direct and indirect costs.

decision makers at the government level, about benefits of employing traffic load monitoring data in structural assessment of existing bridges and, consequently, in making knowledge-based maintenance decisions for an optimum bridge network management. In this way, the practical value of proactive bridge management is clearly demonstrated – embracing innovative tools and methods, as opposed to reactive management – employing visual inspection-based condition assessment.

Further study, in continuation of the presented research, is aimed at creating the database with multiple various bridges and the corresponding measured traffic data. By doing so, the likelihoods of SHM indication used could be estimated with sufficient reliability to be applied in the pre-posterior analysis. Consequently, uncertainties in the analysis (bridge carrying capacity, traffic load effect variability, and estimated costs) would be reduced as more and more bridges are analysed. In order to conduct pre-posterior analysis using the presented algorithm, the decision tree (Figure 7) can easily be modified by adding the probabilistic chance node labelled WIM outcome (SHM Indication) prior to action choice node on branches S_1 and S_2. By doing so, prior probability of failure, obtained without any additional traffic information, and the defined SHM likelihoods, will be sufficient for reliable estimation of probabilities of failure and the corresponding costs and benefits on subsequent branches (with additional traffic information). The estimation procedure can be conducted using the Bayesian updating theory, and will thus provide decision-makers with a value of additional WIM information for selected bridges or bridge network before the information is actually obtained.

Additionally, the presented algorithm can be used for priority ranking of bridges in the network based on urgency of repair. By implementing the time-variant analysis, it is possible to estimate the time period for the realisation of maintenance activities. In order to do so, the estimated costs need to be modified as they are based on present-day values and currently available knowledge. The discounting model proposed by Rackwitz (2006) for industrial countries can be used for modification of future-investment costs.

It is important to note that the accuracy and robustness of the presented algorithm for estimation of all costs and benefits related to the bridge management procedure is closely related to the selection of method and statistical parameters for reliability analysis (Tables 2 and 3), and to the estimation of total costs. Therefore, creation of a

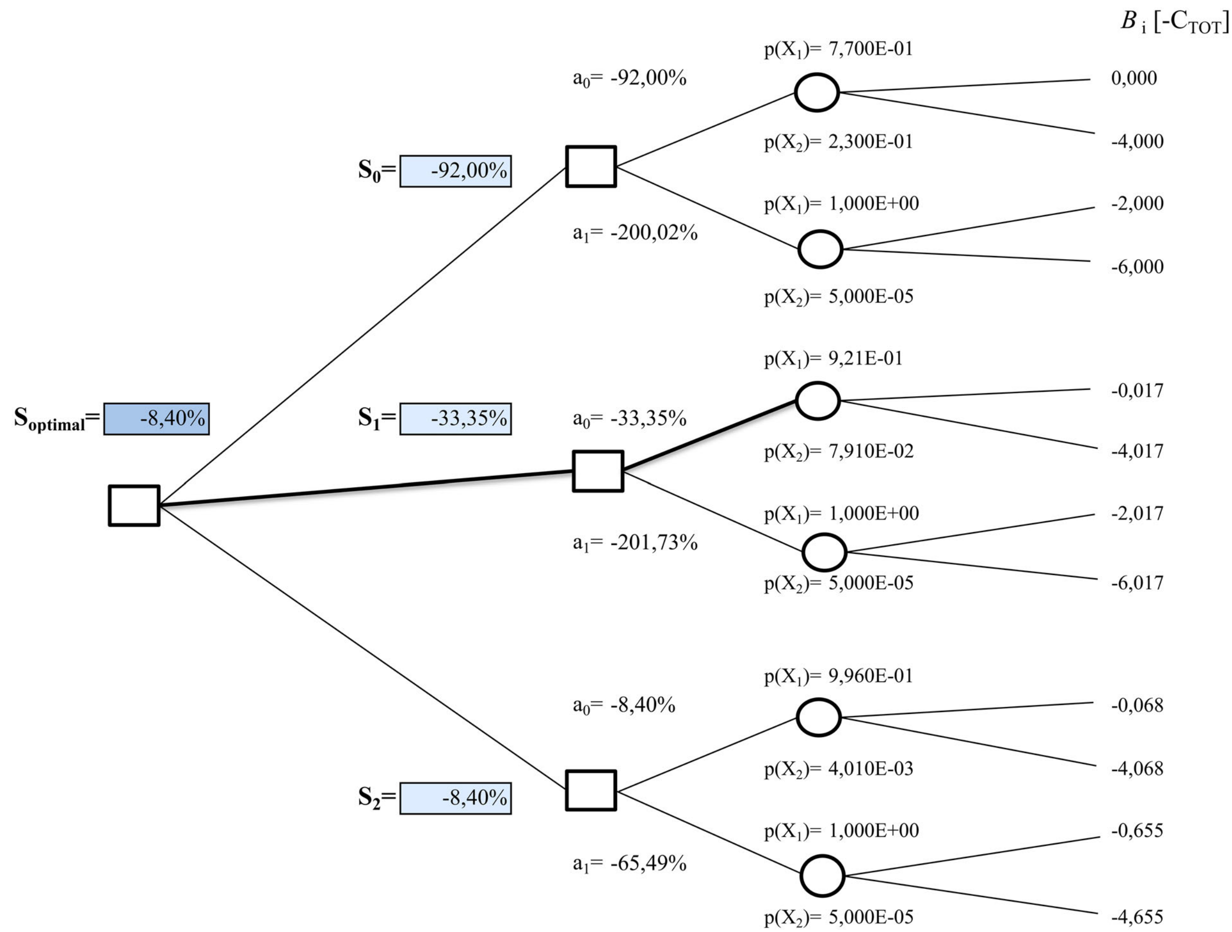

Figure 10. VoI analysis of additional traffic data in assessment of the Case Study bridge, taking into account only direct costs incurred by bridge owner.

Table 9. Summarized results – VoI analysis of additional data – absolute value [EUR].

Assessment strategy	The absolute value of additional SHM information [EUR]: Summarized savings of both bridge owner and its users	The absolute value of additional SHM information [EUR]: Direct savings of bridge owner
Strategy S1: Reduced adjustment factors based on heaviest measured traffic in the country – Level 2	1.438.821,00	342.187,00
Strategy S2: Site-specific traffic load model based on measured WIM data on the road leading to the bridge – Level 3	2.130.081,00	476.087,00

database with realistic parameters for selected bridges would reduce deviation of estimated characteristics and costs from reality, making the proposed method even more suitable for implementation in bridge management systems.

Notations list

AADT	Average Annual Daily Traffic
B-WIM	Bridge Weigh-in-Motion
CDF	Cumulative distribution function
DAF	Dynamic Amplification Factor
FORM	First Order Reliability Method
GSW	Gross Vehicle Weight
JCSS	Joint Committee for Structural Safety
LDF	Load Distribution Factor
LM1	Load Model 1
LSE	Limit State Equation
MC	Monte Carlo method
NCHRP	National Cooperative Highway Research Program
POT	Peaks over Threshold
RC	Reinforced Concrete
SHM	Structural Health Monitoring
SiWIM®	Slovenian Weigh-in-Motion
TS	Tandem System (concentrated traffic load)
UDL	Uniformly Distributed Load (distributed traffic load)
VoI	Value of Information
WIM	Weigh-in-Motion

List of Symbols

a_0; a_1	Actions regarding the bridge (no repair; repair)
$\alpha_{Q,i}$; $\alpha_{q,i}$; $\alpha_{q,r}$	Adjustment factors for traffic load model LM1
β	Reliability index
$\theta_{E,G}$	Random variable of model uncertainties for permanent load effect
$\theta_{E,Q}$	Random variable of model uncertainties for traffic load effect
θ_R	Random variable of model uncertainties for resistance
μ	Mean value

σ	Standard deviation
B_i	Total benefits for strategy S_i
C_0	Bridge construction costs
C_{BV}	Total value of the bridge
C_{FAIL}	Cost of bridge failure
C_{SHM}	Cost of bridge monitoring
$C_{N/A}$	Cost of bridge non-availability
C_{REP}	Cost of bridge repair
$C_{TOT,assessment}$	Total costs for bridge assessment
f_B	Factor for multiplication of bridge value due to its importance
f_C	Construction factor
f_{REP}	Factor for complexity of bridge repairs
G_{AADT}	Grading factor – AADT
G_{DD}	Grading factor – Detour distance
G_{LS}	Grading factor – Largest span
G_{RC}	Grading factor – road category
G_{TL}	Grading factor – Total bridge length
p_f	Probability of failure
S_0; S_1; S_2	Assessment strategies regarding additional SHM data
X_1; X_2	System outcomes (system safe; system not safe)

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Disclosure statement

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