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Proceedings of the

JOINT COST TU1402 – COST TU1406 – IABSE WC1 WORKSHOP

The Value of Structural Health Monitoring for the reliable Bridge Management

2nd – 3rd March, 2017

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Publisher: Faculty of Civil Engineering, University of Zagreb

ISBN: 978-953-8168-08-6

DOI: 10.5592/CO/BSHM2017

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

EUROPEAN COOPERATION IN SCIENCE & TECHNOLOGY

Funded by the Horizon 2020 Framework Programme of the European Union

MAIN CONTENTS

1 WORKSHOP OVERVIEW

In early March 2017 a joint workshop of three associations took place at the Faculty of Civil Engineering of the University of Zagreb. The workshop was organised by two European cooperation and networking projects under auspices of the COST framework and the Working Commission 1 of the International Association for Bridges and Structural Engineering.

COST Action TU1402 strives to enhance the benefit of Structural Health Monitoring (SHM) by novel utilization of applied decision analysis on how to assess the value of SHM – even before it is implemented. COST Action TU1406 aims to bring together, for the first time, both research and practicing community in order to accelerate the establishment of a European guideline in the area of quality specifications for roadway bridges. TU1402 was initiated in November 2014 and TU1406 in April 2015 and together involve about 250 participants from 38 European countries and 15 countries from outside Europe.

IABSE Working Commission 1 provides a forum for discussion on problems related to structural performance and safety and life-cycle cost. WC1 deals with structural performance under various actions, both man-made and environmental, the methodology of structural analysis, assessment of design loads, material properties, structural resistance, and problems related to structural safety and serviceability, for buildings, bridges and other civil engineering structures.

The long-term objective of this workshop was to contribute jointly to the improvement of the bridge management leading to satisfied users and bridge operators, and sustainable development of European road network. More specific objective is to reveal the value of using sophisticated methods of collecting, updating and processing data and subsequently their inclusion in the probabilistic-based reliability analysis.

The workshop gathered 120 participants from 40 countries and was organised through three main sessions:

- Performance assessment of existing Bridges for their reliable management;
- Framework, Strategies and Tools towards the Quantification of the Value of SHM;
- Management and Performance-assessment of Existing Structures

Figure 1: The workshop in Zagreb gathered 120 participants from 40 countries with different professional background

Four important keynote lecturers shared their knowledge on assessment, monitoring and life cycle management of bridges:

- Dan M. Frangopol from the Department of Civil and Environmental Engineering at the Lehigh University in USA lectured on *Risk-, Resilience- and Sustainability-Informed Decision Making for Bridges in a Life-Cycle Multi-Objective Optimization Context*;

- James L. Beck from the Dept. of Computing & Mathematical Sciences at the California Institute of Technology in USA introduced participants to *Bayesian uncertainty quantification and sparse Bayesian learning for model updating in SHM*;

- Ho-Kyung Kim from the Department of Civil and Environmental Engineering at the Seoul National University in South Korea shared the experience in *What We Have Learned from Operational Monitoring and Serviceability Assessment of Long-Span Bridge*; and

- Hitoshi Furuta from the Faculty of Informatics at the Kansai University in Japan gave an overview of *New Technologies for Condition Assessment of Existing Structures*.

Figure 2: Keynote presenters from Lehigh University in USA – Frangopol; from California Institute of Technology in USA – Beck; from Seoul National University in South Korea – Kim; and from Kansai University in Japan – Furuta, shared their knowledge as external advisors

Twenty authors with different backgrounds presented their theoretical and practical knowledge and experience, recent research and application examples in the area of structural health monitoring, performance assessment, quality control, maintenance and management of bridges.

Additionally, technical visit to the Sava river bridges was performed. Altogether 5 bridges on 3 different locations – old composite Sava Bridge, railway steel Green Bridge, arch steel Liberty Bridge, prestressed girder Adriatic Bridge and extrados prestressed Homeland Bridge were visited and their structural specificities, history details and curiosities were explained to the participants.

After the Workshop closing, the Joint Steering committee of both COST Actions TU1406 & 1402 and IABSE WC1 decided to continue the cooperation through

- the joint special issue in the SEI journal,
- selection of joint case study bridge example,
- developing joint training schools and
- organisation of interactive Short term Scientific Missions.

Figure 3: Active discussions during the workshop, particularly during the Closing session initiated future cooperation:

left upper photo: José Matos, chair of the COST Action TU1406, Niels Peter Høj, vice-chair of the IABSE WC1, Sebastian Thöns, chair of the COST Action TU1402;
right upper photo: Michael Havbro Faber, Scientific Chair of the TU1402
left bottom photo: Maria Pina Limongelli, TU 1406 Innovation Committee
right bottom photo: Helder Sousa, TU1402 Innovation Committee

1.1 NOTE FROM THE CHAIR OF THE COST ACTION TU1402

The COST Action TU1402 strives to enhance the benefit of Structural Health Monitoring (SHM) by novel utilization of applied decision analysis on how to assess the value of SHM – even before it is implemented. This improves decision basis for design, operation and life-cycle integrity management of structures and facilitates more cost efficient, reliable and safe strategies for maintaining and developing the built environment to the benefit of society.

Structural Health Monitoring (SHM) can efficiently contribute to an enhanced functioning of infrastructures, by facilitating an improved benefit generation and the reduction of operational costs and risks throughout the life cycle. In this perspective, SHM development and research is traditionally focused on technologies and data analysis approaches providing various information associated to the loading and to the structural conditions. However, the link to a quantification of the enhanced functioning of civil structures and infrastructures provided by SHM is often missing.

The quantification of the Value of the Information provided by SHM necessitates the knowledge and modelling of the life cycle performance of the monitored infrastructure and of the SHM system, their probabilistic characteristics, functionality, benefits, costs, risks, operation and decommissioning. In addition, it necessitates that these models are coupled in the framework of the Bayesian decision theory. Then the effect of SHM on the expected benefits and costs throughout the infrastructure life cycle can be quantified and optimized.

For the specific area of bridges, the COST Action TU1402 seeks liaison with the COST Action TU1406 and IABSE to identify potentials for cooperation and to exchange research and impact visions. The workshop on “The Value of Structural Health Monitoring for the reliable Bridge Management” constitutes an important event in this perspective.

I wish you an enjoyable reading.

S. Th

Sebastian Thöns

1.2 NOTE FROM THE CHAIR OF THE COST ACTION TU1406

COST Action TU1406 aims to address the European economic and societal needs by standardizing the condition assessment and maintenance level of roadway bridges. Currently, bridge quality control plans vary from country to country and, in some cases, within the same country. This therefore urges the establishment of a European guideline to surpass the lack of a standard methodology to assess bridge condition and to define quality control plans for roadway bridges.

Such a guideline will comprise specific recommendations for assessing performance indicators as well as for the definition of performance goals, bringing together different stakeholders (e.g. universities, institutes, operators, consultants and owners) from various scientific disciplines (e.g. on-site testing, visual inspection, structural engineering, sustainability, etc.) in order to establish a common transnational language.

COST Action TU1406 Workshops aim to facilitate the exchange of ideas and experiences between active researchers and practitioners as well as to stimulate discussions on new and emerging issues in line with the conference topics. So far Workshops have been essentially focused on developments performed within WG1 issues, addressing performance indicators, WG2, performance goals, and WG3, establishment of a Quality Control plan, making it possible to assess performance of bridges over their life-cycle.

Information on performance indicators and strategy characterization can be obtained with the implementation of Structural Health Monitoring (SHM) which, with the use of different technologies and algorithms, supply information about the performance of structures over their life-cycle. This is the aim of COST Action TU1402.

There is therefore, a common point of interest between COST Actions TU1406 and TU1402, justifying interactions between both. This collaboration, when extended to IABSE, namely to Working Commission 1, which provides a forum for discussion on problems related to structural performance and safety and life-cycle cost, enriches the work to be achieved by all.

Hopefully, with this eBook, the relevancy of this symbiosis can be shown, not only to those directly involved, but also for the whole engineering community.

José Matos

1.3 NOTE FROM THE VICE - CHAIR OF THE IABSE WORKING COMMISSION 1

The COST Actions TU 1402 and TU 1406 make significant progress in the development and discussion of the quantification of the value of the information and for management of bridge structures in general. The achievements in the COST actions deserve to be disseminated to a broader audience within Europe and worldwide.

IABSE - and in the case specifically IABSE Working Commission 1 " Structural Performance, Safety and Analysis"- has proudly participated in the planning of the workshop "The Value of Structural Health Monitoring for the reliable Bridge Management". From the side of IABSE WC1 we have contributed with views from outside the two COST actions.

IABSE Working Commission 1 provides a forum for discussion on problems related to structural performance and safety and life-cycle cost. The evaluation of Performance includes analytical and experimental approaches. The activities in the Workshop in Zagreb 2 – 3 March 2017 is fitting well in fields of interest of the commission, and the significant results in the COST Actions are foreseen to be followed up the commission – also after the COST actions have been concluded.

Thanks to Ana Mandic for arranging an unforgettable event – we are looking forward to collaboration in the future.

Niels Peter Høj

1.4 NOTE FROM THE LOCAL ORGANISERS CHAIR

Faculty of Civil Engineering under the auspices of University of Zagreb is the oldest civil engineering faculty in Croatia conducting education at the undergraduate, graduate and postgraduate level in all branches of civil engineering. Within Department for structures, Chair for Bridges is the oldest technical Chair of University of Zagreb founded in 1919. It is enrolled as scientific centre in the field of bridges, place where young researchers are educated and at the same time a very productive core where many big bridges are designed. Currently two large bridges designed at the Chair for Bridges, are under construction in Croatia. In addition, Zagreb airport building designed at the Bridge department just open for traffic in April 2017.

Bridge department is traditionally oriented towards design of new bridges by improving analysis and construction methods, but also in developing bridge management system, strategies and working plans. We are involved in technical controls and revisions of projects as well as in the highly demanding expertise and reconstructions. During last decade our research activities are oriented to limit states and sustainable bridge engineering, assessment of existing bridges, numerical modelling of corrosion effect in maritime environment, structural durability. Our main current focus is to improve performance assessment methods within management of bridges considering traffic load enlargement, high wind influence and seismic activity.

As a member of Working Commission 1 of the IABSE, at the end of 2014, I proposed the topic *Performance of existing structures, analysis and safety* to be initiated through Geneva Conference session in September 2015, further on developed through the special issue of the Journal Structural Engineering International (SEI), with the idea to finally generate a particular IABSE Report. When I got involved in the activities of the COST Acton TU 1406 - dealing with quality control of road bridges and TU 1402 - trying to quantify the value of structural health monitoring, I realized we may all work to the joint target - to improve bridge assessment and management leading to satisfied users and bridge operators. Members of those three associations/activities, with diverse knowledge and experience in the area of the structural health monitoring, performance assessment, quality control, risk assessment, maintenance and management of bridges etc., promised an interesting interaction of ideas for further progress.

Having all this on mind, together with Niels P. Høy - a vice chair of the IABSE WC, I suggested to COST Actions' chairs, Sebastian Thöns and José Matos to held the Joint workshop in Zagreb with the Local support of Chair for Bridges. I thank them all for supporting the idea and participating actively in organising this event from abroad. It was challenging but very interesting, and we have learned a lot form this. I would also like to thank our keynote lecturers for sharing their knowledge on assessment, monitoring and life cycle management of bridges from outside the Europe. And finally, I would like to thank all participants of our joint workshop, weather those that present their research or practical contributions or those who actively participated in the discussions and dynamism of this event. Interesting and important work that they have presented made this workshop very successful.

I would like to believe that we triggered the future collaborations and joint activities among different areas of expertise. The special issue of SEI: The Value of Health Monitoring in Structural Performance Assessment, planned for 2018 is our following step.

Ana Mandić Ivanković

Ana Mandić Iv.

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2 KEYNOTE CONTRIBUTIONS

2.1 Risk- and Sustainability-Informed Decision Making for Structures in a Life-Cycle Context (abstract)

Dan M. Frangopol

2.2 Bayesian uncertainty quantification and sparse Bayesian learning for model updating in SHM (paper / presentation)

James L. Beck

2.3 What We Have Learned from Operational Monitoring and Serviceability Assessment of Long-Span Bridges (paper / presentation)

Ho-Kyung Kim

2.4 New Technologies for Condition Assessment of Existing Structures (abstract / presentation)

Hitoshi Furuta

Risk- and Sustainability-Informed Decision Making for Structures in a Life-Cycle Context

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DOI: https://doi.org/10.5592/CO/BSHM2017.2.1

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Abstract. Risk-based performance metrics allow engineers to combine the probability of structural failure with the consequences corresponding to this event. A sustainability performance metric is established considering the risks associated with economic, social, and environmental impacts, utility theory, and the decision maker's risk attitude.

Keywords: risk; sustainability; bridges; multi-hazards, life-cycle; optimization; decision making

1 Introduction

The condition of civil infrastructure systems around the world is degrading due to a variety of deteriorating mechanisms including aging, environmental stressors, man-made hazards (e.g., blasts and fires) and natural hazards (e.g., earthquakes and hurricanes), among others. Consequently, improving the overall condition and safety of deteriorating civil infrastructure systems is a key concern worldwide. For example, in 2017, the American Society of Civil Engineers (ASCE) reported, within the Report Card for America's Infrastructure, that the average age of the United States' 614,387 bridges was 43 years. Additionally, nearly a quarter of these highway bridges were classified as either structurally deficient or functionally obsolete (ASCE 2017). These staggering statistics highlight the dire need to implement rational mitigation strategies that maintain structural performance within acceptable levels through the life-cycle of deteriorating civil infrastructure. In order to ensure adequate life-cycle performance, it is crucial to implement optimal management strategies that maintain performance of infrastructure systems within acceptable levels through their life-cycle (Okasha & Frangopol 2009, Dong et al. 2015). Life-cycle management is widely recognized as an effective tool for maximizing the cost-effectiveness of implementing intervention actions that improve condition and safety, and extend the service life of deteriorating infrastructure systems (Ang 2011, Barone & Frangopol 2014a, Frangopol & Liu 2007).

2 Risk and Sustainability in a Life-Cycle Context

Life-cycle assessment of deteriorating infrastructure systems includes aleatory and epistemic uncertainties associated with natural randomness and inaccuracies in the prediction or estimation of reality, respectively (Ang & De Leon 2005, Ang & Tang 2007). Because of these uncertainties, it is imperative for structural engineers to accurately model and assess the structural performance and expected total cost within a probabilistic life-cycle context (Frangopol 2011).Furthermore, the effects of maintenance, repair, and rehabilitation on structural life-cycle performance must be quantified (Barone &Frangopol 2014b, Frangopol & Soliman 2016, Frangopol et al. 2004, Sánchez-Silva et al. 2016, Zhu & Frangopol 2012, 2013). Approaches for the life-cycle management of infrastructure systems involving reliability performance indicators consider uncertainties associated with loads and resistance, but are not able to account for the consequences incurred from structural failure. Risk-based indicators provide the means to combine the probability of structural failure with the consequences associated with this event (Ang & Tang 1984, Barone & Frangopol 2014b, Decò & Frangopol 2011, Sabatino et al. 2015, 2016). Furthermore, methodologies considering sustainability as a performance indicator are becoming relevant within the field of life-cycle engineering. The incorporation of sustainability in the life-cycle performance assessment and management procedures allows for the effective integration of economic, social, and environmental aspects (Bocchini et al. 2014). A sustainability performance metric may be established considering multi-attribute utility theory, which facilitates the combination of several risks while incorporating the risk attitude of the decision maker. Utility theory is employed herein to incorporate the influence of the decision maker's risk attitude on the relative desirability of lifetime management plans (Keeney & Raiffa 1993).

In general, utility is defined as a measure of desirability to the decision maker. Utility theory is a powerful tool used to conduct rational multi-criteria decision making analyses considering uncertain information.

3 Utility-based decision making framework

The schematic diagram in Figure 1 shows the utility-based decision making framework proposed by the authors. The five interconnected modules shown in Figure 1 perform specific tasks. They were applied to bridges and naval vessels. The first step of the framework involves identifying relevant structural parameters (i.e., module 1) and which stressors affect the investigated system. Some critical parameters include the structural geometry, material behavior, loading due to multiple hazards, information from SHM, and risk attitude of the decision maker. Next, the structural performance is assessed and predicted throughout the life-cycle of the investigated system (i.e., module 2). Within this step, time-variant structural performance is evaluated to determine the current and predict the future condition of various components or the entire structure. Structural performance is assessed in terms of risk and sustainability. After determining the time-variant structural performance of the system, the utility assessment of each attribute investigated within the decision making problem is conducted (i.e., module 3). Utility theory is utilized in order to depict the relative desirability of maintenance strategies to the decision maker. Attributes that are mapped to utility within this study include risk, cost, benefit, and availability. Multi-attribute utility theory is employed in order to effectively capture the sustainability performance of highway bridges and impact of the decision maker's risk attitude.

Once all investigated attributes are mapped to utility, the objective functions may be formulated, and a multi-criteria optimization is carried out (i.e., module 4) with the final goal of determining optimal life-cycle management plans (i.e., when to intervene and which performance measure(s) should be implemented). In general, optimal lifetime intervention plans are obtained by carrying out a multi-criteria optimization procedure

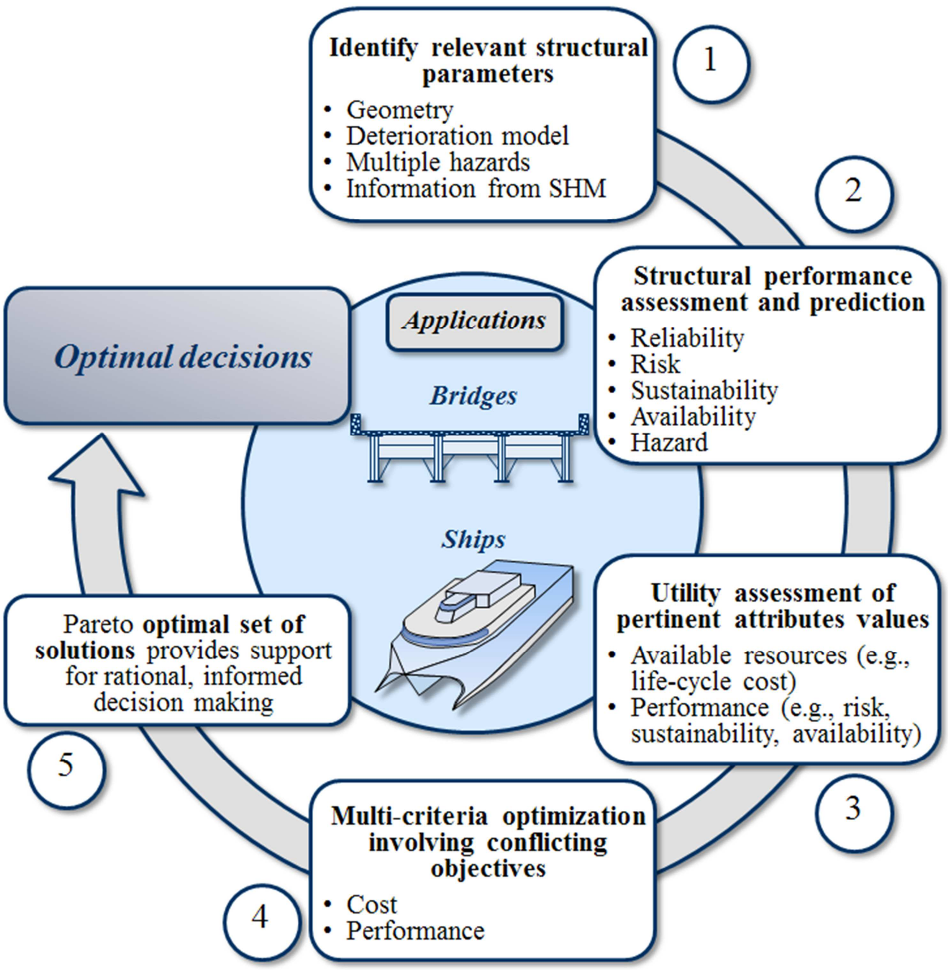

Fig 1. Risk- and sustainability-informed life-cycle decision making framework based on utility.

where the utility associated with total intervention cost and utility corresponding to performance are considered as conflicting objectives. Within this optimization procedure, the utilities associated with total life-cycle cost and performance are simultaneously maximized in order to achieve optimal lifetime management plans. The output of the optimization process, facilitated with genetic algorithms (GAs), is a Pareto optimal set of solutions which provides rational, informed intervention schedule choices to the decision maker that balances both cost and performance (i.e., module 5). Ultimately, based upon the risk attitude, preferences, and budgetary constraints of the decision maker, he/she may choose, amongst a group of trade-off solutions, an optimal intervention schedule for an investigated structural system. Overall, the proposed methodology can be used in assisting decision making regarding intervention actions that improve the performance of structural systems in a life-cycle context. Applications of the proposed utility-based decision framework to bridges can be found in Dong et al. 2015, Frangopol et al. 2017, and Sabatino et al. 2015, 2016. Also a decision making framework for optimal structural health monitoring planning of ship structures considering availability and utility has been recently proposed by Sabatino and Frangopol (2017).

4 Conclusion

All three aspects of sustainability (i.e., economic, social, and environmental) are crucial to the life-cycle performance assessment of civil engineering. Multi-attribute utility theory allows for the quantification of the sustainability performance of civil infrastructure systems and for risk-informed decisions.

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Acknowledgements

The authors are grateful for the financial support received from the U.S. National Science Foundation Grant CMMI 1537926 and the U.S. Office of Naval Research (ONR) Awards N00014-08-1-0188, N00014-12-1-0023, and N00014-16-1-2299. The opinions and conclusions presented in this paper are those of the authors and do not necessarily reflect the views of the sponsoring organizations.

Bayesian uncertainty quantification and sparse Bayesian learning for model updating in structural health monitoring

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DOI: https://doi.org/10.5592/CO/BSHM2017.2.2

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Abstract. The application of interest in this paper is model updating based on vibration monitoring of an instrumented structure, especially to detect and quantify localized stiffness losses as a proxy for damage. Because of its ability to quantify modeling uncertainty, a Bayesian approach is used in which the relative plausibility of each model in a model class (based on parameterized set of structural models) is quantified by its posterior probability from Bayes' Theorem. In addition, the relative plausibility of each model class within a set of candidate model classes can also be assessed. Computation of this posterior probability from Bayes' Theorem over all candidate model classes automatically applies a quantitative Ockham's razor that trades off a data-fit measure with an information-theoretic measure of model complexity, which penalizes model classes that "over-fit" the data. We present our recent progress in exploring sparse Bayesian learning for structural health monitoring, in which we infer spatially-sparse substructural stiffness reductions in a way that is consistent with the Bayesian Ockham razor. Illustrative results validate the capability of the presented sparse Bayesian learning algorithms for structural health monitoring.

Keywords: Structural Health Monitoring, System Identification, Bayesian Updating, Bayesian model class selection, Probability Logic, Uncertainty Quantification, Bayesian Ockham Razor; Sparse Bayesian Learning, Hierarchical Bayesian Model.

1 Introduction

System identification is the key component in model-based inversions for detection and assessment of damage in structural health monitoring. It uses observed structural response data and prior knowledge to update mathematical models of the behavior of a system such as a bridge or building subject to dynamic excitation. In addition to structural health monitoring, the goals of such data-informed modeling might also include providing a better understanding of the structural system's behavior and allowing more accurate predictions of its future response to specified excitations.

One of the main difficulties is that it is impossible to exactly model the full behavior of a structure by using the limited sensor data and prior knowledge available. Since any model gives an approximation to the real system behavior, there are always modeling uncertainties involved; for example, what values of the model parameters are appropriate and how well does the model predict the real system response? Another difficulty is that for complex system models, single-point parameter estimation often gives non-unique results (e.g. multiple least-squares or maximum likelihood estimates). In order to make more robust predictions, one should track all plausible values of the parameters based on the data and also explicitly treat the uncertain prediction errors (the difference between the response of the real system and that of the system model), as well as possible measurement errors. These issues have motivated numerous researchers to tackle the problem of structural system identification from a Bayesian perspective (e.g. Beck, 2010; Green et al., 2015; Au & Zhang, 2016; Huang et al., 2017b).

In contrast to the point estimates of the parameters used in the conventional deterministic or frequentist probabilistic methods, the Bayesian probabilistic framework uses Bayes' Theorem to quantify the relative plausibility based on the data of all possible values of the model parameters via their posterior PDF (probability density function). This procedure is used to learn about all plausible models for representing the system's behavior where each parameter value specifies a possible model for the system. Since there is always uncertainty in which parameterized model class to choose to represent a system, one can also choose a set of candidate model classes and calculate their posterior probability based on the data by applying Bayes' Theorem at the model class level. An information-theoretic interpretation (Muto & Beck, 2008; Beck, 2010) shows that the posterior probability of each model class depends on the difference between a measure of the average data-fit of the model class and the amount of information extracted from the data by the model class, which penalizes model classes that "over-fit" the data. Comparing the posterior probability of each model class therefore provides a quantitative Ockham's razor (Gull, 1989; Jefferys & Berger, 1992; Mackay, 1992), that is, models should be no more complex than is sufficient to explain the data.

Sparse Bayesian learning (Tipping, 2001a) is a supervised learning framework that is very effective at implementing Ockham's Razor by achieving parsimonious (sparse) representations in the context of regression and classification. It was the basis for the introduction of the relevance vector machine (Tipping, 2000) and sparse principal component analysis (Tipping, 2001b). We give an overview of our recent progress of developing sparse Bayesian learning algorithms for system identification, and present illustrative examples to show the capability of these methods.

2 Bayesian system identification and the Bayesian Ockham Razor

Consider the problem of predicting the output $\mathbf{z}(t)$ to some input $\mathbf{u}(t)$ of a real dynamic system over some time interval, $t \in [0, t_f]$, by using a computational model of the system. We use $\mathbf{u}_n = \mathbf{u}(n\Delta t) \in \mathbb{R}^{N_I}$ and $\mathbf{z}_n = \mathbf{z}(n\Delta t) \in \mathbb{R}^{N_o}$ to denote the real system input and output, respectively, at discrete times $t_n = n\Delta t, n \in \mathbb{Z}^+$, and use $\mathbf{u}_{0:n} = [\mathbf{u}_0^T, \mathbf{u}_1^T, \ldots, \mathbf{u}_n^T]^T$ and $\mathbf{z}_{0:n} = [\mathbf{z}_0^T, \mathbf{z}_1^T, \ldots, \mathbf{z}_n^T]^T$ to denote the discrete-time histories of the system input and output up to time t_n.

2.1 Stochastic model class

In modeling the I/O (input and output) behavior of a real system, one cannot expect any chosen deterministic model to make perfect predictions and the prediction errors of any such model will be uncertain. This motivates the introduction of a *stochastic* (or *Bayesian*) *model class* $\mathcal{M}$ (Beck, 2010) that consists of a set of *stochastic I/O models* valid for any $n \in \mathbb{Z}^+$ $\{p(\mathbf{z}_{1:n}|\mathbf{u}_{0:n}, \mathbf{w}, \mathcal{M}): \mathbf{w} \in \mathbf{W} \subset \mathbb{R}^{N_p}\}$ (also called *stochastic forward models*) for a system, together with a chosen *prior probability distribution* $p(\mathbf{w}|\mathcal{M})$ over this set that quantifies the initial relative plausibility of each I/O probability model corresponding to each value of the parameter vector $\mathbf{w}$. Any deterministic I/O model of a system that involves uncertain parameters can be used to construct such a model class for the system by *stochastic embedding* (Beck, 2010) in which the *Principle of Maximum Information Entropy* plays an important role (Jaynes 1983; Jaynes 2003) (see (1) in the next sub-section).

<u>*Remark 2.1*</u>: Probability as a logic provides a rigorous foundation for the Bayesian approach. Probability in *probability logic* is interpreted as the degree of plausibility of a statement on the basis of the specified conditioning information (Cox, 1946,1961; Jaynes, 1957,2003; Beck, 2010). This allows the uncertainty in predictions to be quantified due to our incomplete information because of our limited capacity to collect or understand the relevant information. The probability logic axioms apply to incorporating not only parametric uncertainty (uncertainty about which model in a proposed set should be used to represent the structure's I/O behavior) but also non-parametric uncertainty due to the existence of prediction errors because of the approximate nature of any structural model. This is in contrast to the relative frequency interpretation of probability in Kolmogorov's axioms, which is restricted to "inherently random" physical variables.

2.2 Bayesian updating for a given model class

If sensor data $\boldsymbol{\mathcal{D}}_N = \{\hat{\mathbf{u}}_{0:N}, \hat{\mathbf{y}}_{1:N}\}$ are available where $\hat{\mathbf{y}}_{1:N}$ and $\hat{\mathbf{u}}_{0:N}$ are the measured time histories of the system output and the corresponding measured system input (if available), respectively, sampled at time interval Δt, then a model can be developed to *predict* the *measured* system output $\mathbf{y}_n$ at each time t_n by using:

$$\mathbf{y}_n = \mathbf{z}_n + \mathbf{m}_n = \mathbf{q}_n(\hat{\mathbf{u}}_{0:n}, \mathbf{w}) + \mathbf{e}_n + \mathbf{m}_n \tag{1}$$

where $\mathbf{m}_n$ and $\mathbf{e}_n$ denote the measurement noise and output prediction error at time t_n and the system output equation $\mathbf{z}_n = \mathbf{q}_n(\hat{\mathbf{u}}_{0:n}, \mathbf{w}) + \mathbf{e}_n$ is used where $\mathbf{q}_n$ is the corresponding output of a parameterized deterministic model that can be based on theoretical principles (e.g., a FEM model). A probability model can be chosen for the I/O behavior by selecting a PDF for $\mathbf{e}_{1:n}$ that maximizes Shannon's entropy (a measure of uncertainty) subject to some prior constraints. This procedure is called *stochastic embedding* of the parameterized deterministic model in Beck (2010). A probability model can also be chosen for the measurement error $\{\mathbf{m}_n\}$ based on a separate study of the sensors, where $\{\mathbf{m}_n\}$ is taken independent of the prediction errors $\{\mathbf{e}_n\}$. This leads to a probability model $p(\mathbf{y}_{1:N}|\hat{\mathbf{u}}_{0:N}, \mathbf{w}, \mathcal{M})$ for predicting the sensor output $\mathbf{y}_{1:N}$. In many applications, $\mathbf{m}_n$ is negligible compared with $\mathbf{e}_n$ and so it can be dropped, that is, the difference between the measured system output $\mathbf{y}_n$ and the actual output $\mathbf{z}_n$ is ignored but not the difference between the real system and model outputs, $\mathbf{z}_n$ and $\mathbf{q}_n$.

The data $\boldsymbol{\mathcal{D}}_N$ can be used to update the relative plausibility of each stochastic I/O model $p(\mathbf{y}_{1:n}|\hat{\mathbf{u}}_{0:N},\mathbf{w},\mathcal{M})$, $\mathbf{w}\in\mathbf{W}\subset\mathbb{R}^{N_p}$, defined by the stochastic model class $\mathcal{M}$, by computing the *posterior* PDF $p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M})$ from *Bayes' Theorem*:

$$p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}) = p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M})p(\mathbf{w}|\mathcal{M})/p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}) = c^{-1}p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M})p(\mathbf{w}|\mathcal{M}) \quad (2)$$

where $c=p(\boldsymbol{\mathcal{D}}_N|\mathcal{M})$ is the normalizing constant, which is called the *evidence* or *marginal likelihood* for the model class $\mathcal{M}$ given by data $\boldsymbol{\mathcal{D}}_N$; $p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M})$, as a function of $\mathbf{w}$, is the *likelihood function* which expresses the probability of getting data $\boldsymbol{\mathcal{D}}_N$ based on the PDF $p(\mathbf{y}_{1:N}|\hat{\mathbf{u}}_{0:N},\mathbf{w},\mathcal{M})$ by substituting the measured output data $\hat{\mathbf{y}}_{1:N}$ for $\mathbf{y}_{1:N}$. Note that a model class can be used to perform both prior (initial) and posterior (updated using system sensor data) robust predictive analyses, which can be used during design and operation, respectively, of a structure, based purely on the probability logic axioms (Papadimitriou et al., 2001; Beck & Taflanidis, 2013).

2.3 Bayesian updating for multiple model classes

If $\mathbf{M}$ denotes the proposition that specifies a *discrete* set of candidate model classes $\{\mathcal{M}_m : m=1,2,\ldots,N_M\}$ that is being considered for a system, together with a prior probability distribution $p(\mathcal{M}_m|\mathbf{M})$ over this discrete set, then the posterior PDF $\mathrm{p}(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathbf{M})$ based on $\mathbf{M}$ is given by the Total Probability Theorem:

$$p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathbf{M}) = \textstyle\sum_{m=1}^{M} p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)P(\mathcal{M}_m|\boldsymbol{\mathcal{D}}_N,\mathbf{M}) \quad (3)$$

where the posterior PDF for each model class $\mathcal{M}_m$ in (3), which comes from (2), is weighted by the posterior probability $P(\mathcal{M}_m|\boldsymbol{\mathcal{D}}_N,\mathbf{M})$ computed from Bayes' Theorem at the model class level:

$$P(\mathcal{M}_m|\boldsymbol{\mathcal{D}}_N,\mathbf{M}) = p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m)P(\mathcal{M}_m|\mathbf{M})/p(\boldsymbol{\mathcal{D}}_N|\mathbf{M}) \quad (4)$$

Here, $p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m)$ is the *evidence* for $\mathcal{M}_m$ provided by the data $\boldsymbol{\mathcal{D}}_N$ (additional conditioning on $\mathbf{M}$ is irrelevant), which is given by the Total Probability Theorem:

$$p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m) = \int p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M}_m)p(\mathbf{w}|\mathcal{M}_m)d\mathbf{w} \quad (5)$$

A uniform prior probability distribution can be chosen for the candidate model classes, that is, $P(\mathcal{M}_m|\boldsymbol{M}) = 1/N_M$, if the model classes are considered equally plausible a priori.

The calculation of the posterior probability $P(\mathcal{M}_m|\boldsymbol{\mathcal{D}}_N,\mathbf{M})$ in (4) provides a procedure for *Bayesian model class selection* (or comparison, or assessment), where the computation of the multi-dimensional integral in (5) for the evidence function is vital. If there is no analytical solution for (5), Laplace's approximation method can be used when the model class is globally identifiable based on the available data $\boldsymbol{\mathcal{D}}_N$ (e.g. Beck & Yuen 2004, Beck 2010). When the chosen class of models is unidentifiable or locally identifiable based on the data $\boldsymbol{\mathcal{D}}_N$ so that there are multiple MLEs (*maximum likelihood estimates*) (Beck & Katafygiotis, 1998), only stochastic simulation methods are practical to calculate the model class evidence, such as the TMCMC method (Ching & Chen, 2007), the stationarity method in Cheung & Beck (2010) or the Approximate Bayesian Computation method (Chiachio et al., 2014; Vakilzadeh et al., 2017).

2.4 Bayesian Ockham Razor

Comparing the posterior probability of each candidate model class by (4) automatically implements an elegant and powerful version of Ockham's (Occam's) Razor, known as the *Bayesian Ockham Razor*. The essence of Ockham's Razor has long been advocated for data-based model identification, that is, a simpler model should be preferred over a more complex model if it leads to comparable agreement with the data. However, until recently, the approximate complexity measure for a model did not have a rigorous formulation. Two early attempts are AIC (Akaike, 1974) and BIC (Schwarz, 1978), which trade-off a data-fit measure with a measure of "complexity" proportional to the number of uncertain parameters N_p. Using these simplified criteria for model assessment requires caution, however, because their penalty term for model class complexity depends only on N_p and ignores the effect of the prior distribution.

A recent interesting information-theoretic interpretation (Muto & Beck, 2008; Beck, 2010) shows that the evidence $p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m)$ in (5) explicitly builds in a trade-off between a data-fit measure for the model class and an information-theoretic measure of its complexity that quantifies the amount of information that the model class extracts from the data $\boldsymbol{\mathcal{D}}_N$. This result is based on using (2) in the expression for the normalization of the posterior PDF:

$$\begin{aligned}\log[p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m)] &= \int \log[p(\boldsymbol{\mathcal{D}}_N|\mathcal{M}_m)]p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)d\mathbf{w} \\ &= \int \log[p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M}_m)p(\mathbf{w}|\mathcal{M}_m)/p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)]p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)d\mathbf{w} \\ &= \int \log[p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M}_m)]p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)\,d\mathbf{w} - \int \log[p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)/p(\mathbf{w}|\mathcal{M}_m)]p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)d\mathbf{w} \\ &= \mathbf{E}\left[\log\left(p(\boldsymbol{\mathcal{D}}_N|\mathbf{w},\mathcal{M}_m)\right)\right] - \mathbf{E}\left[\log[p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)/p(\mathbf{w}|\mathcal{M}_m)]\right]\end{aligned} \quad (6)$$

where the expectations $\mathbf{E}[\cdot]$ are taken with respect to the posterior $p(\mathbf{w}|\boldsymbol{\mathcal{D}}_N,\mathcal{M}_m)$. The first term is the posterior mean of the log likelihood function, which is a measure of the average data-fit of the model class $\mathcal{M}_m$, and the second term is the Kullback-Leibler information, or relative entropy of the posterior relative to the prior, which is a measure of the model complexity (the amount of information gain about $\mathcal{M}_m$ from the data $\boldsymbol{\mathcal{D}}_N$) and is always non-negative. This information-theoretic result was first given by Beck & Yuen (2004) for the case of globally identifiable models and then extended to the general case by Ching et al. (2005) where the model may be unidentifiable. The merit of (6) is that it shows rigorously, without introducing ad-hoc concepts, that the log evidence for $\mathcal{M}_m$ explicitly builds in a trade-off between the data-fit of the model class and its information-theoretic complexity. This is important in structural health monitoring applications, since too complex models often lead to over-fitting of the data and the subsequent response predictions may then be unreliable since they depend too much on the details of the specific data, e.g., measurement noise and environmental effects.

3 General formulation of sparse Bayesian learning

3.1 Input-output model specification

Given a set of I/O data $\boldsymbol{\mathcal{D}} = \{\hat{\mathbf{u}}, \hat{\mathbf{y}}\}$, suppose that the model prediction of the output is $\mathbf{y} = \mathbf{f}(\hat{\mathbf{u}}) + \mathbf{e} + \mathbf{m} \in \mathbb{R}^{N_o}$ involving a deterministic function $\mathbf{f}$ of the input vector $\hat{\mathbf{u}}$, along with uncertain prediction error $\mathbf{e}$ and measurement noise $\mathbf{m}$. Assume that the function $\mathbf{f}$ is chosen as a weighted sum of N_p basis functions $\{\boldsymbol{\Theta}_j(\hat{\mathbf{u}})\}_{j=1}^{N_p}$:

$$\mathbf{f}(\hat{\mathbf{u}}) = \sum_{j=1}^{N_p} w_j\,\boldsymbol{\Theta}_j(\hat{\mathbf{u}}) = \boldsymbol{\Theta}(\hat{\mathbf{u}})\mathbf{w} \quad (7)$$

where $\boldsymbol{\Theta}$ is an $N_o \times N_p$ matrix with the basis functions $\{\boldsymbol{\Theta}_j\}$ as columns. Analysis of this model is facilitated by the adjustable parameters (or weights) $\mathbf{w} \in \mathbb{R}^{N_p}$ appearing linearly. The objective here is to infer values of the parameters $\{w_j\}_{j=1}^{N_p}$ such that $\boldsymbol{\Theta}(\hat{\mathbf{u}})\mathbf{w}$ is a 'good' approximation of $\mathbf{f}(\hat{\mathbf{u}})$ and the parameter vector $\mathbf{w}$ is sparse. There has been significant recent interest (e.g., Tropp, 2004; Hastie et al, 2015) in the notion of sparse learning algorithms which promote significant numbers of the parameter components w_n to be zero as a means of providing model regularization during inverse problems. These methods have been applied for compressive sensing (Candès, 2006; Donoho, 2006; Huang et al., 2011; Huang et al., 2014; Huang et al., 2016).

3.2 Sparse Bayesian learning model

Sparse Bayesian learning (SBL) encodes a preference for sparser parameter vectors by making a special choice for the prior distribution for the parameter vector $\mathbf{w}$ that is known as the *automatic relevance determination* (ARD) prior (Mackay, 1992; Tipping, 2001a):

$$p(\mathbf{w}|\boldsymbol{\alpha}) = \prod_{j=1}^{N_p} p(w_j|\alpha_j) = \prod_{j=1}^{N_p} \mathcal{N}(w_j|0,\alpha_j^{-1}) = \prod_{j=1}^{N_p}\left[(2\pi)^{-1/2}\alpha_j^{1/2}\exp\left\{-\frac{1}{2}\alpha_j w_j^2\right\}\right] \quad (8)$$

where the hyperparameter α_j is the prior precision (inverse variance) for w_j. An individual hyperparameter α_j is associated independently with each weight w_j, thereby moderating the strength of the Gaussian prior. Note that an infinite value of α_j implies that the corresponding coefficient w_j has an insignificant prior contribution to the modeling of the measurements $\mathbf{y}$, because it produces essentially a Dirac delta-function at zero for the prior, and so the posterior.

By using *the principle of maximum information entropy* (Jaynes, 1983) and incorporating the first two moments as constraints, the combination of the prediction error and measurement noise $\mathbf{e}$ is modeled as a zero-mean Gaussian vector with covariance matrix $\beta^{-1}\mathbf{I}_{N_o}$, which gives a Gaussian predictive PDF:

$$p(\mathbf{y}|\mathbf{w},\beta) = (2\pi\beta^{-1})^{-\frac{N_o}{2}}\exp\left(-\frac{\beta}{2}\|\mathbf{y} - \boldsymbol{\Theta}(\hat{\mathbf{u}})\mathbf{w}\|_2^2\right) = \prod_{j=1}^{N_p}\mathcal{N}(\mathbf{y}|\boldsymbol{\Theta}(\hat{\mathbf{u}})\mathbf{w},\beta^{-1}\mathbf{I}_{N_o}) \quad (9)$$

By substituting the data $\hat{\mathbf{y}}$ for $\mathbf{y}$, (9) gives a Gaussian likelihood function that measures how well the model for specified parameters $\mathbf{w}$ and β predicts the measurements $\hat{\mathbf{y}}$. A stochastic model class $\mathcal{M}(\boldsymbol{\alpha},\beta)$ is then defined by the I/O predictive model in (9) and the prior PDF on $\mathbf{w}$ given by (8).

3.3 Bayesian updating for given model class $\mathcal{M}(\boldsymbol{\alpha}, \boldsymbol{\beta})$

The posterior distribution $p(\mathbf{w}|\hat{\mathbf{y}}, \boldsymbol{\alpha}, \beta)$ over the weight parameters given by model class $\mathcal{M}(\boldsymbol{\alpha}, \beta)$ is computed based on Bayes' theorem:

$$p(\mathbf{w}|\hat{\mathbf{y}}, \boldsymbol{\alpha}, \beta) = p(\hat{\mathbf{y}}|\mathbf{w}, \beta)p(\mathbf{w}|\boldsymbol{\alpha})/p(\hat{\mathbf{y}}|\boldsymbol{\alpha}, \beta) \tag{10}$$

where $p(\hat{\mathbf{y}}|\boldsymbol{\alpha}, \beta) = \int p(\hat{\mathbf{y}}|\mathbf{w}, \beta)p(\mathbf{w}|\boldsymbol{\alpha})d\mathbf{w}$ is the *evidence* of the model class $\mathcal{M}(\boldsymbol{\alpha}, \beta)$. Since both the prior and likelihood for $\mathbf{w}$ are Gaussian and the likelihood mean $\boldsymbol{\Theta}(\hat{\mathbf{u}})\mathbf{w}$ is linear in $\mathbf{w}$, the posterior PDF can be expressed analytically as a multivariate Gaussian distribution:

$$p(\mathbf{w}|\hat{\mathbf{y}}, \boldsymbol{\alpha}, \beta) = \mathcal{N}(\mathbf{w}|(\boldsymbol{\Theta}^T\boldsymbol{\Theta} + \beta^{-1}\mathbf{A})^{-1}\boldsymbol{\Theta}^T\hat{\mathbf{y}}, (\beta\boldsymbol{\Theta}^T\boldsymbol{\Theta} + \mathbf{A})^{-1}) \tag{11}$$

where $\mathbf{A} = \mathrm{diag}\left(\alpha_j, \dots, \alpha_{N_p}\right)$.

3.4 Hyperparameter learning by evidence maximization

A continuous set of candidate model classes $\mathcal{M}(\boldsymbol{\alpha}, \beta)$ is defined in Subsection 3.2**,** and the robust posterior PDF $p(\mathbf{w}|\hat{\mathbf{y}})$ can be computed by integrating out the posterior uncertainty in $\boldsymbol{\alpha}$ and β as below. We assume that at the posterior $p(\boldsymbol{\alpha}, \beta|\hat{\mathbf{y}})$ is highly peaked at $\{\tilde{\boldsymbol{\alpha}}, \tilde{\beta}\}$ (the MAP (maximum a posteriori) value of $\{\boldsymbol{\alpha}, \beta\}$). We then treat $\{\boldsymbol{\alpha}, \beta\}$ as a 'nuisance' parameter vector and integrate it out by applying Laplace's asymptotic approximation (Beck & Katafygiotis, 1998):

$$p(\mathbf{w}|\hat{\mathbf{y}}) = \int p(\mathbf{w}|\hat{\mathbf{y}}, \boldsymbol{\alpha}, \beta)p(\boldsymbol{\alpha}, \beta|\hat{\mathbf{y}})d\boldsymbol{\alpha}d\beta \approx p\left(\mathbf{w}|\hat{\mathbf{y}}, \tilde{\boldsymbol{\alpha}}, \tilde{\beta}\right). \tag{12}$$

where: $$\left\{\tilde{\boldsymbol{\alpha}}, \tilde{\beta}\right\} = \arg\max_{[\boldsymbol{\alpha},\beta]} p(\boldsymbol{\alpha}, \beta|\hat{\mathbf{y}}) = \arg\max_{[\boldsymbol{\alpha},\beta]}\{p(\hat{\mathbf{y}}|\boldsymbol{\alpha}, \beta)p(\boldsymbol{\alpha})p(\beta)\} \tag{13}$$

If we assign flat, non-informative prior PDFs for $\boldsymbol{\alpha}$ and β, we equivalently just need to maximize the evidence function $p(\hat{\mathbf{y}}|\boldsymbol{\alpha}, \beta)$. The optimization of $\{\boldsymbol{\alpha}, \beta\}$ is the procedure of *Bayesian model class selection* (Beck & Yuen, 2004) from a continuous set of model classes $\mathcal{M}(\boldsymbol{\alpha}, \beta)$. For larger amounts of data (larger N_o), accurate predictions are expected that are typically highly sparse because the maximization in (13) causes many hyperparameters α_j to approach infinity during the learning process. This is the Bayesian Ockham razor (Gull, 1988; Jefferys & Berger, 1992; Mackay, 1992) at work: the maximization of the evidence function $p(\hat{\mathbf{y}}|\boldsymbol{\alpha}, \beta)$ automatically involves a trade-off between the average data-fit of the model class $\mathcal{M}(\boldsymbol{\alpha}, \beta)$ and model sparseness (more sparseness corresponds to less model complexity), as we discussed in Section 2.4.

3.5 Robust predictions

Having found the MAP estimates $\{\tilde{\boldsymbol{\alpha}}, \tilde{\beta}\}$, our approximation to the robust predictive distribution of the system response $\mathbf{y}$ for a given input $\hat{\mathbf{u}}$ would be:

$$\begin{aligned} p(\mathbf{y}|\hat{\mathbf{y}}) = \int p(\mathbf{y}, \mathbf{w}, \boldsymbol{\alpha}, \beta|\hat{\mathbf{y}})\, d\mathbf{w}d\boldsymbol{\alpha}d\beta &= \int p(\mathbf{y}|\mathbf{w}, \boldsymbol{\alpha}, \beta)\, p(\mathbf{w}|\hat{\mathbf{y}}, \boldsymbol{\alpha}, \beta)p(\boldsymbol{\alpha}, \beta|\hat{\mathbf{y}})d\mathbf{w}d\boldsymbol{\alpha}d\beta \\ &\approx \int p\left(\mathbf{y}|\mathbf{w}, \tilde{\boldsymbol{\alpha}}, \tilde{\beta}\right) p\left(\mathbf{w}|\hat{\mathbf{y}}, \tilde{\boldsymbol{\alpha}}, \tilde{\beta}\right)d\mathbf{w} \end{aligned} \tag{14}$$

This robust predictive PDF takes into account all posterior plausible values of the model parameter vector $\mathbf{w}$.

Remark 3.1: A *hierarchical Bayesian model* (Gelman et al., 2013) is involved if we define hyper-priors over the prior precision parameter vector $\boldsymbol{\alpha}$ and prediction error precision parameter β**.** It is typical to assign gamma distributions over $\boldsymbol{\alpha}$ and β (Tipping. 2001a); however, the inverse gamma hyper-prior over $\boldsymbol{\alpha}$ produces a more sparse solution (Babacan, 2010).

Remark 3.2: The learning of the prior precision parameter $\boldsymbol{\alpha}$ is vital to reduce the posterior uncertainties by generating sparse models of $\mathbf{w}$, which leads to higher confidence in the predictions. The treatment of the prediction error precision β also affects the algorithm performance significantly, especially when the original model is only approximately sparse (Huang et al., 2016), which is common for structural health monitoring signals.

Remark 3.3: We have found the SBL algorithm suffers from a robustness problem: there are local maxima for (13) that may trap the hyperparameter optimization if the number of measurements N_o is much smaller than the number of model parameters N_p, leading to non-robust Bayesian updating results (Huang et al., 2014). Several robustness enhancement algorithms (Huang et al., 2014; Huang et al., 2016) have been developed by employing different strategies, with the goal of increasing signal reconstruction accuracy in compressive sensing for structural health monitoring signals.

4 Recent progress in applying sparse Bayesian learning to system identification in structural health monitoring

4.1 Hierarchical Bayesian model class

Suppose that we have a vector of identified natural frequencies $\hat{\boldsymbol{\omega}}^2 \in \mathbb{R}^{N_s N_m \times 1}$ (N_s and N_m are the number of modal identifications performed and number of extracted modes for each identification) and mode shapes $\hat{\boldsymbol{\psi}} \in \mathbb{R}^{N_s N_m N_o \times 1}$($N_o$ is the number of measured degrees of freedom). Since the measured degrees of freedom (DOFs) are usually a smaller subset of the DOFs of an appropriate structural model, we introduce the system natural frequencies $\boldsymbol{\omega}^2 \in \mathbb{R}^{N_m \times 1}$ and system mode shapes $\boldsymbol{\phi} \in \mathbb{R}^{N_d N_m \times 1}$ (N_d is number of DOFs of the structural model) to represent the actual underlying modal parameters of the assumed linear dynamics of the structural system at all DOFs corresponding to those of the structural model.

We choose a set of parameterized linear structural models with classical damping to produce normal modes of vibration where each model has the same known mass matrix $\mathbf{M} \in \mathbb{R}^{N_d \times N_d}$ inferred from structural drawings. Taking an appropriate substructuring (perhaps focusing on likely damage locations), we decompose the uncertain stiffness matrix $\mathbf{K} \in \mathbb{R}^{N_d \times N_d}$ as a linear combination of $(N_\theta + 1)$ substructure stiffness matrices $\mathbf{K}_{j,}\ j = 0, 1, \ldots N_\theta$:

$$\mathbf{K}(\boldsymbol{\theta}) = \mathbf{K}_0 + \sum_{j=1}^{N_\theta} \theta_j \mathbf{K}_j \tag{15}$$

where $\mathbf{K}_j \in \mathbb{R}^{N_d \times N_d}, j = 1, \ldots, N_\theta$, is the prior choice of the j^{th} substructure stiffness matrix and the corresponding stiffness scaling parameter θ_j is a factor that allows modification of the nominal j^{th}substructure stiffness so it is more consistent with the real structure behavior. The stiffness matrices $\mathbf{K}_j$ could come from a finite-element model of the structure, then it would be appropriate to choose all $\theta_j = 1$ to give the most probable value a priori for the parameter vector $\boldsymbol{\theta} \in \mathbb{R}^{N_\theta}$. For damage detection purposes, we will exploit the fact that the onset of stiffness reductions is typically in a small number of locations in the absence of structural collapse, and so the potential change in $\boldsymbol{\theta}$ compared with that of a reference calibration stage is expected to be a *sparse vector* with relatively few non-zero components.

The following joint prior PDF for system parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\phi}$ and stiffness scaling parameters $\boldsymbol{\theta}$ is chosen (Huang & Beck, 2015a):

$$p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\beta) \propto (2\pi/\beta)^{-N_m N_d/2} \exp\left\{-\frac{\beta}{2}\sum_{m=1}^{N_m}\left\|\left(\mathbf{K}(\boldsymbol{\theta}) - \omega_m^2\, \mathbf{M}\right)\boldsymbol{\phi}_m\right\|^2\right\} \tag{16}$$

where the finite value of the equation-error precision parameter β in (16) provides a soft constraint for the eigen-equation and it allows for the explicit control of how closely the system and model modal parameters agree. Note that we can decompose the joint prior PDF $p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\beta)$ into the product of a conditional PDF for any one of the parameter vectors and a marginal PDF for the other two parameter vectors. Although the modal parameters are a nonlinear function of the stiffness parameters, we employ a trick to produce a series of coupled linear−in−the−parameter problems.

We choose the unique MAP value $\hat{\boldsymbol{\theta}}_u$ from applying Bayesian updating using a large amount of time-domain vibration data from the calibration state as pseudo-data to define the likelihood function for $\boldsymbol{\theta}$ as:

$$p(\hat{\boldsymbol{\theta}}_u|\boldsymbol{\theta}, \boldsymbol{\alpha}) = \prod_{i=1}^{N_\theta} \mathcal{N}(\hat{\theta}_{u,i}|\theta_i, \alpha_i^{-1}) \tag{17}$$

Although the conventional strategy in SBL is to use an ARD Gaussian prior PDF (Tipping, 2001a) to model sparseness, here we incorporate the ARD concept in the likelihood function, along with the prior on $\boldsymbol{\theta}$ in (16). Gaussian likelihood functions $p(\hat{\boldsymbol{\omega}}^2|\boldsymbol{\omega}^2, \boldsymbol{\rho})$ and $p(\hat{\boldsymbol{\psi}}|\boldsymbol{\phi}, \boldsymbol{\eta})$ are also defined for system parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\phi}$ with precision parameters $\boldsymbol{\rho}$ and $\boldsymbol{\eta}$, respectively. In addition, we model our prior uncertainty in the equation error precision β by an exponential hyper-prior $p(\beta|b_0)$ with rate parameter b_0. The proposed modeling constitutes a multi-stage hierarchical model as shown in Figure 1. The bidirectional arrow in the graph of the hierarchical Bayesian model represents the information dependence between structural modal parameters $\boldsymbol{\omega}^2$and $\boldsymbol{\phi}$, which comes from the joint prior $p(\boldsymbol{\omega}^2, \boldsymbol{\phi}|\beta)$.

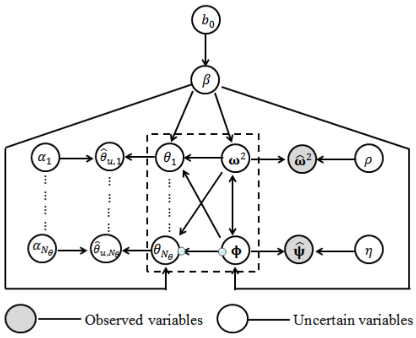


Fig 1. Acyclic graph representing the information flow in the hierarchical Bayesian model for offline SBL algorithms.

4.2 Fast sparse Bayesian learning algorithm

To facilitate the goal of presenting a fast algorithm to perform SBL, we focus on an analytical derivation of the posterior PDF of the stiffness scaling parameter $\boldsymbol{\theta}$ and collect all uncertain parameters except $\boldsymbol{\theta}$ in the vector $\boldsymbol{\delta} = [(\boldsymbol{\omega}^2)^T, \boldsymbol{\rho}^T, \boldsymbol{\phi}^T, \boldsymbol{\eta}, \boldsymbol{\alpha}^T, \beta, b_0]^T$ as 'nuisance' parameters, which are treated by using Laplace's approximation method (their posterior uncertainties are effectively ignored). The stochastic model class $\mathcal{M}(\boldsymbol{\delta})$ for the structural model is defined by the likelihood functions $p(\hat{\boldsymbol{\theta}}_u|\boldsymbol{\theta}, \boldsymbol{\alpha})$, $p(\hat{\boldsymbol{\omega}}^2|\boldsymbol{\omega}^2, \boldsymbol{\rho})$ and $p(\hat{\boldsymbol{\psi}}|\boldsymbol{\phi}, \boldsymbol{\eta})$ and the joint priors given by the product of $p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\beta)$ and $p(\beta|b_0)$. Based on this defined stochastic model class $\mathcal{M}(\boldsymbol{\delta})$, one can use the available modal data $\hat{\boldsymbol{\omega}}^2$ and $\hat{\boldsymbol{\psi}}$ and *pseudo-data* $\hat{\boldsymbol{\theta}}_u$ to update the structural model parameters $\boldsymbol{\theta}$ for system identification purposes. We assume that the posterior $p(\boldsymbol{\delta}|\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u)$ is highly peaked at $\tilde{\boldsymbol{\delta}}$ (the MAP value of $\boldsymbol{\delta}$). We then use Laplace's asymptotic approximation (Beck & Katafygiotis, 1998):

$$p(\boldsymbol{\theta}|\,\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u) = \int p(\boldsymbol{\theta}|\boldsymbol{\delta}, \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u) p(\boldsymbol{\delta}|\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u)\, d\boldsymbol{\delta} \approx p(\boldsymbol{\theta}|\,\tilde{\boldsymbol{\delta}}, \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u) \tag{18}$$

where $p(\boldsymbol{\theta}|\boldsymbol{\delta}, \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u)$ is the posterior PDF for a given model class $\mathcal{M}(\boldsymbol{\delta})$, $\tilde{\boldsymbol{\delta}} = \arg\max p(\boldsymbol{\delta}|\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u)$, and

$$p(\boldsymbol{\delta}|\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u) \propto p(\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u|\boldsymbol{\delta}) p(\boldsymbol{\delta})$$

$$= \int p(\hat{\boldsymbol{\theta}}_u|\boldsymbol{\theta}, \boldsymbol{\alpha})\, p(\hat{\boldsymbol{\omega}}^2|\boldsymbol{\omega}^2, \boldsymbol{\rho}) p(\hat{\boldsymbol{\psi}}|\boldsymbol{\phi}, \eta) p(\boldsymbol{\theta}|\boldsymbol{\omega}^2, \boldsymbol{\phi}, \beta) p(\boldsymbol{\omega}^2, \boldsymbol{\phi}|\beta) p(\boldsymbol{\rho}|\boldsymbol{\tau}) p(\beta|b_0) d\boldsymbol{\theta} \tag{19}$$

where $p(\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u|\boldsymbol{\delta})$ is the evidence function for the model class $\mathcal{M}(\boldsymbol{\delta})$. The full posterior uncertainty in $\boldsymbol{\theta}$ is explicitly incorporated when finding the MAP estimates of all parameters in $\boldsymbol{\delta}$, although it is a nontrivial task. The full details of the fast SBL algorithm are given in Huang et al. (2017a).

Remark 4.1: The maximization of evidence in (19) is effectively implementing the Bayesian Ockham Razor by assigning lower probabilities to a structural model whose parameter vector $\boldsymbol{\theta}$ has too large or too small differences from $\hat{\boldsymbol{\theta}}_u$ identified from the calibration state (that is, the model extracts relatively more or less information, respectively, from the system modal parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\phi}$, and so from the "measured" modal data $\hat{\boldsymbol{\omega}}^2$ and $\hat{\boldsymbol{\psi}}$, which can be seen from the hierarchical model in Figure 1). This process suppresses the occurrence of false and missed alarms for stiffness reductions.

Remark 4.2: It was found that the trade-off stated in Section 2.4 is sensitive to the selection of the equation-error precision parameter β. This motivated us to develop a more sophisticated method, described in the next sub-section, to provide a fuller treatment of the posterior uncertainties, including marginalizing over the posterior uncertainty of β analytically to get a more robust solution.

Remark 4.3: In the fast SBL algorithm, the pseudo-data $\hat{\boldsymbol{\theta}}_u$ is used based on the assumption that it is a unique MAP estimate at the calibration state due to the large amount of time-domain vibration data and identified modal parameters that can be collected. In the next subsection, we relax this assumption by explicitly considering the posterior uncertainty of $\boldsymbol{\theta}_u$ at the calibration stage in case there is not sufficient data to get a posterior on $\boldsymbol{\theta}_u$ that is highly peaked at $\hat{\boldsymbol{\theta}}_u$.

4.3 Sparse Bayesian learning algorithm using Gibbs sampling

The goal of the algorithm presented here is to provide a fuller treatment of the posterior uncertainty by employing MCMC simulation methods, so that the Laplace approximations in the fast SBL algorithm that involve the system modal parameters $\{\boldsymbol{\omega}^2, \boldsymbol{\phi}\}$ and the equation-error precision parameter β can be avoided. Based on the hierarchical model presented in Figure 1, the posterior PDF $p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\theta}}_u)$ can be calculated by marginalizing over the parameters $\beta, \eta, \rho, \boldsymbol{\alpha}$ and b_0 in the full posterior PDF from Bayes' theorem as follows:

$$p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u) = p(\widehat{\boldsymbol{\omega}}^2|\boldsymbol{\omega}^2)p(\widehat{\boldsymbol{\psi}}|\boldsymbol{\phi})p(\widehat{\boldsymbol{\theta}}_u|\boldsymbol{\theta})p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta})/p(\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u)$$

$$= \int p(\widehat{\boldsymbol{\omega}}^2|\boldsymbol{\omega}^2, \rho)p(\widehat{\boldsymbol{\psi}}|\boldsymbol{\phi}, \eta)p(\widehat{\boldsymbol{\theta}}_u|\boldsymbol{\theta}, \boldsymbol{\alpha})p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta}|\beta)p(\beta|b_0)p(\rho, \eta, \boldsymbol{\alpha}, b_0)d\beta d\rho d\eta d\boldsymbol{\alpha} db_0/p(\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u) \quad (20)$$

The resulting expression is intractable because the high-dimensional normalizing integral $p(\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u)$ cannot be computed analytically. Instead, we implement Gibbs Sampling to draw posterior samples from $p(\boldsymbol{\phi}, \boldsymbol{\omega}^2, \boldsymbol{\theta}|\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u)$ by decomposing the whole model parameter vector into the three groups $\{\boldsymbol{\phi}, \boldsymbol{\omega}^2, \boldsymbol{\theta}\}$ and repeatedly sampling from one parameter group conditional on the other two groups and the available data. We can derive the generic form $p(\mathbf{w}_1|\hat{\mathbf{y}}, \mathbf{w}_2, \mathbf{w}_3) = \int p(\mathbf{w}_1|\hat{\mathbf{y}}, \mathbf{w}_2, \mathbf{w}_3, \beta)p(\beta|\hat{\mathbf{y}}, \mathbf{w}_2, \mathbf{w}_3)\, d\beta$ of the conditional posterior PDFs $p(\boldsymbol{\phi}|\hat{\mathbf{y}}, \boldsymbol{\omega}^2, \boldsymbol{\theta})$, $p(\boldsymbol{\omega}^2|\hat{\mathbf{y}}, \boldsymbol{\phi}, \boldsymbol{\theta})$ and $p(\boldsymbol{\theta}|\hat{\mathbf{y}}, \boldsymbol{\phi}, \boldsymbol{\omega}^2)$ by marginalizing over their corresponding nuisance parameters using Laplace approximations (similar to (18) and (19)). The reader is referred to Huang & Beck (2015b) and Huang et al. (2017b) for detailed information of the SBL algorithm using Gibbs Sampling, including the derivation of the generic form of the conditional posterior PDF $p(\mathbf{w}_1|\hat{\mathbf{y}}, \mathbf{w}_2, \mathbf{w}_3)$ and the pseudo-codes.

If the Markov chain created by the GS algorithms is ergodic, samples from the marginal posterior distributions $p(\boldsymbol{\theta}|\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u), p(\boldsymbol{\omega}^2|\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u)$ and $p(\boldsymbol{\phi}|\widehat{\boldsymbol{\omega}}^2, \widehat{\boldsymbol{\psi}}, \widehat{\boldsymbol{\theta}}_u)$ are readily obtained by simply examining the GS samples $\boldsymbol{\theta}^{(n)}$ $(\boldsymbol{\omega}^2)^{(n)}$ and $\boldsymbol{\phi}^{(n)}$, respectively, for larger iteration numbers beyond the burn-in period. Using samples from the marginal posterior PDF $p(\boldsymbol{\theta}_u|\widehat{\boldsymbol{\psi}}_u, \widehat{\boldsymbol{\omega}}_u^2)$ at the calibration stage, we are able to effectively take into account the uncertainty of $\boldsymbol{\theta}_\mathrm{u}$ during the monitoring stage by replacing the MAP value $\widehat{\boldsymbol{\theta}}_u$ with uncertain $\boldsymbol{\theta}_u$, and then drawing samples from the posterior PDF $p(\boldsymbol{\theta}|\widehat{\boldsymbol{\omega}}_d^2, \widehat{\boldsymbol{\psi}}_d, \widehat{\boldsymbol{\omega}}_u^2, \widehat{\boldsymbol{\psi}}_u)$ for the monitoring stage, which is conditional on modal data from both the monitoring and calibration stages.

Remark 4.4: The analytical derivation of the generic conditional posterior PDF $p(\mathbf{w}_1|\hat{\mathbf{y}}, \mathbf{w}_2, \mathbf{w}_3)$ is important for the effectiveness of this Gibbs Sampling algorithm, which leads to a very desirable feature that it is applicable to linear Bayesian model updating problems of arbitrarily high dimensions, in contrast with other MCMC algorithms.

Remark 4.5: In the Gibbs Sampling Algorithm, by marginalizing over β directly to remove it from the posterior distributions, we get the Student-t conditional PDFs that can be sampled in each step of the algorithm. The Student-t PDFs have heavier tails than the Gaussian PDFs sampled in Algorithm 1 and so the algorithm is more robust to noise and outliers.

Remark 4.6: For the updating of the stiffness scaling parameters $\boldsymbol{\theta}$ and system modal parameters $\boldsymbol{\omega}^2$ and $\boldsymbol{\phi}$, the corresponding model classes $\mathcal{M}(\boldsymbol{\gamma}, b_0), \mathcal{M}(\upsilon, b_0)$ and $\mathcal{M}(\tau, b_0)$ are investigated, as seen from the hierarchical Bayesian model in Figure 1. The application of Bayes' Theorem at the model class level automatically penalizes models of $\boldsymbol{\theta}$ ($\boldsymbol{\omega}^2$or $\boldsymbol{\phi}$) that "under-fit" or "over-fit" the associated data $\widehat{\boldsymbol{\theta}}_u$ ($\widehat{\boldsymbol{\omega}}^2$or $\widehat{\boldsymbol{\psi}}$), therefore obtaining reliable updating results for the three parameter vectors, which is the Bayesian Ockham Razor (Beck, 2010) at work.

4.4 Illustrative results of the sparse Bayesian learning algorithms

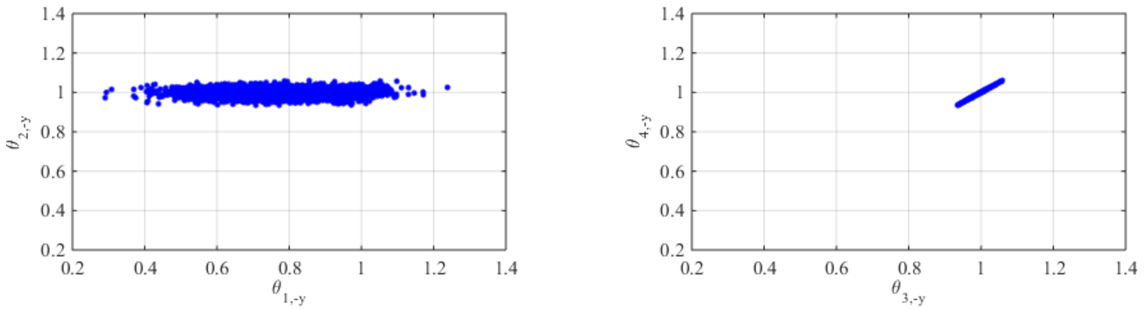
The proposed methodologies are applied to the brace damage patterns in the IASC-ASCE experimental Phase II benchmark problem (Dyke et al., 2003; Ching & Beck, 2003). The benchmark structure is a four-story, two-bay by two-bay steel braced-frame. Three damage configurations (Configs. 4,5,6) and one calibration (undamaged) configuration are investigated in this study. The stiffness scaling parameter vector $\boldsymbol{\theta}$ has 16 components, one for each of the four faces of each of the four stories. The true ratio values for $\theta_{1,-y}$ and $\theta_{4,-y}$ for Config. 4, and $\theta_{1,-y}$ for Config. 5, are 77.4% and the true ratio value $\theta_{1,-y}$ for Config. 6 is 54.9% of the values for the calibration configuration.

In Figure 2, all the samples generated from the Gibbs Sampling algorithm, excluding those in the burn-in period (4000 samples), are plotted in the $\{\theta_{1,-y}, \theta_{2,-y}\}$ and $\{\theta_{3,-y}, \theta_{4,-y}\}$ spaces for Config. 5. They show that the stiffness reduction corresponding to $\theta_{1,-y}$ is correctly identified and quantified as far as the sample means are concerned. Smaller posterior uncertainties can be observed in the stiffness scaling parameters for undamaged substructures, which is a benefit of the procedure of continuous model class selection by learning of the hyperparameters in the SBL formulation.

Figure 3 compares the probability that any stiffness parameter θ_j of a substructure has decreased by more than a prescribed fraction f estimated using the computed posterior PDFs (fast algorithm) or posterior samples (Gibbs Sampling algorithm). It is seen that the two algorithms generate similar results for Config.

4; however, for Config. 5 the posterior uncertainty of the stiffness parameters for the undamaged substructures are smaller for the Gibbs Sampling algorithm. For Config. 6, the occurrence of false damage detections is more unlikely for the Gibbs Sampling algorithm, presumably due to the robust treatment of the equation-error precision parameter β and stiffness parameter vector at the calibration state by a fuller model uncertainty quantification.

Remark 4.7: Much more computing resources are required for the Gibbs Sampling algorithm than the fast algorithm, which is a sacrifice for better posterior uncertainty quantification. Therefore, the choice between these two methods in real applications is a trade-off between the computation time and the level of uncertainty quantification and identification accuracy that the user is willing to accept.



a

b

Fig. 2. Post burn-in samples for some posterior stiffness parameters for the Config. 5 scenario, plotted in: a – $\{\theta_{1,-y}, \theta_{2,-y}\}$; b –$\{\theta_{3,-y}, \theta_{4,-y}\}$ spaces.

5 Concluding remarks

Probability logic combined with a Bayesian approach provides a rigorous framework to quantify modeling uncertainty in model updating in structural health monitoring. It allows plausible reasoning about structural behavior based on incomplete information. A key concept is a stochastic system model class which defines the fundamental probability models that allow robust stochastic structural analyses to be performed. Such a model class can be constructed by stochastic embedding of any deterministic model of the structure’s input-output behavior. One distinguishing aspect of the proposed Bayesian framework is marginalization of posterior PDFs, where instead of seeking to estimate all ‘nuisance’ parameters in the models, we attempt to integrate them out. This allows us to assess the relative plausibility of each model within a set of candidate model classes chosen to represent the uncertain structural behavior. Applying Bayes’ Theorem at the model class level automatically penalizes models that are too simple (“under-fit” the data) and too complex (“over-fit” the data), which is the Bayesian Ockham Razor at work. The quantitative implementation of Ockham's Razor is a natural consequence of applying Bayesian updating at the model class level.

Sparse Bayesian learning is an effective strategy to incorporate sparseness during model updating by automatically implementing Ockham’s Razor. This alleviates ill-conditioning and ill-posedness in the inverse problem in system identification. Recently developed sparse Bayesian learning algorithms for model updating and system identification have been briefly reviewed and illustrated using identified modal data. A promising performance of the algorithms has been shown by the illustrative results.

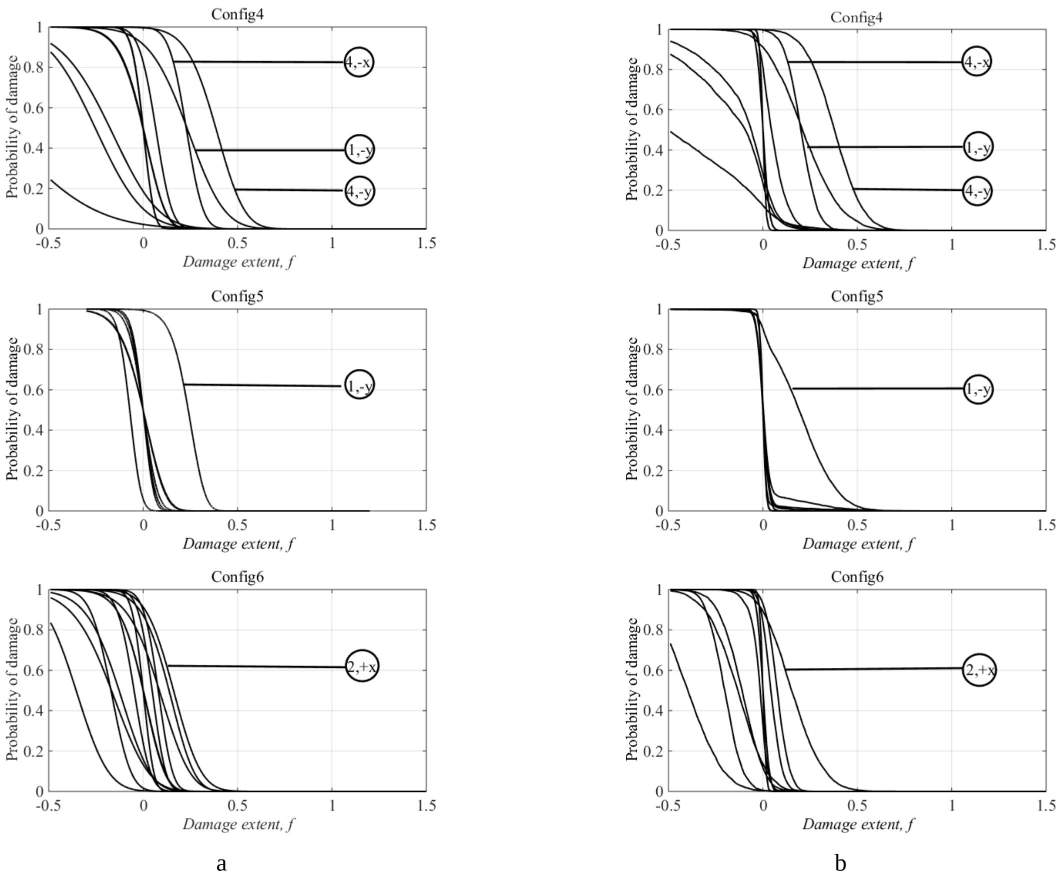


Fig. 3. Estimated damage probability curves for each substructure by running: a –Fast algorithm in Subsection 5.1.2; b – Gibbs Sampling algorithm in Subsection 4.3 using 4,000 post burn-in samples.

Acknowledgement

The second author was supported by the George W. Housner Earthquake Engineering Research fund at the California Institute of Technology and a grant from the National Natural Science Foundation of China (No. 51308161). This support is gratefully acknowledged.

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What we have learned from operational monitoring and serviceability assessment of long-span bridges

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DOI: https://doi.org/10.5592/CO/BSHM2017.2.3

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Abstract. More than 50 cable-supported bridges have been built in Korea since 1980. While maintaining those infrastructures for last 30 years, we have experienced unexpected problems from operational field monitoring of long-span bridges. This paper discusses on what we have observed, learned and how we responded to those events. As an example, a vortex-induced vibration was observed in 2011 in parallel-disposed twin cable-stayed bridges. Operational monitoring and wind tunnel tests successfully identified that the close distance between two bridges was one of the main cause of the vibration. However, an operational modal analysis revealed that unexpected low damping ratio of the bridge was also contributed to the realization of this serviceability problem in an operating bridge. Another observation of vibration in a long-span suspension bridge also attracted our attention on the potential low damping ratio, which may invoke serviceability issues in assessment of bridges. This paper introduces the details of the cause investigations, assessment and mitigation of the serviceability problems, validation through field tests and operational monitoring, and concluding suggestions to bridge owners and engineers.

Keywords: OMA, damping, serviceability, St-Id, structural health monitoring, cable-stayed bridge, suspension bridge, VIV

1 Introduction

The Namhae Suspension Bridge, built in 1973, was the starting point of cable-supported bridge history in Korea. Now we are operating 49 cable-supported bridges (43 cable-stayed bridges and 6 suspension bridges) and 23 more bridges (19 cable-stayed bridges and 4 suspension bridges) are in construction as of Sep., 2015. Fig. 1 shows the bridge data in terms of the length of main span and the year of opening to traffic, and Fig. 2 shows the distribution of the bridges over the Korean peninsula.

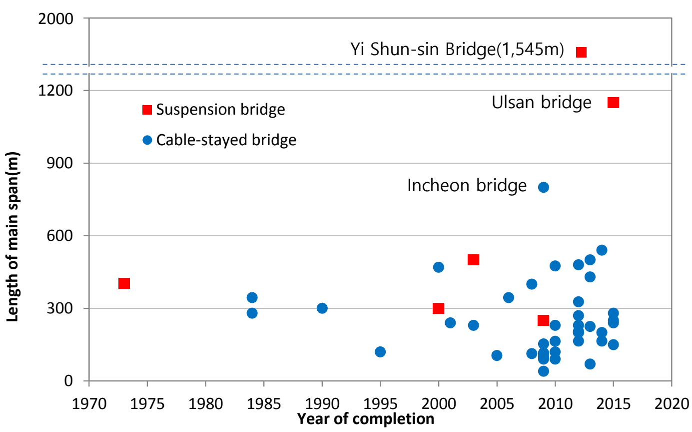

Fig. 1. Data of cable-supported bridges in Korea (*Courtesy of TM-ENC*)

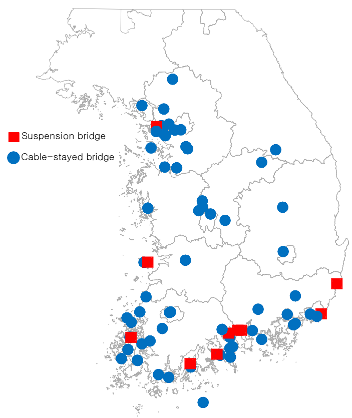

Fig. 2. Locations of cable-supported bridges (*Courtesy of TM-ENC*)

Most of the long-span bridges adopt built-in sensors for structural health monitoring (SHM) during operation. The data collected from the SHM system are accumulating in database of the bridge owners, as shown in Fig. 3. The SHM data of the bridges operated by the Ministry of Land, Infrastructure and Transport, are particularly collected through the Integrated Long-Span Bridge Management Center.

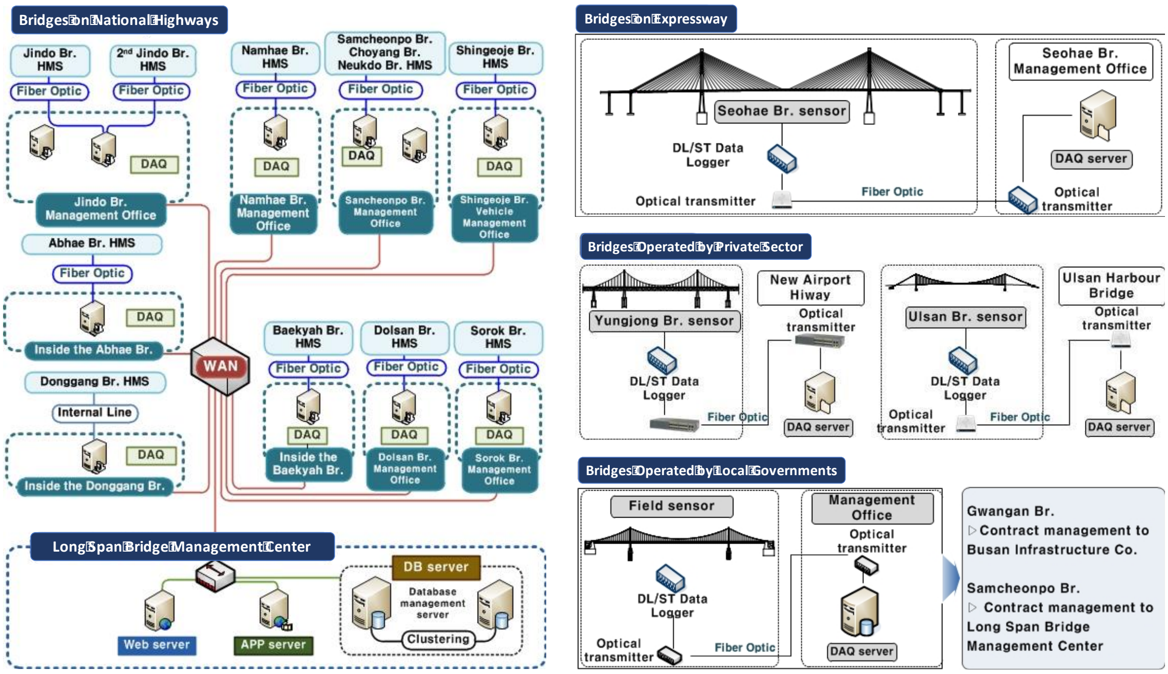

Fig. 3. Data accumulation from SHM system *(Courtesy of TM-ENC)*

Even though careful consideration has been applied for the best deployment of manageable number of sensors, it does not mean all the time that the best data accumulation and utilization are guaranteed from the viewpoint of more comfortable and safe operation of structures. The SHM has focused on the automatic damage detection techniques for a long time (Brownjohn and Carden, 2007; Magalhães et al., 2012; Peeters and De Roeck, 2001). Even though several successful achievements have been demonstrated, the damage detection also showed intrinsic limitation for large-scale infrastructures. Another value of SHM application should be a structural identification (St-Id) of flexible structures mainly targeting identification of dynamic properties. The natural frequencies and corresponding mode shapes are fundamental properties to be well estimated for the assessment of bridge performance not only in dynamic but even in static point of view. The identification of those two parameters are relatively straightforward from operational modal analysis (OMA) by utilizing data obtained from built-in monitoring sensors and/or densely applied temporal sensors for special purposes. Another dynamic parameter is modal damping ratio that is practically important in assessment of vibrational serviceability performance of flexible structures including long-span bridges. The modal damping ratios can be theoretically estimated by applying sweeping test with exciters for harmonic motion. Specially developed exciters were utilized for excitation of actual bridges (Brownjohn et al., 2003; Chen et al., 2016). A sudden release test of heavy weight was also utilized for the excitation of free-decaying motion of bridge structure (Magalhães et al., 2010). However, as the span length increases in recent long-span and super-long span bridges, the use of heavy exciters or sudden release tests are no more applicable from the point of view not only in structural safety but also in securing continuous traffic flows. Accordingly, OMA is becoming the only choice for St-Id nowadays.

This paper presents two events of vortex-induced vibration (VIV) observed in cable-supported bridges. The cause investigations of those unpredicted events reveal the importance of bridge aerodynamics for huge but also flexible structures. However, a more in-depth consideration of the cause of VIV shed light on St-Id of inherent modal damping ratios for more valuable utilization of SHM for the prognosis of potential vulnerability in serviceability performance of flexible structures. This paper demonstrates the role of damping ratios in observed VIV, the uncertainties in identified damping ratios, suggestions in data processing for more reliable St-Id, and finally concluding suggestions to bridge owners and operating engineers.

2 Case Study of VIV in Parallel Cable-Stayed Bridges

2.1 Observed VIV

The Jindo Bridge is composed of two parallel cable-stayed bridges. The first bridge (Bridge 1) was built in 1984 and the second bridge (Bridge 2) was opened to traffic to accommodate the increased volume of traffic in 2005. Two decks are in tandem with a net distance of 9.9m, as shown in Fig. 4. A noticeable VIV was observed for the deck of Bridge 2, located upstream. The mean wind velocity was maintained as 9.8-11.5 m/s for two hours. Since the amplitude of the observed acceleration exceeded the allowable level, as shown in Fig. 5, a series of wind tunnel tests were carried out to identify the main causes of vibration.

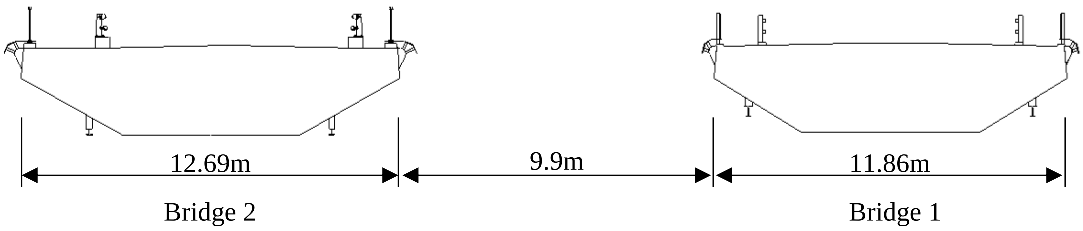

Fig. 4. Two decks of Jindo Bridge in tandem

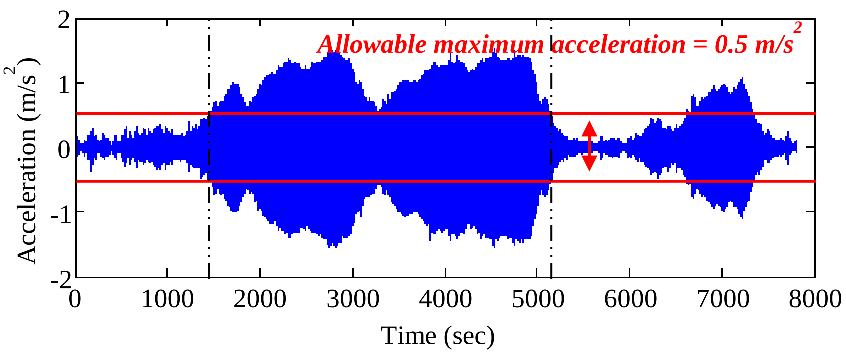

Fig. 5. Vertical acceleration of the deck in Bridge 2 at the center of main span

2.2 Cause Investigation in a Wind Tunnel

The purpose of wind tunnel tests was 1) to confirm the VIV in upstream deck being originated from parallel disposition of two decks, 2) to examine any potential aerodynamic measure including modification of external shape in upstream deck, 3) to identify the flow change in between two decks by applying a particle image velocimetry, and eventually 4) to identify the relationship between amplitude of VIV and damping ratio.

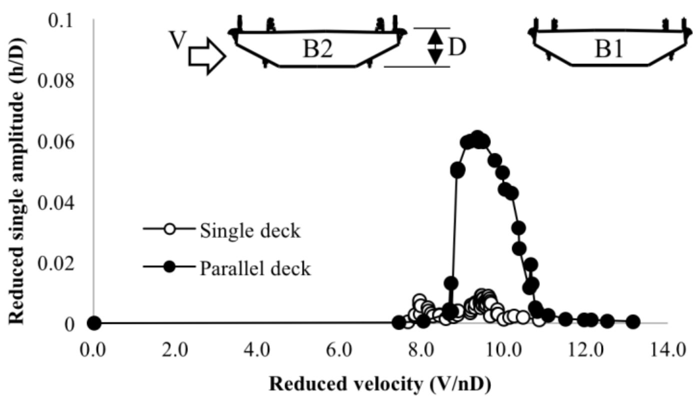

Fig. 6. Amplitude of VIVs in both a single deck and parallel decks

Fig. 6 compares the amplitudes of lock-in VIV for single deck (only Bridge 2) and parallel decks (Bridge 2 in upstream and Bridge 1 in downstream). It is found that the VIV in the upstream deck was amplified due to the presence of another deck in downstream. Every tries in modification of aerodynamic shape of upstream deck were not effective in mitigating the VIV (Seo et al, 2013).

Fig. 7 shows phase-averaged flow streams between two decks at the lock-in wind velocity of parallel-deck case. The Karman-type vortex trails behind the upstream deck were developed to motion-induced vortices, which resulted in interference VIV in upstream deck (Seo et al, 2013).

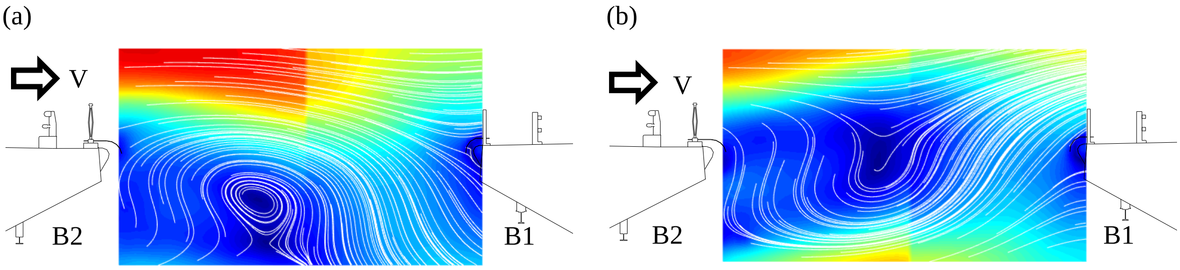

Fig. 7. Phase-averaged flow streams at the phase of (a) $\pi/2$, (b) $3\pi/2$

Fig. 8 shows the relationship between the amplitude of VIV in upstream deck to the damping ratio in wind tunnel setup (Seo et al, 2013). Fig. 8 demonstrates the high sensitivity of VIV amplitude to damping ratio. Based on Fig. 8, the field monitored amplitude of VIV of 0.19m approximately corresponds to a potential damping ratio of 0.21%, which is much less than the design damping ratio of 0.4% recommended in the design guidelines (KSCE, 2006).

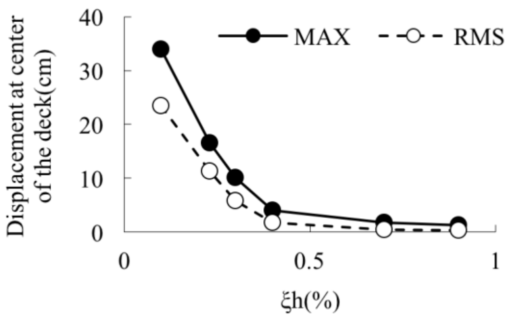

Fig. 8. Amplitude of VIV for set-up damping ratio in wind tunnel

2.3 OMA-Based Damping Identification

Based on thorough wind tunnel tests, two main causes were indicated. The first cause was the amplification of VIV in upstream deck owing to the close parallel disposition between two decks, and the second one was unexpected low damping ratio of Bridge 2. The NExT-ERA technique was applied as an OMA to three-day acceleration data measured at the center of mid-span in both bridges (Kim et al., 2013). Fig. 9(a) shows the identified damping ratios for each segmented 20-min data. A large scattering was observable but the mean value of the damping ratio was 0.28%, which showed a somehow agreement with the predicted one in wind tunnel tests. However, the damping ratios for Bridge 1 was identified much higher than those for Bridge 2 by showing around 0.69%, as shown in Fig. 9(b). In fact, the damping ratio is an inherent property of bridge and, even two similar bridges in terms of materials and overall shapes show a quite different levels of damping ratio.

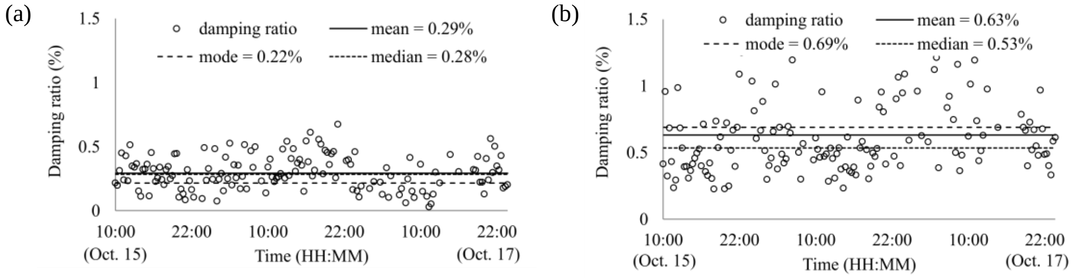

Fig. 9. Estimated damping ratio of (a) Bridge 2 and (b) Bridge 1

3 Case Study of Higher-Mode VIV in a Suspension Bridge

3.1 Observed VIV

The Yi Sun-shin Bridge is a long-span suspension bridge with a main span length of 1,545m, adopting a streamlined twin-deck section. A VIV was observed in October, 2014 for a duration of two hours. At that time, one side of deck was closed for the replacement of epoxy-coated wearing surface. Being completely unaware of aerodynamic severance, the workers applied temporal covers on bridge railings for shielding the curing surface from chilly side winds, as shown in Fig. 10 (b). The time history of measured displacement is shown in Fig. 11. The maximum amplitude reached 0.52m in double amplitude and the motional frequency was identified to be 0.3176Hz, which corresponded to the 4th symmetric vertical mode of the bridge. The wind velocity was estimated to be 5.3~7.0m/s at the deck level.

Fig. 10. Guardrails on decks (a): As-built, (b): Covered by temporal screens

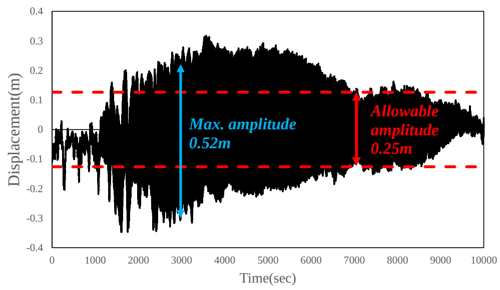

Fig. 11. Time history of VIV

3.2 Examination of Temporal Screen Effect in Wind Tunnel

A series of comparative wind tunnel tests were performed for two section models that simulating as-built and covered guardrails in Fig. 10. Since the amplitude of VIV was critically influenced by the level of modal damping ratio, the set-up damping ratios of the section models were variated for the evaluation of the dependency of VIV to damping ratios. Fig. 12 demonstrates a quite difference in aerodynamic performance for two guardrail conditions. The amplitude of VIV for the as-built guardrails is below the allowance even for the low damping ratio, which shows a stable performance in VIV. However, the guardrails covered by temporal screens leads a magnified VIV at the wind velocity of 4.4~8.3m/s, which confirms to the measured wind velocity range of 5.3~7.0m/s. Even in the design damping level, the amplitude remains in 0.55m in a prototype scale, which correspondent to the measured one with 0.52m.

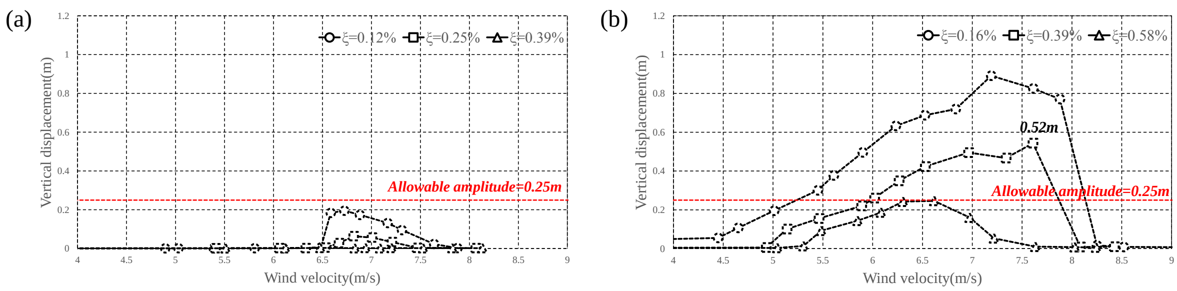

Fig. 12. Amplitude of VIV with (a) As-built guardrails, (b) covered guardrails temporarily

3.3 Damping Estimation from Field-Monitoring Data

To reveal a reason for higher mode VIV, modal damping ratios were identified by OMA. 1-hour ambient vibration data was utilized for damping identification. The Frequency Domain Decomposition (Brinker, 2000) and NExT-ERA were implemented for St-Id. The modal damping ratio of 4^{th} vertical mode was identified to be 0.42%, which was equivalent to a design level, while the modal damping ratios for 1^{st}, 2^{nd} and 3^{rd} mode were evaluated as 1.26%, 2.42% and 0.44%, respectively. This demonstrates that the happening event of VIV for the 4^{th} mode may be potentially related to a relatively low modal damping ratio for the 4^{th} mode.

4 Signal Stationarization in Damping Estimation

Since the modal damping ratios were critical factors in assessment of vibrational serviceability in two demonstrated cases, the authors continued the investigation on the system identification approaches. The modal damping ratios can only be estimated when a bridge construction is being completed. Even though exciters have been utilized as a source of bridge motion for sweeping tests, it is not popular nowadays due to increased size of bridge structures. When the bridge is opened to traffic, the use of exciter is also not feasible when the traffic should be secured all the time. Accordingly, OMA is practically the sole approach we can choose, in which ambient vibration data are collected and modal damping ratios are identified by several St-Id techniques in operating condition. However, in contrast with the identification of natural frequency or mode shape, OMA-based damping estimation is still challenging due to several issues involved (Brewick and Smyth, 2014; Berwick and Smyth, 2014b; Chen et al., 2016; Guo et al, 2012; Kareem and Gurley, 1996; Seo et al., 2013; Kim et al., 2013; Kim et al, 2016; Rainieri and Fabbrocino, 2014; Tamura, 2012).

As was demonstrated in Fig. 9, the estimated damping ratios show a large scattering for each try, even though the most probable value may be represented as the average concept. One of the source of the scattering may be the nonstationarity in ambient vibration signals excited by moving vehicles. When the number of traffic lane over the bridge is only one or two and the traffic volume is not high, the ambient vibration signal at the sensor position show an envelope as a vehicle is approaching and fading away. Since OMA assumes that the signal to be analyzed is a stationary white-noise process, the envelop-like signal obtained from running vehicles can also contribute to scattering in St-Id. Fig. 13(a) shows a sample of traffic-induced vibration obtained from a bridge. The envelop-like responses show a certain level of nonstationary, as shown in Fig. 13(b).

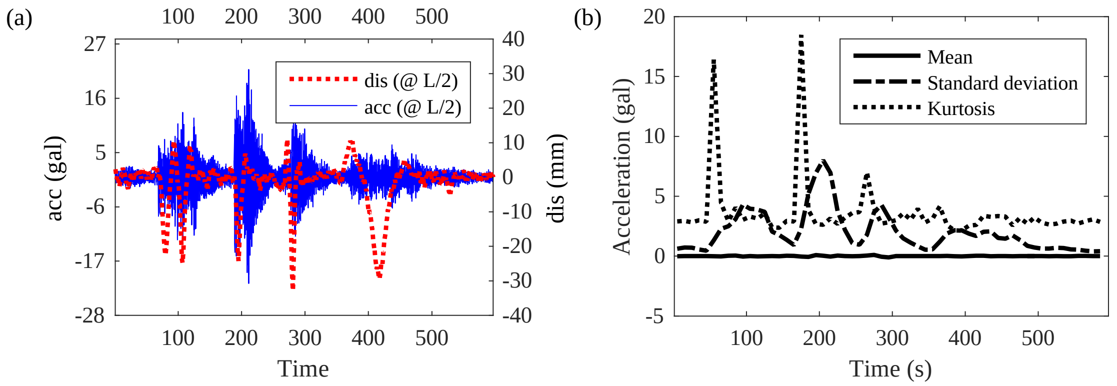

Fig. 13. (a) Measured displacement and acceleration at the center of mid span and (b) moving average, standard deviation and kurtosis of (a)

Lin and Chiang (2012) proposed a practical method to extract the approximate stationary process from nonstationary measurements containing envelope-like function. If the nonstationary loading can be represented by the product of amplitude-modulating function and stationary white noise process, then the envelope function can be evaluated by temporal root-mean-square function of responses. Consequently, the approximated stationary process can be extracted by dividing the measurements with calculated envelope function. Fig. 14 showed that this process reduced nonstationarity in traffic-induced response significantly

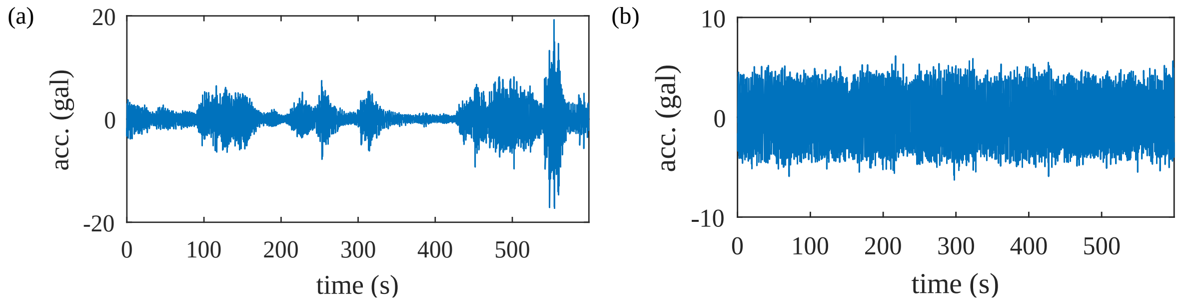

Fig. 14. (a) Measured acceleration and (b) approximated stationary acceleration

The signal stationarization process is applied to the same ambient vibration data utilized for the damping estimation in Fig. 9. The results are shown in Fig. 15. It is found that some highly scattered points were eliminated by the signal stationarization. The coefficient of variation (COV) of estimated damping ratios is reduced from 44.6% to 33.7%, indicating the signal stationarization process is worth in reducing scattering in identified damping ratios from OMA.

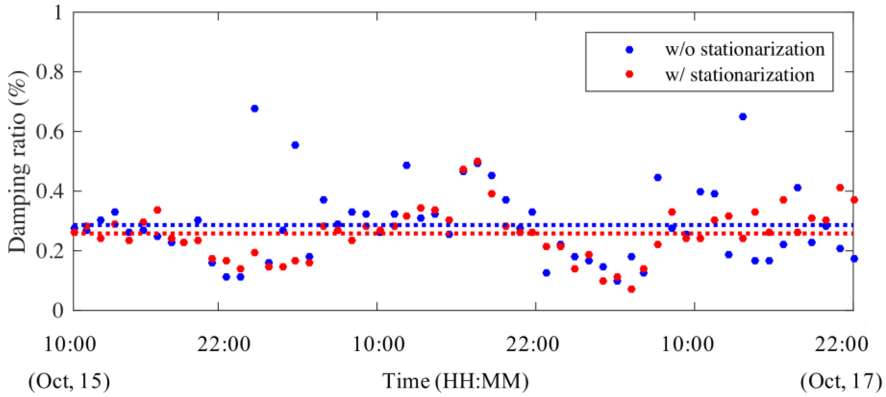

Fig. 15. Estimated damping ratio

5 Concluding Remarks and Suggestions

This paper investigated field-observed VIVs in two cable-supported bridges. One case study was for an interactive VIV in a twin cable-stayed bridge, the first observation in an actual bridge. The observable vibration in the upstream deck was induced by the magnified motion-induced vortex trails generated behind the deck due to the blockage effect by the downstream deck. Accordingly, the interactive VIV was identified to be sensitive to the open gap width between two decks. Meanwhile the close disposition between two decks was an indisputable cause of the observed interactive VIV, another issue of low damping ratio also attracted keen interest of engineers. The damping ratios for both bridges were identified from OMA for the field-monitored data. Even though both bridges were designed as a concept of twin bridges, the estimated damping ratios showed a wide difference. The damping ratio for the upstream bridge, subjected to interactive VIV, was estimated to be 0.28%, while that of the downstream bridge was more than 0.6%. Since the design damping ratio of the steel cable-supported bridge was set to 0.4% in Korean design guideline, the estimated damping ratio was much lower than expected. Another report of VIV in a super-long span suspension bridge was induced by an unexpected happening during the re-pavement work. However, it demonstrated that higher modes could be excited for this sort of very flexible structures and it might be correlated to the lower damping ratio among potential modes.

Two case studies demonstrate the importance of damping ratios in serviceability assessment and securing potential prognosis for enhancement of vibrational resilience for dominant modes. However, a large scattering in identified damping ratio remains to be a challenging issue to bridge engineers. The scattering can be induced from several factors such as nonstationary in monitoring data, but sophisticate and unpredictable relationship of damping ratio to many environmental factors such as amplitude of vibration, temperature change, source of vibration can make the identification of damping ratio to be more challenging by operational monitoring and modal analysis. This paper shows couple of efforts to approach this impending issues by introducing database for probabilistic assessment and data processing to eliminate large scattering in identified damping ratios. At the same time, the authors recommend to bridge owners to secure initial damping ratios for dominant modes and to confirm the validity of wind-resistance design in terms of design damping ratios. If necessary, the level of design damping ratio can be lowered to accommodate a potential possibility of low damping ratios of flexible structures, which adopt higher-strength materials and simpler connections.

Acknowledgements

This research was supported by the Grant (16SCIP-B119960-01) from the Ministry of Land, Infrastructure and Transport of Korean Government through the Korea Bridge Design and Engineering Research Center (KBRC) at Seoul National University; partial supports were received from the Korean Institute of Bridge and Structural Engineers (KIBSE) and the Hyundai Engineering and Construction. The authors particularly thankful to the Korea Infrastructure Safety & Technology Corporation (KISTEK) and the JeollaNamdo for their sharing field-monitored data with us.

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New Technologies for Condition Assessment of Existing Structures

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Abstract. In Japan, a new project called “Cross-ministerial Strategic Innovation Promotion Program (SIP)” supported by Japanese government has started three years ago, one of which deals with the innovative technologies for maintenance, renewal and management of infrastructures. In this program, 60 new technologies have been developed using IT, sensor and material engineering. Representative technologies developed herein will be introduced by paying attention to the condition assessment of existing structures. In addition, an attempt is made to introduce the application of chaos theory into structural damage assessment in non-destructive inspection and vibration-based health monitoring. Firstly, a new impact acoustic method based on attractor analysis is described, which can improve the accuracy of conventional methods of investigating the frequency domain. The proposed method detects exfoliations and confirms the filling conditions inside steel-concrete composite slabs by evaluating the difference of convergence processes of attractors reconstructed from acoustic signals. The effectiveness of applying chaos theory to structural damage assessment is discussed through field experiments and numerical simulations.

3 COST ACTION TU 1406 CONTRIBUTIONS

3.1 Performance indicators for road bridges - overview of findings and future progress (paper / presentation)

A. Strauss, A. Mandić Ivanković, J. C.Matos, J. R. Casas

3.2 Multi-criteria decision making: AHP method applied for network bridge prioritization (paper / presentation)

Z.A. Bukhsh, I. Stipanovic Oslakovic, G. Klanker, N. P. Hoj, B. Imam, Y. Xenidis

3.3 Structural Health Monitoring and Design Code compliance for performance assessment of bridges under scour and seismic hazards (paper / presentation)

M. A. Zanini, F. Faleschini, N. Ademovic, L. J Prendergast, K. Gavin, M. P. Limongelli

3.4 The impact of different maintenance policies on ownerscosts: Case Studies from Croatia and the Netherlands (paper / presentation)

G. Klanker, I. Stipanovic Oslakovic, S. Skaric Palic

3.5 Performance indicators for bridges exposed to a flooding hazard (paper / presentation)

N. Tanasic, R. Hajdin

3.6 Basis for the development of Quality Control Plans for Arch Bridges (paper / presentation)

J. Amado, R. Hajdin

3.7 Bridge inspection quality improvement using standard inspection methods (paper / presentation)

M. Kušar

Performance indicators for road bridges - overview of findings and future progress

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.1

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Abstract. One of the main objectives of COST Action TU 1406 is to build a performance indicator database, in order to develop technical recommendations which will specify the performance goals, aiming to provide a methodology with detailed step-by-step explanations for establishment of QC plans for different bridge types. This paper presents the main findings of Working Group 1: Performance Indicators (PI), based on analysis of documents related to bridge maintenance, assessment and management from different European countries through surveying, clustering, homogenization and categorization. In addition, further steps in order to correlate with the objectives of the Working Group 2, whose work is dedicated to revealing Performance Goals (PG), and the Working Group 3, dedicated to the establishment of Quality Control (QC) plans, if is foreseen.

Keywords: performance indicators, operators' database, research based database, performance levels & aspects, performance goals, performance thresholds, weighting factors

1 Introduction

The main objective of the COST Action TU1406 is to develop a guideline for the establishment of QC plans in roadway bridges, by integrating the most recent knowledge on performance assessment procedures with the adoption of specific goals (Matos, 2016, Matos et al., 2016). This guideline will focus on bridge maintenance and lifecycle performance at two levels: (i) performance indicators and (ii) performance goals. The possibility to incorporate new indicators related to sustainable performance will also be considered. By developing new approaches to quantify and assess bridge performance, as well as quality specifications to assure expected performance levels, bridge management strategies will be significantly improved, enhancing asset management of ageing structures in Europe.

In order to reach this main general aim through more specific objectives and deliverables, the work was structured in several Working Groups. This paper presents the main findings of Working Group 1: Performance Indicators based on analysis of the operators' and research based database. Operators' database was created by surveying documents related to bridge maintenance, assessment and management from different European countries and research based database through surveying scientific documents by answering to several questions. In addition, further steps in order to correlate with the objectives of the Working Group 2 whose work is dedicated to revealing Performance Goals and the Working Group 3 dedicated to the establishment of Quality Control plans is foreseen.

2 Surveying and main findings

Through the WG1 activities, the development of a performance indicators database has been defined as an essential component of the COST Action TU1406. The core of the survey process was structured as a user interface in Excel by storing information in four main groups (Strauss et al., TU 1406 WG1 Report, 2016): Performance level, Damage, Performance indicator/index and Performance assessment. Besides this data, there was an opportunity to add additional references and specific information about a group element (e.g. evaluation process, formula, figure, etc.). The background for this structure comes from screening of the Austrian national document (Bundesministerium für Verkehr, Innovation und Technologie, 2011) and two documents from United Kingdom (County Surveyors Society CSS, 2004). In order to support on the interface in the screening process, a Glossary of key terms is required to store the information and terminology related to Performance Indicators, Performance Goals, Performance Thresholds and Performance Method. It has been prepared on the basis of the information from German and Austrian documents (BASt, 2015, Bundesministerium für Verkher, Innovation und Technologie, 2011).

The selected screening methodology is based on a deep analysis of the existing bridge inspection and evaluation policies in European countries and the main performance indicators used with the objective to define a common group of quality specifications and control plans that can be assumed by all these countries in the next future. This, with the aim to manage the existing roadway infrastructure from an European and not only a country-specific perspective. From the first review of the screening background documents, and the database for performance indicators, main findings are as follows.

The most widely used performance indicator is the condition index, condition rating, deterioration index,…, whatever it is called by different countries and operators, mainly obtained from visual inspection. All surveyed countries have a performance indicator related to this subject. Similar rating system as shown in table 1 is used for many of the countries.

Table 1 Exemplary rating system used in Austria and Croatia

Rating Index	Description
1	No or very slight damage, normal age-related wear and tear, aesthetic damage. *No decrease* in load carrying capacity, serviceability and predicted life time. *No measures* required.
2	Slight damage, production defects with no signs of further deterioration. *No decrease* in load carrying capacity and serviceability. If no suitable measures are taken, the predicted life time will decrease. Repair measures are required in the course of the next maintenance action.
3	Moderate to severe damage with *no decrease* in load carrying capacity and serviceability. *Signs of deterioration* regarding load carrying capacity and serviceability. *Medium-term maintenance* and repair actions are necessary in order to preserve the serviceability and expected life time of the structure.
4	Severe damage, with *no decrease* in load carrying capacity. *Deterioration* in terms of serviceability and expected life time can already be observed. *Maintenance measures* are to be instigated as soon as possible in order to safeguard the serviceability and the expected life time. Such measures may be substituted by additional special inspections within a defined time frame.
5	Extreme damage with impact on the load carrying capacity of the structure. Repair and maintenance measures must be performed immediately.

In many countries, this is the only performance indicator used in practice by bridge owners and operators. However, some countries like Denmark and The Netherlands have started to use other relevant indicators in the assessments made by bridge owners, and not only at a research level. For instance in Denmark, the concepts of remaining service life, safety index-reliability, vulnerability and robustness appear. In The Netherlands the performance is measured in terms of reliability, availability, maintainability and safety/risk (RAMS) among others. The concept of risk is respectively used to define several new indicators: a social indicator, environmental indicator, economic indicator and political indicator (requirements for public image).

Inspection and monitoring strategies for existing bridges, aim at the evaluation and assessment of structural safety and reliability (load carrying capacity, serviceability), with the ultimate objective of determining the traffic safety. Monitoring and evaluation measures are recommended with the aim of improving the understanding and the general assessment of the condition of the structure or also as a special inspection which enables the identification and localization of damage in time. The ultimate objective is to safeguard the performance over the whole life-span. The basis of any kind of monitoring is always a detailed inspection. Such inspections may be subdivided in four time-related categories (e.g. Austria):

- Visual inspections, e.g. yearly basis.
- Simple checks, for instance 3 years after every main inspection.
- In-depth examinations or main inspections, for instance, every 6 years.
- Special inspections, following exceptional occurrences or incidents.

In case of defects or deficiencies, special inspections and further tests or examinations need to be conducted with the aim of assessing whether or not these defects have any impact on the structure serviceability. On this basis, it shall be decided whether the deficiencies and/or damages are to be repaired in the course of the next maintenance action. In general, in-depth examinations should be performed at intervals of no longer than 6 years.

3 Clustering and homogenization of PI database

After collecting the input from different countries, based on surveying of inspection and evaluation documents related to bridge maintenance, assessment and management, it was concluded that results are partly heterogeneous with a number of overlaps. This mainly results from free interpretation leeway and different know-how of experts in visual inspections, performance evaluation, performance assessment and decision making. In some way, there was also some misunderstanding about what are performance indicators and how are they obtained (Strauss et al., TU 1406 WG1 Report, 2016).

defects	related to material properties	related to equipment & protection	geometry changes	related to bearing capacity, structural integrity and joints	related to original construction and design	related to dynamic behaviour	environmental based (common appearance)	rating	cost and importance	loads
abrasion	acids attacks	absence (missing) of equipment component	buckling	absent (missing) structural component	accessibility to damage	atypical vibrations	biological growth	advanced deterioration process	bridge importance (size)	gross weight of a vehicle
absence/ missing	aggregate segregation	approach slab settlement	cross incline of road	accumulated dirt and deposits in joints	bad design	damping	climate change	condition note	element functionality level	permanent loading
aggradation (alluviation)	aging of material	asphalt pavement cracking	deformation	anchorage blocks deficiency	carrying capacity factor	frequency	environmental exposure	condition of a bridge	importance of bridge element	traffic loading
blistering	alkali aggregate reaction (alkali-silica reaction)	asphalt pavement wearing and tearing (rutting, ravelling)	denivelation	anchorage deficiency or failure	concrete cover	noise	freeze-thaw	condition rating	price of the new element	
blocking	alkali aluminium reaction	asphalt pavement wheel tracking and wrinkling and undulation	differential movement	arch ring separation	cracks due to curing and forming	real dynamic behaviour	humidity	damage	sum of costs for repair of individual damages	
bulging	bad concrete compaction	blistering paint	displacement	barrel damage to stone arches	design codes	relative vibrations between elements	moisture	damage assessment	traffic restrictions	
cavitation	bedding mortar failure	cladding damages	distortion	bearing defects	design load	sound	soot	damage degree	traffic volume	

Fig. 1. Cut-out from clustering table of PI related terms for homogenization of the applied database

Therefore, a critical overview of contributions from different countries, with respect to the content and definitions, was necessary. In order to do that, clustering of performance indicators into several groups is suggested. Clustering was guided with the thought that it should allow to more easily identify methods and procedures for revealing and quantifying of performance indicators as well as to define levels of their contribution to a certain structural performance goal. The clustering procedure allowed to reduce the list of terms related to performance indicators in half, from more than 700 hundred of terms into 385. Cut-out from the list of clustered terms is shown at the Figure 1.

Further, the clustering served for homogenisation of the complete European Database, in order to harmonize the Performance Indicators from an European perspective. Example of homogenisation within the Croatian database is shown at the Figure 2. For each available cluster of performance indicators, one example for converting terms from original database into a homogenised one is given. The nominated persons were asked again to verify their performance and damage specific inputs by comparing it with the homogenized terms which are available by a drop-down list in the extended homogenization field. This procedure with the extended homogenized fields by retaining the original information of the databases allows an effective comparison of the performance quantities between countries. Upon homogenisation from all countries the number of indicators was significantly reduced.

A) Performance Level				B) Damage		C) Performance Indicator/Index		D) Performance Assessment		
level	system	compo-nent	material	type	charact-eristic	indicator	detection	thresh-old	goal	
Sub System	All bridge types	Super Structure	Concrete	Damage State	Cracks	Damage degree	Direct Measure-ment	crack width (mm)	Damage Assess-ment	DEFECTS Crack width
Sub System	All bridge types	Super Structure	Concrete	Damage State	Honey-combing	Damage degree	Direct Measure-ment	affected area (m2)	Damage Assess-ment	MATERIAL PROPERTIES bad concrete compaction
Sub System	All bridge types	Super Structure		Damage State	Freeze-thaw	Damage degree	Direct Measure-ment	affected area (m2)	Damage Assess-ment	ENVIRONMENTAL BASED freeze-thaw
Sub System	All bridge types	Super Structure	Brick	Damage State	Disinte-gration of mortar	Damage	Visual Inspection		Damage Assess-ment	STRUCTURAL INTEGRITY & JOINTS disintegration of mortar
Sub System	All bridge types	Railings	Steel	Damage State	Missing parts	Damage degree	Visual Inspection		Damage Assess-ment	EQUIPMENT AND PROTECTION absence of equipment component
System	All bridge types			Damage State	Buckling	Damage degree	Visual Inspection		Damage Assess-ment	GEOMETRY CHANGES Buckling
System	All bridge types		Concrete	Damage State	Execut-ion defects	Damage degree	Direct Measure-ment	affected area (m2)	Damage Assess-ment	ORIGINAL CONSTRUC-TION & DESIGN execution/construction defects
Ele-ment				Damag-ing Process	low damage degree (first phase)	Damage degree	Visual Inspection	Upper limit + Duration of damage phase	Damage Assess-ment	RATING damage degree +damage evolution
Ele-ment						Impor-tance of bridge element		Quanti-tative scale of values	Element impor-tance assess-ment	COST & IMPORTANCE importance of bridge element

Fig. 2. Example of homogenization of terms within the Croatian database

4 From Performance Indicators to Key Performance Indicators

It was agreed, among the TU 1406 community, that Performance Indicator is a measurable and quantifiable parameter related to the bridge performance that can be compared with a target measure of a performance goal or can be used for ranking purposes among a bridge population in the framework of a Quality Control Plan or life-cycle management (which includes decisions and actions involving economic resources).To evaluate certain performance indicator, performance thresholds or criteria must be set. A threshold value constitutes a boundary for purposes such as: a) monitoring (e.g. an effect is observed or not), b) assessing (e.g. an effect is low or high), and c) decision-making (e.g. an effect is critical or not). A criterion is a characteristic that is relevant for the choice between processes e.g. such as maintenance actions or others. Although the interaction of different performance indicators is inevitable, their categorization into technical, sustainable and socio-economic indicators through component, system and network level is proposed in order to more easily identify level of their influence to a certain key performance indicator related to performance goal.

In order to move on with the reduction of the list of Performance Indicators, an Expert Group was asked to specify PIs (YES/NO) according to the following points: Measurable?; Quantifiable?; Target value available?; Valid for ranking?; Allow decision with economic implications?. At the end, approximately 100 extricated PIs are further related with one or more Key Performance Indicators (KPI): Reliability (R), Availability (A), Maintainability (M), Safety (S), Security (Se), Environment (E), Costs (C), Health (H), Politics (P), Rating/Inspection (I). Further the process required the categorization of Performance Indicators in relation to Performance Goals (PG) and Performance Thresholds (PT) at different levels: component (CL), system (SL), network (NL); taking into account different aspects: technical (Tech), sustainability (Sust) and socio-economic (SoEc). Each expert's feedback was systemized as shown in the cut out example at the Figure 3.

Safety, Reliability, Security						
PI	Level Component Level (CL) System Level (SL) Network Level (NL)	Performance indicator PI if Measurable? {Quantifiable? Target value available? Valid for ranking purposes? Allow decision with economic implications?} (YES/No) Technical (Tech), Socio Economical (SoEc), Sustainable (Sust)	PI belongs to the Key Peformance Reliability (R), Availability (A), Maintainability (M), Safety (S), Security (Se), Environment (E), Costs (C), Health (H), Politics (P), Rating/Inspection (I)	Assessment Threshold (T = ...) Goal (G = ...) Rating (R = ...)	rating (1-5)	weighting
concrete cover (insufficient)	CL	Yes, Tech, Sust	R, A, (C, I)	T= thickness (mm), G= assessment of damage and affected area (m2), R=important for durability	2	0,8
origin (e.g. due to loading, due to settlement, due to crumbling of concrete,…	CL, SL		R, A, S, (C, I)	T=width (mm), G=understand origin through the correlation of the observed thickness (mm), length (cm), location/orientation and spacing/pattern, R=key PI to access reliability	3	0,5
fatigue cracking	CL, SL	Yes, Tech	R, A, S, (C, I)	T= number of cracks and affected components; G= local or generalized situation and importance of affected components; R= Key for reliability.	5	0,3
settlement	SL	Yes, Tech	R, A, S, (C, I)	T= dimension (mm) and orientation (º), G= Affected components, stable/evolving, R=key for structural equilibrium	2	1
…..	….	….	…	…	…	…
					total rating	n_{SRS}

Fig. 3. Cut out of the categorization of performance indicators at different levels, taking into account different aspects

Safety, Reliability, Security		
PI	rating (1-5)	weighting
crack width	2	0,8
corrosion	3	0,5
lack of bolts	5	0,3
support damage	2	1
drainage system	2	0,8
fungus appearance (wooden elements)	3	0,5
bugs attack (wooden elements)	5	0,3
rotting (wooden elements)	2	1
overweight traffic	1	1
sediment accumulation	2	0,8
vandalism	3	0,8
	total rating	r_{SRS}

Availability, Maintainability		
PI	rating (1-5)	weighting
concrete florescence		
lack of bolts		
support damag		
drainage syste		
fungus appearance (wooden elements)		
bugs attack (wooden elements)		
rotting (wooden elements)	3	
sediment accumulation	1	
vandalism	3	
	total rating	r_{AM}

Costs		
PI	rating (1-5)	weighting
….	…	…
	total rating	r_C

Environment		
PI	rating (1-5)	weighting
….	…	…
	total rating	r_E

Health, Politics		
PI	rating (1-5)	weighting
….	…	…
	total rating	r_{HP}

Fig. 4. Rating and weighting scheme related to five main groups of key performance indicators

Further process will require allocation of rating value (1-5, as in table 1) and weighting factor (whose values are stil to be defined) to each PIs related to five main groups of Key Performance indicators which are established in relation to requirements of the Working Groups 2 and 3 (Figure 4). The final rating and weighting will reveal overall rating (with rating factors r_{SRS}, r_{AM}, r_C, r_E and r_{HP}) of each of the five most important KPIs groups.

Example is presented with the Figure 5. Green areas represent the most favourable rate and the red areas should alarm the bridge operator and require immediate intervention.

KPI	
Safety, Reliability, Security	r_{SRS}
Availability, Maintainability	r_{AM}
Costs	r_C
Environment	r_E
Health, Politics	r_{HP}

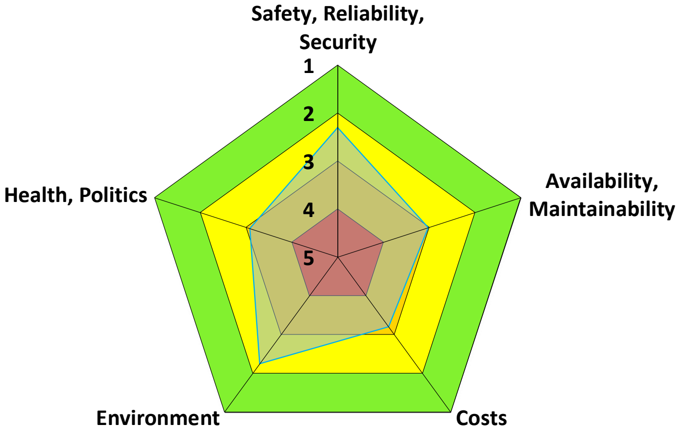

Fig. 5. Overall rating example of each of the five most important KPIs groups

5 Conclusion

The determination of performance indicators for bridge structures from European countries and its harmonization on a European level is complex, extensive, and time consuming process. After collecting the input from different countries, heterogeneous data on bridge performance aspects were systemized through clustering and homogenization of performance related terms. This is followed with the categorization of reduced list of actual PIs at different levels: component, system, network; taking into account different aspects: technical, sustainability and socio-economic. Categorization process is still undergoing, aiming final overall rating of each of the five most important groups of Key Performance Indicators required to define quality specifications and control plans of road bridges at the European level.

Acknowledgements

This paper is based upon work from COST Action TU-1406, Quality specifications for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology).

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Multi-criteria decision making: AHP method applied for network bridge prioritization

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.2

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Abstract. In bridge management systems, multi-objective decision-making has emerged as a decision support technique to integrate various technical information and stakeholder values. Different multi-criteria decision making techniques and tools have been developed in the last three decades. This paper presents an overview of different approaches to multi-objective decision making at the object and network level, with the purpose of incorporating different aspects of bridge performance goals, which may vary according to technical, environmental, economic and social factors. The example of application of analytic hierarchy process (AHP), as one of the multi criteria decision making method, to a illustrative case study is presented in the paper.

Keywords: multi-criteria decision making, bridge management, performance goals.

1 Introduction

Bridges are key structures of a transport infrastructure system. From an economic viewpoint it is crucial that bridges provide their designed function as part of the infrastructure network systems in an efficient manner. Bridges present a vital link in any roadway network. It is estimated that the ratio of expenses per route km of bridges or tunnels is 10 times average expenses per route km of roads (CEDR, 2010). Also the length of bridges compared to the whole length of road network is only app. 2% but at the same time they present 30% value of the whole network (PIARC, 1999). When these statistics are taken into consideration it is easy to understand why, in the past few decades, an increasing number of deteriorating bridges led to the development of a number of Bridge Management Systems (BMS) and life cycle maintenance models like for example Branco&Brito, Frangopol's and Rijkswaterstaat's model (Kaneuji et al. 2006, Airaksinen 2006, BRIME 2000, Noortwijk J.M. et al. 2004).

Profit-driven transport operations, aging infrastructure and adverse climate changes imposed the need of cost-effective and improved maintenance strategies. Infrastructure managers are under a lot of pressure to not only reduce network downtime, but also plan, such maintenance policies that could prolong the overall service lifetime of the infrastructure. According to Van der Velde et al. (2013) budget restraints and increased public demand in terms of service and quality put pressure on government bodies that have to manage transport infrastructure systems while dealing with these dynamics. Looking for ways to cope with this issue the attention from infrastructure operators is increasingly turned to asset management. Applying this approach to infrastructure is seen as a way to deal with the pressures that are present on the management of public transportation infrastructure which were discussed above. Moon et al. (2009) describe the different concepts that are involved with infrastructure asset management. According to them, integrating these concepts should lead to a better infrastructure management. The following concepts are described: 1) Performance based engineering and management leads to the definition of performance objectives and correlated metrics, 2) Structural identification, health monitoring and intelligent infrastructure are used to monitor and forecast metrics, and 3) Life cycle costs and decision making are used to identify trade-offs. These concepts and their interplay are depicted in Figure 1. In the work presented here, the focus will be on multi-criteria decision making while assessing trade-offs based on different aspects of bridge performance.

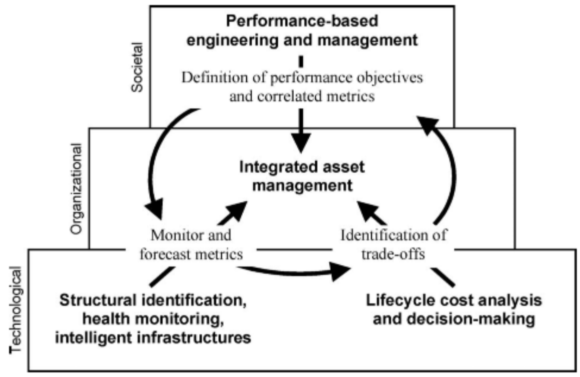

Figure 1: Integration of the different aspects of integrated asset management (Moon et al., 2009)

2 Identification of trade-offs

Bridge maintenance planning is a process of deciding the scope, timing, costs, and benefits of future maintenance activities on a specific bridge. Optimization of maintenance activities regarding technical and economic requirements is essential for road owners to fulfil societal expectations. Due to the long life time of the road infrastructure, especially bridge structures (often longer than 50 years), the assessment of technical and economic performance is necessary in order to optimize budget expenditure. Life cycle cost (LCC) analysis and reliability based concepts are well established methodologies for identification and assessment of trade-offs.

2.1 Object level

In the current state of the practice, most bridge management systems are very effective at storage and retrieval of all the raw data needed for maintenance planning. With these data it is possible to develop plots of performance over time and to identify possible maintenance alternatives. Based on that, life cycle costs for different maintenance alternatives can be calculated and compared. To express and manage this spectrum of possible futures, the concept of a “candidate” is suggested (Patidar et al. 2007). A “candidate” is hereby defined as a life-cycle activity profile for one bridge, consisting of a sequence of agency activities—including do-nothing— in each of a sequence of future time periods. Development of alternative candidates and selection of the best one is a cardinal aspect of decision making by the bridge maintenance planner. The planner decides which of the alternative candidates are worthy of consideration and, over time, narrows the list to just one or a small number that the planner then submits to the next level of bridge management (i.e., programming and budgeting at the network level). An example of comparison of different maintenance options and LCC results is shown in Figure 2. However, the focus of existing bridge management systems is still mainly on owner’s costs, rarely taking into account other impacts of the bridge, such as environmental impacts, availability, importance on the transport network and society as a whole, etc.

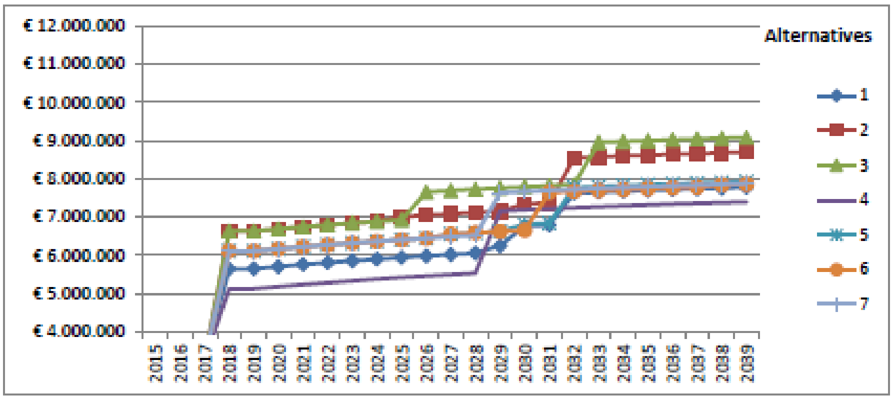

Figure 2: Comparison of different maintenance alternative candidates for life cycle of 25 years (Wolthuis, 2014)

2.2 Network level

In the course of developing network-level bridge programs, program managers typically face a variety of objectives and constraints. Examples of objectives are to maximize cost effectiveness, to minimize vulnerability to damage, to maximize average condition, and to optimize a utility index that combines various objectives. Constraints include budgetary limits that cannot be exceeded or a minimum level of average bridge performance (Patidar et al., 2017).

The foundation of any decision analysis is a clear statement of goals and belonging performance indicators. To describe the consequences of alternative bridge actions and enable trade-offs between competing goals, it is necessary to identify a set of goals and a set of performance indicators for each goal, as it is shown in Figure 3.

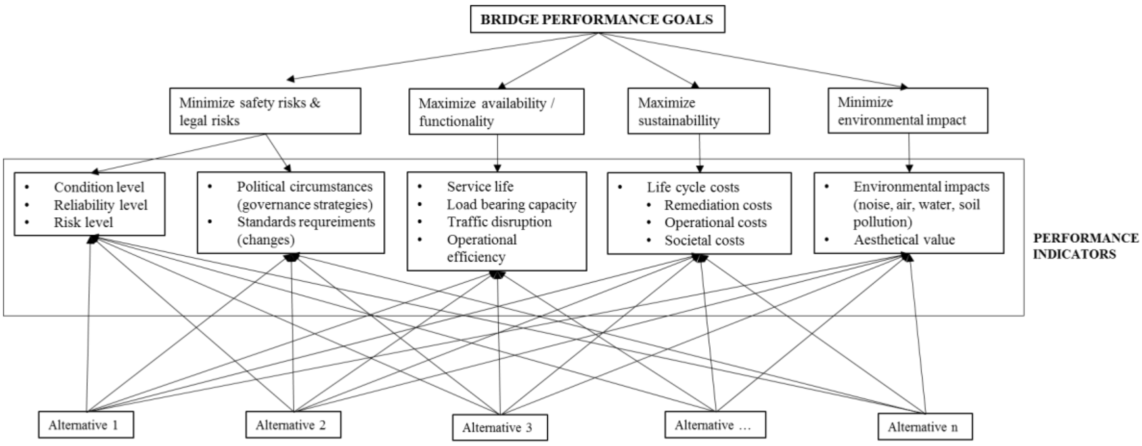

Figure 3: Hierarchy structure for linking multi-objective bridge performance goals and performance indicators (based on Rashidi & Lemass, 2011)

Based on the work which has been so far done in COST TU 1406 WG 1 (WG 1 Report, 2016) and literature review (Stipanovic et al. 2016; Imam, 2016) for the purposes of this study, a list of general bridge performance goals is proposed, as given in Table 1. A set of performance indicators for each goal clarifies the meaning of each goal and is required to measure the consequences of alternative bridge maintenance actions. Performance indicators are also sometimes referred to as measures, attributes or criteria.

Table 1: Overview of performance goals and performance indicators

Performance Goal	Performance aspect	Performance indicators*
To provide safe and reliable network	Reliability	Condition Rating
		Reliability Rating
	Safety	Number of casualties caused by traffic accidents
To protect from extreme events	Safety	Scour Vulnerability Rating
		Earthquake Vulnerability Rating
		Other Disaster Vulnerability Rating
To provide responsive and sustainable network	Availability	Availability of the road (% of time)
		Downtime (Traffic delays caused by maintenance works)
To minimize agency costs	Economic aspect	**Owners costs** (LCC, initial costs, maintenance costs, replacement costs etc.)
To minimize its negative impacts on users, local communities and the environment	Societal and environmental aspects	**Importance on the network** (Traffic intensity)
		User delay costs
		Societal costs
		Environmental impacts

* list not complete, only selected examples provided here

3 Multi-criteria decision making

Implicit in any decision-making process is the need to construct, either directly or indirectly, the preference order, so that alternatives can be ranked and the best alternative can be selected. For some decision-making problems, this may easily be accomplished. For example, in case of a decision based on a cost-minimization rule (where the lowest-cost alternative is chosen), the preference order is adequately represented by the natural order of real numbers (representing costs). Hence, in such a case, the preference order need not be constructed explicitly (Patidar et al., 2007).

Multi-criteria decision-making (MCDM) provides a systematic approach to evaluate multiple conflicting criteria in decision making. Conflicting criteria are typical in evaluating options: cost or price is usually one of the main criteria, and some measure of quality (performance level) is typically another criterion, usually in conflict with the cost. MCDM is used to identify and quantify decision-maker and stakeholder considerations about various (mostly) non-monetary factors, in order to compare alternative courses of action (Kabir et al., 2014). Alternatively, the multiple performance criteria can be combined into a so-called utility function, in which all the criteria are brought into a single scale.

In the course of developing network-level bridge programs, program managers typically face a variety of objectives and constraints. Examples of such objectives are given in Table 1, and constraints usually include a budgetary limitation that may not be exceeded or a minimum level of average bridge performance. The overlap between objectives and constraints is a key issue to practical multi-objective optimization of an asset management program.

An important class of decision-making techniques that attempt to construct the preference order by directly eliciting the decision maker’s preference is predicated on what is known as utility theory. This, in turn, is based on the premise that the decision maker’s preference structure can be represented by a real-valued function called a utility function. Once such a function is constructed, the selection of the appropriate alternative can be done using an optimization method. Broadly speaking, this technique involves three steps (Patidar et al., 2007):

1. Weighting: This assigns relative weights to the multiple criteria.

2. Scaling: Because the performance criteria can be of different units, scaling provides a common scale of measurement and translates the decision maker’s preferences for each performance criterion on a 0–100 scale. This involves developing single-criterion utility functions.

3. Amalgamation: Amalgamation is combining the singlecriterion utility functions using the relative weights into one measure based on mathematical assumptions about the decision maker’s preference structure. This involves deriving the functional forms of multi-criteria utility functions.

3.1 Analytic Hiearchy Process

The Analytic Hierarchy Process (AHP) is a structured technique for organizing and analyzing complex decisions, based on mathematics and psychology. It was developed by Thomas L. Saaty in the 1970s and has been extensively studied and refined since then. It has particular application in group decision making, and is used around the world in a wide variety of decision situations, in fields such as government, business, industry, healthcare, shipbuilding and education.

Rather than prescribing a "correct" decision, the AHP helps decision makers identify the decision that best suits their goal and their understanding of the problem. It provides a comprehensive and rational framework for structuring a decision problem, for representing and quantifying its elements, for relating those elements to overall goals, and for evaluating alternative solutions.

The analytic hierarchy process aims to arrive at the relative weights for multiple criteria in a realistic manner while allowing for differences in opinion and conflicts that exist in the real world. The analytic hierarchy process can handle quantitative, qualitative, tangible, and intangible criteria. The process is based on three principles: decomposition, comparative judgments, and synthesis of priorities. It constructs a hierarchy and uses pairwise comparisons at each level to estimate the relative weights.

The procedure for using the AHP can be summarized as (Saaty, 2008):

1. Model the problem as a hierarchy containing the decision goal, the alternatives for reaching it, and the criteria for evaluating the alternatives.
2. Establish priorities among the elements of the hierarchy by making a series of judgments based on pairwise comparisons of the elements. For example, when comparing potential purchases of commercial real estate, the investors might say they prefer location over price and price over timing.

3. Synthesize these judgments to yield a set of overall priorities for the hierarchy. This would combine the investors' judgments about location, price and timing for properties A, B, C, and D into overall priorities for each property.
4. Check the consistency of the judgments.
5. Come to a final decision based on the results of this process.

3.2 Illustrative case study for AHP method

In this section, an example case is provided to demonstrate the multi-criteria optimization for the bridge maintenance planning. A road network has a number of bridges which might be exposed to deteriorating condition. Due to budget limitations, a decision has to be made regarding the selection of bridge(s) for maintenance. Therefore, the objective of this example is to illustrate the decision making procedure for a bridge(s) selection, where the cost and downtime due to maintenance is aimed to be kept minimum.

For the sake of demonstration, we have considered five bridges depicted as A, B, C, D and E. Each bridge on the network has a number of related attributes such as location, geometry, condition etc. Before assessing the bridge performance on the network level, it is assumed that for each bridge an optimal maintenance alternative has been selected, based on life cycle cost calculations for several alternatives, e.g. three alternatives: do nothing, minor repair, major repair. For the purpose of ranking the bridges in the terms of maintenance prioritization, the following performance indicators have been selected (see Table 1, marked bold): reliability level, maintenance cost, downtime due to the maintenance works, and importance on the network.

Table 2. Representational data for illustrative case

	Reliability	**Economy**	**Availability**	**Society**
Bridges	Reliability level	Maintenance cost	Downtime	Importance on the network
Name	Score card	Euros	Hours	Traffic Intensity (# cars / day)
A: B101	3	500k	30	9000
B: B109	4	1000k	70	10000
C: B207	4	200k	60	13000
D: B307	5	800k	180	15000
E: B150	3	500k	40	5000

The related data for five bridges is provided in Table 2, where indicators are defined as follows: *Maintenance cost* is a direct (owners) cost which is expected to be incurred during the maintenance procedure. *Downtime* is defined as the unavailability of the bridge due to planned maintenance activity. The *reliability level* of a bridge is a constructed value based on the condition indexes. The reliability level only involves those bridges that are critical to perform their intended function. The score card depicting the reliability levels are shown in Table 3.

Table 3 : Reliability level score card

Reliability Level	**Description**
1	Very Good (no faults)
2	Good (minor faults well within tolerance)
3	Fair (tolerable faults, no restriction in use necessary)
4	Poor (significant structural defects)
5	Very poor (seriously deficient, mitigation measures necessary)
6	Out of service (on high risk of failure, mitigation needed urgently)

Only those assets that are in the optimum range of serviceability, i.e. 1-5 can be considered in the maintenance planning procedure. Bridges having the reliability level of 6 are considered too critical to delay the maintenance actions. Finally, the network importance is defined by the intensity of traffic passing over the bridge each day.

3.3 Preference structure

Subjective judgment in the decision making methods plays vital role. The preference structure and procedures for this example is based on analytical hierarchy process proposed by Saaty (1988). To assign the ratio scales to the subjective judgments of comparison, a fundamental scale for pairwise comparison is used. By assigning the ratio scale values, the importance of one attribute (e.g. condition) become clear to another attribute (e.g. cost).

Based on the scale of pairwise comparison[1], a relative importance of each attribute to define the decision criteria is defined in Table 4. Considering the objective, the decision criteria are to select a bridge for maintenance that has minimum cost, minimum downtime, minimum reliability level, and minimum traffic intensity. Therefore, in comparison of maintenance cost and downtime the intensity of their relative importance is equal to 1. While, in comparison of maintenance cost and reliability level, the maintenance cost is moderately more important than reliability level with intensity of 3. It is important to notice that all the comparisons among attributes and their data is made by authors of this paper for demonstration purposes.

Table 4: Subjective judgment to ratio scales

Criteria		**Importance & Intensity**	
A	B	Imp	Intensity
Maintenance cost	Downtime	A	**1**: Equal importance
Maintenance cost	Reliability level	A	**3:** Moderate
Maintenance cost	Traffic Intensity	A	**5:** Strong
Downtime	Reliability level	A	**3:** Moderate
Downtime	Traffic Intensity	A	**3:** Moderate
Reliability level	Traffic Intensity	A	**7:** Very Strong

The subjective judgment to ratio scales provides a decision criteria matrix. The step of pairwise comparison is repeated four times to compare data values for each bridge to another. For instance, the maintenance cost for each bridge is compared to another and their relative importance is defined. The low maintenance cost will get high intensity value as compared to the higher maintenance cost. These pairwise comparisons of data values yielded four comparison matrixes.

To derive the final weighted scale, the Eigen vector method is used to first reduce the matrix into a value function ranging from 0 to 1 and then to find the largest Eigen value. Irrespective of matrix type either as decision criteria matrix or comparison matrix, the steps to reduce to Eigen vector are similar. The following steps are taken to find the best bridge for maintenance that incur minimal maintenance cost and downtime. To represent the calculation procedure, some steps are adopted from (Ma, Ma, Zhou, & Ma, 2015):

a) normalize each matrix

$$\overline{e_{ij}} = \frac{e_{ij}}{\sum_{k=1}^{n} e_{ij}} \quad (1)$$

Where e_{ij} represents an element in the matrix M. $\bar{e}_{ij}$ represents an element of normalized matrix which is resulted by dividing e_{ij} to sum of its column value. This step results into a normalized matrix.

b) To calculate the geometric mean of each matrix, add all the elements in a row
Then divide the sum of values in a row to the number of rows

$$\overline{w_i} = \sum_{j=1}^{n} \overline{e_{ij}} \quad (2)$$

[1] Online link to scale: http://bit.ly/2gEluQX

This step represents the final weighted score for a matrix, represented as S_m. The calculation of geometric mean reduces the matrix size to 4*1 and 5*1 for decision matrix and comparison matrix respectively.

$$S_m = \frac{\overline{w_i}}{\sum_{j=1}^{n} w_j} \tag{3}$$

c) The final step is to calculate the largest value that represents the aggregated preference of reduced maintenance cost and downtime.

$$M_{jm} = (S_{mc}, S_d, S_{rl}, S_{ti}) \tag{4}$$

Where S_{sc}, S_d, S_{rl} and S_{ti} represent the geometric mean values for maintenance cost, downtime, reliability index and traffic intensity respectively. The final score is obtained by multiplying the judgment matrix represented as M_{jc} to the decision criteria matrix represented as M_{dc}.

3.4 Results

The result shows the scoring of selected bridge maintenance alternatives for five bridges, when taking into account four different aspects, measured by cost, downtime, reliability level, and traffic intensity at the network level. Prioritization of bridges for maintenance will be done based on the objective to keep the maintenance cost and downtime minimum. The final obtained results are illustrated in Figures 4 and 5.

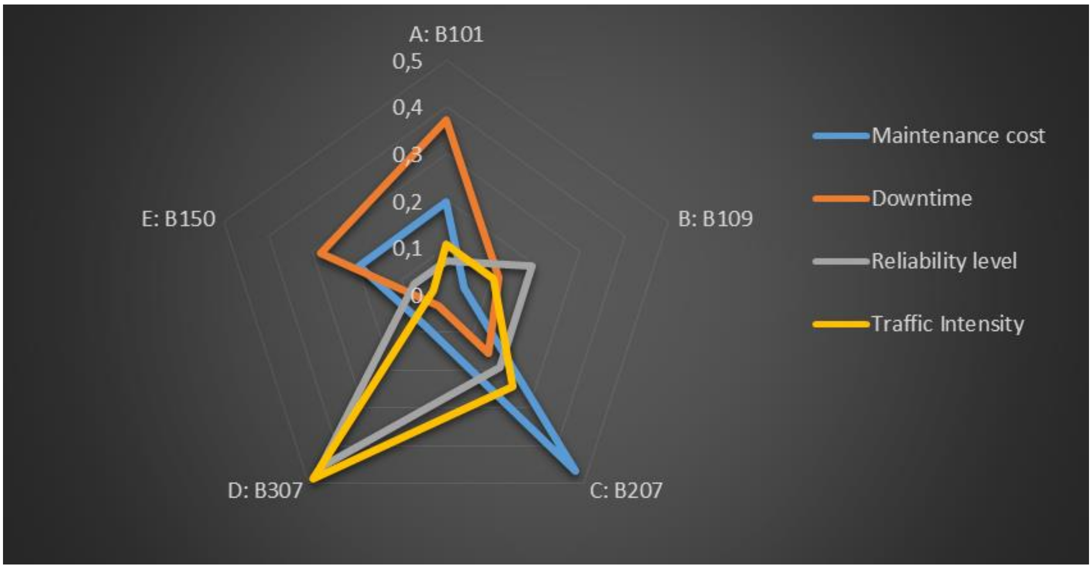

Figure 4: Comparison of performance aspects for five bridges

Before providing the final selection of a single bridge, Figure 4**Error! Reference source not found.** graphically represents the equation 4 where the final score (S_m) is combined to form a judgment matrix (M_{jm}). The higher the value of an attribute the more it is preferable with regard to the defined objective. The graph shows that bridge *C:B207* is most preferable selection in terms of reduced cost, while bridge A:B101 is best in terms of reduced downtime. Similarly, more conclusions can be drawn e.g. for the maximization of reliability level then bridge *D: B307* is scoring the best.

It is important to notice that based on the attribute nature the applied function is defined. For instance, for the maintenance cost and downtime the minimization function is applied whereas for the reliability level and traffic intensity the maximization function is used.

Figure 5 shows the final prioritization result based on the AHP process for bridge maintenance planning, when the objective is set to minimize maintenance costs and minimize downtime. It is shown that bridge *C:207* is most preferred in terms reduced cost and downtime whereas B: 109 is least preferred bridge for maintenance.

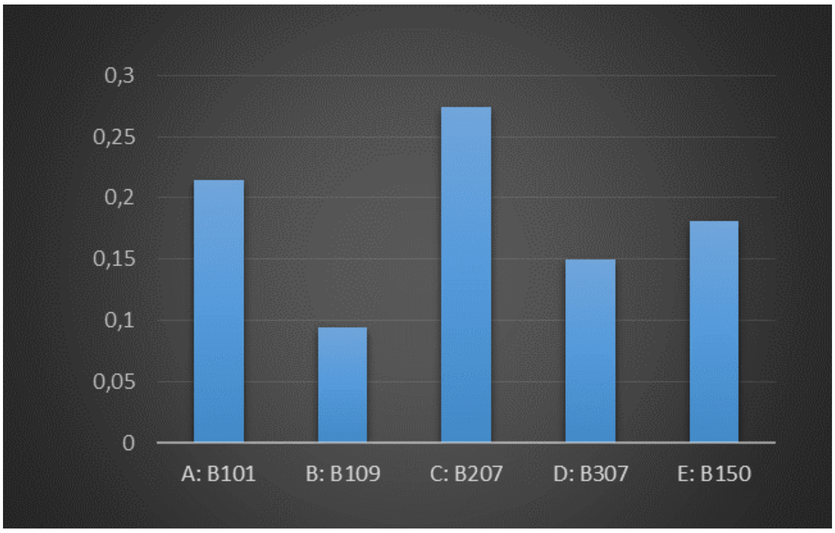

Figure 5: Bridge scoring based multi-criteria decision anaysis

4 Discussion and Conclusion

One of the main challenges in future research is how to quantify performance goals other than technical, and how to link network level to the performance requirements on the object level. Network or even societal goals tend to be rather broad in their definition. Furthermore, there is often no exclusive relationship between performance indicators set at a lower level and goals at a higher level. An important notion is that in many countries, the main focus of bridge management is still the condition assessment of the particular objects or elements thereof.

This study provides some guidance on how a program manager can implement multiple performance goals by establishing a multi-criteria decision making framework, based on the performance indicators linked to the performance goals and aspects. In the paper a set of performance goals is proposed, based on the existing practices and literature review, which are linked to the set of performance indicators that include reliability, availability, economy and societal aspects. For the purpose of showing how multi-criteria decision making can be performed, an illustrative case study has been done with the application of AHP method. With this method it was possible to evaluate multiple performance aspects of multiple bridges in order to rank bridge maintenance activities on the network level.

Acknowledgements

This article is based upon the work from COST Action TU1406, Quality specifications for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology).

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Structural Health Monitoring and Design Code compliance for performance assessment of bridges under scour and seismic hazards

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.3

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Abstract. When dealing with asset management of infrastructural systems, maintenance planning of bridges and other critical structures has to be thought against natural deterioration due to environmental conditions, but also taking into account potential criticalities induced by the occurrence of seismic and flood hazards. The aim of this work is to focus on potential damage scenarios, assessment methods for common bridge structures potentially subject to earthquake loading and scouring phenomena due to flooding hazards.

Keywords: bridge performance assessment, natural multi-hazards, scour, seismic actions, structural health monitoring

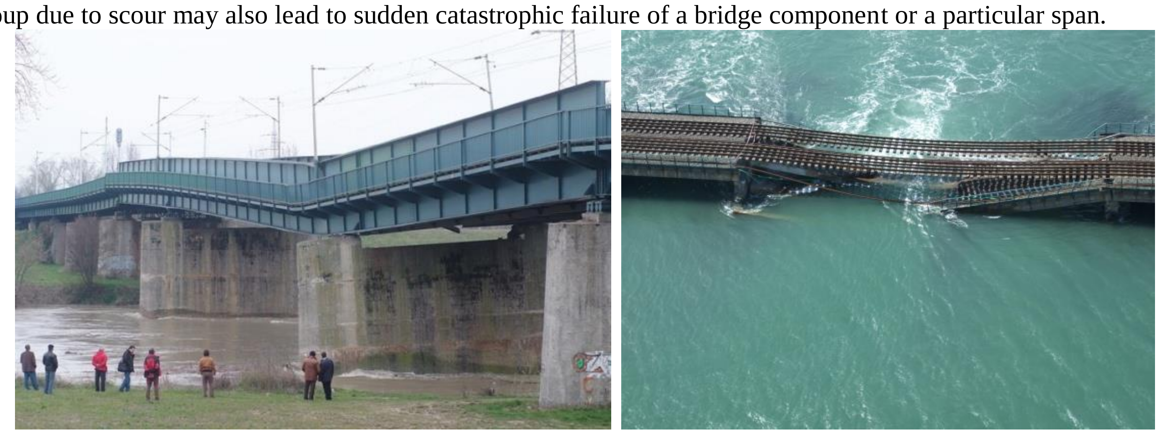
1 Introduction

In infrastructural networks, bridges can be considered key elements and their functionality must be preserved. Bridge structures have therefore to be adequately maintained, since structural deterioration combined with the occurrence of hazardous events like earthquakes or floods can compromise structural stability, and lead to bridge failure (Deng et al. 2016). In recent years bridge failures induced by natural hazards were drastically increased, due to lack of adequate monitoring and preventive maintenance actions, but also for the higher number of natural disasters that yearly hit different regions worldwide. With reference to bridges and viaducts, floods and earthquakes could be considered among the most critical events causing significant damage. Flooding phenomena usually affect substructures of bridges crossing rivers, causing local scouring phenomena at the base of pier foundations set in the river bed. This can be exacerbated by natural channel evolution and the resulting settlements may affect the whole structural stability. Consequences of such events and potential critical points in road networks can be previously identified based on weather forecasts, thus reducing the probability of human and economic losses. Scouring effects could be particularly dangerous for ancient (e.g. masonry arch) bridges due to the shallow foundations (Zanini et al. 2016). The increase of the hydraulic outflow speed causes turbulences and vortex shedding (HEC18, 2001) close to the bridge piers, generating local scouring at the base of pier foundations. Scour of bridge foundations is one of the most frequent causes of structural collapse in United States, with about 600 bridges failed during the last 30 years (Briaud et al. 2005), but also in Europe, particularly in the United Kingdom (Maddison 2012) and central Europe (Tanasic 2016). In regions prone to seismic hazard, ground motions may induce damages on bridge structural components like piers, abutments and bearing systems. Several regions in Europe have both seismic and scour hazards however the traditional approach to bridge design does not take into account the increased hazard induces by the joint action of the two phenomena. The two types of hazards are actually independent as to the generation process but the loss of surrounding soil due to scour may significantly reduce the lateral strength of pile foundations thus increasing the earthquake damage potential (Song et al., 2015). In recent years researchers have begun investigating the performance of bridges, or of bridge components under multiple scour and seismic hazard (Haney al., 2010, Alipour et al, 2013, Ganesh et al, 2013).

In this contribution a brief overview of potential damage scenarios induced by flooding and seismic actions and by the combined action of the two, is first illustrated. Assessment procedures currently in use against such natural hazards are briefly reviewed and some current research trends reported.

2 Damage scenarios due to scour and seismic actions

Both scour and earthquake events can cause heavy damages to bridges, leading in some cases to the structural failure. Scour has a deleterious effect on the stability and capacity of bridge foundations and can give rise to several major forms of damage, see Figure 1 for an example of two bridge failures due to scour. Depending on the type of bridge and the nature of the foundation this damage can be extremely detrimental to the operational capacity of the structure and may result in serviceability or ultimate failure. For bridges founded on shallow foundations, scour undermining the foundation can give rise to adverse settlements which can lead to cracking at the deck level and at other supports. In masonry arch structures, this damage can be even more severe and may compromise arch stability. Even when scour does not undermine a bridge pad foundation, the reduction in soil level around the foundation can give rise to geotechnical stability problems in the remaining soil and exacerbate flow conditions around the foundation element. For piled foundations, there are a number of critical damage scenarios. The loss of lateral pile support may give rise to the possibility of pile buckling. This could cause a very sudden and severe issue to arise and may result in total bridge collapse. For severe scour around piles, the loss in shaft resistance may result in adverse settlement issues, which has ramifications for the bridge in terms of crack propagation in the superstructure. Differential settlement of different foundations may lead to severe cracking, deck buckling or total failure, whereas global settlement may induce serviceability failure with unacceptable settlement to the deck, for example. For pile groups, the possibility of differential block settlement may arise inducing unacceptable tilting of the supported pier or abutment. This tilt may cause a deck to slide on its supports or buckle, depending on the nature of the structural connection. Moreover, block failure of a pile group due to scour may also lead to sudden catastrophic failure of a bridge component or a particular span.



(a) (b)

Figure 1 Failure due to scour – (a) pier settlement, Croatia 2009; (b) pier failure, Dublin 2009

Regarding the damage scenarios induced by earthquake occurrence, past earthquakes have shown that for common girder bridges failure may occurr due to: collapse of the piers for bending or even for shear if capacity design prescriptions are applied; collapse of the pier foundations if a capacity design is not applied or collapse of the deck due to unseating induced by high seismic displacement. Scour may exacerbate the dynamic behavior of bridges under seismic actions since it causes the loss of lateral support and of axial friction from the soil at the level of the bridge foundations. This, beyond producing a change in the capacity of the foundation, may alter the hierarchy of failures required by capacity design thus leading the foundation to collapse before the flexural failure of the columns takes place. On the other hand the increase of modal periods induced by the reduced stiffness of the foundations can even have a beneficial effect in terms of reduction of the inertia forces - similar to the effect provided by an isolation system - before scour induces the wash out of the foundation. The final effect of scour on the seismic fragility of bridges depends on which effect dominates the response: degradation of the foundation or increased flexibility of the piers (Wang et al. 2014).

(a) (b)

Figure 2 Failure due to seismic actions – (a) slab unsitting, Japan 1964; (b) slab unsitting, USA 1989

Beyond the ones related to the bridge columns and to the foundations, seismic actions may activate other collapse mechanisms such as the unseating of bridge deck (see **Figure 2**) that could be affected by scour as well. The increased flexibility due to scour increases the maximum displacement of the deck induced by seismic actions thus increasing the probability of failure due to unseating of the deck. For masonry arch bridges, the main issues are related to the loss of equilibrium, rather than to the failure of the material for stresses higher than the ultimate resistance. Failure mechanisms induced by seismic actions can be categorized in this case in in- and out-of-plane mechanisms. The first ones can be local if only arches are subjected to the creation of a kinematic mechanisms or global when also piers are involved in the failure. The latter can be represented by the failure of the spring walls (local) or the global out-of-plane when slender piers are present. For masonry bridges situated in river beds, where a residual scour depth can be observed after the transient flooding phenomena, if any maintenance action is made, a worsening of the seismic response can be observed in case of earthquake occurrence.

3 Performance assessment through Structural Health Monitoring (SHM)

3.1 SHM for seismic actions

Structural health monitoring is an important tool in seismic areas for both rapid post-event assessment and also for a prompt assessment of damage before the structure reaches a critical state. Traditional methods of damage detection based on walk-through visual inspections or experimental techniques such as radiography or ultrasound require that the vicinity of damage is already known and easily accessible. These techniques may be costly, taking a long time to be performed and impractical to detect damage in long bridges. Furthermore they may fail if damage is not visibly evident. A promising alternative, able to provide information on the structural health consists in the use of responses recorded by digital accelerometers commonly installed on instrumented bridges. Several monitoring programs are running all over the world providing valuable data that are currently used for development and validation of damage identification methods, to assess the bridge performance, to provide real-time information for safety assessment in the aftermath of an extreme event (e.g. Mufti 2002, Smyth et al, 2003, Pezeshk at al 2004, Ko and Ni 2005, Celebi, 2006)

After an earthquake, basing on the responses retrieved from the sensors and applying appropriate damage detection techniques, a quick assessment of the damage state of the bridge can be obtained. In the last twenty years significant technological progress has been made in the area of commercially available innovative sensors (e.g. fiber optic sensors) capable of providing reliable information about loading, environmental effects and structural health. Long term monitoring networks of sensors installed on bridges may include several types of sensors (strain gauges, accelerometers, displacement transducers, GPS, fiber optic, tilt-meters, seismometer, video cameras and temperature sensors) and techniques for data fusion are needed in order to exploit efficiently the large amount of different data retrieved by the sensors network.

For seismic SHM in general three main categories of responses in terms of accelerations are sought (Celebi, 2006):

1. Response of the superstructure (deck, piers, towers) to retrieve the fundamental modal parameters and of the foundation (base of piers, abutments) to provide information on the soil-structure interaction and on the spatial variation of the ground motion.
2. Strong motion recorded in the free-field close to the structure
3. Ground failure arrays in the vicinity of the structure

If responses are made available and analyzed in real time by proper damage identification algorithms, the SHM system can be used to make informed decisions related to the performance of the bridge such as inspection or closing for maintenance or reparations. In the last twenty years several approaches have been proposed for damage identification based on the analysis of responses to vibrations recorded on the structures. In references Yan et al., 2007 and Fan et al. 2011 comprehensive state-of-the-art are reported. A very diffuse approach to the problem of damage detection is based on the analysis of changes of modal characteristics between the original (undamaged) state and the (possibly damaged) current state. Methods based on frequency changes can be reliably applied to detect damage but they are hardly able to give information about the location of damage. To this aim are more effective methods based on the analysis of changes of modal (Fan et al. 2011) or operational shapes (Limongelli 2014) or of their derivatives.

In addition to information on the global behavior such as increased flexibility due to damage or dependency of the modal parameters on the amplitude of the excitation, monitoring networks can give local information for example about possible malfunction of unintended-function of the bearings and of the connections (Fujino et al, 2008).

3.2 SHM for scour

The assessment of scour around critical bridge infrastructure to date has broadly been undertaken using visual inspections of the foundation condition by trained divers. Visual inspections typically involve rating a structure based on the perceived condition, whereby the rating denotes the necessity for intervention or the time before the next inspection. A variety of non-scour related defects are widely measured in this way such as cracking, water ingress, concrete spalling, etc., see (Irish National Roads Authority 2008a; Irish National Roads Authority 2008b). Visual inspections for scour aim to assess the nature and magnitude of a scour hole around a critical foundation element such as a pier or abutment foundation, and alert bridge managers when a scour hole is deemed to have surpassed some pre-determined threshold condition. Although no specific rating schemes are globally in existence for this, a number of individual rating systems have been implemented (by universities and local authorities) on various networks throughout Europe.

Table 1. Overview of some instruments capable of direct scour measurement

Type	System	Primary Operation	Advantages	Disadvantages
Single-use device	Float-Out Device	Buried at specific depth and floats out when scour reaches level	Simple operation method, indicates float-out by triggering a switch	Require expensive installation, have only a single use, can only detect scour at installation depth
Pulse/radar device	Time-Domain Reflectometry (TDR)	Uses changes in the dielectric permittivity constants between materials to determine a depth of scour at a particular location	Easy to read results – provides direct view of scour depth	Requires long probes installed in riverbed, measurement accuracy affected by channel temperature variation
Drive/Buried Rod Systems	Magnetic Sliding Collar	Physical probe positioned around a rod augured into the soil moves downward with scour progression closing magnetic switches	Easy to read data, direct measurement of scour	Only detects scour condition at location of sensor, may miss global effect
Sound-wave devices	Sonic Fathometer	Fixed to bridge pier, these emit sonic pulses to continuously establish the water-sediment interface	Continuous scour measurement in vicinity of bridge pier with easy to read data output	Susceptible to measurement error from entrained air in highly turbulent flow

These surveys also look for secondary damage as a result of scour such as cracking of foundations and visible damage to piles with compromised lateral stability among others. While relatively straightforward to undertake, there are a number of critical drawbacks to these types of surveys, most notably the subjective nature of the information gathered and the discrete nature of the assessment process. In addition, unlike typical bridge damage such as crack formation, scour can occur quite rapidly under increased flow conditions (as during a flood) and it is usually not possible to undertake diving inspections during these critical timeframes. To overcome limitations in this assessment process, significant efforts have been made to design remote systems capable of relaying information about scour reducing the need for manual intervention. These systems can be broadly categorized as follows (Prendergast & Gavin 2014): single-use devices (NCHRP 2009; Briaud et al. 2011), pulse or radar devices (Forde et al. 1999; Yu 2009; Yankielun & Zabilansky 1999), buried or driven rod systems (NCHRP 2009; De Falco & Mele 2002; Zarafshan et al. 2012; Fisher et al. 2013), sound-wave devices (Nassif et al. 2002; Fisher et al. 2013; Anderson et al. 2007), fiber-Bragg grating devices (Lin et al. 2006; May et al. 2002) and electrical conductivity devices (Anderson et al. 2007). Table 1 gives an overview of the function and detection methodology of a number of types of instrument available.

In recent years, direct measurement of the response features of a bridge to varying scour conditions has come to the fore as a potentially more reliable indicator of the performance. The compromised stability of a bridge foundation due to adverse scour hole formation gives rise to a change in the response features of a bridge such as increased pier tilting, pier settlement, changes to dynamic characteristics, increased strain at deck level, crack formation due to differential settlement among other indicators. Primarily, the change in support conditions gives rise to a change in the dynamic characteristics (Doebling et al. 1996; Prendergast et al. 2016b; Prendergast et al. 2016a; Foti & Sabia 2011; Elsaid & Seracino 2014; Briaud et al. 2011; Prendergast et al. 2013), and this may become a primary indicator of performance. Vibration-based scour detection is a significant advance over the instruments discussed in Table 1, in that it focuses on measuring the response of the structure itself to the formation of scour in lieu of inferring the bridge stability from a measured scour depth. This is primarily due to the fact that a scour depth measuring instrument may miss the global (true) effect of scour due to poor positioning and natural channel evolution. By monitoring the change in a dynamic parameter of interest (natural frequency, damping, mode shapes, mode shape curvature and flexibility-based deflection among others) perhaps before and after a major flood event, it may be possible to detect a loss in structural performance arising due to compromised foundation capacity as a result of scour. These methodologies may become increasingly used in national bridge inspection guidelines, and in terms of scour, are much more applicable than visual rating-based inspections.

3.3 SHM for seismic actions and scour: future developments

For the time being, to the knowledge of the authors, integrated monitoring systems for bridges under seismic actions and scour are still in the research phase. Based on the current applications in the two different fields a common denominator is found which consists in the use of vibration based techniques for the identification of changes in modal parameters. Both scour and seismic actions may induce variation of modal frequencies that can be detected using accelerometers installed on the bridge. The availability of a network of accelerometers installed on the deck of the bridge could probably allow localizing variations of the deformed shapes induced by the increased flexibility of the piers due to scour or to a seismic damage. The more refined inverse methods based on a calibrated model of the bridge, could be the tool to assess the severity of damage e.g. the depth of the scour or the amount of stiffness reduction induced by an earthquake.

4 Performance assessment according to Design Codes

4.1 Methods for seismic assessment

Modern design codes are based on Performance Based Design (PBD) method requiring the structure to achieve an expected level of performance. This method is formalization of the objectives of designing structures to withstand minor or frequent earthquake shaking without damage, moderate levels of shaking with only non-structural damage and severe shaking without collapse and a threat to life safety (ATC, 1978). PBD entered into the practical engineering filed at the beginning of the 20th century, in New Zealand. The new philosophy is to design structures in such a way that a specified performance limit state is obtained under a specific level of seismic intensity (Calvi G.M., et al, 2008). The fundamental component of PBD is nonlinear dynamic analysis where an attempt is made to capture the real behavior of the structure by explicitly modeling and evaluating post-yield ductility and energy dissipation when subjected to actual earthquake ground motions. PBD can be force or displacement based. The main difference between the two approaches is that Force Based Design (FBD) uses displacements to perform a final check of the structural performance while Displacement Based Design (DBD) uses displacements (or strains) as the target performance.

It should be mentioned that DBD procedures are now well-established for buildings (Kappos 2010), however for bridges, besides its numerous advantages, the procedure still has some disadvantages. Their application is mainly limited to bridges that can be modeled by single degree of freedom for calculating seismic demand and its applicability is in the scope of preliminary design. Reasoning behind this lies in the importance of the higher modes in the transverse response of bridges even of some relatively short ones (Paraskeva and

Kappos, 2010), which complicates the proper assessment of the displaced shape of the bridge and the target displacement. In this case not one target displacement is required but a target displacement profile.

Current design codes are based on a FBD utilizing the behavior factor *q*, to reduce expected elastic levels of base shear strength to acceptable design levels taking into account the ductile behavior of the structure. In many European countries, up to the 20th century bridges were designed without taking into account any seismic actions and only in the last 20-30 years codes based on seismic FBD were enforced in regions with medium to high seismicity. The Italian Standard Code (NTC 2008) and European Standard, Eurocode 8 (CEN,2004) define general criteria for the seismic assessment of common bridge types, with special focus on simply supported – continuous girder bridges. Scaling factors for uniform hazard spectra are proposed with reference to each bridge type and indications on the most suitable type of structural analysis are provided. Prescriptions for the assessment are provided, evidencing how the engineer has to check the compliance to the capacity design rules, the dimensions of the plastic hinges and additional criteria related to the execution of retrofit intervention, e.g. the insertion of isolators or dissipation devices. At research level, other probabilistically-based indicators can be considered for the assessment of bridge seismic response, like seismic vulnerability (Zanini et al. 2013), through the construction of fragility functions and resilience indexes.

4.2 Methods for scour assessment

The performance of bridges under scoured conditions is dependent on the nature of the situation, such as type of bridge, nature of traffic, foundation type, redundancy, and others. New bridges are specifically designed with scour in mind whereas existing or legacy-era bridges may be subjected to retrofitting to account for scour occurrence. Typically, new bridges are designed with a design scour depth incorporated. A variety of methods exist to calculate a design scour depth based on a given flow condition, see (Kirby et al. 2015; May et al. 2002). These broadly empirical methods, an example of which is the Colorado State University (CSU) method (Arneson et al. 2012), derive a design scour depth based on the geometry of the pier, the depth of flow upstream, the flow velocity, the flow angle to the bridge, and the nature of the bed material among other variables. The bridge geometric parameters are then optimized to reduce the design scour depth below some threshold value, such as 2.3 times the bridge pier diameter for example. In Italy few prescriptions are provided by the Italian Code for Constructions (NTC 2008). When designing new river bridges, it is necessary to provide a hydrological report and an associated hydraulic one mainly focused on the design solutions adopted against scouring phenomena. The Code suggests to avoid, where possible, to design bridges with piers placed in the river bed, indicating when necessary a distance between consecutive piers of at least 40 m. At European level, Eurocode 1 in Section 4.9 suggests how to take into account actions without giving specific formulations for the assessment of scour depths.

4.3 Methods for joint seismic and scour assessment: current research trends

Fragility curves describe the relationship between a certain intensity measure and the probability of failure. Ter Huene (2014) describes the development of fragility curves for bridge scour, the form is shown in **Figure 3**.

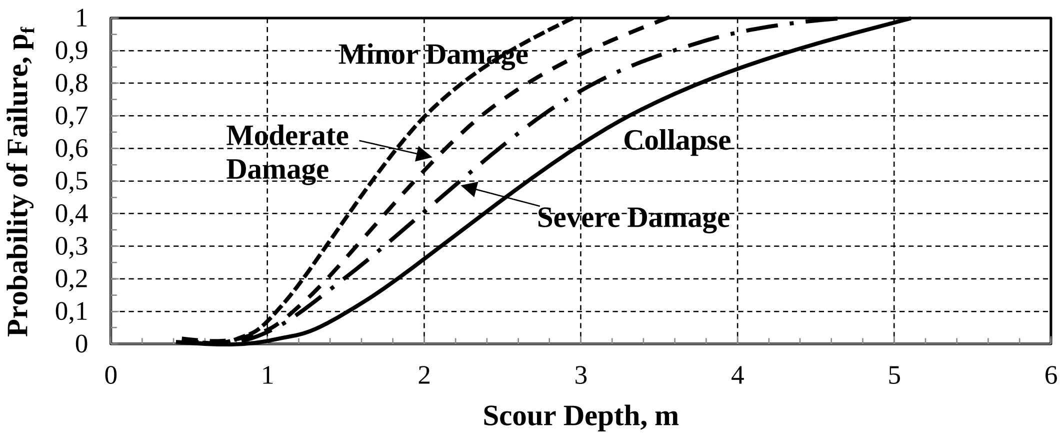


Figure 3 Development of fragility curves for the effect of scour on bridges

The approach is particularly useful for considering separate damage (limit states), in this case the impact ranging from minor damage to complete collapse to the probability of failure of a bridge due to scour depths of up to 6m. Whilst the approach is commonly applied to single hazard analysis, and is particularly useful in considering the effect of hazards such as earthquakes, it has only recently been applied to consider joint multiple hazards occurring simultaneously or with some small time-lag.

The importance of analyzing multiple-hazards, both natural and man-made is widely recognized. An approach that calculated the reliability indices of a bridge to combined effects from earthquakes, wind, scour and vessel collision over some time interval ΔT was developed by Ghosn et al. (2003). Prasad and Banerjee (2013) note that flood induced scour is not in itself a load event, rather it is the result of a flood event and its effect is to amplify the impact of other load events on the bridge performance. Therefore, superposition of individual load events is not a reliable means of assessing impact; instead a hazard-specific analysis is required to consider the bridge performance. The authors performed assessments of the effects of combined scour and earthquakes hazards on the dynamic response of four reinforced concrete bridges with spans. The bridge supports were piled and the spans varied from 2 to 5 and the response of the pile groups used at support locations were modelled using p-y springs acting on a single equivalent pile. Scour was modelled by incrementally removing springs. Five damage states were considered, ranging from no damage to complete collapse, the boundaries between damage states were based on the displacement ductility. Modelling was performed based on the assumption that some flood induced scour precedes an earthquake event. The study found that non-linear changes of the seismic fragility characteristics of the bridge occurred as scour progressed. Changes were rapid during the early stages of scour and for the systems considered became negligible for scour depths greater than 3m. The diameter of the pile foundation was seen to have an effect on the response with larger piles mitigating the effects of scour, whilst the bridge length was also shown to impact the fragility response.

Wang et al. (2014) also modelled the effect of earthquakes following a scour event on the seismic response of three forms of reinforced concrete bridges; single frame box-girder, a three span simply supported girder and a three-span continuous girder bridge. Finite element models were developed for each of the bridges and scour was induced by removal of lateral (p-y) and axial (t-z) soil springs. The analyses showed that the periods of the first few vibration modes increased with scour for all bridge types considered. The degree of change in natural frequency was dependent on the bridge type, with the three-span simply supported structure being more sensitive to change in scour depth. Interestingly when considering component analysis of the bridge columns, in some cases (for the three-span bridges) scour had a beneficial effect on the column response due to the much longer vibration periods that result.

Gehl and D'Ayala (2015) propose a component based approach to multi-hazard fragility analysis of road bridges. Damage dependent fragility curves are derived at a component level and a Bayesian Network approach is used to assemble these component level curves into a system level assessment. As a result multi-variation fragility functions can be derived wherein each input variable represents an intensity measure for a specific hazard. The hazards considered included earthquakes, ground failures and fluvial floods.

5 Discussion and conclusions

In recent years failures of bridges have increased due to natural disasters on one hand and to the lack of adequate monitoring and preventive maintenance actions on the other. The combined action of different types of hazard, such as scour and seismic actions, may have significantly higher impact damage on bridges with respect to the separate actions and can even lead to service limit states or partial or total collapse. Scour may exacerbate the effect of the seismic actions causing the loss of lateral support and of axial friction from the soil at the level of the bridge foundations. This may lead to the occurrence of the collapse of the foundation before the bending failure of the columns thus altering the hierarchy of failures required by capacity design.

Structural health monitoring has been identified as an effective tool for performance assessment of bridges under seismic actions and several monitoring and NDT techniques are currently used to investigate scour. Vibration-based monitoring systems appear promising for the joint monitoring of seismic and scour effects but at the time being a very limited number of applications exist.

Also, design codes define general criteria for the seismic assessment of common bridge types and some prescriptions are provided regarding protection from scour but design procedure for the joint action of the two actions are not yet available.

Recently several researchers have tackled the problem through numerical analysis focusing on the use of fragility curves to study the seismic behavior of bridges degraded by scour. Results show that there is a strong impact of scour on the seismic behavior of bridges thus, considering the two sources of hazard separately, may underestimate the effect of their combined action. Furthermore the fragility curves for seismic hazard are affected non-linearly by the amount of scour and the combined effect of the two actions may significantly depend on the structural type of the bridge. This points out the urgent need of further investigations in the field of both structural health monitoring techniques and of design procedures effective in taking into account the joint hazard of seismic actions and scour and allowing the computation of performance indicators able to properly describe the performance of a bridge with respect to the combination of the two actions.

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The impact of different maintenance policies on owners costs: Case Studies from Croatia and the Netherlands

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.4

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Abstract. Optimisation of maintenance planning is an important part of bridge management. The efficiency of condition assessment process is directly influencing the choice of repair and maintenance technique, i.e. measures which have to be undertaken. This paper examines the effect of two opposing maintenance strategies, corrective and preventive maintenance. The condition rating and planned maintenance costs for structural elements of bridges at different ages are compared. In case studies from Croatia and the Netherlands, data from previously performed inspections was compiled in order to correlate age, condition and costs for structural elements. The case studies have been used to assess the impact of different maintenance policies (corrective vs. preventive) on the occurred maintenance costs.

Keywords: bridge management system, condition assessment, maintenance options, corrective vs. preventive approach, costs.

1 Introduction

Functional and serviceable road infrastructure presents one of the most important predispositions for economic growth of countries around the world. One of the most important and critical parts of road infrastructure are bridges which present a vital link in any roadway network. It is estimated that the ratio of expenses per route km of bridges is 10 times the average expense per route km of roads. The length of bridges compared to the whole length of road networks is only approximately 2% but at the same time they present 30% value of the whole network (PIARC,1999). During the last decade a new network management philosophy, Asset Management has emerged, which puts the customer - the traffic user - into the centre, and, at the same time, methods to allocate the socio-economic optimal amount of money to each specific asset (roads, bridges, tunnels, lighting, signs, guard rails, etc.). When these developments and statistics are taken into consideration it is easy to understand why, in the past few decades, an increasing number of Bridge Management Systems (BMS) and life cycle maintenance models were developed like for example Branco & Brito, Frangopol’s and Rijkswaterstaat’s model (Kaneuji et al. 2006, Airaksinen 2006, Astudillo Pastor et al. 2000, Noortwijk J.M. et al. 2004).

A Bridge Management System (BMS) is defined as a rational and systematic approach to organizing and carrying out all the activities related to managing a network of bridges, including optimizing the selection of maintenance, repair and rehabilitation actions in order to maximize the benefits while minimizing life cycle costs (Hudson et al., 1992). Mainly there are three aspects addressed by BMSs found in literature: condition assessment, modelling future degradation and optimisation of maintenance, repair and rehabilitation decisions and actions. The heart of a BMS however is a database derived from the regular inspection and maintenance activities. The integrity of a BMS is directly related to the quality and accuracy of the bridge inventory and physical condition data obtained through inspections (AASHTO, 1994). Additional information such as the bridge name, location, and construction date are stored in the system.

The BMS can be used as a tool to allow bridge managers to be fully informed about the bridge stock under their control so that they can make informed and optimal decisions about future maintenance and repair activities.

2 Maintenance strategies and maintenance planning

Any BMS is used as a tool as part of a broader bridge management strategy. Depending on the bridge inventory, constraints and societal demands, a road agency will lay out a strategy aiming to achieve optimal results within the limitations posed by the constraints.

Maintenance decisions are based on the deterioration level of an object. Degradation or deterioration of infrastructure is one of the main factor that triggers maintenance. With regard to maintenance planning, two approaches may be applied, a corrective versus a preventive maintenance strategy. In a corrective strategy, maintenance is planned after a certain amount of damage has occurred, whereas a preventive approach aims to plan maintenance so as to prevent unaccepted damages from occurring. In practice, often a combination of both strategies is followed, aiming for an optimal balance between costs and performance, see Figure 1. A good understanding of bridge condition and future degradation is necessary for this optimisation.

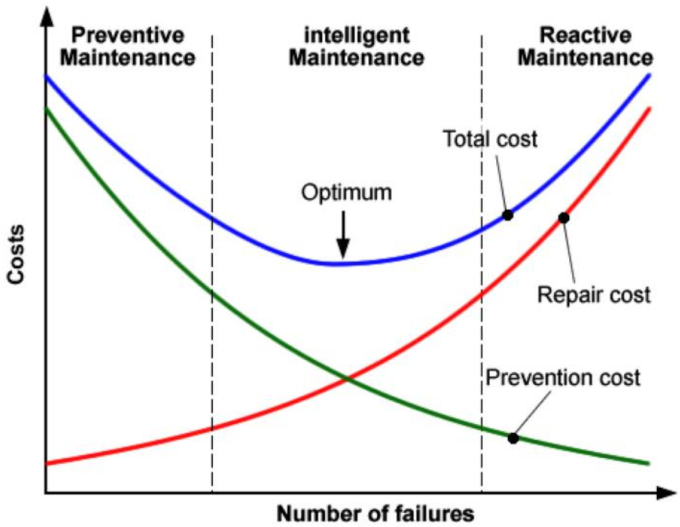

Figure 1: Optimization of Maintenance Costs

Much literature can be found related to condition assessment and modelling future degradation but only few studies have been directed at optimizing the decisions to the maintenance or repair of bridges, especially on the material level (Elbehairy 2007). Frangopol et al. (2001) stated that additional research is required in order to develop a better life cycle so that the costs and benefits can be quantified. Moreover, costs and effects of the interventions should be assessed more accurately based on real data (Bocchini et al., 2010). Best practices call for including all costs incurring through the life of a bridge.

2.1 Maintenance plan

A maintenance plan should be based on a decision making system which enables choosing the best repair option considering multiple performance aspects, for example safety, durability, functionality and economy. A first maintenance plan could be made up based on the design of the structure and be periodically updated during the service life based on inspection and maintenance data. Figure 2 presents a general framework for the life cycle of concrete structures, where the strategy of interventions (maintenance, repair, rehabilitation) is consisting of:

1. Defining the set of requirements to be fulfilled by the structure;
2. Performing a technical and economical analysis through the use of requirements and performance indicators, for example by a life-cycle cost analysis (LCCA); and
3. Selecting among the options identified the optimal repair or strengthening method.

Finally, the repair has to be executed and the maintenance needs of the repaired structure have to be established. Although it seems that a life-cycle cost analysis, or LCCA, would be the most suitable methodology for comparing different repair options, a LCCA is very often not feasible because of lack of experience or of reliable data. (Andrade & Martinez, 2009; CONREPNET 2007, Andrade & Izquierdo 2005; Frangopol et al. 2001). The process of choosing the best repair option is often expert based.

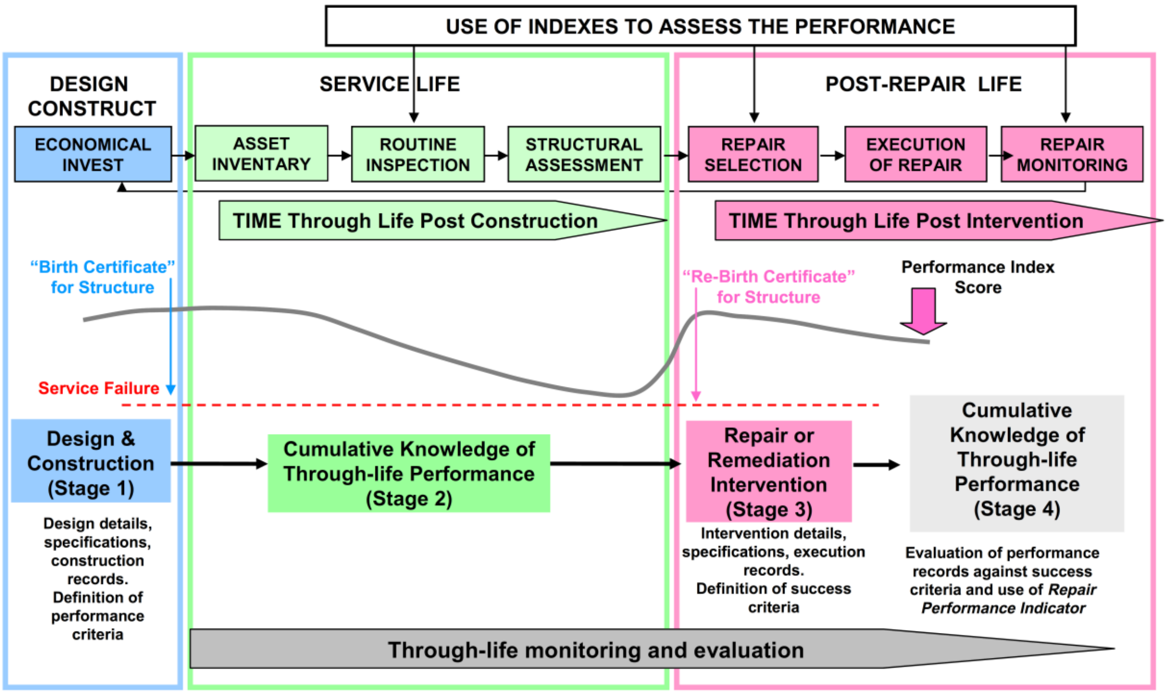

Figure 2: General framework for concrete structure maintenance (Andrade & Martinez, 2009)

2.2 Maintenance options

Common maintenance measures do not need structural design and are generic. They are correlated to the type of elements when these concern the bridge's equipment (eg. joints, bearings) or the type of material when these concern the other elements. More important maintenance measures, like external pre-stress, are correlated to the structural design of the bridge. The European standard EN 1504 gives guidelines for the choice of repair materials and systems that are appropriate for rehabilitation and maintenance of concrete structures. These standards describe the main points of rehabilitation of a damaged concrete structure:

- Assessment of the registered state of a concrete structure
- Determination of the courses of damage
- Determination of the objective of the rehabilitation of a damaged concrete structure
- Choice of relevant principles for rehabilitation of a damaged concrete structure
- Choice of methods for rehabilitation of a damaged concrete structure
- Definition of the properties for repair materials and systems for rehabilitation of a damaged concrete structure or its members
- Specification of requirements for the maintenance that should always follow rehabilitation of a damaged concrete structure or its members.

Table 1: Maintenance strategies

No	**Options for protection and repair**
1	Do nothing for a certain time Postpone the repair work, but monitor the degradation process
2	Re-analysis of structural capacity, possibly leading to downgrading of the function of the concrete structure
3	Prevention or reduction of further deterioration, without improvement of the concrete structure
4	Improvement, strengthening or refurbishment of all or part of the concrete structure
5	Reconstruction of part or all of the concrete structure
6	Demolition of all or part of the concrete structure

According to EN1504-9, six different maintenance strategies are possible. Table 1 indicates the strategies that should be evaluated to decide upon the right maintenance for the structure. It should be emphasised that more than one strategy may be relevant and also that each structural part should be given a separate evaluation. Selection of right maintenance strategy and repair method should at least be based on owners requirements, as built documentation, evaluation of the bearing capacity, cost benefit analyses and a detailed condition assessment.

3 Case study – Croatia

Croatia has started implementing a bridge management system at Croatian Roads and Highway agency about 20 years ago. Input data for bridge condition are based on visual inspections conducted by specially trained engineers, using procedures and aids defined in the management system, HRMOS (Tenzera et al., 2012). Bridge condition is rated through interpretation of damages that have been identified and registered, then these ratings are combined into a single rating valid for the entire structure. Bridge maintenance activities and costs should be planned by setting priorities and anticipating the future life of the structure. Unfortunately, the introduction of HRMOS did not ensure that the activities related to bridge maintenance are carried out regularly, but on an 'as-needed' basis with inspection and testing undertaken only when structural damage becomes self-evident. This is an example of corrective maintenance policy. To illustrate the grave consequences of neglecting bridge maintenance activities, several cases in the past have shown that if we choose to act only when damage of a bridge structure becomes self-evident, the repairs are not only expensive but very difficult to perform. (Radic et al. 2007, Stipanovic et al. 2008).

In Figure 3 a relation has been established between degradation category and direct cost of repair method, based on the historical data from Croatian case studies (Skaric Palic et al. 2008, Stipanovic Oslakovic et al. 2008). For example cost for category IV and V is the same but risks are different because category V means that stability of element or the whole structure is endangered. The methods and principles used are some of repair methods from standard EN 1504-9:2001. Direct costs included in the analysis present current costs on Croatian market and were collected directly from manufacturers and contractors.

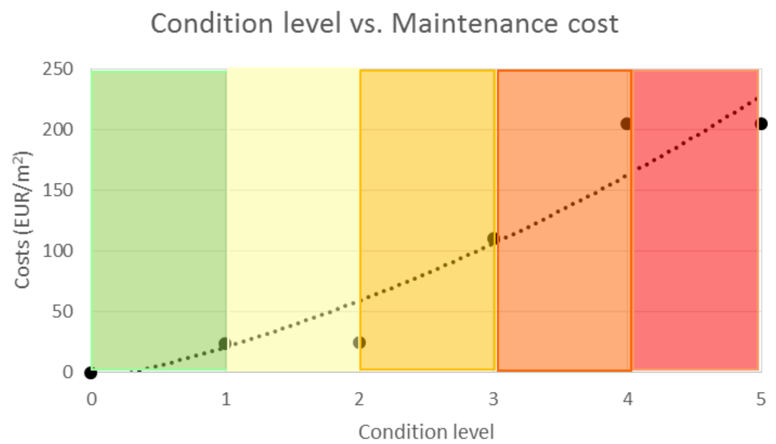

Figure 3: relation between degradation category and direct costs of the repair method

A case study was performed on 12 bridges (viaducts) inspected in 1998 and in 2010. The analysed viaducts are listed in Table 2. Only routine maintenance was performed on viaducts from the year of construction until the final inspection in year 2010. In 1998 and 2010 visual inspection were performed on all structural elements (bridge equipment, substructure and superstructure elements). All defects (delamination, spalling, segregation, corrosion of reinforcement, wet areas, mechanical defects, cracks) were recorded and categorized accordingly. Upon the performed condition assessment a list of repair works needed to bring the structural elements (substructure and superstructure in this analysis) back into their original condition (when constructed) was made with direct costs of all activities.

In Table 2 direct costs of repairs performed in 1998 and 2010 are presented. They are compared and also transformed into the unit value €/m^2. The averaged value of direct repair cost in relation to the age of the bridge is presented in Figure 4. It is obvious that after app. 17 years of usage direct repair costs enhance rapidly.

Table 2: Analysed viaducts from Croatian case study

No	Year of construction	Length (m)	Technical characteristics	Bridge area (m^2)	Total repair cost in year 1998 (€/m^2)	Total repair cost in year 2010 (€/m^2)
1	1988	186 (6 spans)	4 precast prestressed “T” girders (H=1,70m, L=29,85m) + continuity slab 24 cm (precast slabs 6 cm + in-situ concrete 18 cm) • Cantilever head beam • 2xO column • Precast curbs and cornices	1971,6	3,7	149,0
2	1988	189 (6 spans)		2003,4	3,4	31,9
3	1988	186 (6 spans)		1971,6	3,2	68,0
4	1988	186 (6 spans)		1971,6	4,5	15,2
5	1988	127 (4 spans)		1346,2	3,4	16,0
6	1988	480 (16 spans)	4 precast prestressed “I” girders (H=1,70m, L=29,85m) + continuity slab 24 cm (precast slabs 6 cm + in-situ concrete 18 cm) • Cantilever head beam • ☐ column • Precast curbs and cornice	5088,0	6,9	27,1
7	1981	377 (14 spans)	6 precast prestressed box girders (H=1,50m, L=29,10m) • Cantilever head beam 2xO column	4410,9	0,2	49,8
8	1981	74 (4 spans)	6 precast prestressed box girders (H=1,10m, L=17,30m) • Cantilever head beam • ☐ column	843,6	0,5	37,8
9	1981	240 (8 spans)	4 precast prestressed “I” girders (H=1,86m, L=29,75m) + continuity precast slab 17 cm • Cantilever head beam • Octagonal column • Monolithic pedestrian ways	2760,0	18,4	152,6
10	1981	122 (6 spans)	4 precast prestressed “I” girders + continuity precast slab • Cantilever head beam • 2x☐ column • Precast curbs and cornices	1725,0	2,1	19,6
11	1981	150 (5 spans)	4 precast prestressed “I” girders (H=1,86m, L=29,75m) + continuity precast slab 17 cm • Cantilever head beam • Octagonal column • Monolithic pedestrian ways	1342	23,9	144,2
12	1981	50 (1 span)	4 precast prestressed “T” girders (H=1,70m, L=29,85m) + monolithic continuity slab 17 cm Monolithic pedestrian ways	575,0	24,3	159,3

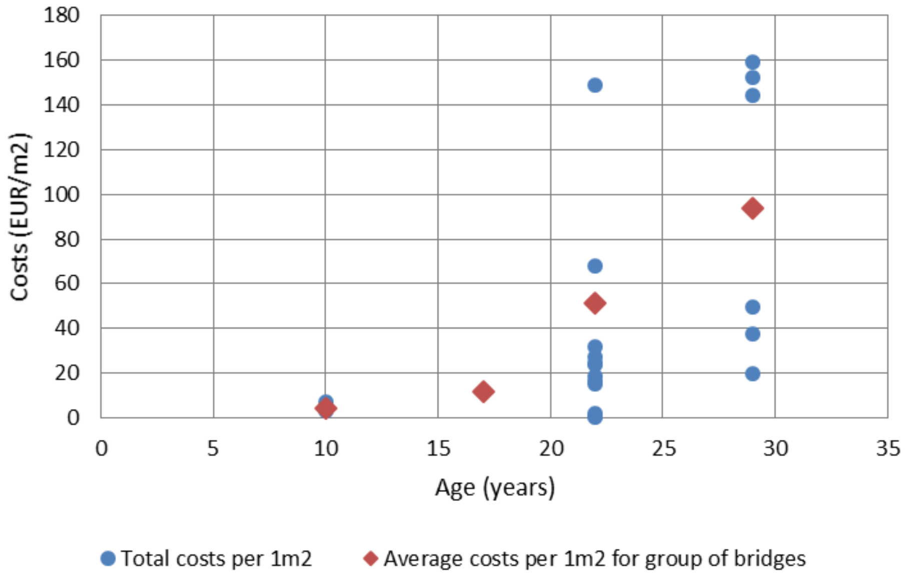

Figure 4: Direct costs established on a case study of 12 viaducts

4 Case study – the Netherlands

The maintenance strategy in the Netherlands can be described as a preventive approach. Bridges are inspected on a regular basis and based on the information collected a risks are assessed and maintenance tasks planned to manage and mitigate risks. Maintenance is planned so as to prevent the occurrence of unaccepted risks, the occurrence of unwanted situations is thus prevented. A description of the inspection practice can be found in Bakker et.al. (2012).

During inspections a maintenance strategy is used which states general requirements for different (structural) elements (eg. joints, superstructure, bearings) and materials (eg. concrete, steel). Suggested maintenance options and associated costs are also included in the maintenance strategy. The inspection process is further supported by the DISK Bridge Management System. To rate the condition of bridge elements, a 7 point scale is used (table 4). When an element reaches or approaches a condition rating of 3 a maintenance tasks is to be planned. Note that also at condition 3 requirements are still met. Maintenance tasks are aimed to be preventive.

Table 4. Condition rating in the Netherlands

Condition rating	Description
0	excellent, as new
1	very good
2	good
3	average, risk of meeting requirements
4	under average, requirements are not meet
5	poor
6	bad

As a case study, 24 randomly chosen bridges with different periods of construction were analysed (table 5), all part of the main highway network. For each bridge, the results of two inspections (with an interval of circa 6 years) were compared for the main structural elements, with the first inspection between 2008 and 2011 and the second inspection 2014 and 2016.

Table 5. Characteristics of the 24 bridges in the case study, incl. costs and condition rating for from two inspections

Bridge No.	Year of Construction	Length (m)	Width (m)	Deck area (m^2)	Object type	Inspection #1 Costs (€/m^2)	Inspection #1 Condition Rating	Inspection #2 Costs (€/m^2)	Inspection #2 Condition Rating
1	1967	49	15	732	Viaduct in motorway	0,97%	3	0,26%	3
2	1979	44	15	641	Viaduct in motorway	0,00%	2	0,00%	2
3	1979	47	15	692	Viaduct in motorway	0,00%	2	0,00%	2
4	1975	48	19	908	Viaduct in motorway	0,00%	2	0,08%	4
5	1975	99	15	1.496	Bridge	0,00%	2	0,00%	2
6	1987	40	15	623	Viaduct in motorway	0,00%	3	0,00%	2
7	1995	42	15	625	Viaduct in motorway	0,00%	2	0,00%	2
8	1987	76	27	2.009	Bridge	0,00%	2	0,00%	2
9	1987	119	14	1.637	Bridge	0,00%	2	0,00%	2
10	1979	29	18	535	Bridge	0,00%	2	0,00%	3
11	1979	66	20	1.344	Viaduct in motorway	0,00%	2	0,00%	2
12	1979	66	21	1.408	Bridge	0,00%	2	0,00%	2
13	1995	43	23	970	Viaduct in motorway	0,00%	2	0,33%	2
14	1995	166	14	2.403	Viaduct in motorway	0,00%	2	0,24%	2
15	1995	32	23	725	Bridge	0,00%	1	0,00%	2
16	1967	36	21	764	Viaduct in motorway	0,00%	2	0,06%	3
17	1979	22	23	502	Bridge	0,00%	3	0,11%	3
18	1967	37	17	611	Bridge	0,76%	2	0,38%	3
19	1967	112	18	1.988	Bridge	0,00%	2	0,00%	2
20	1967	49	13	653	Viaduct in motorway	0,00%	2	0,00%	2
21	1967	44	15	678	Viaduct in motorway	0,00%	2	1,13%	3
22	1967	66	17	1.093	Viaduct in motorway	0,00%	2	0,00%	2
23	1967	35	19	660	Viaduct in motorway		2	0,00%	2
24	1967	33	17	564	Viaduct in motorway		3	0,65%	3

First, the condition rating and costs for planned maintenance tasks for the main structural elements were compared for two inspections for each bridge. Note that for most bridges the condition indicated a rather good condition (rating of 1 or 2) in which cases no maintenance tasks were planned. For the last inspection, 8 out of 24 bridges showed a condition worse than 2, for 9 bridges maintenance tasks were planned (table 5).

Secondly, the relation between the age of the bridge at the moment of inspection and both condition and costs for maintenance was analysed. The age of the bridges at the moment of inspection ranged between 13 and 49 years. Most maintenance was planned for bridges constructed over 36 years before the moment of inspection. The exception being three bridges were maintenance was planned after 20 years.

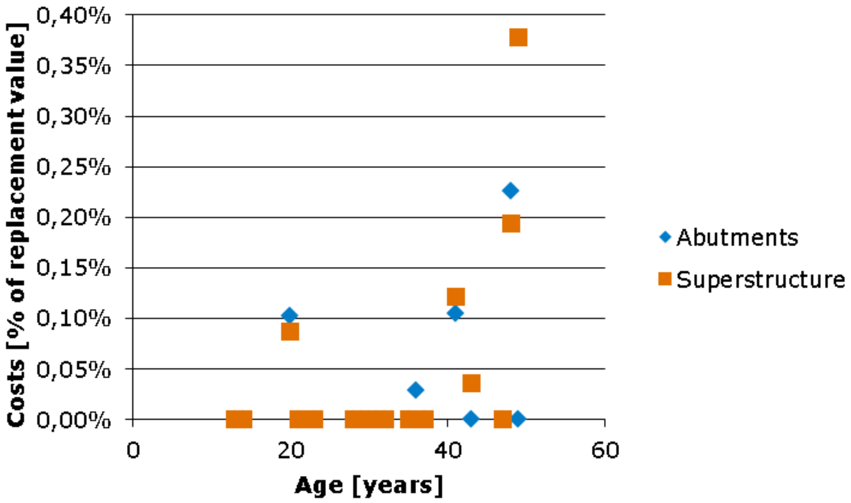

Figure 5. Costs of maintenance tasks relative to bridge age

5 Discussion and conclusion

The efficiency of condition assessment process is directly influencing the choice of repair and maintenance technique, i.e. measures which have to be undertaken. This paper examines the effect of two opposing maintenance strategies, corrective and preventive maintenance, from Croatia and the Netherlands respectively. The condition rating and planned maintenance costs for structural elements of bridges at different ages are compared. In case studies from Croatia and the Netherlands, data from previously performed inspections was compiled in order to correlate age, condition and costs for structural elements. Although both cases are limited to only a small number of bridges, it can be argued that there is a relation between the age of the structure and maintenance costs. Also, the increase in maintenance costs appears to be later in the case of the Netherlands than in the Croatian practice. Finally, based on these results if can be concluded that a preventive approach to maintenance planning supported by an inspection procedure will, in the longer term, be more efficient.

Acknowledgements

This article is based upon the work from COST Action TU1406, Quality specifications for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology).

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Performance indicators for bridges exposed to a flooding hazard

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.5

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Abstract. The data on performance indicators from relevant national bridge inspection documents have been collected in a survey within the COST TU1406 action. Among this data, there are essential information on roadway bridge management practice in Europe related to a flooding hazard, that waits to be identified. This is one of the main tasks of the work group 3 which goal is to facilitate establishment of quality control plans for girder, frame and arch bridges. The review of the collected data has been performed to reveal the performance indicators for the cases of flooding and related scour at bridge substructures. The essential information was extracted and complemented with the relevant information necessary to establish relationships between performance indicators and performance goals. Here, the vulnerability assessment is suggested as a convenient approach as it accounts both for the probability of a bridge failure and related consequences. In the approach, the emphasis is to account the resistance of a bridge to scour, which is not thoroughly considered in the current practice. This is one of the main aspects that should be elaborated and included in the structure of the future QC plans for bridges exposed to flooding hazards.

Keywords: survey of performance indicators, flooding hazard, scour, vulnerability, resistance of a bridge to scour, quality control plans

1 The status of the COST TU1406 and its Work Group 3

The relevant information on bridge performance indicators (PI-s), their thresholds and related goals have been recently collected in the survey of the Work Group 1 (WG1) within the Cost action TU1406. In the scope of the survey were nationally applied bridge inspection manuals/guidelines in Europe and pertinent research papers. The gathered PI-s from 29 countries have been collected in an Excel database, homogenized into ten groups, and the Glossary of country specific terms was provided (Strauss & Mandić-Ivanković, 2016).

At the recent action meeting in Delft, it was underlined that there are nine key performance indicators (KPI) which are the most relevant for this project: Cost, Availability, Reliability, Safety, Maintainability, Economy, Security, Health, Politics. The future task of the Work Group 2 (WG2) is establishing of connections between the collected information on PI and the KPI-s while the Work Group 3 (WG3) works on elaboration of quality control (QC) plans for most common bridge types: arch, girder and frame, affected by various interceptable (i.e. slow) and non-interceptable (i.e. sudden) processes.

One of the main tasks within the WG3 (Task no. 4) is to investigate and account the dynamics and uncertainty of the sudden processes, focusing on extreme flooding events that may significantly affect a bridge performance. Among the collected data in the survey there are information on bridge management (BM) practice related to a flooding hazard, which wait to be pointed out and clarified. Although hazards were not the main topic of the survey, almost every country provided information on appraisal of a flooding impact, namely scour & erosion at bridge substructures. As discussed in (Tanasic & Hajdin, 2016), the most of the approaches that account this hazard impact on transportation infrastructure in the current BM practice are qualitative and do not provide reliable/optimal solutions for mitigating the factual threat of a bridge failure. So far, a quantitative methodology was suggested, and here the core of this process is the vulnerability assessment. In order to conduct this type of assessment, a minimum set of data i.e. PI-s and specific observations/findings is necessary.

In this paper, the surveyed data are reviewed in line with the COST WG3 framework (Figure 1) and the additional data which is necessary for evaluating of the relevant PI and its connection to the KPI-s are discussed. The principal topic in structuring of adequate QC plans, the consideration of a bridge resistance in a flooding event, is emphasized.

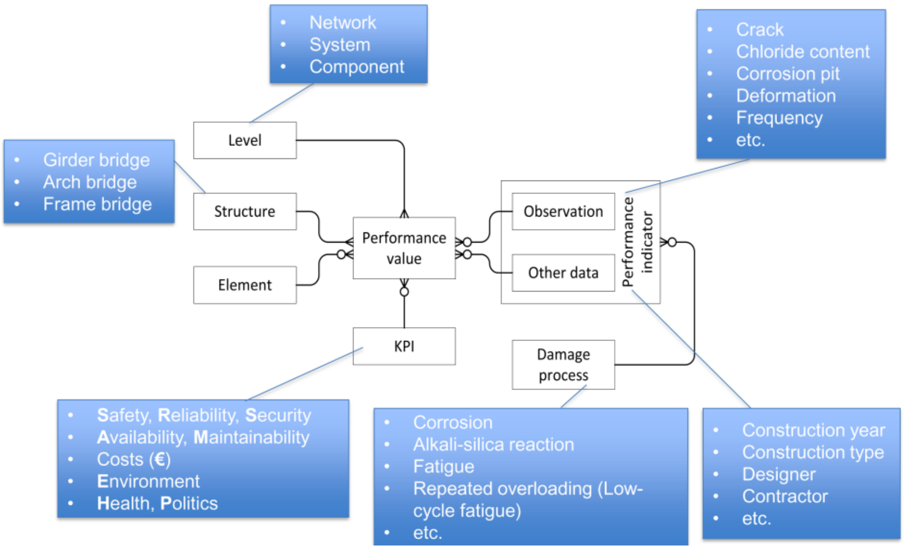

Figure 1 The entity relationship diagram of WG3 approach (Hajdin, 2016)

2 Review of the survey - the information related to flooding hazard & scour

The aim of the COST survey was to collect as much as information possible from the relevant national documents - bridge inspection manuals & guidelines. However, it is a fact that when it comes to application of procedures & actions for timely mitigation of hazard related consequences, not much concrete information can be found in the documents. This has been confirmed by the review of the database information which was here performed, and in the following text the main findings are presented and discussed.

The scour as the main culprit of bridge failures in a flooding event have been mentioned in almost every national guideline. The similar term reported in Glossary is erosion near piers and abutments, while under the group *performance criteria*, the terms that are related to a flooding are: collapse, river bed deterioration & aggradation and special main underwater inspection. Here, only a general piece of information is given thus these terms should be further investigated from the provided references. Also, vulnerability to natural hazards is mentioned by the authors of this paper as a performance indicator, and its application in the scope of the action will be discussed in the next paragraphs.

Interestingly, the term flood is only mentioned in the survey by a few countries as well as the term sudden event (Greece, Germany, Portugal, Croatia, Ireland). With exception of Sweden, France, the Netherlands, Hungary, Finland and Poland, all other countries reported that scour/erosion is considered in their guidelines. Eighteen countries indicated that there is visual inspection performed to confirm the adverse effect of flowing water on pier & abutment foundations or embankments. Here, specific classes/indexes (e.g. from 1 to 5) are used to grade scour criticality/impact, but the specific information on actual grading was not provided (i.e. no reference is given). Although these countries reported that they use damage catalogue for this matter, it is not clear if the grading accounts for previously observed failure modes, visual appearance at the time of inspection (e.g. exposed foundations) or it is consequence driven (as reported for Latvia).

The direct measurement of scour was indicated by five countries (Czech Republic, Croatia, Germany, Lithuania, Greece). Here, either the scour depth is measured or a monitoring technique is applied. The assessment of scour at a bridge is performed by estimating scour affected area in m^2 in two countries (Croatia, Lithuania), but here the provided references must be checked for specific information on the used thresholds. Only a few countries (Lithuania, Ireland, Germany, Greece) provided a specific reference related to the assessment of scour. Only one country (Spain) reported the use of a formula for evaluation of a scour depth.

Besides the term scour, the associated terms hydraulic inadequacy and hydraulic performance were reported by Greece and Israel respectively. The performance goals related to scour evaluation and assessment are provided only by a few countries (Germany, Denmark, Greece) and these include: traffic safety, ULS, DLS and service life. Scour countermeasures are not reported by any country, only in the survey from Greece, a term *hydraulic protection system* is provided.

It is unlikely that among surveyed countries there are those that do not appraise flooding hazard or scour at existing bridges. For example, in Melville & Coleman, 2000 it is stated that Sweden and Netherlands have manuals of practice and design guides for bridge scour, where scour estimation is covered, but this was not reported in the survey. The contents of the reported damage catalogues used for grading scour impact on bridges, remains unknown for now. The information provided on methods for assessment and monitoring of scour depth is vague as well as the assessment of the reported performance goals. The relevant research documents on the topic were also reviewed, but in these there are only two that elaborate adverse action of scour at bridges (Greece and Serbia). The two other culprits for bridge inadequate performance besides scour, i.e. overtopping and washing away of access roads were not identified in the survey. Clearly, the relevant data on flooding hazards in BM practice in Europe must be further investigated.

Now, there is sufficient information to structure a questionnaire that will clarify the contents of the surveyed data. It is envisioned to disseminate it to those countries which have provided the most relevant information and to other that show interest in it. Its aim would be to collect specific data related to scour assessment and related BM methodology. The questionnaire will address the following topics:

- Lessons learned from the past failures – crucial information on possible failure modes
- Methodology for the scour assessment and thresholds which are considered
- Equipment and its deployment procedures in measuring of scour depth
- Availability of the sufficient data to conduct quantitative assessments e.g. risk/vulnerability
- BM practice regarding a climate change - the needs and shortcomings

Currently, the main task of the WG 3 is to make use of the available database information in structuring guidelines that will facilitate establishment of QC plans. In the next paragraph, the relevant PI for flooding hazard are discussed.

3 Relevant PI for flooding/scour hazard and other relevant data

In the line of the WG3 framework, the results of the survey are structured in lists and presented in Figure 2. Here, all the terms reported (at least by one country) are in gray color, while those not reported and the additional relevant data/parameters are given in white. As seen, this additional data is not coming out of the survey and in fact represent missing links between observations/indicators and the KPI-s. It is clear that the following task is to describe relationship between the data in the lists, thus facilitate evaluation of its impact on the KPI.

		Performance indicators			
Structure	**Elements**	**Observation**	**Other relevant data**	**Damage process**	**KPI**
All bridge types and materials	*Foundations*	*Scour depth*	*Bridge geometry & dead load*	*Flood/Scour*	*Reliability*
	Embankment	*Scour affected area*	*Type of foundations*	*Erosion*	*Safety*
	Scour Countrameasures	*Exposed foundation*	*River bed properties*		*Availability*
	Substructure	*Eroded embankment*	*Foundation soil properties*		*Cost*
	Bearings/Joints/Hinges	*Hydraulic performance*	*Flood magnitude*		*Maintainability*
	Superstructure	*Specific damage location & severity*	*Debris/ice potential*		*Economy*
		Condition state	*Traffic data*		

Figure 2. List of key terms for a bridge exposed to a flooding hazard and scour

All bridge types regardless of age, static system or materials may be affected by a flooding hazard. Here, foundations of substructures and bridge embankments are exposed to the process of scouring. There is removal of soil at foundations while the supporting soil and bridge structure jointly resist this adverse action until the bridge fails under its own dead load (Tanasic, 2015). This resistance is not adequately accounted in the current bridge management practice, which is resulting in an overestimation of the factual threat of a failure. The resistance of a bridge to a flooding event is primarily governed by the assumed hazard scenario (e.g. scour at a middle pier). Possible failure modes are governed by the combined resistance of soil-bridge system and here the foundation soil properties have the leading role. Secondary, but not unimportant is the engaged superstructure resistance governed by bearing, hinge or joint properties.

Indirect observations which can point out problem with scour are pier/abutment settlement/rotation and resulting localized damage (e.g. cracks) at joints, bearings, hinges. These indicate that a failure mode has already occurred, which requires immediate attention i.e. adequate repair actions. These types of observations are not in the scope of this task within WG3. However, any type of damage at structural elements, which is not a result of a foundation displacement/rotation, are of interest as it may decrease bridge resistance to an oncoming flooding event. Here, the importance of two parameters: damage location and its severity (e.g. area/depth affected) must be recognized for every bridge type and element in order to conduct comprehensive analysis on possible bridge failure modes.

The scour countermeasures at substructures and their condition state is important for the scour assessment. Protective structures against erosion (e.g. gabion piles and walls), mitigate the threat of failure but also a structure left embedded in the soil after foundation construction (e.g. perimeter wall of Larsen talpes) should be considered as eligible to reduce the threat as well.

The reported terms in the list *Observations* are overlapping as they use similar or the same information for their assessment. It has to be clear in which cases (e.g. certain bridge types, foundations, etc.) these terms are eligible for evaluation of a specific KPI. The scour depth is an observation/indicator that may be directly measured, monitored or indirectly evaluated by empirical formulas. For the latter, parameters from the first six groups of data in the list O*ther relevant data* (excluding dead load) is necessary. Similar goes for the assessment of hydraulics performance, but this is an observation which indicate that there is going to be significant erosion at foundations, i.e. complementary to the observation of evaluated scour depth. The reported observations: exposed foundations, eroded embankment and scour affected area are not by themselves an effective PI. They are assessed in visual inspection, and only if noticed timely may signalize for a potential future threat (i.e. failure scenario).

For the assessment of scour at substructures, it is essential to know the type of foundations (e.g. shallow RC footing, wooden piles, RC caisson, etc.), their position/orientation with regard to river bank and exposure to an extreme flooding magnitude given as function of flow and duration. Also important are the river bed properties i.e. slope and Manning coefficient as well as foundation soil properties (i.e. erodibility and geotechnical properties), which are not usually stored in bridge databases but exist up to an extent in bridge design documentation. Knowledge on debris/ice potential at the bridge site provide supplemental information in the assessment of scour at substructures. Information on bridge geometry, dead load and traffic data are generally known or may be easily surveyed. Traffic data are used to calculate the costs related to inadequate bridge performance due to a scour related failure, which makes them a valuable piece of information. The reported performance goals from the survey relate to the KPI-s of Reliability, Safety and Availability. This suggests that none of the countries systematically evaluate the monetized consequences which are result of an inadequate bridge performance due to flooding hazards.

As seen from the survey and its review, in the Europe there is a variety of BM practices related to a flooding hazard. It is clear that an adequate approach, that will account all relevant data, must be chosen to facilitate elaboration of QC plans for different bridge types, which is now discussed.

4 The impact of PI-s for flooding hazard on the KPI-s and structuring of adequate QC plans

In order to account the impact of scour on bridges and related consequences, an adequate qualitative or quantitative approach should be chosen to relate the performance values of a relevant PI to as much as possible KPI-s.

The example of a qualitative approach for assessment of hydraulic vulnerability of existing bridges is given in (NYSDOT, 2003). Here, the most of the data in Table 1. is considered in the evaluation of a rating score, but the main shortcoming of the approach is that the superstructure resistance is not accounted i.e. the failure modes and related consequences are only generally addressed. In the approach of the U.S. Federal Highway Association (FHWA), the ratings in the National Bridge Inventory (NBI) database for the Item 113 - Scour critical bridges, are given based on engineering judgement supplemented by visual inspection, field review, indirect evaluations and condition state of applied countermeasures (Pearson et al., 2002). Here, the mentioned consequences (bridge closure) and failure modes (stability endangered) are primarily considered in the light of the evaluated local scour depth and available information on a foundation type/depth. The superstructure resistance is not considered in assigning the rating score. Although comprehensive, in the two mentioned approaches there are no explicit connections of the KPI-s (or none at all) standing between their PI values (i.e. rating score) and related QC plans.

However, for a quantitative approach e.g. the suggested vulnerability assessment, the performance values of PI-s can be directly related to the KPIs: Reliability, Availability, Maintainability, Economy, Cost and Safety. In this assessment, the two values are essential, the probability of a bridge failure due to a certain magnitude of a flooding event and the related total consequences (direct & indirect). Their relationship is via a failure mode which is dependent from the evaluated scour depth and the resistance of a soil-bridge system to the related removal of supporting soil.

The main idea in the action is that the QC plans should be tailored for a certain type of bridges, elements, observations and other relevant data. Besides the currently reported and obvious differences in QC plans based on bridge foundation types (i.e. shallow/deep), there are other relevant terms which should be accounted. They reflect on how much of a superstructure resistance can be engaged in a certain failure scenario and an extent of damage:

- Detailing of a foundation affected by scour
- Type/properties of the joints at a pier/abutment top (e.g. free, fixed, pinned)
- Type/properties of a superstructure and a number of spans

In some cases, there is no need for accounting either soil or a superstructure resistance (Figure 3), as a local failure of the foundation may govern the failure mode.

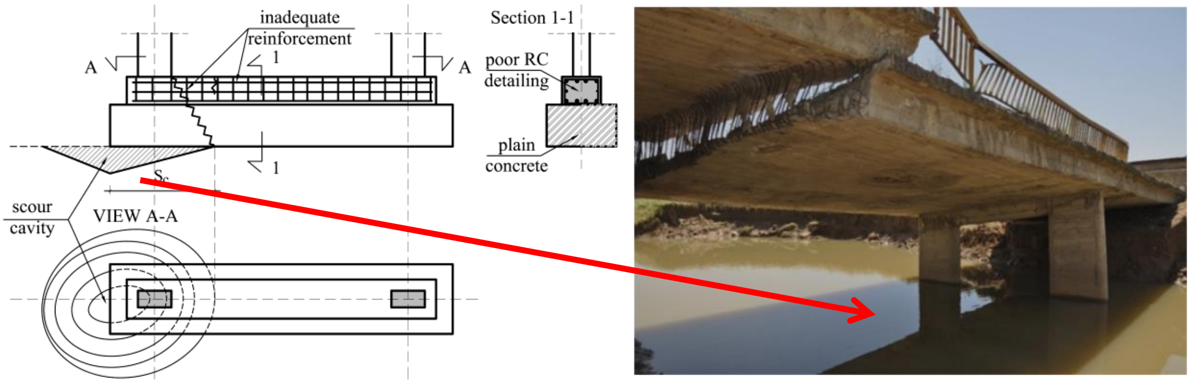

Figure 3 A brittle failure governed by poor foundation detailing (Tanasic, 2015)

5 Conclusion and further steps

The survey of performance indicators from the national bridge inspection manuals/guidelines is performed within the COST TU1406 action. The collected data was reviewed and discussed in the light of the Task 4 of the Work Group 3, which is related to non-interceptable processes – a flooding hazard and scour. It is concluded that the available data is not sufficient to fully comprehend the procedures in bridge management related to mitigating the threat of oncoming flooding events.

Although the scour at bridge foundations is recognized as a damage process by almost every surveyed country, the reported information on indicators/observations, which are used to identify and assess the severity of this threat to a bridge, are vague. The details on specific assessment procedures & equipment were not in the scope of the survey, but it is a fact that not all countries equally account for this hazard. The most of the countries simply rely on visual inspection but some take a step further by making direct measurements and indirect calculations of a scour depth. There is an opportunity to gather essential data on BM practices related to flooding in Europe by structuring and disseminating a simple yet sufficiently comprehensive questionnaire to facilitate and supplement the work of the WG-s within the action.

The most relevant indicators and terms are pointed out from the survey. This data is complemented with other relevant information related to flooding hazard, soil foundation, bridge structure and traffic. The information on past scour from visual inspections at bridge substructures is important in the definition of a failure scenario, but solely not sufficient for BM in face of oncoming flooding events. It is suggested to use vulnerability assessment as it comprise the most comprehensive information on the factual threat by accounting probability of a failure, failure modes and related consequences. The connection to the relevant KPI-s are in this case straightforward.

The current QC plans for bridges exposed to flooding hazard are primarily based on a foundation type, but this cannot be regarded as a general rule. The situations in which the types of superstructure and bearings provide additional resistance to a bridge in a flooding event, must be thoroughly elaborated in the future COST TU 1406 QC plans.

Acknowledgment

The authors would like to acknowledge that the ongoing research on the presented topic is motivated by goals of the COST action TU1406, specifically the tasks of the Work Group 3 „Establishment of a Quality Control Plan “

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Basis for the development of Quality Control Plans for Arch Bridges

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.6

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Abstract. This work contributes to the systematization of arch bridges specificities, required by a framework to develop Quality Control Plans. The objective of the WG3 of the Cost Action TU1406 is to provide a methodology with detailed step-by-step explanations for establishment of QC plans for different types of bridges, based on the results of WG1 and WG2 as well as on survey of existing approaches in practice. The main concepts presented are therefore those elaborated in the published report of the WG1 of this Cost Action, the developments already made within the scope of WG3, and a literature survey regarding this specific type of bridges. The main challenge of a QC plan is the necessity to connect general data about each bridge, observation findings and other performance indicators with a set of key performance indicators that can be directly related to performance goals, fulfillment of which ensure sufficient quality. A general approach for QC plans was already developed in the scope of WG3 for all types of structures, and is now being analyzed within the scope of arch bridges.

Keywords: arch bridges, quality control plans, performance indicators, performance goals, damage process

1 Introduction

Efficient bridge management may ultimately be understood to promote the well-being of general public, both in economical and societal sense, and coping at the same time with environmental and sustainable challenges.

Although bridge management systems are widely spread, at different sophistication levels that may include lifecycle analysis, prediction models and optimization algorithms, Quality Control (QC) Plans varies greatly from country to country and sometimes within the same country.

This Cost Action aims to contribute to the development of a guideline for the establishment of QC plans for roadway bridges, in order to reduce the disparity of bridge management quality, through the standardization of the condition assessment and maintenance strategies.

Quality control plans usually rely on the gathering of a set of Performance Indicators (PI), from observations, measurements or other data such as bridge type or year of construction. These PIs are used to obtain Key Performance Indicators (KPI), such as safety, availability or maintainability, among others. In turn, KPIs are compared to Performance Goals (PG), which if fulfilled ensure the sufficient quality of service for a given bridge. How these PIs correlate with each other's and how are KPIs derived from PIs, are the main challenges for the practical implementation of this effort.

While the survey of the established practices among the Cost countries was the objective of the WG1, WG2 address the challenge related with the establishment of performance goals and therefore the KPIs at different levels, namely for the network, for the system (bridge) and for the component. Correlation between this variables and thresholds for performance goals will be analyzed in WG3, through a common framework for different types of structural systems.

2 Common Framework for the Development of Quality Control Plans

A common framework for the development of QC plans for different types of structural systems was proposed in the Cost TU1406 workshop held in Delft in October 2016.

As summarized in Figure 1 this framework presents relationships between the entities considered fundamental for bridge management throughout lifecycle (Hajdin, forthcoming).

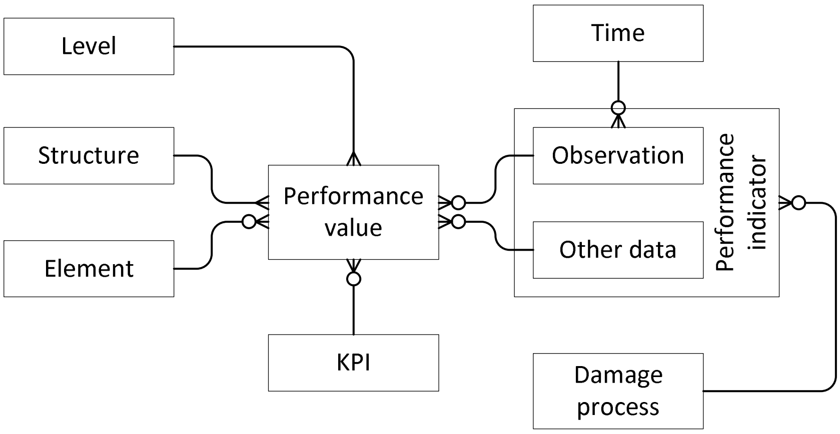


Fig. 1. Common framework for the development of QC plans (adapted from Hajdin, *forthcoming*)

According to the diagram the entity referred as *Structure* encompasses different structural systems. Each structure type may include different *Elements* (deck, columns, beams, etc.) which, in turn, can have attributes such as material properties or construction type.

During its service life, a bridge can experience investigations that may reveal findings and measurements, represented together as entity *Observations*. *Time* is therefore an attribute of these observations that might be unique or periodical during the service life of a structure.

Construction year or material properties are examples of data to include in the entity represented in the diagram as *Other Data*. This data, when related with the *Observations,* might indicate the existence of a particular *Damage Process* whose evolution can be tracked, both in a qualitatively or quantitatively manner, by the means of a *Performance Indicator*.

For each of the considered *Performance Indicators,* which is as explained above related to *Observations* and *Other Data* in a specific moment in *Time* and in case of forecast to a *Damage Process*, a *Performance Value* can be determined. *Time,* and therefore *Performance Values,* can be related to past and present events, as well as future predictions according to degradation models for damage processes.

Finally, determined performance values are used to derive Key Performance Indicators, which are directly used to be compared with Performance Goals, thus mirroring management goals and quality requirements. According to the considered time-frame, structure improvement, rehabilitation or maintenance can be planned both for short or long-term, or even demolition in case of structure obsolescence.

3 Quality Control Plans for Arch Bridges

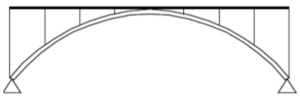
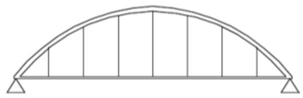
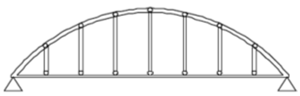
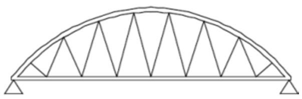
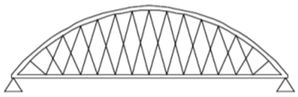
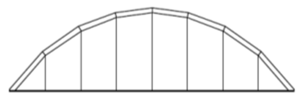
Arch bridges represent a significant percentage in most European countries national inventories, with special emphasis on masonry structures. It is important to be aware that many of them are centenarian and the oldest structure type of the bridge population (UIC, 2011). For this reason, there is a significant degradation of the properties of their materials, due to inherent wear and tear over time. In addition, some of this these bridges may be subject to higher loads than those that were anticipated at the time of construction. These circumstances justify the relevance that should be given to the optimization of the maintenance strategies of arch bridges (Aníbal Costa et al., *forthcoming*).

The selection of the best maintenance strategies for arch bridges requires a good knowledge of the structural behavior, material properties, as well as a correct interpretation and evaluation of the revealed defects and general findings. Structural behavior depends on the geometric and mechanical characteristics of the materials, the structural elements and their related connections, as well as the size of the structures and their environmental exposure (Aníbal Costa et al., *forthcoming*).

3.1 Element

Distinguished proposals regarding arch bridges taxonomy can be found in literature, in most cases related with the main material in use. Seeking a systematic approach for all arch bridges a classification system for arch bridge was recently proposed (Han, Sim & Kim, 2016). This work, exemplified in Table 1, defined nine different categories, including the material, number of span, road position, support condition, rib shape, rib alignment, stiffening girder type, spandrel type and arch type.

Table 1. Classification code system for arch bridge (adapted from Han, Sim & Kim, 2016).

Sort/ Material	Classification		
	Concrete	Steel	Composite
Spandrel			
	Open Spandrel		Closed Spandrel
Arch type			
	Langer	Lohse	
			
	Nielsen	Warren	Non Formal

This taxonomy mirror the complexity of this structural type of bridges for the three most common structural materials. However, some improvements can be made, such as a narrower differentiation of arch types. For instance, in case of masonry structures, three common arch types can be added to this taxonomy, namely: semicircular arch, segmental arch and three centered arch.

3.2 Time

In the scope of a bridge quality control plan the *Time* factor is related with the loadings or events affecting the structure during its service life, as well as to the evolution of deterioration processes acting upon structure. The understanding of the degradation phenomena also allows predicting its evolution. Therefore, *Time* is related to forecasts.

The survey performed by WG1 of the Cost Action TU1406 and published in the Technical Report (Strauss and Ivanković, 2016) documented a common understanding on investigation workflow to assess bridge condition. This report clearly identified detailed inspections as the basis of any assessment, sub-dived in four time-related categories, namely: visual inspections, e.g. yearly basis; simple checks, for instance 3 years after every main inspection; in-depth examinations or main inspections, for instance every 6 years; special inspections, following exceptional occurrences or incidents. In respect to monitoring techniques the report emphasizes that under no circumstance this actions can replace detailed structural inspection, as they should be always complementary to inspections, adding additional information.

3.3 Observations and Other Data

Data that might interest during bridge service life should be recorded in the bridge inventory. General information such as construction year, designer, contractor, scope of rehabilitations, etc., are important features that can be correlated with performance.

Concerning observations taken in the periodic inspections, Bień and Gładysz-Bień (2016) proposed six basic types of defects that can affect arch bridges:

- Deformations: changes of the structure geometry, incompatible with the project, with changes of mutual distances of structure points;
- Material destruction: deterioration of physical and/or chemical features of structural material with relation to designed values;
- Material losses: decrease of designed amount of structural material;
- Material discontinuity: inconsistent with a project discontinuity of a structure material;

- Contaminations: appearance of any type of an organic or inorganic dirtiness or non-designed plant or other organisms existing on the structure;
- Position changes: dislocation of a structure or its part incompatible with the project, also restrictions in designed displacement capabilities.

References to design values underlying original project can be understood as the as-build condition, with special attention to centenarian bridges where a formal project design may never have existed.

3.3.1 Performance Indicators

Based on the survey conducted by WG1 of Cost Action TU1406, regarding bridge inspection and evaluation documents, eleven clusters of performance indicators were identified, related to: defects; material properties, loads, environment, cost and importance, rating, dynamic behavior, original construction and design, bearing capacity, structural integrity and joints, equipment and protection, geometry changes.

Specifically related to masonry arch bridges an analysis to the WG1 database was already accomplished, according to Matos et. al. (2016), linking damages or defects with performance indicators such as structural safety (ULS), serviceability (SLS) and durability (D). According to the results presented in the aforementioned work and shown in Table 2, most common damages can be related with all three referred performance indicators. In addition, the main processes of identification and evaluation for each damage are also presented in the same table.

Table 2. Performance indicators for masonry arch bridges (Matos et. al., 2016).

Damage characteristic	Performance Indicator				
	Detection	Evaluation	Level		
			D	SLS	ULS
Joints deficiency	VI	DC	X	X	X
Joints leaking	VI	DC	X	X	X
Dewatering deficiency	VI	DC	X	X	X
Contamination	VI	I	X	X	X
Cracks	DM	DC	X	X	X
Spalling	VI	DC	X	X	X
Deformation	VI	DC	X	X	X
Displacement	DM	I	X	X	X
Loose of stones/bricks	VI	DC	X	X	X

DM – direct measurement; VI – visual inspection; DC – damage catalogue; I - inspection

Future work will include a review of Performance Indicators form WG1 database, regarding all types of arch bridges. With this systematization performance thresholds or criteria can be searched, in order to assist decision making regarding choices such as maintenance actions, tests, monitoring or others.

3.4 Damage Process

The existence of damages can be related with two different types of processes, as proposed by Hajdin (2016): the interceptable (observable) processes and non-interceptable processes (accidents, earthquakes, etc.).

Interceptable (observable) processes in masonry arch bridges, as proposed by UIC (2006), were categorized accordingly their relation with demand or damaging processes (Amado, forthcoming).

More generally for the three most common structural material of arch bridges, masonry, steel and concrete, Bień and Gładysz-Bień (2016) proposed, as shown in Table 4, three main groups of the degradation mechanisms: physical, chemical and biological.

The purpose of indicating thresholds or goals for performance indicators requires a deep understanding of the phenomenon affecting bridges, their causes, consequences, actual degree or extend and possible progression. Therefore, the calculation of a certain performance value always depends on the consideration of certain observations and other relevant data, correlated with the knowledge of the damaging processes affecting bridges.

In other words, diagram entity *Damage Process* is primarily related with *Observations* or *Other Data*, and not directly connected to *Performance Values* or KPIs.

Table 3. Degradation mechanisms according to material of arch bridge (adapted from Bień & Gładysz-Bień, 2016)

	Degradation mechanisms	Material of arch bridge		
		concrete	steel	masonry
Physical	Accumulation of inorganic contamination	●	●	●
	Freeze/thaw actions	●	○	●
	Erosion	●	○	●
	Crystallization	●	X	○
	Extremal temperature influence	○	●	○
	Rheological processes	●	○	○
	Overloading	●	●	●
	Leaching	●	X	●
	Fatigue	○	●	○
	Changes of geotechnical conditions	●	●	●
Chemical	Carbonization	●	X	○
	Corrosion	●	●	X
	Aggressive environmental impact	●	●	●
	Reactions between material components	●	○	○
Biological	Accumulation of organic contamination	●	●	●
	Influence of microorganisms	●	●	●
	Influence of plants	●	○	●
	Influence of animals	○	●	○

● - basic mechanism; ○ - supplementary mechanism; X - not applicable

3.5 Level

Consequences of a certain phenomenon can be analyzed at different levels. If in a close look the functionality of a single element is affected, it might have an effect on the whole structure stability and therefore contribute to a certain performance of the network, depending on the bridge importance or redundancy on the network. Three levels were identified and analyzed in WG1 survey, and they should be considered in the establishment of QC plans: component level, system level and network level. However, it is possible to consider that performance goals will not have to be established at all levels, as a correct modeling may mirror the propagation of effects from the element level to the network level.

Obviously, these levels do not depend from the structural type, nevertheless, for arch bridges, heritage concerns might be much more common than for other bridge types, even if not considering landmark bridges. Heritage might therefore be a factor at the network level, whose primary goal to be reached is priority repair ranking among a set of bridges with detected necessities. This ranking should be based on bridge condition assessment, accomplished through standard inspection and evaluation procedures, with additional evaluation of bridge importance in the network (Strauss and Ivanković, ed., 2016).

The structure level is related with bridge functionality as whole, in order to assess the impact of the damaged element to the entire structure. The importance of the bridge element can be evaluated according to the structural safety, serviceability, traffic safety and durability (DIN 1076 1999).

If one considers bridge inspections as the basis of a systematic management, the element level will be the ground level as in general inspections include observations detected on bridge elements. This level comprises damage detection, identification, evaluation (comparing with a certain threshold) and more detailed assessment such as testing, if necessary.

At the element level it's essential to know, depending on bridge type, that the possible damages and degradation processes, on element level may compromise the whole structure

3.6 Performance Value

The *Performance Value,* as represented in the Fig. 1. diagram, puts together all relevant data that has impact on the performance indicator. On the other hand each KPI has to consider the correlation of the performance values contributing to this KPI. For each *Performance Indicator* a value will reflect the findings, usually through the use of a rating index.

As stated in WG1 Technical Report, a similar approach is used in several countries, and a five-level rating system is presented as a typical example. In this example, extreme grades represent, respectively, "No or very slight damage" and "Extreme damage". Middle grades are summarized as "Slight damage", "Moderate to severe damage" and "Severe damage".

3.7 Key Performance Indicators

The main idea of the Cost Action TU1406 is to provide a set of standardized tools to support the establishment of QC plans for roadway bridges. These tools are considered to be performance indicators and performance goals, which may differ for each type of bridges. The survey performed in the scope of WG1 revealed, after additional clustering and homogenization, that the most widely used performance indicator is obtained by visual inspections, despite the name vary from condition index, conditions rating, and deterioration index, among others (Strauss and Ivanković, ed., 2016).

Cleary, while some countries rely on this indicator to perform their management activities others have started to consider other relevant indicators, seeking to express concepts such as remaining service life, safety index-reliability, vulnerability or robustness. These terms, or others that can be defined, are always dependent of observations and related with other bridge data, previously also defined as performance indicators. Therefore, these top-level definitions are usually considered as Key Performance Indicators (KPIs). Finally, in a QC plan these KPIs are directly used to be compared to Performance Goals. The PG can be extremizing goals meaning that the corresponding KPI has to be maximized or minimized, or satisfying goals represented as thresholds.

The establishment of QC plans, seeks to identify crucial KPIs, based on most relevant performance indicators for arch bridges that might, or not, be common for bridges in general.

4 Future Developments

A review of the WG1 Technical Report is underway in order to identify most important performance indicators for arch bridges. From WG2 is expected the identification of KPI's that can easily be adopted by roadway agencies in different countries. Subsequent action will be to model the impact of different combinations of structure types and elements, observations and other relevant data with KPI's.

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Bridge inspection quality improvement using standard inspection methods

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DOI: https://doi.org/10.5592/CO/BSHM2017.3.7

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Abstract. There are many approaches in structural damage detection and assessment. However, for a large number of bridges to be periodically inspected only visual inspection and non-destructive testing are suitable. European countries use different regular bridge inspection protocols resulting in divergent data quality. To improve it in countries with underperforming bridge inspection methodologies, guidelines need to be defined that will help develop appropriate visual inspection protocols. In order to accurately determine the risk of intense or concealed damage non-destructive tests have to be employed in addition. These tests should be time and cost efficient, and above all, complementary with visual inspection.

Keywords: bridge inspection, damage detection, visual inspection, non-destructive testing

1 Introduction

The bridge inspection process is critical to ensuring the safety of bridges, identifying repair and maintenance needs and determining appropriate allocation of funds. As a result, the quality of the data produced during the inspection process is extremely important (Washer & Chang, 2009). The inspection process is the foundation of the entire bridge management system. Data accuracy has to be consistent throughout the period when inspections are conduced. Only then, it is possible to get the best possible information describing the bridge current state, performance, deterioration rate and similar, over the years of use.

To ensure quality data acquisition and processing a quality control plan has to be developed. This is one of the main objectives of COST Action TU1406. Its development is planned within Work Group 3 (WG3) and its framework has already been outlined (fig.1). It consists of several interconnected and interdependent parts, with the final goal to determine the performance value of each element of the individual bridge and each bridge within the addressed network as well as network as a whole.

As it can be seen (fig.1), one of the components in the proposed quality control plan is observation. The primary objective of this activity is to register type, extent and intensity for every damage recorded on each element of the bridge. During field investigations, every element is being examined separately and at the bridge component level (there are no complex analyses to be performed on this level), one of the most important goals to be reached is objective damage assessment. There are four main approaches in damage detection and assessment (Strauss et al., 2016):

- visual inspection,
- non-destructive testing,
- probing and
- structural health monitoring.

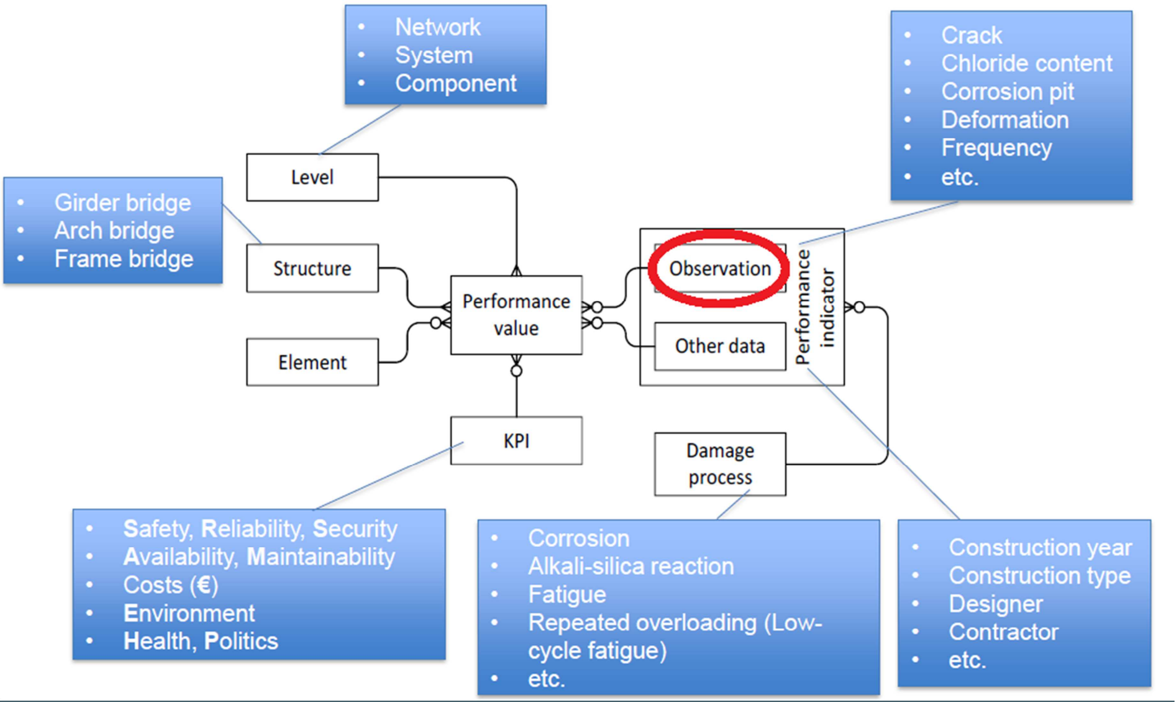

Fig. 1. Quality control plan framework (Hajdin, 2016)

Structural health monitoring (SHM) is generally performed on the bridges of utmost importance for the road network. Equipment acquisition, its maintenance, data collection and analysis require financial assets that are not affordable for large scale use. Therefore, SHM is in most cases used for bridges with large spans only. Probing provides the most reliable results regarding the state of the bridge and its individual components. Its biggest weakness is the fact that its implementation causes a certain damage to the construction. In most cases, it is performed when remediation or reconstruction of a specific bridge is already envisaged, however more accurate information on the state of the bridge components is still needed.

The use of SHM and probing is therefore not suitable for large-scale periodical damage detection and assessment. Although somewhat less reliable, for long-term data acquisition regarding the bridge state and its changes over time, two types of data collection techniques remain available: visual inspection and non-destructive testing (NDT). Both approaches have advantages and disadvantages from the viewpoint of data acquisition, reliability, work pace, required equipment etc. Most importantly, visual inspection disadvantages can be eliminated, to a large extent, with the complementary use of NDT. The use of both methods does not guarantee the quality of acquired data by itself. Appropriate quality plan and working methods have to be developed for this purpose. One of the WG3 tasks in COST Action TU1406 is to define the most appropriate use of resources available (i.e. human and equipment) for bridge inspection practice. Challenges related to this task fulfillment are addressed in the following sections.

2 Visual inspection

The majority of the existing bridge maintenance systems in the past were based primarily on information obtained through visual inspections (Gattulli & Chiaramonte, 2005). Although documented past (Phares et al., 2004) and ongoing experience (Kušar, 2014) reveals that this type of inspection is often unreliable, it will remain the main aid for collecting data due to its simplicity and cost effectiveness (Tenžera et al., 2012). This point of view was accepted as an undisputed fact in WG3 meetings of COST Action TU1406 in April 2016 (Belgrade, Serbia) and October 2016 (Delft, Netherlands).

Data reliability can always be improved in two basic ways: by improvement of existing protocol of visual inspection or by use of additional methods of examination. The countries participating in the COST Action TU1406 have different visual inspection protocols. They vary greatly in manner of implementation and consequently extent and quality of the collected data. In order to develop guidelines for visual inspections, with the aim to uniform (standardize) them as much as possible at a European level, a survey of best protocols has to be performed and subsequently, within the WG3, the issues listed in Table 1 along with some other ones need be defined.

Table 1. Some of the issues regarding visual inspections to be tackled

Frequency of inspection Should it be fixed or dependent upon the bridge state/importance?
Inspector qualification and experience What is the minimum formal education, when is an inspector considered experienced, do higher risk bridges need to be examined by experienced inspectors only?
Data input Should report be completed on field or in office?
Inspectors rotations Should inspections on the selected bridge be performed always by the same inspector or should inspectors' rotation be mandatory?
Office review Which parts of inspection reports always need review, e.g. critical findings, recommended actions, whole reports?
Field review What percentage of bridges investigated should be reviewed? How do we select the bridges to be reviewed? By random sampling, critical bridges only, do all inspectors need to be reviewed?
Refresh training Training extent (theoretical, practical), frequency?
Performance indicators Should some performance indicators (e.g. scour, settlement) be given more emphasis during inspection?

There are numerus inspection manuals, presentations and inspection practices available from members of TU1406 as well as on the internet. Additionally, the majority of members have experience on the subject, therefore the only challenge is the unification of members' regarding each issue under consideration as possible methods of implementation have already been addressed in the past.

3 Non-destructive testing

As long as bridges exhibit no significant damage, deformations or other irregularities, visual inspections are sufficient for determining slow continuous processes of degradation. Problems occur when it becomes necessary to accurately determine the risk of intense or concealed damage (Kušar & Cmok, 2016). Although NDT is not regularly integrated in regular bridge inspection processes, their application brings valuable additional information on the current condition of the structure and should be applied when degradation processes intensify.

Certain NDTs used for detailed examination are frequently undertaken because of their simplicity (e.g. rebound-hammer, cover meter) while other methods are sophisticated and applied for special investigations or scientific use only. Reliable, relatively simple and if possible inexpensive NDT should be used for routine inspection practice. Advanced methods are not suitable for large-scale implementation, since they are in most cases expensive. An overview of selected NDT is already available in literature (e.g. Sousa et al., 2009). Additionally, a survey of over 30 methods is currently in progress (Table 2). The aim is to identify methods appropriate for use from the viewpoint of time consumption, cost efficiency and reliability of results. They must also be able to cover the shortcomings of visual inspections.

Table 2. NDT methods survey

1. NDT for measurement of: cracks, leaking, mechanical damage, scaling, segregation…			
Image Pro Plus (IPP)	Acoustic emission (AE)	Impact echo	Infrared thermography
Impulse response	Radiography	Petrography	Lamb wave Theory

2. NDT for measurement of: compressive strength, surface, hardness, adhesion			
Rebound hammer	Ultrasonic pulse velocity (UPV)	CAPO test	Probe penetration
Micro-coring	Pull-of test		

3. NDT for measurement of: chloride concentration			
Quantab test	Potentiometric titration	Fast chloride test	

4. NDT for measurement of: corrosion			
Galvanostatic pulse method	X-ray diffraction and atomic absorption	Electrical potential measurement	Time domain reflectometry (TDR)
Ultrasonic waves	Linear polarization resistance		

5. NDT for measurement of: carbonation (concrete pH)			
Phenolphthalein indicator test	Rainbow indicator		

6. NDT for measurement of: internal damage and defects, delamination			
Vibration based damage identification	Seismic refraction method	Ultrasonic longitudinal waves	Ultrasonic continuous spread spectrum signal

7. NDT for measurement of: internal damage and defects, delamination			
Water permeability test	Initial surface absorption test (ISAT)	Covercrete absorption test (CAT)	

For each test presented in Table 2 the following content is to be addressed: performance indicators assessed, method application, its advantages and disadvantages, possible other issues. The main goal is to assess compatibility and complementarity of the discussed NDT with visual inspection.

4 Conclusions

In Europe, the state of bridges with largest spans are in most cases controlled by structural health monitoring systems, while the damage state of vast majority of bridges is determined by conducting periodical regular inspections only. Detecting damage during these inspections will remain in the domain of visual inspection, however evaluating its extent and intensity is most likely to be shared with selected NDTs.

Although somewhat contradictory to the above conclusions, it should be noted that a large number of bridges to be periodically inspected, in combination with limited financial resources, demands only reasonable and not best possible effort for quality data acquisition. The latter will stay in the domain of detailed bridge inspection. Therefore, selection of the best suitable protocol for visual bridge inspection could be extremely demanding. Use of NDTs, on the other hand, has to be thoroughly examined in order to select the methods that are most suitable, i.e. time and cost efficient, reliable and complementary with visual inspection.

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4 COST ACTION TU 1402 CONTRIBUTIONS

4.1 Observation-based Decision-making for Infrastructure (paper / presentation)

E. Chatzi, D. Straub, R. Hajdin

4.2 Monitoring during life cycle of bridges to establish performance indicators (paper / presentation)

P. Haardt, R. Holst

4.3 Application of DIC to monitor reinforced concrete structures (paper / presentation)

L. Saucedo-Mora, C. Andrade Perdrix, C. L. Hombrados, J. Barroso, A. Z. Bragado

4.4 System Reliability of Bridge Structure Subjected to Chloride Ingress (paper / presentation)

B. J. Leira, S. Thöns, M. H. Faber

4.5 Quantifying the value of SHM for emergency management of bridges at-risk from seismic damage based on their performance indicators (paper / presentation)

P. Omenzetter, M. P. Limongelli & U. Yazgan, S. Soyoz

4.6 Application of Bridge Weigh-in-Motion measurements in assessment of existing road bridges (paper / presentation)

D. Skokandić, A. Žnidarič, A. Mandić Ivanković, M. Kreslin

4.7 Optimal decisions based on monitoring – case study of steel roof (paper / presentation)

D. Lenzi, D. Diamantidis, M. Sykora

Observation-based Decision-making for Infrastructure

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.1

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Abstract. The aim of this review paper is to summarize available options for decision-support toward the optimal management of infrastructure systems under consideration of uncertainties. Thereafter, we elaborate on a particular variant, namely Partially Observable Markov Decision Processes (POMDP), as a flexible framework for the incorporation of information from observations (visual inspections, non-destructive testing and monitoring) in the context of optimal inspection and maintenance planning. Examples from the literature are presented, demonstrating the potential of planning under uncertainty and the links to the Value of Information (VoI) from Structural Health Monitoring (SHM).

Keywords: Optimal Inspection and Maintenance Planning, Structural Health Monitoring (SHM), Value of Information (VoI), Markov Decision Processes (MDPs), Partially Observable Markov Decision Processes (POMDPs)

1 Introduction

Structures and infrastructure systems face challenges due to aging, deterioration and adverse operational conditions. Recent technological advances have allowed for development of Structural Health Monitoring (SHM) systems, which provide information on the “health” state of structural systems, and may be exploited to derive indicators of corresponding performance. SHM systems may therefore serve for supporting decisions regarding the management of infrastructure systems throughout their life-cycle. Such decisions pertain to the planning of appropriate inspection and maintenance actions in evidence of damage or deterioration. To address these challenges effectively and scientifically, new methods and tools are needed to quantify and optimize the Value of Information (VoI) from the SHM systems.

As part of COST Action TU1402, Working Group 3 aims at identifying developing and critically overviewing methods and tools required for the utilization of the theoretical VoI framework in infrastructure practice. Such methods take basis in modern methods of probabilistic systems analysis including Fully and Partially Observable Markov Decision Processes, Semi Markov Decision Processes, Bayesian Networks, Monte Carlo simulation schemes, Stochastic Meshing Algorithms, First Order Reliability Methods, and combinations thereof. The Influence Diagram offered in Fig. 1 illustrates the separate components involved in the Value of Information framework within the context of SHM (Straub et al., 2017).

The goal of such a framework is to provide support for optimal maintenance and intervention planning. Consider the example of a bridge object, where the goal lies in assurance of a desired service quality with minimum interruptions. In such a case, bridge owners launch preventive actions when the risk of service impairment, interruption or losses in life cycle costs reaches some predefined level. Implicitly the owners define the accepted risk based on socio-economic equity principles. This accepted risk depends upon the established performance goals for each component or combination of bridge components and together with the costs implied for every action (in the form of inspection or intervention) governs the policy to be followed for management of the system. In this context, as part of COST Action TU1406, Working Group 3 aims in defining the steps required for setting up Quality Control (QC) plans for diverse types of roadway bridges.

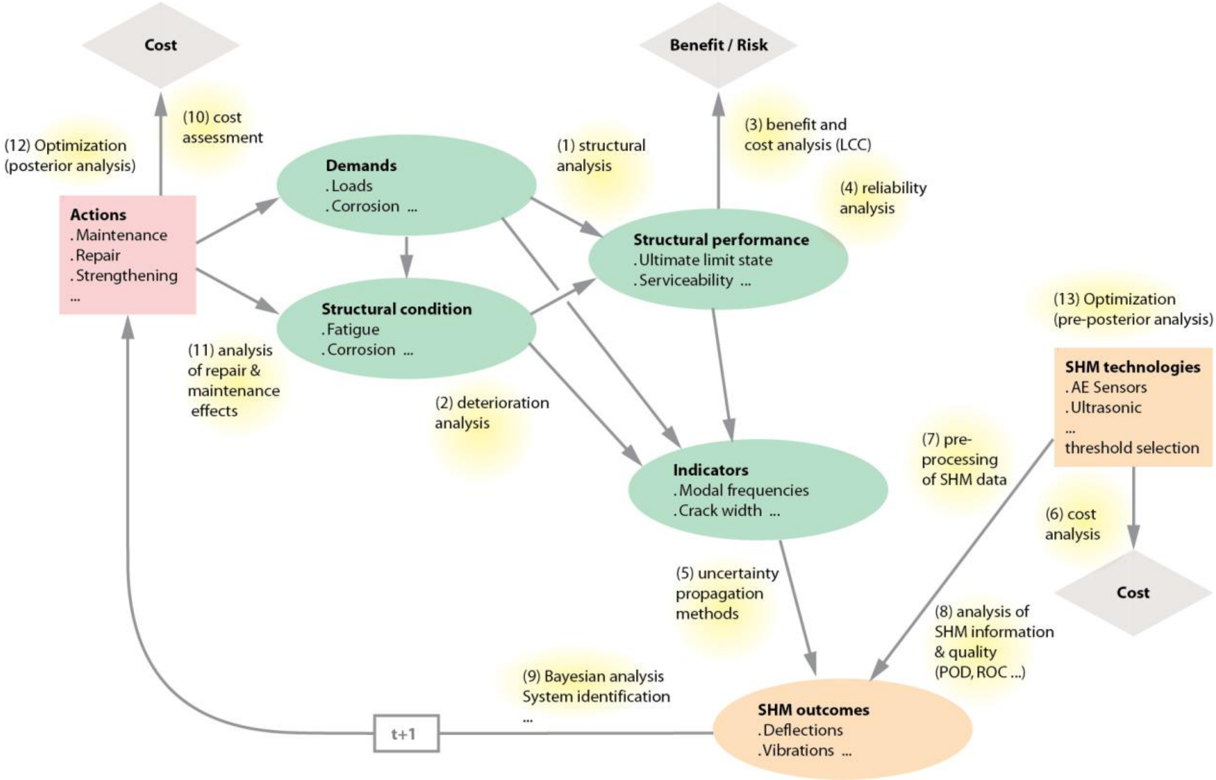


Fig. 1. Influence diagram (ID) of the Value of Information for SHM. Structural parameters and models are indicated in green, SHM parameters and models are denoted in orange, while repair, maintenance and related actions are marked in red. The yellow bubbles are the analysis methods and tools used in the different parts of the process. The box [t+1] indicates that the edge is from one time step to the next, hence this ID represents a decision process in time. Figure from Straub et al. (2017).

In existing literature, several approaches have been formulated to tackle the previously described problem of decision-making for infrastructure. A first take lies in casting this in an optimization framework, where the decision maker may choose to either optimize for separate objectives, usually tied to corresponding performance indicators, e.g. condition, availability, safety, or durability (Liu et al., 1997; Miyamoto et al., 2000; Furuta et al., 2004), or instead decide to simultaneously treat conflicting objectives. When adopting a Multi-Criteria Decision-Making (MCDM) approach, the preferred policy structure of the decision maker is adopted to transform the multiple objectives into a single optimization function. Approaches of this class, as summarized in Adiel Teixeira de Almeida et al. (2015), include single criterion-synthesis methods such as the Multi-attribute Utility Theory and the Multi-attribute Value Theory (Keeney & Raiffa, 1976), outranking methods such as Elimination and Choice Expressing Reality (ELECTRE) described in Figueira et al. (2005) and the Preference Ranking Organization Method for Enrichment of Evaluations (PROMETHEE) overviewed by Brans & Mareschal (2005), as well as further alternatives. As another option, multi-objective optimization may be employed to tackle multiple objectives, delivering a set of compromising decision options along the so-called Pareto front. To this end, Liu and Frangopol (2005) employ multi-objective optimization to solve the combinatorial optimization problem of annual prioritization of maintenance efforts for deteriorating components of concrete bridges. Taflanidis & Beck (2008) proposed a robust stochastic design framework in the context of reliability analysis and related decision support, where probabilistic models of excitation uncertainties and system modeling uncertainties can be introduced. The two-stage framework implements Stochastic Subset Optimization (SSO) for identifying a region of interest in the design space, and a stochastic optimization algorithm to finally identify the optimal solution.

An issue that inevitably enters the decision-making process is that of uncertainties (aleatory or epistemic). In this respect engineering decision problems may be classified into three main categories, namely those of prior, posterior, or pre-posterior decision problems, as elaborated upon by Faber (2005). Pure prior and posterior decision problems, as for instance calibration of code format or service life extension, may be solved using standard approaches of Quantitative Risk Analysis (QRA) and Structural Reliability Analysis (SRA). However, efficient treatment of the broader pre-posterior decision class, which includes the problem of inspection and maintenance planning, necessitates dedicated tools (Faber, 2003; Sørensen, 2009), such as Bayesian Probabilistic Networks and Influence Diagrams (Heckerman, 1995). In this work, we elaborate on a candidate approach, namely Partially Observable Markov Decision Processes (POMDPs). Markov Decision Processes (MDPs) have been widely adopted across diverse domains of engineering and applied sciences in the context of decision-making. A POMDP is in essence a generalization of a Markov Decision Process (MDP), where the decision-maker does not hold complete, i.e., deterministic, information on the system state (condition), as well as on the outcome of the performed inspections/actions.

2 An MDP approach to Decision-making

The motivation behind utilization of the POMDP framework lies in its capability to (i) incorporate stochastic models and uncertain data based on firm mathematical foundations, (ii) to optimize across long-term objectives, and (iii) to incorporate near-real-time observations allowing for near-real-time optimal decision support.

2.1 Markov Decision Processes (MDPs)

A MDP comprises the following basic components: i) the states, which in the context of infrastructure may be tied to indicators of the system's performance or condition (Limongelli et al., 2005); ii) a set of actions, commonly pertaining to interventions (repair, retrofitting, replacement); iii) the consequences of actions on the system's state; and iv) the value/cost of these actions (Cassandra et al., 2005). The MDPs abide to the Markov assumption, implying that the state at a given decision (time) step is uniquely determined by the previous state. Various solution methods have been developed for MDP problems, with Value Iteration (Bellman, 1957) and Policy Iteration (Howard,1960) algorithms holding the lead.

MDPs have long served as tools for decision support. As an example of such a use-case, the Swiss Federal Roads Office employs MDPs for bridge management, as part of the KUBA software (Hajdin, 2008). In KUBA-MS, the assessment units are structural elements, which may be further divided into segments. An influence indicator is linked to each segment, according to its exposure to environmental influences. Prevalent deterioration processes are identified and resulting states are rated according to a five-scale classification system, ranging from 1 (good condition, i.e., no damages) to 5 (alarming condition, i.e., urgent actions necessary). Markov chains are used to obtain condition forecasts and MDP use used to determine optimum preservation policies (Fig. 2). The transition matrices result from statistical analysis of inspection data (Hajdin & Peeters, 2008).

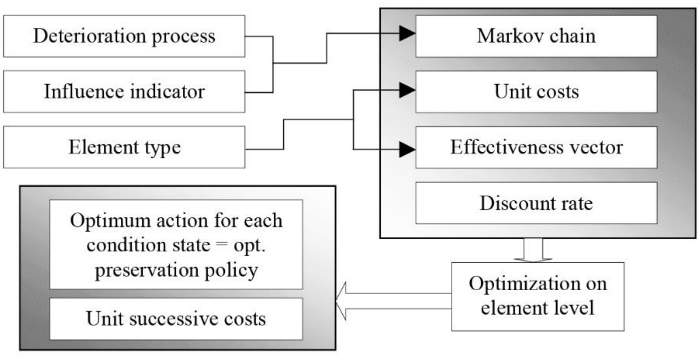


Fig. 2. KUBA-MS Optimization on the element level. Figure reused from (Hajdin, 2008).

2.2 Partially Observable Markov Decision Processes (POMDPs)

MDPs operate on the assumption of complete (deterministic) knowledge of the system's state, which is hardly ever the case. An educated estimate on the condition of the system may be delivered via inspection, non-destructive evaluation or monitoring technologies; none of these options however provide complete and exact information on the system's state. To address this challenge, Partially Observable Markov Decision Processes (POMDPs) relax the MDP assumption to consider probabilistic knowledge of the state, allowing for the analysis of decision-making under uncertain observations.

The POMDP describes the interaction of an agent, in this case a structure, with its surrounding world via the tuple $\{S, A, T, \Omega, O, R\}$:

S : A set of (usually discrete) system states.
A : A set of possible, usually discrete, control actions.
T : A transition model describes the evolution of the system from a state s_i at step i to a future state s_{i+1}, as a consequence of an executed action a. The system's state is updated using the conditional probability $p\left(s_{i+1} \mid s_i, a\right)$.
Ω : A set of (usually discrete) observations describing the outcome of an inspection or monitoring method.
O : The observation model defines the probability of obtaining an observation outcome o when the system lies in state s, i.e., $p\left(o \mid s\right)$.
R : A reward (or cost) function outputs a reward value as $r \in \Re$. The reward may be modeled as dependent upon the current state, the assumed action or a combination of both.
The POMDP process is summarized as indicated in Fig. 3. The agent initiates at a state s_i. An observation o_i is taken, motivating execution of action a_i, shifting the system's state to s_{i+1}, which results in a reward r_i. The state is only partially known due to the uncertainties involved in the system's observation, thus it may only be described in terms of a belief (a probability distribution) over the state-space. Once the system evolves due to the agent's action, the belief state is updated based on the previous belief state, the executed action, and the received observation. This is performed by employing Bayes' rule:

$$b^{a,o}(s_{i+1}) = \frac{p(o \mid s_{i+1})}{p(o \mid \mathbf{b}, a)} \sum_{s_i \in S} p(s_{i+1} \mid s_i, a) b(s_i) \tag{1}$$

where a, o designate the executed control action and inspection (observation), respectively, $b\left(s_i\right)$ is the current belief state describing our confidence in the system lying in state s_i.

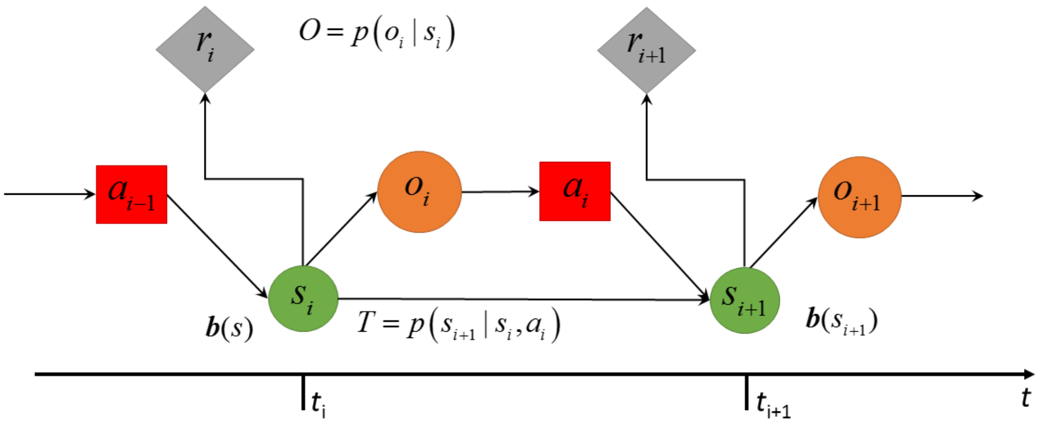


Fig. 3. The POMDP sequential decision process with alternating actions and inspections.

3 POMDP Solution Methods

Discrete POMDPs may be solved via a number of available solution algorithms (Poupart and Boutilier 2004, Sondik 1971). Most of these methods are approximate and rely on sampling of the belief space and propagation of these samples through the sequential planning process (observations, actions). Given the belief state ***b*** in a time horizon *n*, i.e., when *n* decision steps or decision intervals are left, and the updated belief state in horizon *n-1*, the optimal value function at horizon *n* is calculated in a recursive manner, as $V_n(\boldsymbol{b}) = \max_a Q_n(\boldsymbol{b},a)$, where the expected reward Q_n for a given belief state $b(s)$ and action a is $Q_n(\boldsymbol{b},a) = \sum_{s\in S} r_a(s)b(s) + \gamma \sum_o p(o\,|\,\boldsymbol{b},a)V_{n-1}(\boldsymbol{b}^{a,o})$, where $\gamma \in [0,1)$ is the discount factor, which weighs the significance of the immediate in comparison to the delayed reward, $V_{n-1}(\boldsymbol{b}^{a,o})$ is the optimal value function of the previous horizon depending on the belief state $\boldsymbol{b}$, the current action a, and the observation o.

The goal of planning is to maximize the future expected reward by selecting an appropriate policy π^*, i.e., a sequence of actions and observations that maximizes the value function.

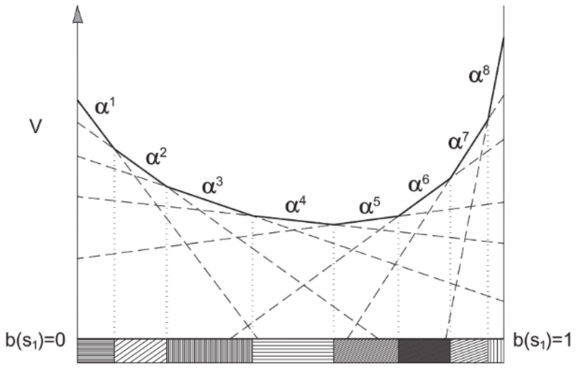


Fig. 4. Optimal a-vectors plotted in the belief space. Figure reused from Papakonstantinou & Shinozuka (2014a)

Sondik (1971) proved that the recursive calculation of the value function in a discrete system degenerates in the search for the so-called optimal a-vectors, each representing the value function for an optimal strategy at the current planning horizon n and for a set of belief states $b(s)$.

The belief space is a simplex, and each vector defines a region over the simplex, which represents a set of belief states. The value function is generally defined as the upper surface of these vectors.

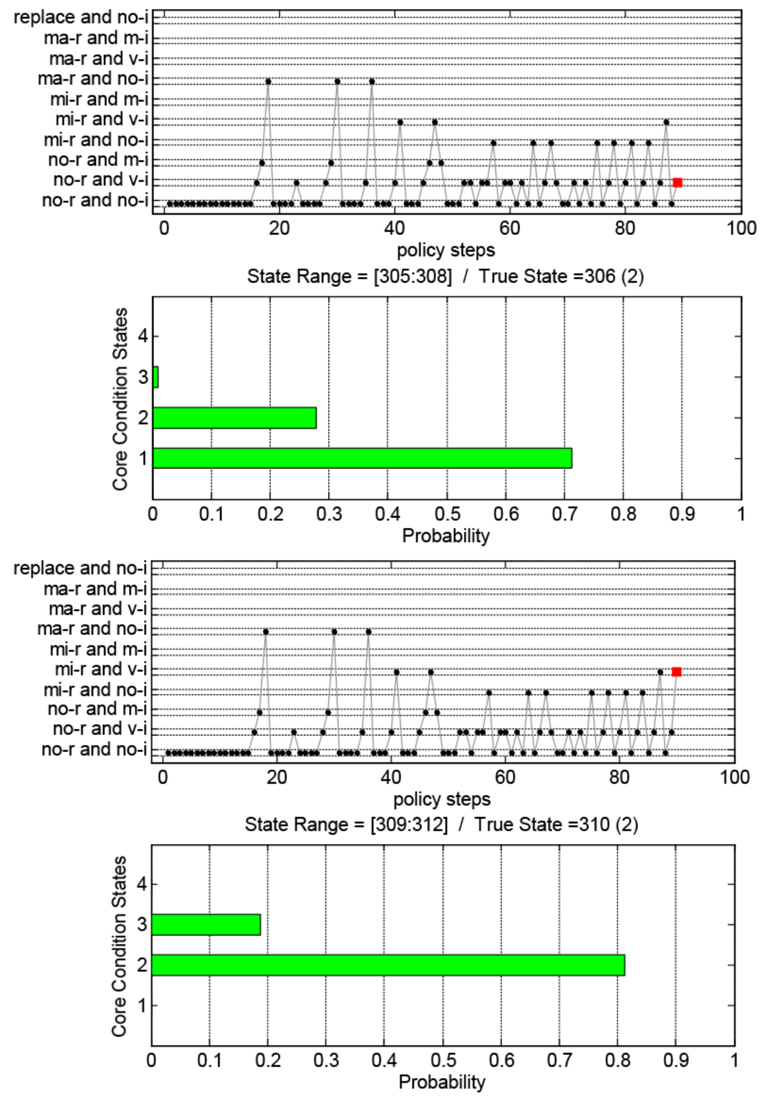
The problem may be solved by means of Value iteration and Policy iteration algorithms; further alternatives are described in Papakonstantinou & Shinozuka (2014a). The POMDP problem may also be cast in the continuous space, as elaborated upon in the work of Schöbi & Chatzi (2016). This allows for more flexible formulations able to account for generic, nonlinear actions and observations. In this case, solution methods include policy search (Ng & Jordan, 2000), or grid- and point-based value iteration algorithms, that are extended to fit the continuous space (Porta et al., 2004).

4 POMDP Implementation in Infrastructure Management

POMDPs admittedly remain less popular than their MDP alternative in the domain of infrastructure planning and policy making, largely owing to their higher computational complexity. However, they offer numerous advantages, some of which are inherited from MDPs, such as flexibility in terms of formulation, functionality in both a discrete or continuous setting, inclusion of periodic and aperiodic inspection intervals, perfect and imperfect inspections, deterministic and probabilistic actions, stationary and non-stationary environments (also treatable within a semi-Markovian context), as well as planning in an infinite or finite decision horizon. In an early work, Madanat & Ben-Akiva (1994) adopt POMDPs for decision-making for highway-pavement networks. Ellis et al. (1995), and Corotis et al. (2005) demonstrate use of POMDPs for bridge inspection planning. Papakonstantinou & Shinozuka (2014a, 2014b) provide a thorough overview of solvers suited for solution of large-scale and more realistic problems.

In an example from (Papakonstantinou & Shinozuka, 2014b), a POMDP policy is described for decision support in the face of corrosion of the steel in a reinforced concrete wharf deck slab. A spatial stochastic corrosion model is used, as specified in (Papakonstantinou & Shinozuka, 2013), which defines the transition probability matrix. In accordance with AASHTO specifications, four discrete states (conditions) are assumed: condition 1 (less than 10% damage), condition 2 (damage between

10%- 25%), condition 3 (damage between 25%-50%), and finally condition 4 (over 50% damage). Implementation of the POMDP algorithm is exemplified in Fig. 5 for two consecutive steps of one possible state evolution scenario. In decision step 119 (top plot) the structure actually lies in condition 2, while the decision-maker attributes a mere 28% to this condition. The POMDP computed policy indicates a visual-inspection at the beginning of the next step. Once this is carried out, the belief is then updated to 81.20% with respect to the possibility that the system (indeed) lies in condition 2. For this belief state, the computed policy plan suggests execution of a minor repair action and subsequently indicates that visual inspection should be chosen as a cost-effective observation option, which will eventually witness the shift of the structure in the improved condition level 1.



Available policy options:

replace and no-inspection
major-repair and monitoring-inspection
major-repair and visual-inspection
major-repair and no-inspection
minor-repair and monitoring-inspection
minor-repair and visual-inspection
minor-repair and no-inspection
no-repair and monitoring-inspection
no-repair and visual-inspection
no-repair and no-inspection

Fig 5. Detailed look at two consecutive policy steps (top to bottom) according to the POMDP computed policy. Figure reused from (Papakonstantinou & Shinozuka, 2014c).

The sequence of policy steps illustrated in Fig. 5 reflects a typical policy planning scenario for infrastructure management; the optimal policy typically favors major-repair actions in early stages, taking advantage of the improved and low deterioration rates. Instead, as demonstrated in a related example (Papakonstantinou et al., 2016), in the case where availability of permanent monitoring data is assumed, the algorithm will tend to exploit the more precise (in comparison to the visual-inspection) information provided by the monitoring system, and strategically suggest more frequent and inexpensive minor repair actions. The latter pro-actively prevents the evolution of severe damage, thus alleviating eventual need for replacement.

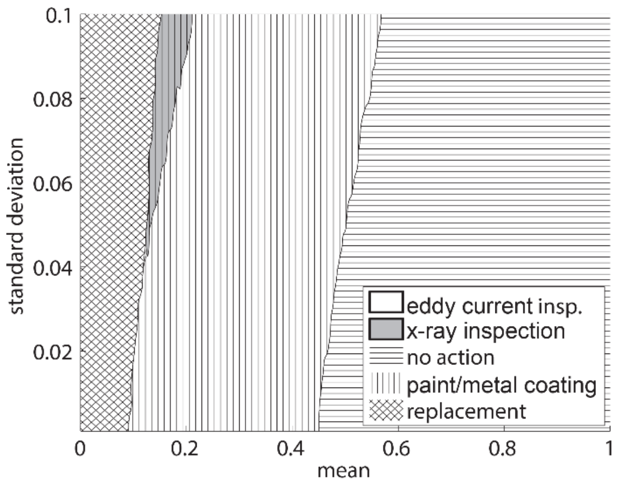


Fig. 6. Policy map for horizon 3 of the bridge maintenance problem in Schöbi & Chatzi (2016).

A valuable outcome of the POMDP policy planning process, lies in the possibility to produce decision maps, as illustrated for the continuous POMDP equivalent in Schöbi and Chatzi (2016). Once an indicator of the system's condition (state) is established, then the belief vector may be summarized via the first two moments (mean and variance).

Consequently, in a finite horizon problem setting, an optimal policy map may be formulated per horizon *n*, as illustrated in Fig. 6, indicating a recommended action (intervention) and corresponding inspection. Such policy maps may be exploited as decision support tools by operators and decision-makers.

POMDPs may further be extended to tackle problems of multiple components or systems, as exemplified in Memarzadeh & Pozzi (2016). The provided example on a farm of 25 wind turbine components reveals the importance of availability of observations from these components, in the form of inspections, for cost-effective management of the farm. However, POMDP are limited when considering the joint optimization of inspections and monitoring in a redundant structural system, due to computational costs. In this case, approximate solutions through heuristic policies may provide a pragmatic solution (Luque and Straub, under review).

5 Conclusions & Outlook

This review work outlines available alternatives for decision-making under uncertainty, shedding focus on the POMDP variant. This tool employs estimates on the condition of the system, extracted by means of uncertain or incomplete observations, for policy planning under the influence of deterioration processes, while assuming availability of actions whose effects are stochastic. Although not so far exploited in this sense, POMDPs and their computed optimal policy trajectories could be utilized in order to feed the components of pre-posterior analysis tools for quantifying the Value of Information of Structural Health Monitoring systems. This is to be explored as a next research direction by the authoring team.

Acknowledgements

The authors would like to acknowledge COST Action TU1402 on "Quantifying the Value of SHM" and COST Action TU1406 on "Quality specifications for roadway bridges, standardization at a European level" for making this work feasible through promotion of an extended network of collaborators across diverse domains.

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Monitoring during life cycle of bridges to establish performance indicators

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.2

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Abstract. In future, additional and more detailed data are needed about the current conditions of bridges for preventive maintenance management. Monitoring procedures are not merely able to provide key performance indicators for a specific point in time, but also over a period. These KPIs must be selected in such a way as to permit substantiated statements about the present and future condition of bridges. For this reason, greater efforts must be made to define the significant KPIs for the various types of bridges, and show how these figures can be reliably determined. Both the COST Action TU1402, and TU1406 offer important approaches which, properly combined, can deliver substantial added value to the calculation and description of the condition of bridges in the interest of proactive maintenance management.

Keywords: preventive maintenance, bridge inspection, monitoring, life cycle, performance indicators

1 Introduction

An efficient road transport infrastructure is the prerequisite for guaranteeing mobility and economic growth. The increase in goods traffic recorded in recent years has led to a large proportion of bridges being pushed to the very limits of their capacity, and a further rise in traffic volume is predicted. It is also important to note that many road bridges are now getting on in years, and several of the old bridges are displaying specific structural shortcomings. Their increased age is accompanied by their deteriorating condition. All in all, these two opposing trends of strong increases in traffic volume on the one hand and increasing age on the other are producing a growing need for action regarding the maintenance and upgrading measures required. A wider scope of reliable information, which in part goes far beyond that traditionally provided, is required for the targeted planning, assessment and implementation of building measures in the life cycle of bridges. Here, it is important to identify the key performance indicators (KPIs) and to be able to substantiate those using verified figures. This is where monitoring can make an important contribution. The principal aspects of integrating monitoring into the life cycle of bridges are examined below.

2 Monitoring as part of the current bridge maintenance regulations

In addition to providing inventory data, the inspection of bridges in accordance with DIN 1076 [1] is of fundamental importance to the maintenance of road bridges from a legal, technical and fiscal perspective. The RI-EBW-PRÜF [2], in which the formula for assessing the condition of bridges during the bridge inspection under DIN 1076 has been documented, has been introduced as a further policy for road bridges. The bridge inspection is essentially conducted visually and at close hand. In line with the above policy, its purpose is to record and describe damage to structural elements and to assess the damage in terms of stability, safety for traffic and durability in conjunction with a condition assessment of the entire construction. The maintenance plans of the road authorities result from an analysis of these findings, and can therefore be viewed to be damage-based and thus reactive. This approach is documented for national highways in the rules on “systematic maintenance planning”. DIN 1076 is considered to be the generally acknowledged technical standard, and the primary functions of bridge inspections are fixed in it:

- To detect defects and damage on time;
- As a result, to enable responsible bodies to rectify the above defects before greater damage occurs or traffic safety is impaired.

A specialist engineer who can evaluate both the structural stability and the constructional state of the bridges must be entrusted with the inspections. It must also be considered, however, that the result largely depends on the experience of the test engineer, and is therefore subjective. Added to this is the fact that the bridge inspection under DIN 1076 only represents a snapshot. Transient action (impact) and changes in condition are not taken into consideration. Due to the rigid inspection intervals, there is a temporal delay when detecting damage. Damage inside the bridge structures is also only diagnosed once it has become visible.

Legal aspects of the bridge inspection are also of key importance when classifying bridge inspections. For this reason, the federal government and “Länder” road laws and administrative regulations contain rules on the responsibilities of road authorities. These results in a special responsibility within the bridge inspection in that the bridge inspector must, where necessary, intervene directly [3]. This means that instrumental monitoring cannot be a complete substitute for bridge inspection at close hand, but must rather supplement it with aspects that are important in view of the growing need for information.

Particularly in view of the increased need for maintenance and upgrading of bridges, it is essential to acquire further information about action and resistance together with the associated findings derived from this, for example regarding system behavior and condition, reliability and the remaining service life. Continuously incorporating this information in the decision-making process makes it possible to move from a damage-based, reactive approach to fore-cast-based, preventive management maintenance.

Known faults in existing bridges and the growing traffic burden have in the recent past already led to a change in maintenance management at a national level. The calculation and upgrading to safeguard future requirements also takes place in bridges in the federal trunk road network where there is no specific damage pattern. Without supplementary regulations, the current calculation standards designed for the building of new bridges are unsuitable for evaluating the load-bearing capacity and serviceability of older bridges. The assessment must also consider the regulations and requirements at that time, the load-bearing system used, the materials deployed, the present condition of the bridge and practical experience to be able to make an accurate judgment about existing constructions; calculation guidelines have been compiled for this purpose [4]. These provide a self-contained method for calculating existing road bridges that makes use of the advantages of the semi-probabilistic approach in the current calculation standards. Spare structural capacity and building materials can be further exploited without compromising the level of reliability [5]. It is characterized by a graduated procedure in which verification management is modified step by step. Aspects of monitoring may be relevant in Steps 2 to 4.

- Step 1: Verification in accordance with the Eurocode, i.e. as for a new build;
- Step 2: Verification with specific rules added to Step 1 that looks into action and resistance;
- Step 3: Verification taking account of the measurement readings taken on the bridge;
- Step 4: Verification using scientific methods.

Every calculation concludes with an engineer’s assessment of results and assignment to verification classes:

- A (no restrictions regarding load-bearing capacity and serviceability with calculation according Step 1);
- B (no usage restrictions when applying the rules according to Steps 2 to 4);
- C (restricted conditions of use or compensation procedures).

The calculation results feed into a feasibility study for bridge upgrading based on the short-comings identified. If the results from the bridge inspection or usual engineering models only deliver imprecise or unsatisfactory information in terms of load-bearing capacity, serviceability or possible remaining service life, reliable and substantiated information can be obtained using monitoring. Additional information on action and resistance and on the system behavior obtained through monitoring may be integrated in calculation Steps 2 to 4 or calculation classes B and C. Monitoring as a compensation procedure is limited to calculation class C.

The cause of calculated excessive stress may lie in modeling that is too crude or not sufficiently realistic. Frequently, the load transfer on the actual system is more favorable than in the idealized model. Stress peaks may arise at individual points in calculation models, e.g. producing support joints that never occur in real life. The rigidity ratios of individual supporting elements or connections to them may also substantially influence the stress distribution. Measuring the actual strain that occurs under moving traffic produces clarity about the real load transfer, the position of the strain neutral axes of individual girders, contributory parts of structures etc.

With fatigue monitoring, not only the amount of stress or the stress intensity factor range are significant, but also the number of associated load changes. The actual traffic is greatly simplified for the calculated verification, however the differences between calculated and measured fatigue stress may be considerable. It generally makes sense to measure the strain or stress intensity factor ranges that actually occur under moving traffic. It is possible to conduct the verification of fatigue and remaining service life directly with the composite stress factors determined during measurement.

Where the calculated exceeding affects several structural elements of a construction or several components along a route section, it may be worth checking whether the load model used (e.g. LM1, BK60/30) corresponds to the actual traffic present. This should be recorded as accurately as possible to adjust the load model to the local traffic load.

The scientific methods in Step 4 are frequently based on complex modeling using very powerful finite element programs. The more complex a model of this kind is, the more opportunities there will be when entering diverse parameters for structural elements or building materials about which there is often no precise knowledge (e.g. Poissons ratio, the bond conditions etc.). Construction measurements are similarly effective when calibrating these complex models using the actual load-bearing or deformation behavior of the construction.

In the case of smaller bridges and especially when there are no as-built documents, load tests may be the last resort when it comes to verifying sufficient load-bearing capacity. Load-bearing capacity is generally calculated by recording a load deformation curve and monitoring the formation or initiation of cracks in critical areas during a cyclical increasing of the load. Tests of this nature require the definition of abort criteria which reliably exclude failure but are still close enough to the maximum load to produce a meaningful result. In addition to the load equipment, a corresponding monitoring system is also necessary.

3 Monitoring during the life cycle of bridges

Application areas for monitoring bridges are varied and occur throughout the service life; in practice, growing use can be seen in the following cases:

- Bridges where there is incomplete knowledge of risks (e.g. geological, seismic, environmental, during construction and during operation);
- Candidates for replacement or upgrading (e.g. with the aim of evaluating the need for intervention or assessing the efficacy of measures);
- Bridges with known shortcomings to extend the service life (e.g. as compensation procedure);
- Bridges of particular significance at the network level (e.g. the road network would be restricted to an unacceptable degree in the event of failure or limited availability);
- New construction methods, innovative designs, techniques or materials (e.g. as the basis for consents in an individual case, for approvals and to add to experience).

The incorporation of monitoring in maintenance management based on life cycle offers opportunities and benefits for road authorities and users of the road infrastructure in terms of safeguarding availability and extending the service life of bridges. Extending this to the entire life cycle entails looking at monitoring possibilities even during the planning phase, measuring and testing during construction and complete monitoring of the bridge in operation. The following potential is recognized in [6]:

- Guaranteeing the quality and checking the functions of bridges and their safety-related structural elements through continuous monitoring;
- Assessing the actual condition of bridges or bridge components based on neutral measurement data;
- Increasing safety due to the early detection of safety-related changes;
- Facilitating reliability analyses to safeguard availability;
- Optimizing repair and maintenance measures through the monitoring of ageing behavior and the development of damage patterns over time;
- Verifying the effectiveness of maintenance measures (including upgrading);
- Monitoring the conditions of use (e.g. complying with the maximum permitted gross vehicle weights);
- Determining the actual action and stress data to produce an input variable for further analyses of bridge management (e.g. fatigue monitoring, standardized calculations, updating of load models).

For informed statements and forecasts to be made about the condition, the load-bearing capacity and the remaining service life of the structural element/construction, it is crucial that the data collected are subjected to a structural analysis. As well as pure data logging, a monitoring system must always be seen in conjunction with an adapted data analysis and targeted assessment procedure.

BASt currently has a research objective which involves developing innovative monitoring systems to provide extensive information in real time and for an integrated evaluation as the foundation for smooth operation and optimized maintenance management throughout the life cycle. This will deploy suitable sensor technology in sensor networks, combined with data analysis and assessment procedures. The use of measuring technology will provide previously unavailable information regarding the availability of the bridge and the damage and changes in condition to be expected. In combination with forecasting models and including the information from databases that has been available until now, the foundation will be laid for a preventive maintenance management focusing on reliability. Where necessary, an appropriate software system - or expert system - will give automated warnings to users, authorities and owners (adaptive structure, “Smart bridges”) [7].

4 Use of monitoring during the life cycle of bridges

4.1 Monitoring for quality assurance and function checks

The life cycle management of bridges, in conjunction with the development of management systems (e.g. BMS [8]) and specific life cycle analysis procedures [9], has become a focus of attention. Life cycle management is understood here to be the integrated collection and analysis of all measures needed to construct, use and demolish a bridge. It is described as the realization and optimization of the analyzed processes of planning, construction and operation/maintenance, with the aim of maintaining the best possible condition of a bridge or a group of bridges in the network throughout their use phase at the lowest possible cost. Environmental, user and societal aspects may be relevant here in addition to the costs for the operator. Life cycle management comprises a calculation of service life during the planning phase, an adaption of service life models to practical implementation during the construction phase and structural inspections at intervals supplemented by the permanent monitoring of critical areas of the construction and the continuous updating of the forecast service life based on this during the operating phase.

Monitoring can, for example, be the foundation for optimized planning of maintenance measures to restore durability. The functionality and efficacy of safety and maintenance measures can then be checked even in inaccessible areas of the bridge, enabling verification about whether the success of a maintenance measure has in fact materialized and is having a lasting effect. Areas of use include maintenance measures aimed at preventing the penetration of pollutants or at maintaining or restoring the passivity of the reinforcement [10].

A further example concerns the area of bridge upgrading. The monitoring and checking of the effectiveness of the reinforcement measures conducted can, for example, include a survey of bridge reactions conducted for a limited period, combined with an assessment to evaluate the structural measures taken. This approach is important when there is a new construction method and innovative designs for which no reliable experience is yet available, whereby monitoring concepts centered on specific objects are needed here [11].

4.2 Monitoring of action (impact)

As well as providing information about traffic statistics, accurate knowledge of traffic load action is useful when deriving realistic load models and as a basis for a realistic estimate of the capacity utilization of bridges. In this context it is important for the determination of the reliability and remaining service life of the bridge. Another objective could be the direct registration of extreme loads and the derivation of traffic management measures. The information collected involves load spectra, vehicle loads, vibration coefficients, other effects such as temperature, wind speed and moisture as well as their classification according to intensity and frequency. With the help of special weighing equipment, e.g. WIM (Weigh-in-Motion) systems, weights and vehicle types can be determined, giving a good indication of load action [e.g. 12]. The accuracy that can be achieved largely depends on the accuracy of the sensors and on a reliable estimate of vehicle speeds. The sensor elements also react to dynamic effects arising from vehicle/carriageway interaction. By taking the additional logging of axle distances into consideration, e.g. using induction loops in the carriageway, it is also possible to obtain information about the axle loads of vehicles. Alternatively, deflection and strain measurements on bridge expansion joints can also be carried out to determine the traffic load [e.g. 13]. Speed can be measured using radar measurements or a number of laser photoelectric barriers.

During the practical application of bridge monitoring, impact is frequently not recorded directly but rather indirectly through the reactions of structures (deformation of supports, strain, open cracks) [refer to 12 for example]. The structural response regarding the causal loads in terms of size and pressure points is calculated for this. Previous approaches to calculate traffic load were restricted to linear elastic systems and to bridges with a narrow span. Complex algorithms are needed to analyze the measurement data, because the reactions of structures for several vehicles may overlap. This method has, however, now achieved a high degree of accuracy and reliability. The acting traffic loads are determined by permanently measuring the strain using a calibration function that describes the structural properties, for example the influence line. The influence line can be calculated in advance using a test load or mathematically, and then classifying the general load data obtained in this way. If the dynamic system behavior is known, the traffic load can additionally be calculated from the dynamic structural response of the bridge, e.g. by working backwards on the basis of model calculations.

4.3 Monitoring of the condition

Bridges must be monitored regularly so that damage to them can be detected at an early stage and so their condition can be recorded. These tests and monitoring are currently carried out in accordance with DIN 1076. The use of bridge monitoring arises in the following cases [14]:

- Excessive deformation of the deck (measurement of deflection, strain, incline: thresh-old monitoring);
- Moisture penetration/efflorescence/washout on the deck (e.g. multi-ring electrodes: measurement starting from new build, frequency once a month, possibly threshold value procedure);
- Flaking with exposed corroded reinforcement and reduced cross-section through to the partial failure of the reinforcement (e.g. corrosion sensor: measurement starting from new build, frequency once a month);
- Depth of ingress of the chloride front (e.g. chloride sensors: measurement starting from new build, frequency once a month, possibly threshold value procedure);
- Checks of cracks and crack widths, taking impact areas into consideration (spray mist/sprayed water area/other area) through to crack movements under traffic loads (e.g. inductive path recorded, threshold monitoring);
- Leaky seal (e.g. multi-ring electrode: measurement starting from new build, frequency once a month.

Damage characterized, for example, by a change in material parameters and possibly in cross-section dimensions, e.g. due to corrosion of the reinforcement or concrete flaking, changes the resistance of the structural element at certain times and is therefore also relevant to load-bearing capacity. Common monitoring systems deliver results from which the damage on a bridge can either be obtained directly or can be used to draw conclusions about existing damage. No examples are known using reproducible information about the variables of a calculation on an undamaged system and the variables to be calculated from the measurement. There are no evaluations that provide stochastic descriptions of action or resistance. Monitoring systems leading to reliable statements regarding load-bearing capacity and remaining service life are only available for certain aspects [14].

4.4 Monitoring safety-related system elements

Monitoring with an early warning function is of great importance for bridges with an especially high risk and damage potential. The fundamental prerequisite here is a precise knowledge of the limit state as well as the non-destructive online analysis of the condition of the construction using objective and dependable measurement methods and decision-making tools. The type of monitoring strategy to be applied will depend on the specific problem. The following monitoring strategies are basically possible [15]:

- Predictive strategy: this requires adaptive models which adjust or can be adjusted to the respective state of the bridge; or
- Threshold monitoring: in this case, there is continuous monitoring of threshold values (e.g. limit to strain, cracks) without the need for a model.

The condition of the bridge can be assessed according to whether a defined threshold value (test threshold or alarm value) has been exceeded or fallen short of. Threshold values can, for example, be defined by the maximum permitted number of individual events or number of times they exceed or fall below a permitted range. It is usually necessary to know the zero state here, however. The threshold value may be directly defined for the measurement or a benchmark. If it is not possible to measure the benchmark directly, a corresponding measurement must be found. The important factor here is an unambiguous and so reversible connection between benchmark and measurement, using which the alarm threshold values of the measurement can also be transferred to the benchmark (and vice versa). A complex correlation often exists between the measurement and the benchmark or target value, which must be accounted for by adjustment or using a differentiated specification of the threshold values concerned, e.g. by specifying seasonal or temperature adjusted threshold values for deformation and strain. Alarm figures may be stipulated using upper and lower threshold values according to the marginal conditions and impact based on proof load(s) and on parameter or sensitivity studies.

In this context, the realistic simulation of the damage and bond behavior of reinforced concrete based on suitable material laws is an important aspect in establishing the limit or alarm thresholds. It is necessary to think about how the monitoring measure might influence the assessment of damage spread. Frequently, it is possible to achieve a good linear approximation of the damage trends between the as-built state and the end of use, so that the mathematically identifiable residual safety can be used as a good prediction of the remaining service life.

In the course of the monitoring concept there is also a need to establish whether and how an alarm (e.g. by email, text message, barrier blocking system, traffic lights etc.) may be triggered when the threshold value has been exceeded. This must also contain plausibility checks, control measurements or redundant measurement procedures which preclude a false alarm. The online monitoring must always be coupled with an emergency or action plan that states where the relevant measures are to be initiated if a limit value has been exceeded. The emergency plan in the form of a traffic light system in Figure 1 [16] offers an example of this.

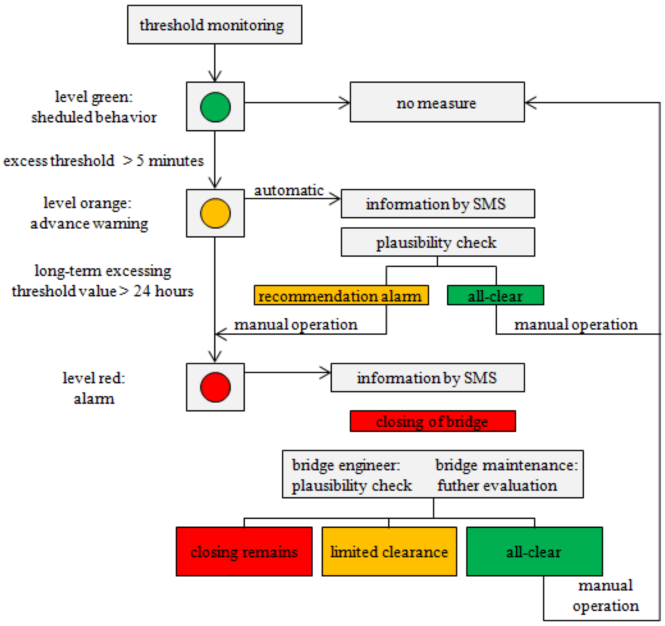

Fig. 1. Example of an alarm plan to regulate the responsibility in the case of exceeding limits for the threshold monitoring of a bridge [16]

The extraction of features from the measurement data recorded and the subsequent diagnostics constitute a key point of the monitoring system. There are basically two available models for monitoring constructions. When physical models are used, a parameter-based calculation model is adapted based on measurements of the actual structure. Due to their complexity, bridges are generally modeled using the FE method. The idealized bridge parameters such as dimensions and materials mean that these models are not able to reflect all aspects of actual structural behavior, however, so that the analytical forecasts then deviate from the data measured on the bridge. An updating and calibration of the unreliable parameters in the FE model (Finite Element Model Updating, model adjustment) is therefore required to achieve a further improved approximation of the actual structure; see [17].

The non-physical model is a data-based method used to recognize patterns in the measurement data. Non-physical methods require training using learning techniques before their introduction. Mathematic algorithms are deployed that detect changes in a system using measurement data. For this, training data must first be accumulated through static or dynamic measurements to practice the reference condition. New measurements can subsequently be compared to this condition. Variations from the reference state suggest a change to the structure. Raw measurement data (e.g. vibration accelerations or structural element strain) or alternatively existing processed information (e.g. modal properties) is used to extract features. In order to obtain as many events as possible (impact of temperature and traffic), data from a sufficiently long period of time are needed. For data evaluation using cluster analysis, neural networks and regression analysis, see [17].

The advantage of the physical models lies in the potential realistic reflection of the construction to be monitored. This enables damage or a change in the behavior of the structural element to be detected, located and quantified. The preparation of physical models (and particularly the comprehensive FE models) is extremely time-consuming. There is moreover a risk of obtaining no clear solution when identifying the bridge. Each individual bridge demands its own model adaptation, making it impossible to find any generalized monitoring measure. By contrast, measurement data can be evaluated extremely quickly using non-physical models, making these models suitable for use in continuous monitoring of a bridge, where necessary in real time. One disadvantage is the large amount of measurement data needed for system training. As for implementation in practice, this model can largely be deployed independently of the bridge, and can therefore be used anywhere.

4.5 Monitoring as compensation procedure according to the calculation guidelines

The calculation guidelines have given engineers a tool for assessing the load-bearing capacity and serviceability of existing bridges. Compensation procedures are specified in the guidelines to safeguard further traffic use. A distinction is made between traffic compensation procedures and compensation monitoring procedures. Whereas the traffic compensation procedures are described in detail, the guidelines contain no explanations about compensation monitoring procedures and their effects on the reliability of the bridge.

In line with the reliability theory in civil engineering as the foundation for valid design rules, the necessary target reliability can be achieved again by gaining added safety through monitoring, thus compensating for the insufficient reliability level of the defective bridge. Initial approaches depending on the intensity of the monitoring are described in literature [e.g. 18], although the target values are only defined intuitively. Generally applicable methods for the application of measuring technology assisted monitoring concepts as compensation procedure have been developed in [19].

The key requirement for a compensation monitoring procedure is that it generates a sufficient safety gain. To support engineering practice, a quantification of the safety gain in a semi-probabilistic assessment concept is useful. In this concept, the safety gain is expressed through the influence of reduced partial safety factors and adapted characteristic variables. Based on probability theory, reduced partial safety factors for the “threshold monitoring” and the “action monitoring” were determined subject to the reliability of the monitoring itself.

The threshold monitoring describes the direct monitoring of the defective limit state. Exceeding a previously defined limit triggers an alarm mechanism. In the probabilistic approach, and assuming an adequate reaction to successful detection of damage, the probability of failure with threshold monitoring ($P_{f,Ü,S}$) results from the intersecting set of the probability of failure without compensation procedure (P_f) and the probability of failure of the threshold monitoring itself ($P_{f,S}$); see Figure 2 (left).

$$P_{f,Ü,S} = P_f \cdot P_{f,S} \tag{1}$$

Threshold monitoring affects the actual limit state and therefore also all input parameters, enabling the partial safety factors to be reduced according to the probability of failure of the threshold monitoring. Provided that the structure failure itself is independent of the probability of failure of the threshold monitoring and with the help of full probabilistic analyses and consideration of the safety gain, the partial safety factors result as shown in Figure 2 (right).

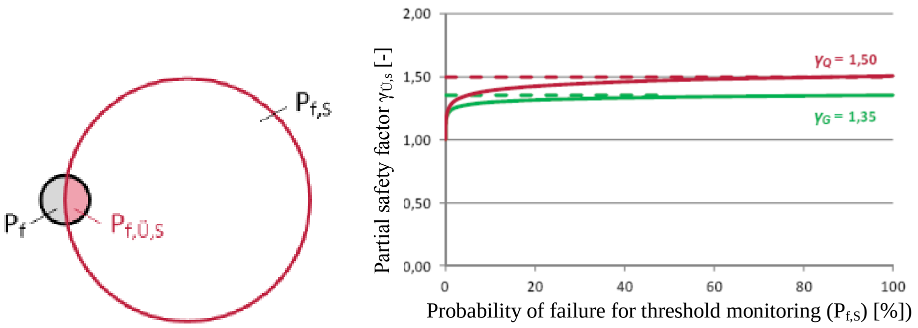

Fig. 2. Probability of failure of the threshold monitoring (left) and reduced partial safety factors γ_Q and γ_G due to threshold monitoring (right) [19]

The measuring technology design of threshold monitoring depends on the shortcoming in the existing bridge that needs to be compensated for. Experiences when calculating concrete bridges showed typical shortcomings in the SLS when verifying decompression and limiting crack widths and also in the ULS when verifying shear strength, transverse bending load-bearing capacity, fatigue and stress crack corrosion. Depending on the shortcoming, performance indicators with corresponding benchmarks or measurements must be identified and the corresponding measurement ranges defined. Suitable sensor technology that is coordinated to this must be selected (see [19]).

With impact monitoring, potential impact from traffic is detected before it can actually affect the bridge. In the event of exceeding a previously defined limit, the bridge is closed. From a probabilistic perspective, impact monitoring means the elimination of uncertainty on the impact side above the prescribed impact limit, leaving a reduced probability of failure in relation to the probability of failure of the monitoring procedure itself; see Figure 3 (left). Reduced partial safety factors $\gamma_{Qi,Ü,E}$ in relation to the prescribed impact limit Q and the probability of failure of the impact monitoring $P_{f,E}$ are specified in Figure 3 (right). Here too, the calculation is made with the help of full probabilistic analyses and consideration of the safety gain.

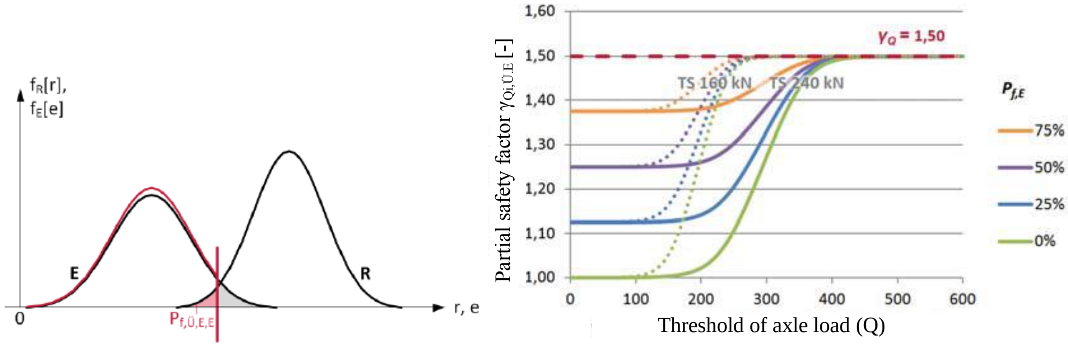

Fig. 3. Probability of failure during impact monitoring (left) and reduced partial safety factors due to impact monitoring (right) [19]

A series of case-by-case assessments – also concerning the reliability of the monitoring - is required so that the partial safety factors can be adapted to certain situations. Only with experience in practical applications is it possible to prepare a practical course of action and supplement the calculation guidelines with an annex of compensation monitoring procedures in the same way as already exists for traffic compensation procedures. Guidelines on specifying and monitoring threshold values are also highly relevant to practical application. Further research and above all more practical application are needed in the area of compensation monitoring procedures.

5 Conclusions

In future, additional and more detailed data are needed about the current conditions of bridges for preventive maintenance management. Monitoring procedures are not merely able to provide key performance indicators for a specific point in time, but also over a period. These KPIs must be selected in such a way as to permit substantiated statements about the present and future condition of bridges. For this reason, greater efforts must be made to define the significant KPIs for the various types of bridges, and show how these figures can be reliably determined. Both the COST Action TU1402, and TU1406 offer important approaches which, properly combined, can deliver substantial added value to the calculation and description of the condition of bridges in the interest of proactive maintenance management.

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Application of DIC to monitor reinforced concrete structures

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.3

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Abstract. The reinforced concrete structures need to be monitored to ensure their structural integrity, but sometimes those measurements are very local and the instrument is complex to locate physically in the structure and may interfere on it. Digital Image Correlation, DIC, is a non-contact and non-destructive experimental technique capable to measure the displacement field in a big region of a structure with a great accuracy. This allows extracting valuable information from the fracture processes of reinforced concrete structures. Critical for the evaluation of the structural integrity. The identification of the energy dissipated by the structure is essential for the identification of the strength mechanisms that are failing in the structure, and to identify a proper repair. In this paper the penetration of a prestress rebar in concrete is measured with this technique and the energy dissipated by different fractures is fully observed. Comparison is made with traditional measurement techniques. Also, using Fracture Mechanics other valuable information is extracted from the fracture processes of the reinforced concrete beam, such as the Mode I and Mixed Mode fracture energy released at each loading step, which is essential to evaluate the elastic energy that the structure can accumulate before collapse. The examples enable to anticipate the importance of DIC for future studies at large scale of fracture in concrete and other materials related to construction.

Keywords: Digital Image Correlation, Fracture Mechanics, damage

1 Introduction

Digital Image Correlation (DIC) is a robust, non-destructive and non-contact experimental technique widely used in the last decades for micromechanical characterization of materials (Chu et al 1985). It uses images of a surface to measure its local displacements, being the input of DIC a pair of images (i.e. the reference and deformed), which are divided in windows to correlate them and measure its displacements (Pan et al 2009). The use of this technique was extended, although still is very scarcely employed, to concrete structures in the last years with successful results. The great complexity and size of the concrete elements were a drawback for the use of DIC, but the information that can be extracted with it is essential for the full characterization of the fracture processes of concrete, which has encouraged its application. Thus, among others, in 2006 Küntz et al. applied DIC to measure the stability of a shear crack in a reinforced concrete bridge during loading. In 2012 Lee et al. compared the measurements done with DIC in a reinforced concrete structure with foil and vibrating wire strain gauges. They concluded that DIC is more versatile and as accurate as the others. In 2013 Dutton et al. used DIC to measure the curvature of a reinforced concrete beam of 3.8m long. They did it imaging 2 regions of 600x400 mm were the deflection was calculated. And in 2014 Fayyad and Lees used it to measure the opening of a crack in a notched beam of 0.8m long.

This paper will focus on exploring the possibilities of application of DIC as a health monitoring technique for civil structures, mainly motivated by its not contacting and non-destructive nature. The use of DIC to characterize the fracture of a reinforced concrete beam is also very useful to understand the complex fracture processes (Bazant and Planas 1997) and the evolution of the crack behaviour. Its combination with fracture mechanics allows us to extract more characteristics of the fracture processes such as the relationship between Mode I and Mode II and the transition between both. For this the stress intensity factors are used and calculated from the strain field ahead of the crack tip. There are many solutions for this calculation such as the interaction integral technique (Dolbow et al 2002) or the method of Yoneyama el at. In 2007, but in this paper we are using the classical polar equations of the linear elastic fracture mechanics (Shah et al 1995). In this paper we are able to measure the discontinuity created by a crack, the crack opening along the height of the beam and to characterize a Mode I and a Mixed-Mode fracture. All those aspects, with the Fracture Process Zone (Saucedo-Mora et al 2012), are the keys to characterize the integrity of the reinforced concrete structures.

2 Experimental: Fracture test of a prestress beam

In this case the DIC method was applied to a three point bending test in a concrete beam with 4 prestressed rebars and a span of 700 mm and a 250x200 mm section. The concrete has a compressive strength of 75 MPa, and the beam has 4 prestressed longitudinal rebars of a 1670C steel with a diameter of 7.5mm located 2 at 50mm from the top of the beam, and other 2 at 40mm from the bottom. Without any shear reinforcement. The imaged area was 390x260mm located beneath the loading point in the centre of the beam front. The images were taken with a camera Nikon D7200, with a size of 6000x4000 pixels (i.e. 24 Megapixels). Resulting in a pixel size of 0.065 mm. The measurement has an error of 5 µm, measured with an undeformed image displaced with a Rigid Body Movement. After the calculation with the deformed images, the displacement field was corrected for Rigid Body Movements, rotation and out of plane movements. Figure 1 is an example of a DIC treatment of an image at an early stage of the cracking process (i.e. half of the peak load), it shows the relative strain between the loaded and the unloaded beam, without considering its initial prestress strain. At the bottom the crack initiation can be identified as a hot spot in Figure 1a. Also, as expected, there are compressive strains in the top part of the beam. Figure 1b shows the deflection of the beam, measured through DIC at the bottom of the image, and it's comparison with the analytical deformation given by the beam theory. In this case any big discontinuity in the deflection can be clearly identified, as in Figure 4a for higher loads.

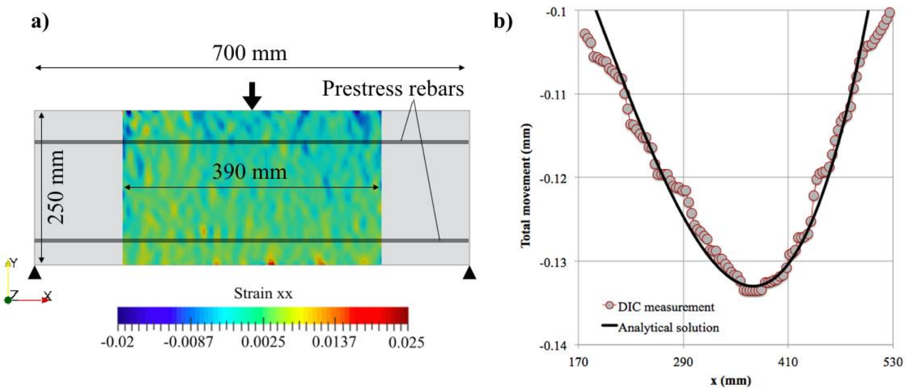

Fig. 1. DIC of the beam at 220 kN (45% of the peak load); a) geometry of the beam and region imaged with the strain in x and b) deflection of the beam measured with DIC compared with the analytical solution.

3 Results

The beam was tested with load/unload cycles with increasing peaks. For higher loads (Figure 2), the crack grows from the location spotted in Figure 1a at an earlier stage with the DIC analysis. Because the displacement field of the imaged region of the beam is extracted with DIC, this displacement can be magnified (100 times in) to plot the deformed shape of the beam with the strain field superimposed, as in Figure 2. This amplification shows that the displacements measured are not noisy, producing a homogeneous deformation. Also, applying an arbitrary threshold of 0.1 in the strain, the crack can be identified. Figure 2 shows the strain concentration at the crack tip and the Fracture Process Zone, as well as the regions under compression on the top of the beam.

Figure 2 shows the interaction between the Mode I crack originated beneath the loading point, and the compressive region on the top of the beam. As the crack grows from 280 kN to 320 kN the compressed region is displaced from the centre of the beam, growing the crack around this region at 360 kN. In Figure 2 the regions in purple are confined under compression and their displacement to one side of the loading point creates non-symmetry in the curvature of the beam. Also the displacement of the compressive regions leads to an increment of the shear in the right side of the beam that ends with a shear crack, which will be later introduced.

This analysis allows us to characterize in detail the crack. Figure 3a shows an analysis of the orientation of the maximum principal strain in a 100x100 mm region of the crack zone at 300 kN. Here, because the eigenvectors are scaled with the value of their associated eigenvalue, we can see how the orientation of those vectors defines the fracture path. As well, it can be identified how the strain of the crack decreases as it approaches the crack tip. In the rest of the sample the eigenvectors of the principal maximum strains with low values are not oriented in any preferably direction. It shows the complex elastic deformation of concrete due to the different local stiffness of the aggregates and matrix that conform the microstructure.

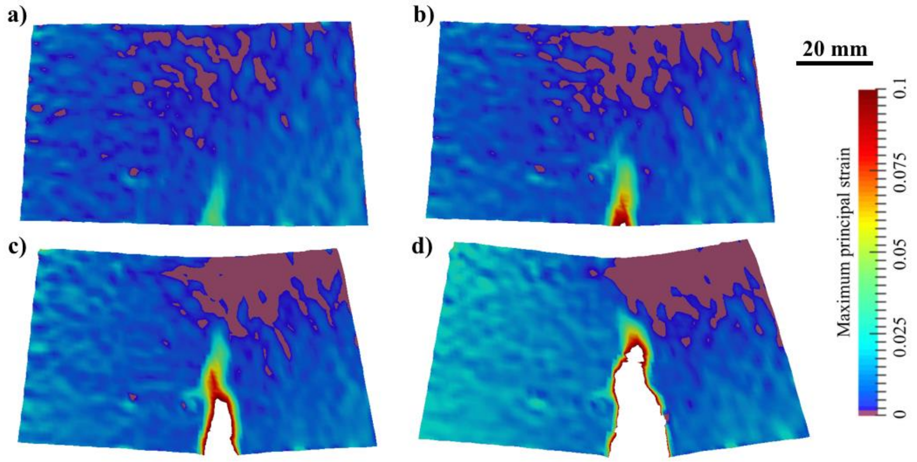

Fig. 2. Deformed shape magnified 100 times and superimposed strain field of the beam loaded at: a) 260 kN, b) 280 kN, c) 320 kN and d) 360 kN.

The regions with higher strains of Figure 3a are not the real strains of the material and are just the derivative of the jump in the x displacement between the crack faces. Only in the elastic regions those strains are the real strains of the material. Extracting the jump between the crack faces along the beam height for different loading steps we can measure the crack opening shown in Figure 3b. In this figure the crack tip was identified as a kink in the small linear elastic deformation that generates the variation of the crack opening plotted in the figure. As the load increases, the tip (in Figure 3b only the open crack is plotted) moves closer to the top of the beam, and the slope of the opening becomes less inclined.

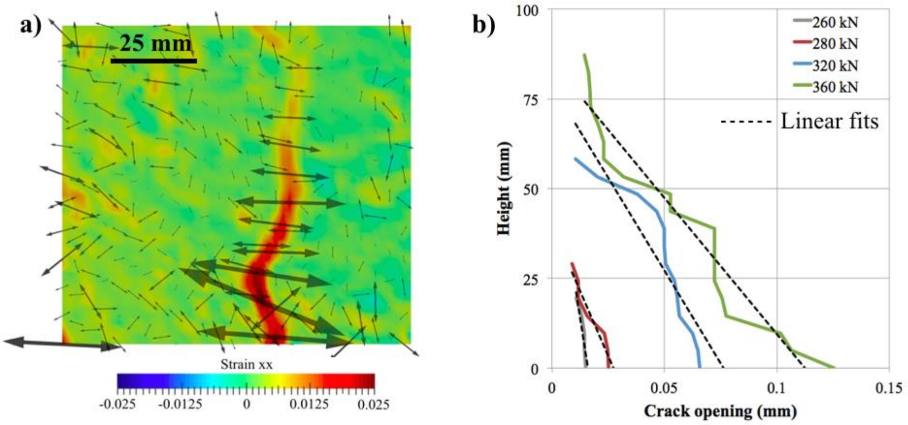

Fig. 3. Analysis of a region of interest of 100x100 mm around the crack: a) strains field in x and eigenvectors of the maximum principal strain and b) crack opening versus the vertical position in the beam.

The crack opening of Figure 3b is just the jump of the total displacement along the beam. Figure 4a shows the total displacement measured at the bottom of the beam at 260 kN, 280 kN, 320 kN and 360 kN, close to the final load of 490 kN. Figure 4a is the evolution of Figure 2b, remarking the effect of the damage. The main difference introduced by the crack is that in Figure 3b almost all the displacement is given by the *y* deflection, and in Figure 4a the jump between the crack faces, mainly in *x* for a Mode I crack, is governing the total movement along the bottom of the beam. Measurements at each load/unload cycle were done with a gauge of 80mm at the left and right of the Mode I crack, showing in Figure 4b the evolution of the residual strains in the beam. This result shows that apart from the main crack, the damage is spread by the rebar around the main crack region. This is a direct effect of the pull out of the rebar from the crack faces and the transfer of stresses between them, which doesn’t relax the regions around the crack faces as in the materials without any kind of reinforcement, prestress rebars in this case. The loaded results of Figure 4b are a combination of the elastic deformation and the opening of the microcracks shown during the unload.

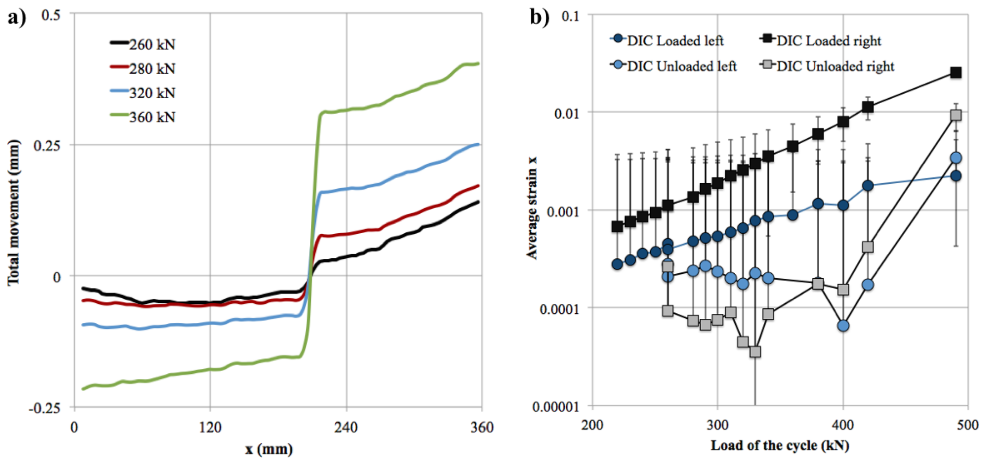

Fig. 4. a) movement profile along the bottom of the beam for different loads and b) measurement of the average strain on the left and right of the crack with a gauge of 80 mm for the beam loaded and unloaded.

70 kN before the final load, at 420 kN, another crack appeared on the right of the Mode I crack previously described. This crack goes from the region with high shear strain at the bottom of the beam to the region on the top confined under compression (Figure 2d). This crack grows dynamically, appearing with its 85 mm length between 400 kN and 420 kN. As a comparison, the Mode I crack of the center grows 20.1 mm in this gap of 20 kN.

4 Conclusions

The paper presents the potential applications of the Digital Image Correlation Technique to monitor structural concrete, paving the way for future developments and applications in civil structures. Complex mechanisms that involve damage and the lost of capabilities of the structural element are measured successfully using this technique, with a simple setup. The novelty of the methodology described in this paper is the capability to describe not only the crack location and opening, also the way that the elastic energy is released at each fracture step. Also the non-contact and non-destructive nature of the technique enhances its applicability to structures in extreme environments and locations complex to reach, showing the potentiality of this technique to be used for structural health monitoring.

Acknowledgements

The authors want to acknowledge Rogelio Sánchez Verdasdo for his support during the test with the manipulation and setup of the cameras. Luis Saucedo-Mora wants to acknowledge the financial support given by the Secretariat of State for Research, Development and Innovation of the Spanish government under the grant IJCI-2014-19362 associated to a Juan de la Cierva Incorporation Fellowship.

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System Reliability of Bridge Structure Subjected to Chloride Ingress

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.4

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Abstract. The parameters relevant for prediction of service lifetime with respect to chloride ingress are associated with large uncertainties. Full-scale measurements are in demand for conditions which are as homogeneous as possible. The present paper first summarizes statistical distributions which are obtained based on measurements from the Gimsøystraumen bridge in Norway. A large number of chloride profiles are available, and for each of these the diffusion coefficient and surface concentration (due to sea-spray) are estimated. Extensive measurements of concrete cover are also performed. These probability distributions are subsequently employed as input to a prediction model for chloride concentration at the steel reinforcement for a single but arbitrary position along the reinforment. Since the input parameters are represented in probabilistic terms, the chloride concentration is also a stochastic quantity. Furthermore, introducing the critical chloride concentration on a similar form, the probability of exceeding the critical threshold is determined as a function of time.

In order to address chloride attack on the entire bridge, a system model with 90 components is next introduced. This model is employed in order to perform reliability updating based on observations at a number of sites along the bridge. First-order (FORM) reliability methods typically become inaccurate for large systems of this type. Crude Monte Carlo Simulation (which can be more accurate) will easily demand impractical efforts in terms of CPU-time, and a more efficient Monte Carlo simulation method is accordingly applied. It is shown that this typically reduces computation times by a factor of around 10.

Keywords: System reliability; Enhanced Monte Carlo; Chloride ingress; Bridge test data.

1 Introduction

A large number of chloride profiles have been obtained from the Gimsøystraumen bridge which is located in the Northern part of Norway. For the superstructure profiles from 725 locations were collected. For the columns sampling was performed for 168 locations (Skjølsvold, 2001). For each of the profiles, the corresponding diffusion coefficient and the chloride surface concentration were estimated. Extensive measurements of concrete cover were also performed. (Note: The values for statistical values given herein may deviate slightly from those of (Skjølsvold, 2001) due to further refinement of the chloride profile data in that report).

The corresponding probability distributions are subsequently employed as input to a model for prediction of chloride concentration at the steel reinforcement. As the input parameters are represented in probabilistic terms, the chloride concentration accordingly becomes a stochastic quantity. The critical chloride concentration is also introduced on a similar form. As the next step, the resulting probability that the concentration at the reinforcement exceeds the critical threshold is then determined as a function of time, see also (Hynne et. al., 2001). Parameter variations are performed with respect to the input statistical models. In particular, the effect of introducing a diffusion coefficient which varies with time is investigated.

In order to address chloride attack on the entire bridge, a system model with 90 components is next introduced. This model is employed in order to perform reliability updating based on observations at a number of sites along the bridge. The computations are performed by application of the so-called enhanced Monte Carlo simulation method (Næss et. al., 2009 & 2012).

2 Probabilistic modelling based on full-scale measurements

2.1 General

The Gimsøystraumen bridge is located in the Lofoten area in Northern Norway, see Figure 1. This bridge has served as a "test bridge" for many purposes including assessment of different types of repair methods.

Fig 1. The Gimsøystraumen bridge in Northern Norway (Lofoten area)

The objective of the present study is to assess the merits of relevant probabilistic models based on full-scale data and to show how they can be applied for the purpose of lifetime assessment with respect to chloride ingress.

Furthermore, it is intended to illustrate how information from monitoring and inspection can serve the purpose of reliability updating. In order to achieve a realistic model of the entire bridge structure, a system model is subsequently established. As computation of the corresponding system reliability as a function of time easily becomes quite demanding, it is also demonstrated how so-called enhanced Monte Carlo Techniques can serve to make calculation of the structural reliability more efficient than the crude Monte Carlo techniques (abbreviated simply as MC)

2.2 Statistical analysis of test data

For each of the three parameters that were measured or estimated based on the measurements (i.e. diffusion coefficient, surface concentration and concrete cover), the applicability of various analytical probability distributions were tested by plotting in different types of probability paper. A ranking was performed based on the regression coefficients. As an example, a summary of the results are shown in Table 1 for the diffusion coefficients obtained for the east side of the columns.

Table 1. Diffusion coefficient (Multiplication by 10^{-12} gives the values in m^2/s)

Prob. model	Regression line	Mean value	Standard deviation	Sample variance	R^2
Normal	y = 1.6051x – 2.0384				0.9815
Gamma	y = 0.7388x – 0.6466				0.7934
Gumbel	y = 2.1481x – 2.1085	1.27	0.64	0.41	0.9899
Weibull	y = 2.3307x – 0.8316				0.9948
Lognormal	y = 1.9765x – 0.1641				0.9809

As observed, the Weibull distribution gives the highest regression coefficient, R^2. The measured and analytical distribution functions as plotted in Weibull probability paper are compared in Figure 2. However, in general all the different distributions give quite high values for the regression coefficient.

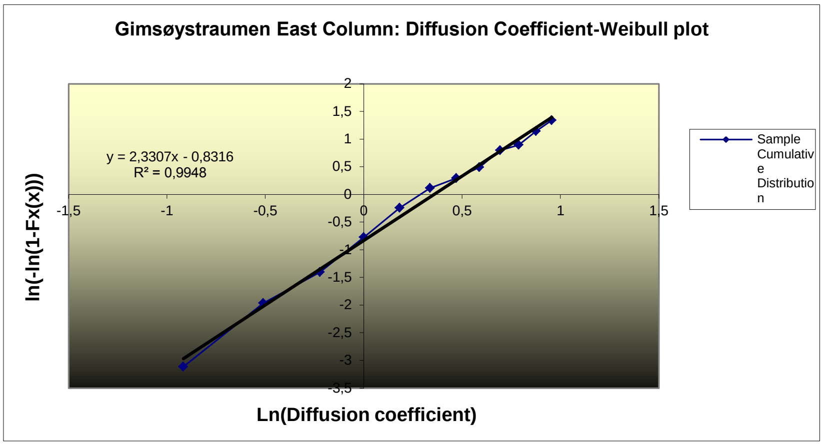

Fig. 2. Comparison between sample distribution function and fitted Weibull distribution for the diffusion coefficient, east side of columns.

A more direct comparison between the analytical model and the observed data is provided by considering the density function, i.e. the expected number versus the observed number of samples within each discretized interval. Such a comparison is provided by Figure 3. The overall comparison is quite good, but with some "oscillations" around the theoretical curve

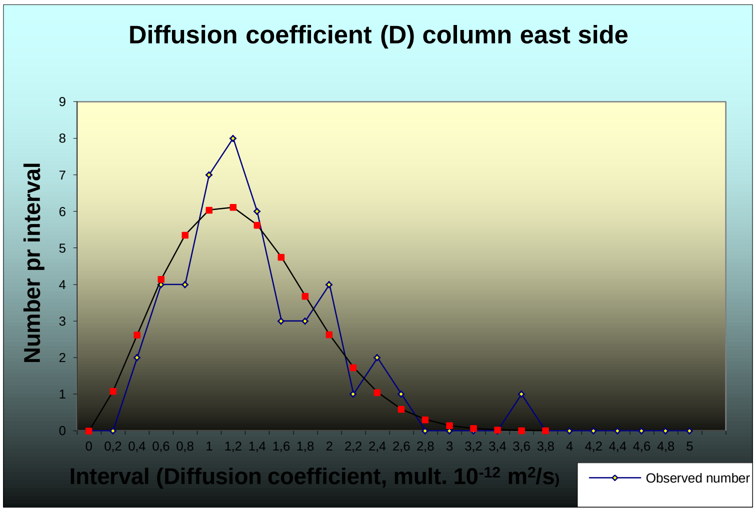

Fig. 3. Observed versus predicted number of samples for the diffusion coefficient within each interval, east side of columns. Theoretical model is based on regression curve in Figure 2.

Although the Weibull model gave the best fit for this specific case, it is found that on the average, the lognormal probability distribution gives the best fitting. Furthermore, there are reasons of convenience for selecting this model when calculating the probabilistic lifetime distributions. Hence, the lognormal distribution is applied for the present calculations of lifetime distributions.

The regression coefficients obtained from a similar fitting of probability distributions for the *chloride surface concentration* are shown in Table 2. It is observed that the lognormal distribution gives the highest value for the regression coefficient. However, all the distributions have regression coefficients higher than 0.9, which in general is quite acceptable.

Table 2. Surface concentration, C_s (% of concrete weight)

Probabilistic model	Regression line	Mean value	Standard deviation	Sample variance	R^2
Normal	y = 3.5447x – 1.5979				0.9156
Gamma	y = 1.4932x – 0.3422				0.9352
Gumbel	y = 4.0841x – 1.3635	0.50	0.34	0.11	0.9716
Weibull	y = 1.9038x + 1.2355				0.9338
Lognormal	y = 1.4879x + 1.3571				0.9826

The corresponding sample distribution function and the fitted lognormal model are shown in Figure 4. It is seen that the upper part of the empirical distribution (which is most relevant for the shortest lifetimes) is also fitted well by this analytical model.

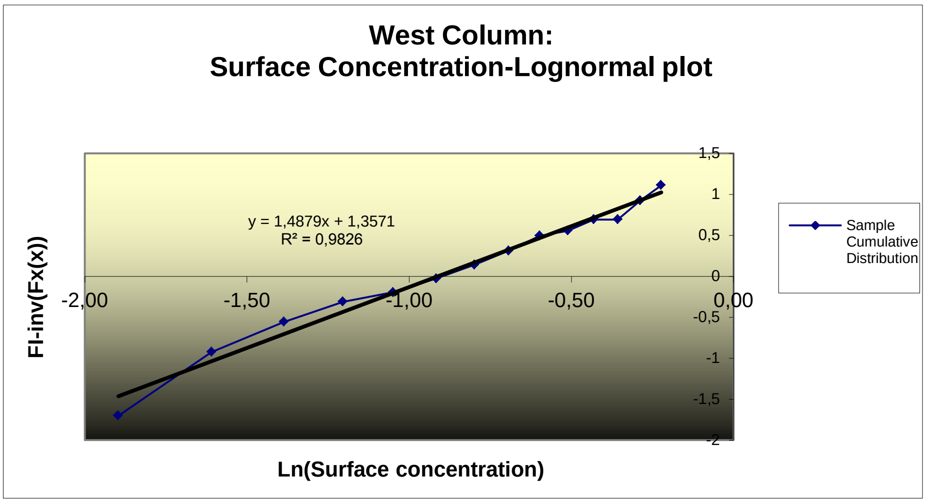

Fig. 4. Cumulative distributions for measured surface concentration plotted in lognormal scale, and resulting fitted lognormal model. West side of columns.

Measurements of concrete cover depth were also performed. A lognormal model was found to give the best fit to the measurements. Based on the full-scale measurements and consideration of the additional parameters entering into the computation of chloride lifetime, corresponding probabilistic models are established. The relevant parameters are defined in relation to the solution of Fick's second law for the chloride concentration c(x,t) at position x and at time t:

$$c(x,t) = c_i + (c_s - c_i)\cdot \operatorname{erfc}\left(\frac{x}{2\sqrt{D\cdot t}}\right) \qquad (1)$$

where c_i is the initial chloride concentration in the concrete, c_s is the chloride concentration at the surface, and D is the diffusion coefficient. The concentration at the position of the reinforcement is subsequently compared to the critical chloride concentration for onset of corrosion. The diffusion coefficient may furthermore be time-dependent. The time variation is here expressed by the so-called alfa-factor (Maage et. al., 1994 and Poulsen, 1996). The value of the alfa-factor influences the time variation of the diffusion coefficient through the following expression: (talfa). Accordingly, alfa equal to zero corresponds to a constant diffusion coefficient while alfa = 1 corresponds to a linear increase with time etc.

The probabilistic models which applied for the superstructure are summarized in Table 3. The model uncertainty factor which occurs both in Table 3 and Table 4 is introduced in order to account for deviations between model predictions and observed diffusion rates. The lowest value is taken to represent lifetime calculations performed for the bridge from which the measurement were performed. The highest value could e.g. represent a situation where these particular data were applied for calculations of a "similar" bridge.

Table 3. Statistical distributions for superstructure

Statistical variable	Distribution type	Mean value	Standard deviation
Surface concentration	Lognormal	0.25 (% concrete weight)	0.18 (% concrete weight)
Diffusion coefficient	Lognormal	0.88 (m2/sec, mult 10^{-12})	0.68 (m2/sec, mult 10^{-12})
α-factor (time-var. of diff. coef.)	Deterministic	0.0	-
Initial concentration	Normal	0.015 (% concr. wght.)	0.0015 (% concrete weight)
Concrete cover	Lognormal	23 mm	6 mm
Critical chloride concentration	Lognormal	0.18 (% concr. weight)	0.06 (% concrete weight)
Model uncertainty	Normal	1.0	0.01/0.10

Corresponding models which apply to the columns are given in Table 4. As observed, both the diffusion coefficient and the surface concentration are higher for this case. However, the concrete cover is also considerably thicker than for the superstructure.

Table 4. Statistical distributions for columns

Statistical variable	Distribution type	Mean value	Standard deviation
Surface concentration	Lognormal	0.50 (% concrete weight)	0.34 (% concrete weight)
Diffusion coefficient	Lognormal	1.27 (m2/sec, mult 10^{-12})	0.64 (m2/sec, mult 10^{-12})
α-factor (time variation)	Deterministic	0.0	-
Initial concentration	Uniform	0.015 (% concrete weight)	0.0015 (% concrete weight)
Concrete cover	Lognormal	45 mm	6 mm
Critical chloride concentration	Lognormal	0.18 (% concrete weight)	0.06(% concrete weight)
Model uncertainty	Normal	1.0	0.01/0.10

3. Lifetime distributions based on the measured data

3.1 Base case analysis

The cumulative distribution functions for chloride lifetime which are obtained by calculating probabilities of the type: P(chloride concentration at reinforcement at time t < critical chloride concentration). These probabilities are computed repeatedly for a number of different values of the time parameter. The calculations are performed by application of so-called First Order Reliability Methods (FORM), see e.g. (Madsen et. al. 1986).

The probability distribution that results from reliability analysis based on the input data given in Table 3 (superstructure), is shown in Figure 5. The corresponding probability density function is obtained by numerical differentiation and is given in Figure 6. As observed, the peak of the latter occurs for a lifetime of 6 years. However, the shape of the upper tail is such that it decays very slowly. This implies a large standard deviation for the lifetime. This is also reflected by the distribution function rising very slowly.

This distribution function obtains a value of 0.4 for a duration of 80 years. This implies that the probability for the lifetime to be smaller than this value is 40%.

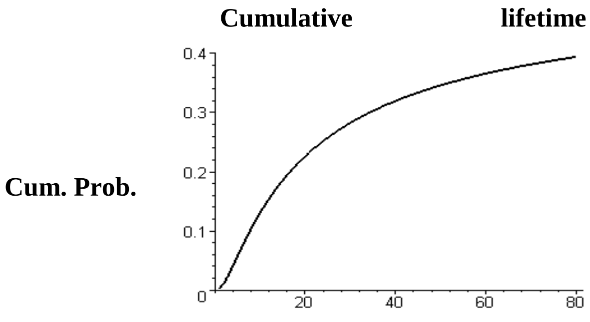

Fig. 5 Probability distribution of lifetime (superstru[illegible]nding to input statistical models given in Table 3.

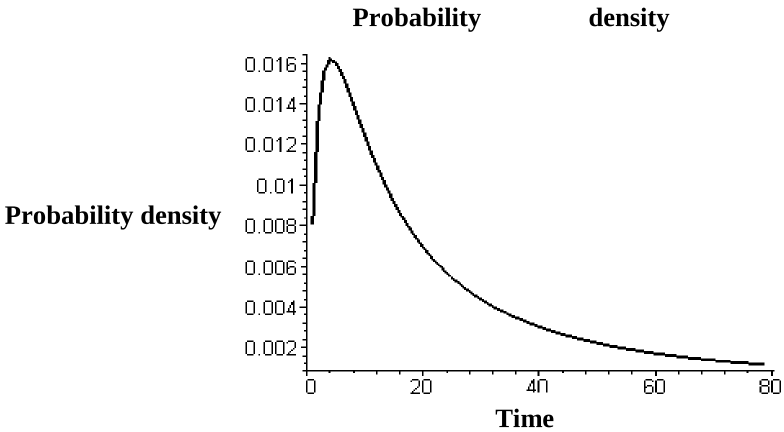

Fig. 6. Probability density function obtained by differentiation of distribution function in Figure 5.

3.2 Parametric variations

The effect of varying the statistical parameters of the input models can be readily studied. The effect of including a probabilistic time varying diffusion is accounted for by introducing the alfa-parameter as discussed above. This is presently done by modelling this parameter as a random variable. The mean value is taken to be 0.4, and the standard deviation is 0.1. A lognormal distribution is assumed to apply. The resulting cumulative distribution of the lifetime and the corresponding density function are shown in Figures 7 and 8. These should be compared to the distribution and density functions presented in Figures 5 and 6.

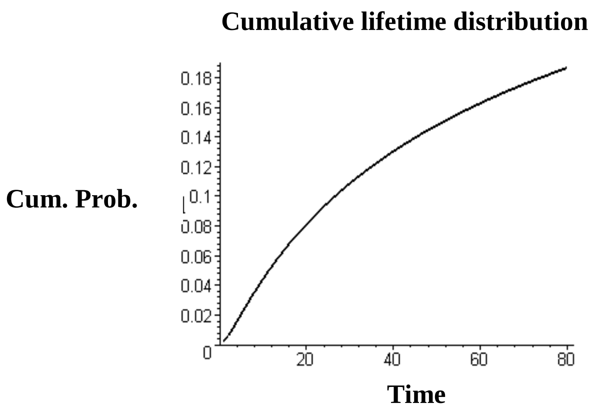

Fig. 7. Cumulative distribution function for chloride lifetime. Alfa-parameter which defines variation of diffusion coefficient with time is represented by a lognormal distribution with mean value 0.4 and a standard deviation of 0.1

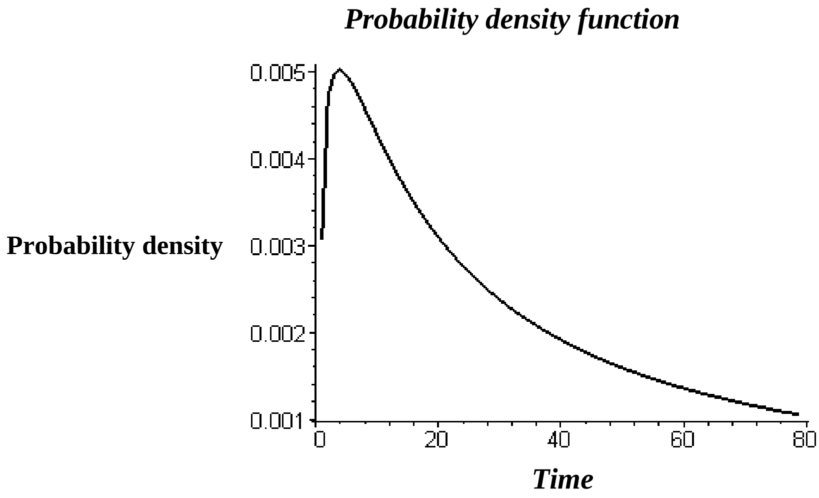

Fig. 8. Probability density function corresponding to the distribution function in Figure 7.

The peak of the density function is still located at 6 years. However, the peak is now much smaller than in Figure 6. The upper tail of the density is also higher, and the corresponding distribution function in Figure 7 is “stretched” towards higher lifetimes as could be anticipated.

4. Reliability updating for system model of entire bridge structure by enhanced Monte Carlo simulation

4.1 General

The analysis so far has basically been relevant for only a single “spot” or “component”. A more realistic model corresponds to analysis of the whole bridge structure, which implies that assessment of the corresponding system reliability needs to be made.

This requires a more complex analysis where FORM/SORM techniques easily become inadequate, or at least inaccurate. Resort must typically be made to Monte Carlo simulation methods which can provide more accurate results, but which at the same time are quite demanding with respect to computation time.

This calls for more efficient simulation methods, and in the present study the enhanced Monte Carlo simulation technique (which was referred to above) is applied. In general, this approach is based on introduction of a scaling parameter λ for the limit state function. A scaling factor of 1.0 corresponds to the “true” failure function while a value smaller than one leads to higher failure probabilities (i.e. a less reliable structural system). In the present application, a similar scaling is also introduced for the “observation function” which represents additional information that has become available based on e.g. monitoring or inspection of the structure.

4.2 Simplified system model of bridge superstructure

The bridge superstructure is considered to consist of 3 sites (i.e. “components”) in the transverse direction and 30 segments in the longitudinal direction, giving at total of 3x30 = 90 components. In the transverse direction, each component represents the chloride ingress for one of the “faces” of the box girder, i.e. the windward face, the downward face and the leeward face. The roadway itself is not included as the surface chloride concentration is much lower for this part then the others. In the longitudinal direction, each component represents a certain length segment of each of the faces.

A simplified analysis is applied where only the surface concentration is represented as a random variable (while the other random variables are modelled as deterministic quantities equal to the mean values of the corresponding variables in the table above). The initial chloride concentration at the steel armour is set to zero.

For this purpose, the surface chloride concentration is represented by a mean value of 0.14% and a standard deviation of 0.028 %. (Instead of lognormal model, a Gaussian model is applied which is truncated at a value of 0.1% for the surface concentration). These values correspond to a situation where the failure probability for a single component as well as the entire bridge system is much smaller than for the previous case where models based on full-scale observations for the particular bridge were applied.

Presently, identical values are applied for the surface concentration of all "components" and accordingly the failure functions are the same for all the components. However, the concentrations at different sites are assumed to be completely independent from each other which implies that 90 independent random variables are introduced.

4.3 Reliability updating based on inspection of surface concentration

First a system reliability analysis is performed based on the assumptions described above. Furthermore, a system failure probability is evaluated at a time in operation of t= 60 years. The corresponding failure probability is shown as a function of the scaling parameter in Figure 9 below for the case that no additional information from monitoring or inspection is available. The 95% confidence band is also shown as represented by the upper and lower curves.

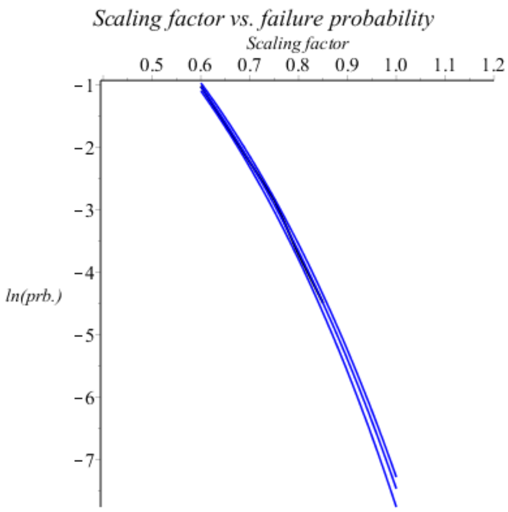

Figure 9. Failure probability as a function of the scaling parameter at a time of t = 60 years without any additional information (from monitoring or inspection) being available.

The corresponding estimated failure probability for the system with 90 components (i.e. for the scaling parameter λ= 1.0) is computed as 5.72e-4 with the 95% confidence interval being (4.78e-4, 6.58e-4). This implies that the coefficient of variation for the estimated failure probability is around 5%. The total number of samples is 32000, which corresponds to a reduction by a factor of around twenty as compared to what would be required by crude Monte Carlo simulation in order to achieve the same level of accuracy.

It is next assumed that the surface concentrations for half the components are found to be lower than the mean value plus two standard deviations, i.e. 0.196%. The results based on enhanced Monte Carlo simulation for the new updated failure probability at t = 60 years are shown in Figure 10 for increasing values of the scaling parameter(which is applied both for the failure function and for the observation function).

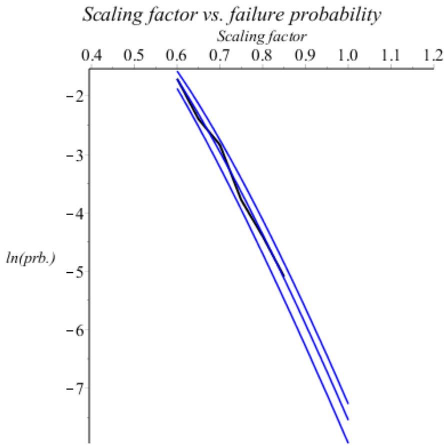

Figure 10. Failure probability as a function of the scaling parameter for system with 90 components subjected to chloride ingress. Failure probability at t = 60 years for the case that the surface concentrations for half the components are found to be smaller than 0.196%.

The estimated failure probability for the system with 90 components (i.e. for the scaling parameter λ= 1.0) is now found to be 5.26e-4 with the 95% confidence interval being (3.53e-4, 6.93e-4). This implies that the coefficient of variation for the estimated failure probability is around 15%. The total number of samples is 16000, which also now corresponds to a reduction by a factor of six as compared to what would be required by crude Monte Carlo simulation.

We next assume that the surface chloride concentration is less than the critical value 0.18% (for half the "sites/components"). The results for this case are shown in Figure 11.

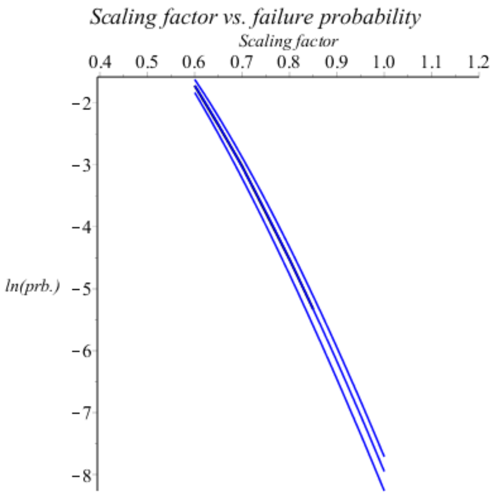

Figure 11. Failure probability as a function of the scaling parameter for system with 90 components subjected to chloride ingress. Failure probability at t = 60 years for the case that the surface concentrations for half the components are found to be smaller than 0.18%.

The estimated failure probability for the system with 90 components (i.e. for the scaling parameter λ= 1.0) is now found to be 3.52e-4 with the 95% confidence interval being (2.57e-4, 4.48e-4). This implies that the coefficient of variation for the estimated failure probability again is around 15%. The total number of samples is 20 000, which still corresponds to a reduction by a factor of six as compared to crude Monte Carlo simulation.

For the present analysis, independence between the "components" was assumed. The corresponding effect of having additional information on the resulting system reliability was a reduction of the failure probability roughly by a factor of three. If correlation between the components was introduced the effect would be much more pronounced. If full correlation would apply, this essentially means that there is only a single component in the system rather than 90. Accordingly, the failure probability would be reduced by a similar factor even if only a single component was inspected.

5. Concluding remarks

In the present paper, probabilistic models based on full-scale measurements from the Gimsøystraumen bridge are addressed. These models apply to the diffusion coefficient, the chloride surface concentration and the concrete cover. Based on these models and supplementary models for other parameters affecting chloride diffusion, probabilistic lifetime calculations are performed.

A system reliability analysis method was introduced and subsequent reliability updating was performed by means of enhanced Monte Carlo simulation. As a general observation, it was found that the computational effort (as measured by CPU-time) was typically reduced by a factor of six.

There are clearly multiple future research topics that should be addressed. Examples are: The effect of correlation between the system components in connection with the updated reliability, the effect of non-identical system components, combination of parallel and series system models of bridge systems and Ultimate Limit State criteria in addition to Serviceability criteria.

Acknowledgements

The data basis for the present work was established during the project "The lifetime of concrete structures" (Betong-konstruksjoners Livsløp) which was performed by the Norwegian Public Roads Administration. The digital background material for the present paper was provided by a number of persons, in particular Finn Fluge, Ola Skjølsvold, Trine Hynne, and Jan-Erik Carlsen. Their contributions are greatly acknowledged.

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Quantifying the value of SHM for emergency management of bridges at-risk from seismic damage

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.5

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Abstract. This paper proposes a framework for quantifying the value of information that can be derived from a structural health monitoring (SHM) system installed on a bridge which may sustain damage in the mainshock of an earthquake and further damage in an aftershock. The pre-posterior Bayesian analysis and the decision tree are the two main tools employed. The evolution of the damage state of the bridge with an SHM system is cast as a time-dependent, stochastic, discrete-state, observable dynamical system. An optimality problem is then formulated how to decide on the adoption of SHM and how to manage traffic and usage of a possibly damaged structure using the information from SHM. The objective function is the expected total cost or risk. The paper then discusses how to quantify bridge damage probability through stochastic seismic hazard and fragility analysis, how to update these probabilities using SHM technologies, and how to quantify bridge failure consequences.

Keywords: Bridges, pre-posterior analysis, seismic damage, seismic risk, seismic structural health monitoring, value of information

1 Introduction

Structural health monitoring (SHM) has gained considerable interest in the technology research and development community. Because of this technology push, SHM has made a transition from the laboratory to the real world and many in-situ structures, notably bridges, have been instrumented. However, most of such monitoring exercises are academically driven and practitioners, asset managers and emergency response authorities (e.g. those charged with ensuring adequate post-earthquake actions) remain indifferent to the practical usefulness and value of SHM. At the same time, strong assertions can be heard about the value and expected benefits of SHM. It is thus important that the claims of the value of SHM be backed up by quantitative evidence, otherwise the idea of SHM may be seen by sceptics, not just opponents, as belonging largely in the post-truth world.

The broader motivation behind using SHM is to collect information about structural performance and condition, that would otherwise be unavailable or of insufficient accuracy or precision, and use this information for managing the risk of infrastructure failure or underperformance. If so, the concept of risk can be, as a function of both the probability of failure and its consequences, utilized in quantifying the value of SHM given the many uncertainties encountered in processing SHM data for structural failure prediction, SHM system performance (e.g. accuracy of the data measured and models used) and failure consequences. A useful tool, which utilizes the concept of risk, is the Bayesian pre-posterior decision analysis combined with the decision tree representations, as this enables calculating the value of SHM information even *before* one procures and installs an SHM system. The fact that we are trying to evaluate the performance and economic benefit of an SHM system that has not yet been deployed on a structure is critical to appreciate the use of pre-posterior decision analysis, but it may initially elude the reader. However, it is, in fact, not dissimilar to, e.g. seismic risk analysis, where we try to model probabilistically what could happen should an earthquake occur, but we do so *before* the actual event. Indeed, performance-based seismic design or assessment of a structure is a similar undertaking, where we try to envisage what could happen to a structure that now only ‘exists’ in the designer’s minds, and make decisions about what to do to manage the risks potentially eventuating. In *all* those cases, we deal with significant uncertainties.

In this paper, the Bayesian pre-posterior decision analysis is employed to propose a framework for quantifying the value of using SHM in the context of detecting damage to bridges subjected to strong ground motion for achieving better-informed post-event decisions such as those pertaining to the continuation of full or limited emergency operations or bridge closure because of safety concerns. The framework uses the established seismic structural risk analysis principles based on site hazard probabilities and structural vulnerabilities, and absorbs SHM information into the process. An important aspect is that aftershock induced hazard is considered. After the occurrence of a mainshock earthquake, the affected area will often experience an increased level of seismic activity with a potential large number of strong aftershocks. Such sequences of aftershock events may continue for several months in case of large magnitude mainshock events. A bridge exposed to the mainshock or earlier aftershocks may have been damaged by them and will now have increased vulnerability to future tremors. Thus, one example scenario where SHM could make a difference is detecting such existing damage so that the weakened, but still operating, structure does not fall in an aftershock, leading, e.g., to new casualties or injuries amongst its users and other avoidable consequences. We assume that only seismic risk is considered, i.e. the bridge will not fail under traffic or other loads, but the framework can be extended to include multiple hazards, as it can to consider also structural deterioration with time due to corrosion, fatigue or scour.

2 Framework for quantifying the value of seismic SHM of bridges

This section presents a process of building a decision tree for the Bayesian pre-posterior analysis (Raiffa & Schlaifer, 1961) for quantifying the value of seismic SHM of bridges. It starts with a decision problem whether a bridge should be closed or kept in service for a structure subjected to the mainshock and a single aftershock when SHM is not used. It then considers how additional information from SHM may be used in emergency decision making. The evolution of the damage state of the bridge with an SHM system is cast as a time-dependent, discrete-state, observable, stochastic dynamical system. An optimality problem is thus formulated how to decide on the adoption of SHM and how to manage traffic and usage of a possibly damaged structure incorporating SHM data where it is available. The objective is to find a set of decisions that lead to the minimum expected total cost including the price paid for installing and maintaining SHM system and the probable losses that ensue due to the operational decisions made.

2.1 Decision problem for continuing operations of a bridge without an SHM system subjected to the mainshock and a single aftershock

The decision tree used in the situation described in the section title may be build up as a collection of the basic blocks shown in Figure 1. On the left, the detail of the basic building block is shown, and on the right, its abridged symbolic representation. Squares denote decision nodes and circles represent random outcome nodes. To keep the schematic representation uncluttered, only some branches of the tree are shown; similar simplifications will be used throughout the paper. The generic symbol E (also when used as a superscript) refers to a particular *event*: E=M for the mainshock, and E=A for the aftershock, respectively. TR_i^E refer to *traffic restriction* actions taken by the authority after the seismic event E. There may be $K+1$ different actions, with TR_0^E corresponding to uninterrupted operations, and, at the other end, TR_K^E corresponding to the full closure of the bridge; the other actions could be restricting the use to only light vehicles and/or restricting speed, allowing only use by emergency vehicles, etc. Note, these decisions must be reached, in the scenarios considered in this section, using only the information which is available without a dedicated SHM system installed on the bridge. DS_i^E refer to levels of *damage sustained* by the structure during seismic event E. The level of damage is often expressed by assigning the structure to one of the L+1 discrete damage states, ranging from, e.g. no/negligible damage, to light damage, to moderate damage, to severe damage, and eventually to the total collapse. Alongside the different levels of damage, shown are the probabilities of their occurrence, P^{E}_{DSi}.

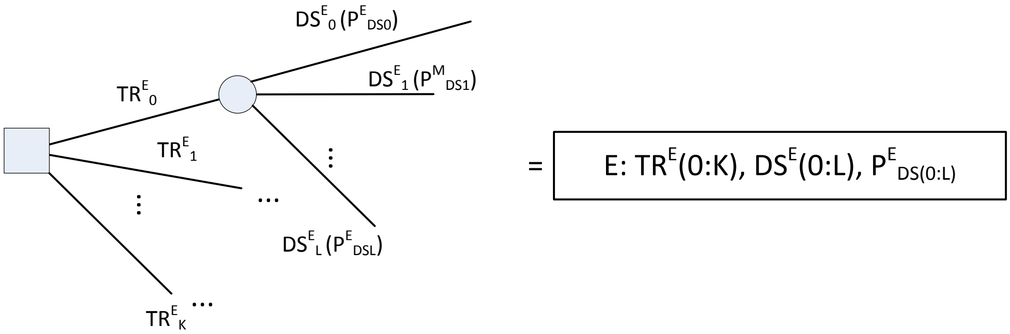

Fig. 1. Basic building block of decision tree to manage bridge usage

The full decision tree for continuing operations of a bridge without an SHM systems subjected to the mainshock and a single aftershock is shown in Figure 2. Here, in the building blocks for the aftershock events (denoted by symbol *A*), the probabilities $P^{A|M}_{DS_i|DS_j}$ of bridge sustaining a given level of damage, DS_i in the aftershock are conditional on the level of damage, DS_j sustained in the mainshock, i.e. they are transition probabilities. That in fact cast our problem as a dynamical, discrete-state stochastic system. Without monitoring, the system is not observable, but once an SHM information is included, which is explained in the following section, it will become observable. The system can be though as time dependent, although this is now hidden in the occurrences of the mainshock and the aftershock. This also expresses the fact that damage will accumulate over consecutive earthquakes. On the very right of Figure 2 are consequences related to each combination of actions and random outcomes (states of nature), $C_{ijkl} = C\left(TR_i^M, DS_j^M, TR_k^A, DS_l^A\right)$, ($i$, k=0, 1, … K; j, l=0, 1, … L). For example, closing the bridge altogether to traffic after the mainshock or the aftershock, when in fact it can be used without restriction or perhaps at least for emergency services, will entail economic losses because of delays, loss of service etc., and will possibly also mean delays in getting the injured to a hospital worsening their condition. On the other hand, a bridge that is unsafe but allowed to operate may collapse leading to additional economic losses or even casualties or new injuries.

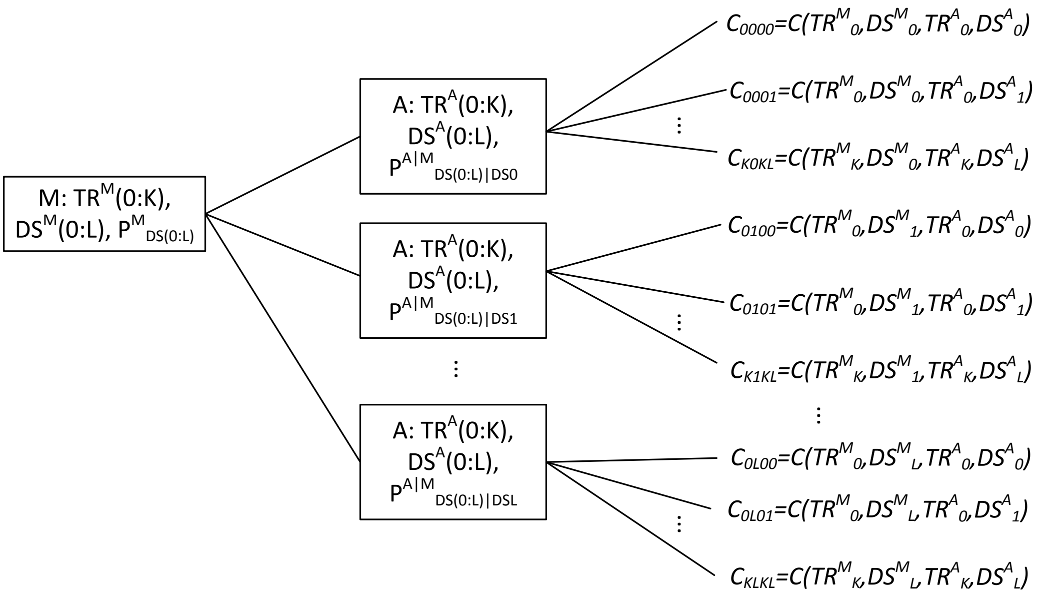

Fig. 2. Decision tree for continuing bridge operations for bridge without SHM system subjected to mainshock and aftershock

The optimal pair of actions $\left(TR^M, TR^A\right)_{opt}$ after the mainshock and the aftershock is the one that minimizes the overall risk:

$$\left(TR^M, TR^A\right)_{opt} = \min_{i=0,1\ldots K} E_{DS_j^M} \min_{k=0,1\ldots K} E_{DS_l^A|DS_j^M}\left[C_{ijkl}\right] \tag{1}$$

Here, E[] denotes the expected value operator.

2.2 Decision problem for continuing operations of a bridge with an SHM system subjected to the mainshock and a single aftershock

To handle the scenario where an SHM system is to be adopted, another basic decision tree building block is adopted as shown in Figure 3. Here, decisions to adopt a *health monitoring* system before seismic event E are denoted as HM_i^E. There may be N+1 such decisions, each corresponding to the adoption of a particular SHM system or technology, with HE_0^E corresponding to the decision to not adopt any. Note that the superscript E is still present as we envisage monitoring may be adopted before the mainshock but alternatively only after the mainshock to monitor the structural performance and damage in the aftershock of the bridge weakened in the mainshock (in which case it would be replaces by superscript A). The cost of each system is indicated by C_{HMi}, with C_{HM0}=0. It should be noted that for a fair assessment of the cost involved in monitoring a structure not only the cost of hardware (capex) must be included but the whole life-cycle cost needs to be quantified (design, installation, operational costs including maintenance, decommissioning, etc.), and the cost of data analysis and integration of the SHM information into the emergency response process. DD_i^E refer to *damage detected* by the monitoring system. Again, it is envisaged that based on the SHM system indication, the structural state will be mapped into one of the L+1 discrete detected damage states. The probabilities of indication of the different levels of damage are indicated as P_{DDi}^E. Note these probabilities include correct as well as incorrect detected damage state classifications with respect to the actual damage states the structure will find itself in.

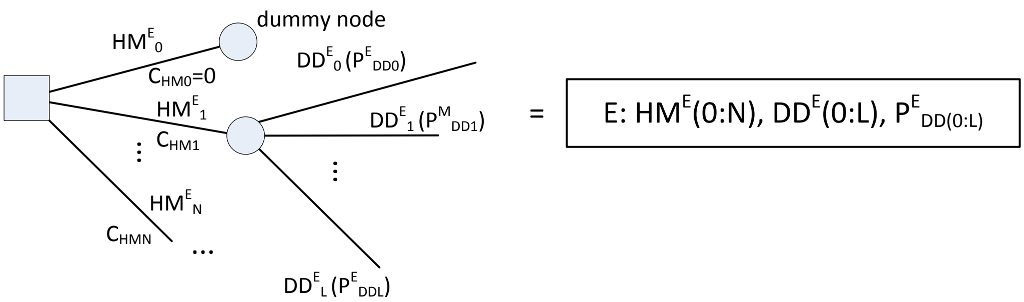

Fig. 3. Basic building block of decision tree for SHM system adoption

With the newly introduced additional building block, we can now formulate the full decision tree for adoption of an SHM system. It is shown in Figure 4. The consequences at the far-right end, $C_{ijklmnpr} = C\left(HM_i^M, DD_j^M, TR_k^M, DS_l^M, HM_m^A, DD_n^A, TR_p^A, DS_r^A\right)$, ($i$, m=0, 1, … N; j, l, n, r=0, 1, … L; k, p=0, 1, … K), depend now also on the additional decisions to adopt or not an SHM system, and if so which, and random outcomes include damage detection alerts issued by the SHM system. As one moves from left to right, the probabilities of each damage state being indicated or actually sustained depend on the entire history of preceding decisions and random outcomes.

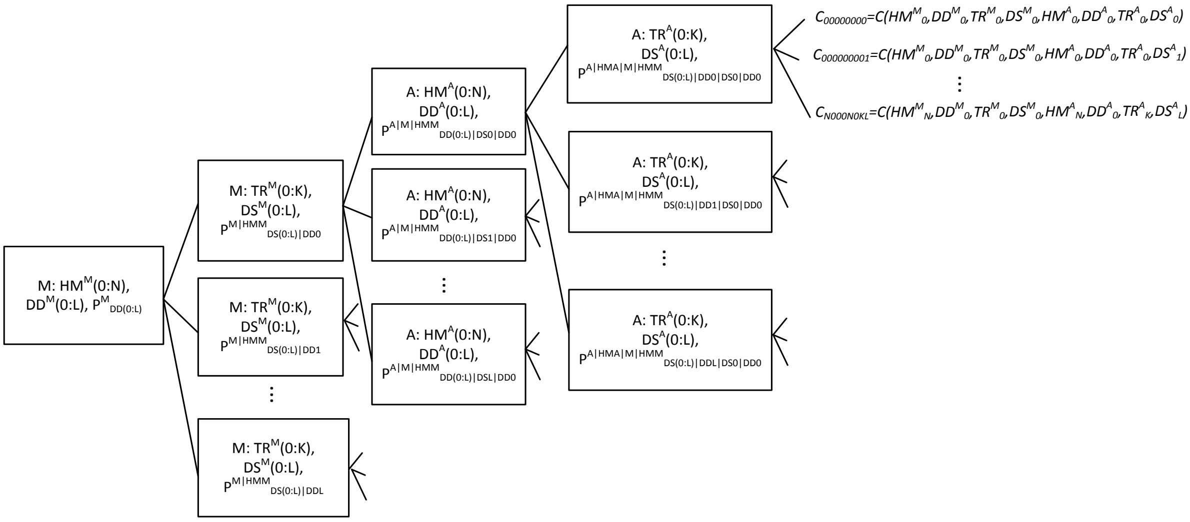

Fig. 4. Decision tree for continuing bridge operations for bridge with SHM system subjected to mainshock and aftershock

The conditional probabilities $P_{DSi|DDj}$ of damage state DS_i having actually been sustained when damage state DD_j has been indicated by the SHM system appearing in the decision tree may be found from the state probabilities $P^M_{DS_i}$ and state transition probabilities $P^{A|M}_{DS_i|DS_j}$ (i, j=0,1,…L), and the probabilities $P_{DD_j|DS_i}$ of correct/incorrect indications of damage states by the monitoring system, for example:

$$P^M_{DD_j} = \sum_{i=0}^{L} P^M_{DS_i} P^M_{DD_j|DS_i} \tag{2}$$

$$P^{M|HM^M}_{DS_i|DD_j} = \frac{P^M_{DS_i} P^M_{DD_j|DS_i}}{P^M_{DD_j}} \tag{3}$$

The optimal set of actions $\left(HM^M, TR^M, HM^A, TR^A\right)_{opt}$ is the one that minimizes the overall risk:

$$\left(HM^M, TR^M, HM^A, TR^A\right)_{opt} = \min_{i=0,1\ldots N} E_{DD^M_j} \min_{k=0,1\ldots K} E_{DS^M_l|DD^M_j} \min_{m=0,1\ldots N} E_{DD^A_n|DS^M_l|DD^M_j} \min_{p=0,1\ldots K} E_{DS^A_r|DD^A_n|DS^M_l|DD^M_j}\left[C_{ijklmnpr}\right] \tag{4}$$

3. Bridge seismic risk modelling: hazard and fragility for

The probability $P^E_{DS_i}$ of a bridge sustaining damage state DS_i when subjected to an earthquake during its expected service life is a critical parameter in the proposed framework (see Figure 1). This probability is a function of hazard at the site and fragility of the bridge. The probability $P^E_{DS_i}$ can be estimated using the following expression:

$$P^E_{DS_i} = \int_0^{+\infty} \left[F_{D|IM}\left(d_i|x\right) - F_{D|IM}\left(d_{i+1}|x\right)\right] \left|\frac{\mathrm{d}\lambda_{IM}(s)}{\mathrm{d}s}\right|_{s=x} dx \tag{5}$$

In the expression above, $F_{D|IM}(.|.)$ is the cumulative conditional probability distribution of peak demand, D, imposed on the bridge conditioned on the intensity measure, IM, of strong ground motion at the site. Variables d_i and d_{i+1} are the demand levels (e.g. strains, curvatures, displacements) corresponding to the onset of damage states DS_i and DS_{i+1}, respectively. The expression $|\mathrm{d}\lambda_{IM}/\mathrm{d}s|$ is the absolute value of the derivative of the estimated seismic hazard λ_{IM}. Typical IM parameters are pseudo-spectral acceleration of the equivalent damped single-degree-of-freedom system, $S_a(T)$, peak ground velocity, PGV, and peak ground acceleration, PGA. IM establishes the connection between the hazard and the vulnerability. Therefore, it is critical to adopt a measure that can effectively capture the seismic behavior of the bridge and can be probabilistically estimated with an acceptable level of uncertainty. Benefits and limitations of alternative *IM*s are discussed by Weatherhill et al. (2011). In the following, potential strategies for estimating the seismic hazard, λ_{IM}, and the fragility, $F_{D|IM}$, will be presented.

The seismic hazard at the site of the bridge can be estimated by performing a probabilistic seismic hazard assessment (PSHA) as proposed by Cornell (1968). In PSHA, the rate, λ_{IM}, at which the strong motion intensity, *IM*, at the site is expected to exceed a specific level, *s*, within a fixed time is assessed. The rate λ_{IM} is evaluated using the following expression:

$$\lambda_{IM}(s) = \sum_{i=1}^{n_s} \nu_i \int_{m_{\min}}^{m_{\max}} \int_{0}^{r_{\max}} P\left[IM > s \middle| m, r\right] f_{R|M}\left(r \middle| m\right) f_M\left(m\right) dm\, dr \tag{6}$$

where n_s is the number of seismic sources that are expected to induce significant shaking at the site, ν_i is the rate of earthquakes that occur at the *i*-th source and which have magnitudes within the range bounded by the minimum magnitude, m_{min}, and the maximum magnitude, m_{max}. The term $P[IM > s|\, m, r]$ is the conditional probability of shaking intensity *IM* at the site exceeding level *s*, given that the site is excited by an earthquake of magnitude *m* and with a rupture plane that lies at a distance *r* from the site. This probability is estimated using ground motion prediction equations which aim at capturing the expected attenuation or amplification of the seismic waves which propagate along the path from the source to the site (Kramer, 1996). Probability density $f_M(m)$ is equal to the relative likelihood of magnitudes of earthquakes that occur within considered time being equal to *m*. Likewise, $f_{R|M}(r|m)$ is the conditional probability of the source-to-site distance being equal to *r* for an earthquake with magnitude *m*.

In the proposed framework, seismic hazards associated with two different types of earthquakes are considered, namely the mainshock and the aftershock earthquakes. Large magnitude earthquakes are often preceded and succeeded by smaller magnitude events that occur at the proximity of each other and within a short period. An entire sequence of earthquakes is referred to as a cluster. Within a cluster, the event with the greatest magnitude is named the mainshock and all the following earthquakes are called aftershocks. Existing earthquake catalogs suggest that mainshock earthquakes often occur at a relatively constant rate at seismic source zones. Accordingly, these events are typically modelled as a homogeneous Poisson processes in the conventional PSHA. Hence, the probability $P^{M}_{DS_i}$ - related to the mainshock - can be obtained using λ_{IM} obtained from Equation (6) and considering structural vulnerability or fragility.

The aftershock earthquakes occur at a rate that decays with time elapsed since the mainshock. The characteristics of this decay were first systematically investigated by Omori (1894). Even today, Omori's model is frequently used for modeling the decaying of rate of aftershocks. Since the rate of aftershocks is not constant over time, the aftershock events are modelled as a non-homogenous Poisson processes in the PSHA. Yeo and Cornell (2009) proposed a modified version of PSHA that considers the time dependent decay of the rate of events. Recently, Müderissoglu and Yazgan (2017) developed a modified version of this approach, which enables making use of mainshock strong motion recordings in updating the uncertainty associated with the expected attenuation of the aftershock induced shaking. This updating results in changing of the conditional likelihood $P[IM>s|m,r]$ in Equation (5). In case of bridges designed and constructed according to modern seismic codes, the primary source of uncertainty associated with the expected performance is that due to uncertainty of the estimated hazard. Therefore, such an updating of the uncertainty associated with the hazard estimate would often lead to a considerable change in the predicted seismic performance.

The aftershock hazard assessment method developed by Müderrisoglu and Yazgan (2017) is especially suitable for bridges which have free-field strong motion recoding instruments. In the context of the framework proposed here, such instrumentation may be conceived as a part of the monitoring system. Using the method, the ground motion recorded by the free-field sensor can be utilized to revise the uncertainties associated with the expected level of attenuation. Thus, the aftershock hazard conditional on the recorded mainshock motion can be obtained. When compared to the case with no instrumentation, this conditional hazard estimate would result in higher or lower exceedance rates.

This difference depends on the motion intensity level registered during the mainshock event. The aftershock damage probabilities, $P^{A|M}_{DS_j|DS_i}$, corresponding to the decision tree branch in Figure 4 related to not adopting any monitoring system (i.e. HM_0^M) may be evaluated using the conventional aftershock hazard assessment approach by Yeo and Cornell (2002). On the other hand, the probabilities $P^{A|M}_{DS_j|DS_i}$ corresponding to the branches related to adopting a monitoring system (i.e. HM_i^M, i=1,2,…N) can be evaluated by substituting the λ_{IM} estimates obtained using the method by Müderrisoglu and Yazgan (2017) into Equation (5).

The conditional probability of a bridge sustaining damage state DS_i when subjected to a given level of shaking intensity is referred to as the seismic fragility. This conditional probability is represented by the term $F_{D|IM}(.|.)$ in Equation (5). There exists a large variety of methods proposed for assessing seismic fragility of structures (Porter, 2003). In the proposed framework, an approach that can be applied to individual structures is needed. Moreover, the approach should enable rational consideration of various sources of uncertainty that have significant impact on the estimated likelihood $F_{D|IM}$. Based on these constraints, the 'analytical approach' for fragility modeling is particularly suited to the framework presented here.

In the analytical fragility modeling approach, a basis numerical model of the bridge is developed for seismic response analysis. The uncertainties associated with the model are assessed and probability distributions are established to capture their random variability. Typically, the existing recommendations (e.g. JCSS, 2001) are utilized for this purpose. A set of alternative models are generated using these probability distributions. Subsequently, a suite of strong ground motion records is established. The records are selected to capture with a required accuracy the mean value and dispersion of the seismic response of the bridge that will be exhibited when it is subjected to the expected seismic events during its service life (Kalkan & Chopra, 2010). For each randomly generated model with a ground motion, incremental dynamic analysis (Vamvatsikos & Cornell, 2002) can be performed. In this process, the response of the bridge to the specific ground motion is simulated by gradually scaling up the ground motion to different IM levels. The record is scaled to the level when the computed demand becomes just equal to the threshold d_i associated with the onset of damage state DS_i. The intensity level x'_{di} that correspond to this threshold is determined for all model realization and ground motion record pairs. Subsequently, the fragility is evaluated as follows:

$$F_{D|IM}\left(d_i|x\right)=\Phi\left(\frac{x-\mu_i}{\sigma_i}\right), \text{ where } \mu_i=\frac{1}{n_m}\sum_{j=1}^{n_m}x'_{di}\left(j\right) \text{ and } \sigma_i=\sqrt{\frac{1}{n_m-1}\sum_{j=1}^{n_m}\left[x'_{di}\left(j\right)-\mu_i\right]^2} \tag{7}$$

In the equation above, $\Phi(.)$ is the standard normal distribution function, μ_i and σ_i are the mean and standard deviation of the IM levels that correspond to the onset of DS_i, and n_m is the total number of model and record pairs. The fragility estimates related to both damage state DS_i and the next more severe one DS_{i+1} needs to be substituted into Equation (5) in order to evaluate the probability $P^M_{DS_i}$ of the bridge sustaining damage state DS_i. The damage probability $P^M_{DS_i}$ is obtained by considering the response of the intact bridge to the mainshock event.

The likelihood $P^{A|M}_{DS_i|DS_j}$ of the mainshock induced damage grade DS_i progressing to a higher grade DS_j because of aftershock induced shaking is needed in the proposed framework. Evaluation of the conditional probability $P^{A|M}_{DS_i|DS_j}$ for a bridge is a more challenging task compared to evaluation of $P^M_{DS_i}$. In this evaluation, the fragility analysis needs to be performed using a damaged bridge model rather than an intact one. Specifically, the damage imposed on the model should be of grade DS_i. The actual mainshock motion that will impose this damage during the expected service life is not available at the time of assessment. The damage grade is a global measure of damage while the actual seismic response is sensitive to all local damages within critical locations combined. Thus, different ground motion records may damage critical zones of the bridge to varying extents as they impose the same global damage state DS_i. In the evaluation of conditional likelihood $P^{A|M}_{DS_i|DS_j}$, this record-to-record variability of mainshock motions that impose the same DS_i grade needs to be considered. One strategy to achieve this is to establish a set of mainshock motions and identify the scaling factors for each of these motions that correspond to the onset of damage state DS_i. Subsequently, aftershock fragility analysis is performed by simulating the response of each randomly generated structural analysis model to sequences of ground excitations. This sequences should consist of the mainshock shaking that imposes damage state DS_i followed by an aftershock excitation (Ryu et al., 2011). The specific aftershock shaking intensity level x'_{dj} that corresponds to the onset of damage state DS_j, is identified by repeating this analysis for a range of aftershock scaling factors. In this analysis, the polarity of aftershock excitation should be randomized as recommended by Ryu et al. (2011). It should be born in mind that the process entails considerable computational effort. To reduce this effort, an approach based on nonlinear regression recommended by Alessandri et al. (2013) may be adopted.

After the intensity levels x'_{dj} are identified for all the mainshock-aftershock sequences, Equation (7) may be utilized to establish the aftershock fragility of the bridge. In this case the resulting fragility $F_{D|IM;DSi}(d_j|x;DSi)$ is conditioned on the mainshock induced damage state DS_i. The required conditional probabilities $P^{A|M}_{DS_i|DS_j}$ can be obtained by substituting $F_{D|IM;DSi}(d_j|x;DSi)$ into Equation (5).

4. Probability of damage state classification and integration of SHM data into bridge reliability assessment

Quantifying the value of SHM via the Bayesian pre-posterior analysis as described in this paper and integration of SHM data into bridge reliability assessment requires probabilities $P^{E}_{DD_i|DS_j}$ of classification of structural states based on the indication from the SHM system. These can generally be found from probability distribution functions of a damage indicator corresponding to the different actual damage states (Omenzetter et al. 2016). These probability distributions will be dependent on the particular SHM system adopted. Here, we need to consider the whole process of SHM data collection and processing which output a damage state indicator. There are a number of challenges at this point as discussed below.

The various structural damage states are known to correlate better with measures related to structural displacements or rotations and associated ductilities, the latter particularly relevant for modern structures designed for seismic regions. For example, Table 1 (Banerjee & Shinozuka, 2008) shows classification of damage into several states depending on the rotational ductility demands. Yet measuring displacements or rotations in-situ for large structures presents a considerable practical challenge, mostly because a fixed reference base is difficult to find for contact measurement technologies, such as linear variable displacement transducers. Non-contact devices will often require a stable base too, which may not be easily available in seismic monitoring, and unobstructed line of sight, which is often unavailable due to vegetation, complex terrain or in densely built-up environs. The global positioning system does not yet offer accuracies required in our context. Strain gauges, and other types of attachable sensors for that matter, will not survive in the areas of large deformations – where we would ideally like them to be placed - because of cracking and spalling. On the other hand, the type of measurements that are more readily available, notably accelerations, do not yield features that readily map quantitatively into structural damage states. Double integration of acceleration time histories to obtain displacements is fraught with drifts. Any practically useful framework for quantifying the value of seismic SHM must recognize such practicalities.

Table 1. Damage states and corresponding rotational ductility demands (adopted from Banerjee & Shinozuka, 2008)

Damage state	Rotational ductility demand
None	<1
Negligible	1-1.52
Minor	1.52-3.10
Moderate	3.10-5.72
Major	5.72-8.34
Collapse	>8.34

A damage detection/classification and future reliability prediction solution that uses acceleration measurements combined with structural model updating and nonlinear time history analysis to establish the probabilities of correct and incorrect classification of structural state based on the indication from the SHM system is proposed here by extending the earlier work of Soyoz and his collaborators (Soyoz et al., 2010, Kaynardag & Soyoz, 2015; Özer & Soyoz, 2015). The approach adopted comprises the following steps:

- A nonlinear finite element (FE) model of the bridges is formulated. This model may also include effects such as soil-structure interaction if deemed important.
- When acceleration data captured by and SHM system becomes available it is used as input to a system identification algorithm to determine modal properties (natural frequencies, damping ratios and mode shapes). Note the type of data applicable for this step is from low level excitations such that the linear response regime prevails. It may be an output-only system identification, but if ground motion sensors are installed next to the bridge and/or on its foundations as part of the SHM system, input-output methods can be adopted that can improve the reliability of results. Enhanced system identification approaches may include considering environmental and operational effects on the responses, such as temperature or presence of vehicles on the deck.
- The FE model initial stiffness is calibrated (updated) against the identified modal parameters. Note because of the linearity limitation above other model parameters that govern the nonlinear part of the response cannot be inferred directly using this approach.
- The updated model is run for nonlinear time history analyses to identify the fragility of the calibrated model. In these analyses, the damage states are established based on, e.g. ductility of the numerically simulated response (Table 1).

Some sources of uncertainties propagating into potential misclassification errors and affecting $P^{E}_{DD_i|DS_j}$, such as the level of noise in acceleration sensor measurements, can be garnered from laboratory trials and previous field applications. In a similar way, uncertainties in modal system identification results (Chen et al., 2014; Chen et al., 2015) and numerical model updating procedures (Shabbir & Omenzetter, 2016) can be assessed. A 'trial' monitoring system can be installed to gather more site-specific data and reduce uncertainties, but a decision to do so should then be assessed for cost-benefit within the proposed decision making framework. However, beyond those the methodology will have very limited access to experimental validation data. Note we try to make inferences about the performance of an SHM system before we actually deploy it on the structure, thus have no 'hard' measured data. Since large structures such as bridges are unique, even available data or experience from 'similar' structures will have limitations. In any case, there is very little monitoring data available thus far from bridges that actually sustained seismic damage. Circumventing this major challenge will require relying on extensive probabilistic numerical simulations, where the given structural system with all expected uncertainties will be simulated for random combinations of structural properties and ground motion inputs to determine its 'virtual' acceleration responses. These responses will then be fed into the bullet-point procedure outlined above to obtain the detected damage state DD_i results for each response simulation. Afterwards, the resulting detected damage states will be compared to the 'actual' damage states DS_i obtained directly from the structural model obtained using ductility thresholds such as those in Table 1. It is clear that many an assumption will be made in this approach, and that formidable computational effort must be reckoned with in the pre-posterior analysis stage to map the measurements to failure probabilities. However, it should also be recognized that the actual operation of the damage classification system does not necessarily entail running the time consuming nonlinear time history analyses. Based on such analyses during the decision-making stage, relationships, e.g. utilising artificial neural networks, can be built between the identified stiffness loss, or even just recorded ground and response intensity measures like *PGA* and peak structural response acceleration, and failure probabilities for quick, near real-time estimation of the associated risks (de Lautour & Omenzetter, 2009).

5. Bridge seismic risk modelling: consequences of bridge failure

A broad overview of the various bridge failure consequences is presented in Imam and Chryssanthopoulos (2012), and this short discussion is based on their work, while more emphasis is placed here on these aspects that are of particular importance or are more specific to seismic failure consequences. It must be made clear at the onset of any consideration of bridge failure consequences that their modelling is multifaceted, complex and inherently uncertain.

The consequences can be categorized into four main groups: human, economic, environmental and social. Example of the most important consequences in each category are shown in Table 2.

Table 2. Consequences of bridge failure (adopted from Imam and Chryssanthopoulos (2012))

Category	Example
Human	Deaths Injuries Psychological trauma
Economic	Repair or replacement costs Loss of functionality/downtime Traffic delay/re-routing/management costs Clean up costs Rescue costs Regional economic losses Loss of production/business/opportunity Investigations/compensations Loss of other infrastructure services (e.g. electricity, communication cables carried by the bridge)
Environmental	CO_2 emissions Energy use Pollutant releases Environmental clean-up/reversibility
Social	Reputational damage Diminished public confidence in infrastructure Undue changes in professional practice

One important factor that influences bridge seismic damage consequences is that earthquakes affect larger areas simultaneously. Thus, e.g. casualties and injuries can be not only to those who happen to be on, under, or in the vicinity of the collapsing structure, but the loss of functionality of a bridge located on a critical route to a hospital can lead to further human consequences. Furthermore, a single structure is normally just one node of an interdependent transportation network. Other bridges located in the same area will also be exposed to seismic risk, and their potential loss of functionality will affect the traffic demands imposed on our focus structure. To quantify the expected number of people in need of hospitalisation in an aftermath of an earthquake, it will thus be necessary to perform a seismic risk study for the entire area the bridge may be expected to serve in such emergency (e.g. to estimate the number of collapsing buildings) and also simulate the functionality of the transportation network in the earthquake aftermath. Similarly, the direct cost to repair or even replace a bridge may be relatively low for a small and simple structure, but if the structure is located on an important route in a transportation network with poor redundancy, which furthermore can be impaired because of seismic damage to other bridges, the resulting economic losses due to traffic delays, detours and loss of business can be much more significant. These costs can also be widespread, affecting negatively the economy of entire regions, if, for example, the bridge is on a route serving a major sea port. Larger timescales, in the order of several years, for the consequences to unfold may need to be considered as rebuilding after earthquakes can take a significant amount of time.

6. Conclusions

We have outlined a framework for quantifying the value of information from SHM technology installed on a bridge. The general case we consider is that of a bridge structure that may sustain damage in the mainshock and further progressing damage in an aftershock. The value of SHM information is computed using the Bayesian pre-posterior approach to decision making. The evolution of the damage state of the bridge with an SHM system is conceptualised as a time-dependent, stochastic, discrete-state, observable dynamical system. Optimal decisions whether to adopt SHM and how to restrict traffic on a potentially damaged structure is formulated to minimise the expected total cost or risk. The paper then discusses how to estimate the bridge damage probability through stochastic seismic hazard and fragility analysis, and how to update these probabilities using SHM data through an approach that combines modal system identification, structural model updating and nonlinear time history simulations. Finally, a brief overview of quantifying bridge failure consequences is included.

Acknowledgements

Piotr Omenzetter works at the Lloyd's Register Foundation Centre for Safety and Reliability Engineering at the University of Aberdeen. The Foundation helps to protect life and property by supporting engineering-related education, public engagement and the application of research. The COST Action TU1402 on Quantifying the Value of Structural Health Monitoring is gratefully acknowledged for networking support.

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Application of Bridge Weigh-in-Motion measurements in assessment of existing road bridges

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.6

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Abstract. Application of current European standards (Eurocode) for the design of new bridges in assessment of existing ones proved non-efficient due to conservative assumptions regarding applied load and subsequent response of these bridges. That is why decisions regarding existing bridges should be based on probabilistic approach, combining advanced methods of analysis with real values and data gathered with on-site measurement and laboratory testing. These types of data are gathered trough Structural Health Monitoring tools as it allows us to observe and record various data over a period of time in order to estimate bridge current condition and to track eventual changes in its behavior. Bridge Weigh-in-Motion measurements, as a part of structural health monitoring of existing bridges, provide us with detailed information regarding volume and weight of traffic on the bridge, while also tracking its structural response. Site specific load models for examined bridges can be developed from collected traffic data, using various extrapolation methods. Simple and fast, and yet precise method of traffic data processing is presented in first part of this paper, while its application, along with bridge structural response data, in optimized bridge assessment is presented in the second part of the paper.

Keywords: optimized bridge assessment, bridge weigh-in-motion, site specific traffic load models.

1 Introduction

Current standards and codes for design of new bridges are based on conservative assumptions regarding load and resistance modelling in order to be applicable on different bridge types. Although those codes result in creation of safe and cost-effective new bridges, use of same standards for assessment of existing bridges may show that many of these bridges need to be strengthened or even replaced (Wiśniewski et al. 2012; Šavor & Novak 2015; Žnidarič et al. 2016). Available standards for existing bridges are less conservative, but are also general, while research showed that site specific bridge assessment, based on measured traffic, can lead to reduction in maintenance costs and extension of remaining service life of specified bridges (Žnidarič et al. 2012; Žnidarič et al. 2016).

Weigh-in-Motion (WIM) (Žnidarič et al. 2016)is a procedure that measures axle weight and gross weights as a vehicle drives over measurement site in full speed, without the need for slowing down or stopping. Stationary WIM systems today apply sensors built into the pavement, with exception of bridge based WIM systems, called Bridge Weigh-in-Motion (B-WIM), which use instrumented bridges as weighing scales. Main advantage over the stationary WIM systems is that they are fully portable, and during installation and maintenance they do not interfere with traffic flow (Žnidarič et al. 2012). Data collected with WIM measurements can be used for a number of applications such as traffic analyses, pavement and bridge design and/or assessment, selection of overloaded vehicles, etc. Site-specific traffic load models, developed from WIM data, are a key input for optimized assessment of existing bridges (Žnidarič et al. 2016). Development of site specific traffic load models using simple and fast method proposed in (Žnidarič et al. 2012) is presented in the chapter 2.

Data gathered with B-WIM measurements also provides realistic structural response of the bridge, which can be used to improve numerical models used in analysis. These types of data enable us to optimize load carrying capacity assessment, as it can be used to discover any type of degradation (even those non visible), such as cracking, which will affect transverse load distributions. Furthermore, realistic support conditions, which can have significant effect on bridge internal forces and bending moments, are also provided with B-WIM measurements. Application of B-WIM data in optimization of numerical bridge models is described in chapter 3.

2 Post processing of B-WIM data

The main challenge in development of site-specific load models is extrapolation of measured WIM data, and consequently, accurate estimation of the maximum load effect on a bridge (bending moments and shears) in certain time period/bridge lifetime. There are number of proposed methods, based either on fitting statistical distribution to the calculated on collected data or using simulations, such as Monte Carlo method to extrapolate limited traffic information. Method used in this paper is suitable for short and medium span bridges, with independent traffic lanes, and is proposed in (Žnidarič et al. 2012). It is based on convolution method (Sivakumar et al. 2011; Žnidarič & Moses 1997) and an assumption that highest load effect is achieved when two trucks from independent traffic flows are placed on the bridge side by side in each traffic lane, at the place of maximum action, what is defined as a loading event. Calculation of load effects of each vehicle passing the bridge, from data obtained with B-WIM measurements, is conducted using the influence lines method, where results are presented in terms of maximum expected moments and shear forces of the critical part of the bridge for a specified time period. Proposed method is based on the one proposed by Moses and Verma in (Moses & Verma 1987), and can be divided in five steps presented in sections 2.1. – 2.5.

2.1 Collection of B-WIM data

B-WIM measurements provide multiple parameters for every vehicle passing the bridge, including timestamp and lane position of every vehicle passage, gross weight, weight of every axle, axle number and spacing etc. In measurement process, vehicles weighing less than 3,5 tones are not taken into account, as they have very low impact on bridge performance and assessment. Every vehicle that is not classified automatically by the software can be manually checked and placed in the right class. Typical data acquired from B-WIM measurement is presented in Table 1 (only as an example, a single pass of two axle truck is showed).

Table 1 - B-WIM output example

Time stamp	Lane	Speed [m/s]	Class	Number of axles	GSW [kN]	AW1 [kN]	AW2 [kN]	Axle spacing [m]
2007-03-22-00-39-28-955	1	17,5	41	2	123,8	37,07	86,69	6,07

Collection of traffic data is conducted using commercial B-WIM system *SiWIM*®(Žnidarič et al. 2011), developed in Slovenia as an outcome of COST 323(Jacob et al. 2002) action and EC 4th Framework project Wave (Jacob 2002).

2.2 Calculation of load effects

Calculation of load effects (bending and shear at critical bridge sections) is conducted using bridge influence lines. Theoretical influence lines for bending and shear of the bridge are easily created, depending on a bridge span and support conditions. B-WIM measurements also can provide realistic influence lines (Žnidarič et al. 2010), which can differ from theoretical ones (due to age and deterioration of bridge bearings etc.) and would result in lower values of traffic effects (described in detail in chapter 3). Static load effect of each vehicle is defined with:

$$Q_s = \sum_{i=1}^{n} A_i \cdot I_i \tag{1}$$

where:

A_i is weight of the axle i,
n is number of axles of each vehicle,
I_i is value of the influence line due to the axle i at location x

Example of calculated static load values for two vehicles is given in Table 2, where vehicles are presented with axle number (n), weights (W_i), spacing ($A_{i,i+1}$) and maximum static load effect (bending M_{Max} and shear V_{Max}) with associated axle positions ($x_{i,M}$ and $x_{i,V}$).

Table 2 - Calculated values of bending and shear for single vehicle

n	W1 [kN]	W2 [kN]	W3 [kN]	A_{1-2} [m]	A_{2-3} [m]	M_{Max} [kNm]	$x_{1,M}$ [m]	$x_{2,M}$ [m]	$x_{3,M}$ [m]	V_{Max} [kN]	$x_{1,V}$ [m]	$x_{2,V}$ [m]	$x_{3,V}$ [m]
3	46,18	52,37	42,85	6,04	1,26	143,78	11,55	5,51	4,25	72,21	6,15	0,11	0,00
2	37,07	86,69	/	6,07	/	237,62	11,55	5,48	/	102,39	6,15	0,08	/

As total amount of vehicles data collected with B-WIM measurements is very large (it depends on the measurement duration, traffic flow and volume), theirs static values (Q_s) are combined into histogram showed on Figure 1 (same procedure is the same for shear force values, but due to the length of the paper it is not presented), separately for each lane. Load intervals (bins) on x axis must be small enough to provide good resolution of the relative frequency histograms. Minimal number of bins is not strictly defined, but 60 to 100 intervals should provide sufficient quality of histograms for further calculations. Furthermore, abscissa must cover values at least 10% above the maximum calculated load effects in order to model the tails of distributions, according to (Žnidarič et al. 2012).

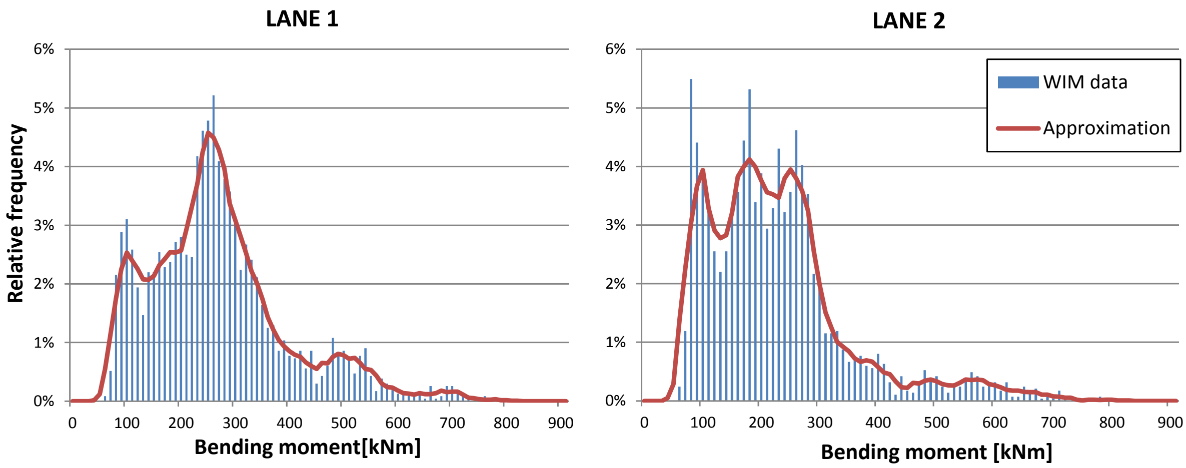

Figure 1 - Relative frequency histograms of bending moments for each lane

In order to smooth distribution curve and to extend it beyond the measured values, as extreme load events are infrequent (Figure 1), modified “moving average” approximation is applied on histograms. This approximation averages the selected number of values before and after averaged value, and is shown on Figure 1 with thick red line. Number of points to average depends on the reliability of traffic information, and can go up to ± 10, but in this example ± 3 points are used.

2.3 Generation of load effects histograms

Next step in presented procedure is convolution of histograms for each lane to simulate loading effect for an event comprised of vehicles from both lanes, which will be presented with probability mass function (PMF) and the corresponding cumulative distribution function (CDF).

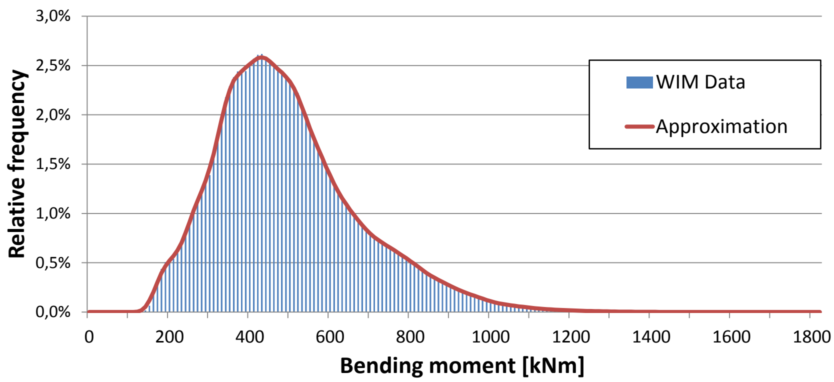

Figure 2 - Probability mass functions for loading event (bending moment)

As distributions of load effects for lanes 1 and 2 are independent, as we assumed before that traffic in one lane does not affect other, PMF of event for both lanes is defined as:

$$f_z = \sum_{k=1}^{m} f_x(k) \cdot f_y(z-k) \tag{1}$$

where f_x and f_y are PMFs (approximated histograms) of load effects for lanes 1 and 2, and f_z is the PMF of the load effects for an event comprising vehicles in each lane (Figure 2). As X and Y are independent and its PMFs f_x and f_y have m bins, Z equals to (X+Y) and f_z has the length of $(2 \cdot m\text{-}1)$ bins.

2.4 Cumulative distribution functions and maximum load effects

Cumulative distribution function (also called convolution curve (Žnidarič et al. 2012)) for a single loading event F_Z is derived from PDF on Figure 2. Expected maximum load effects F_{max} for different time periods are created using extreme value theory (Ang & Tang 1975):

$$F_{\max}(z) = F_Z^N(z) \tag{3}$$

where N is number of expected multiple presence events (when vehicles from both lanes meet on the critical section of the bridge) in associated time period. Convolution curves for bending moments, for different time periods, are presented on Figure 3. Evaluation of expected loading events N is described in the following section.

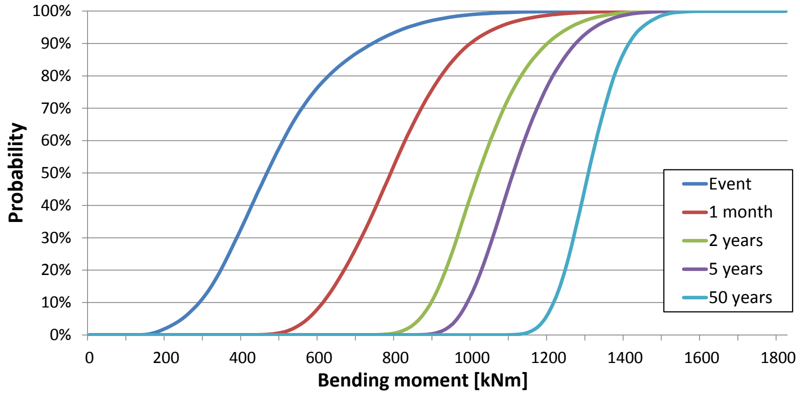

Figure 3 - Convolution curves for maximum bending moment in relevant time periods

Median and characteristic values of function $F_{\max}$ are easily calculated from convolution curves, while total predicted maximum load effects

$$Q = Q_S \cdot DAF \cdot g \tag{3}$$

where Q_S is static load effect from Equation 1, DAF is dynamic amplification factor, representing the dynamic amplification of the traffic loading, and g is girder distribution factor that represents proportion of total traffic load carried out by a critical cross section under analysis. Detailed information on calculation and proper selection of DAF and g values can be found in (Žnidarič et al. n.d.). Mean values (m) and standard deviations (σ) for maximum expected load effects are derived from convolution curves on Figure 3 and presented in the Table 3. Upper and lower characteristic values (5% and 95 % quantiles) can be easily calculated with the same procedure (Žnidarič et al. 2012).

Table 3 - Statistic parameters of maximum expected load effects

Time period	**Mean value [kNm]**	**Standard deviation [kNm]**
Single event	463,60	169,65
One month	922,14	125,72
Two years	1016,96	116,97
Five years	1107,48	115,54
Fifty years	1304,17	76,06

2.5 Selection of proper number of expected events *N*

Number of expected multiple presence events N directly affects maximum expected traffic loading on the bridge, in selected time period. As more multiple presence events occur on the bridge higher will be maximum load effect in selected time period. There are two approaches to calculate, generating headway histograms (Moses & Verma 1987) and gaps between vehicles, or deriving N directly from WIM data sample (if time stamp is provided to at least 1/100 of a second) (Žnidarič et al. 2012). In this example we use the latter, but due to the length of the paper it is not presented in detail, but can be found in (Žnidarič et al. 2012).

3 Application of B-WIM in assessment of exiting road bridges

Assessment of existing bridges using detailed numerical models of those bridges is very common approach in order to determine its load distribution, modal shapes, deflections and other bridge parameters that are needed to establish its load carrying capacity. These models are mainly developed using Finite Elements Method (FEM), and when original design plans are available, they can be developed to a very precise level, even with taking into account certain amount of degradation trough time.

Nevertheless, without data from on-site measurements and monitoring, it is very hard to simulate realistic behavior of existing bridge on its numerical model, as certain parameters are hard to determine only with visual inspection of the bridge. Along with determination of site specific load models, explained in previous chapter, B-WIM measurements can also provide additional data, such as realistic influence lines and transverse load distributions. Combination of these additional parameters with numerical models of the bridge, developed in *Sofistik* software (SOFISTIK AG, 2014) for structural analysis, is presented in following sections with load carrying capacity assessment of Case Study Bridge (Skokandic 2016).

3.1 Case Study Bridge

Case Study Bridge is simply supported highway bridge with a single span of 24,8 meters, its superstructure is composed of five prefabricated I-type girders connected with a monolithic concrete deck (Figure 4). Original design plans, along with built in reinforcement, were available from the archives and 3D FEM model is developed in *Sofistik* and showed on Figure 5.

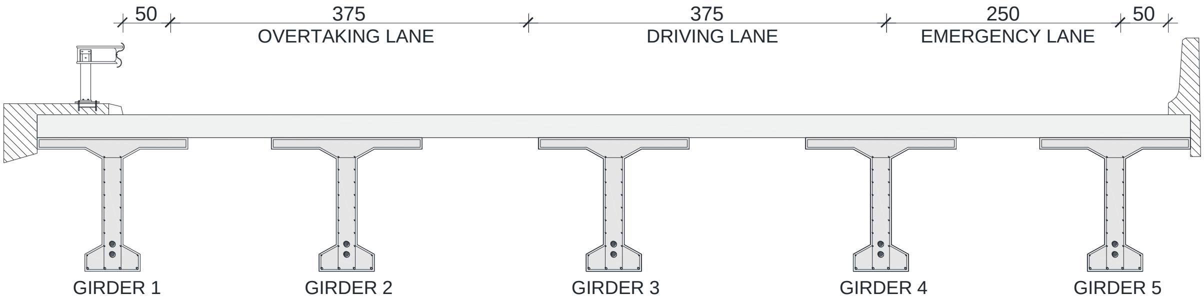

Figure 4 - Cross section of Case Study Bridge with built in reinforcement

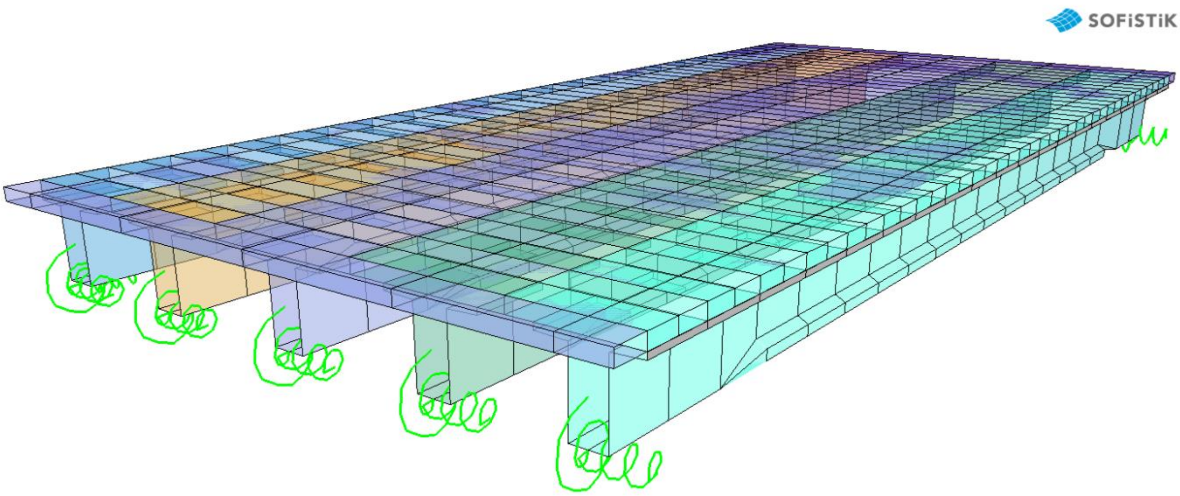

Figure 5 - 3D FEM model of Case Study Bridge

In initial step of the assessment procedure linear analysis was conducted, taking into account self – weight, additional permanent load and traffic load according to EN Load Model 1 (Eurocode 2004a). Bending moments at the middle of the span are calculated with load factors of 1,35 for all loads. Cross section resistance to bending is calculated using original design plans and built in reinforcement according to (Eurocode 2004b). Comparison of the results of initial assessment and the later steps, using B-WIM data, are presented in Table 4 at the end of the paper.

3.2 Application of additional B-WIM data

3.2.1 Realistic Influence lines

Based on the original design plans, bridge model is developed as simply supported single span bridge, with theoretical influence lines for bending moment in the middle of the span are showed on Figure 6.

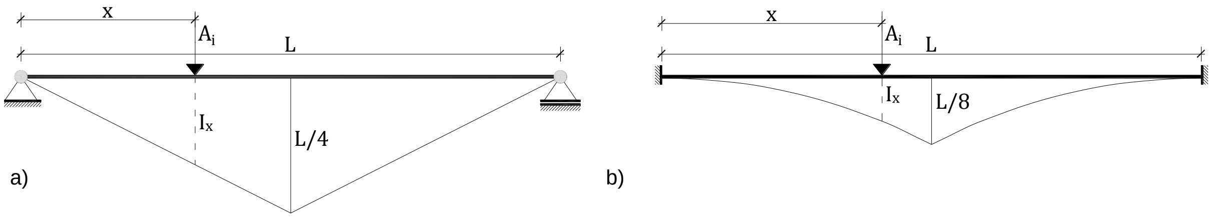

Figure 6 – Theoretical Influence lines for simply supported (a) and fixed bridges (b) (Žnidarič et al. 2012)

As B-WIM technology uses influence lines to calculate static load effect of each passing vehicle (as described in section 2.2.), it has to be calibrated on every bridge before the measurements process begin. Calibration is performed with number of vehicles with familiar axle weight, spacing etc. in order to adjust the measuring sensors. As an output of calibration process measured (realistic) influence lines are obtained, which can differ from theoretical ones due to changes in bridge support conditions (O'Brien et al. 2008; Karoumi et al. 2006). Comparison of theoretical and measured influence lines for bending moment in the middle of the span for Case Study Bridge is showed on Figure 7.

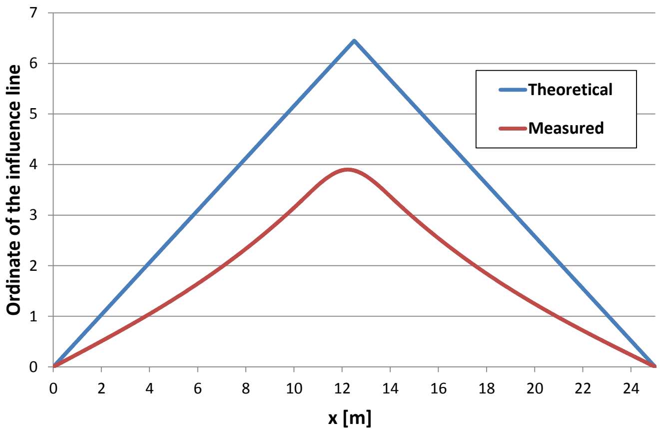

Figure 7 – Comparison of influence lines – Case Study Bridge

Measured influence line, showed above, revealed that realistic bridge behavior is not simply supported, and that the bridge is partially fixed on supports, probably due to degradation of bearings trough time. As a result, bending moments in the middle of the span are reduced compared to theoretical bridge model, adding to resistance/load ratio of bridge cross section.

3.2.2 Transverse load distribution

Distribution of total load on the bridge on its girders in basic numerical model is taken based on bridge cross section geometry and stiffness if the girders. As a part of B-WIM measurements, sensors in transverse direction are placed on every girder, providing its realistic deflection, which enables to determine amount of load taken by specific girder (Žnidarič et al. 2010).

These types of data reveal eventual differences between theoretical and realistic stiffness of bridge elements, pointing to some type of degradation, even those not visible, such as cracking of concrete element, yielding of reinforcement etc.

Figure 8 presents comparison of measured and theoretical load distribution factors (in percentages) for traffic load, showing that there is no significant difference in distribution. These results for Case Study Bridge were expected, due to no visible signs of degradation discovered during initial visual inspection.

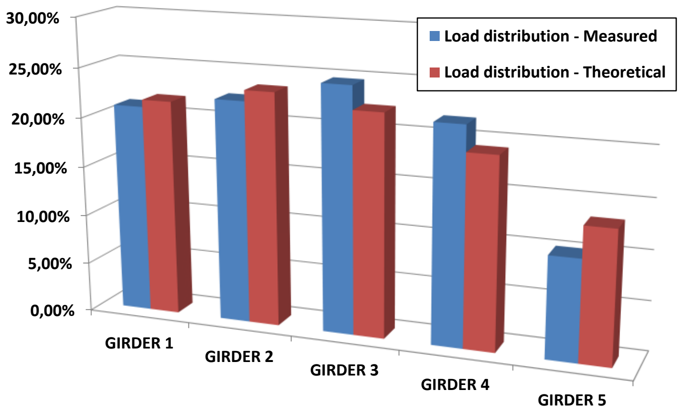

Figure 8 – Comparison of load distribution on each girder – Case Study Bridge

3.2.3 Site-specific traffic load models

Traffic load effects (bending moment in the middle of the span), determined from B-WIM data as explained in chapter 2, are applied on bridge model. Period of 75 years is chosen for extrapolation of available traffic data, as it represents remaining design service life of the bridge. Comparison of these traffic load effects with bending moments calculated in *Sofistik* using EN Load Model 1 (Eurocode 2004a) is presented in Figure 9.

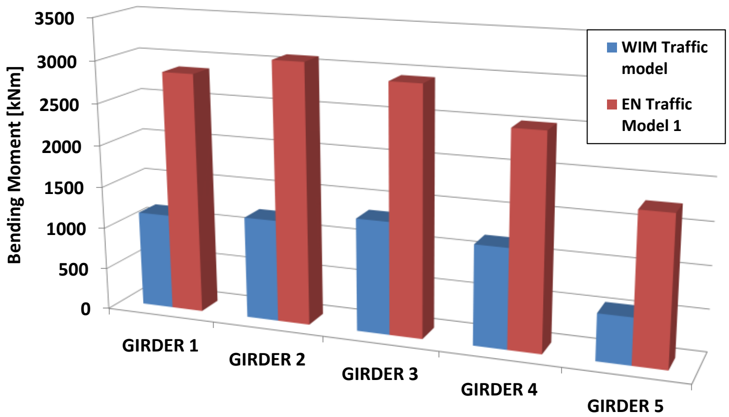

Figure 9 – Comparison of traffic load effects – bending moment in the middle of the span

3.3 Analysis of the results

Results of load carrying capacity of Case Study Bridge using B-WIM data and standards for the design of new bridges are presented in Table 4, as a comparison of load/resistance ratio of every girder. Same analysis can also be conducted using probabilistic approach, as site-specific load effects are defined with statistical parameters, mean value and standard deviation.

Table 4 – Analysis of the results (Skokandić 2016)

	Girder 1	**Girder 2**	**Girder 3**	**Girder 4**	**Girder 5**
M_{Rd}/M_{Ed} – based on current standards	1,046	0,992	1,022	1,124	1,390
M_{Rd}/M_{Ed} – based on proposed method	2,036	1,965	1,894	2,042	2,797

As shown in the table above, application of B-WIM data resulted in reduced bending moment on the Case Study Bridge, increasing the resistance/load ratio for around 100 %.

4 Conclusion

Results of Case Study Bridge assessment (Table 4) show how the classified by the current design standards as inadequate for use, because of insufficient resistance to bending of girder 2, bridge can be reassessed as safe with additional B-WIM data.

Economic aspect of this type of assessment can be defined as a comparison of initial investments in B-WIM measurements and reduction of bridge repair and maintenance costs as a result of more detailed bridge assessment.

Furthermore, beside the load carrying capacity assessment, WIM data can also be used for early discovery of non-visible degradations in bridge elements, as described in section 3.2.2.

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Acknowledgements

This article is based upon STSM performed in Slovenian national Building and Civil Engineering Institute under auspices of COST Action TU 1402 Quantifying the Value of Structural Health Monitoring, supported by COST (European Cooperation in Science and Technology).

Optimal decisions based on monitoring – case study of steel roof

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DOI: https://doi.org/10.5592/CO/BSHM2017.4.7

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Abstract. Monitoring of structures and related decisions based on cost optimization are discussed in this contribution. Many research publications and experimental data are currently available on inspection and on monitoring and they represent the outcome of the remarkable work done. Not all the topics of interest are widely debated and implemented in standards. The current state-of-practice in standards is summarized and aspects to be included in future recommendations are proposed.

The implementation of risk-based decisions is illustrated in a case study dealing with a stadium roof in Northern Italy. Snow actions are important especially in northern and mountainous regions where heavy snowfalls and related accumulation result to considerable loads. As the roof structure fails to comply with the requirements of Eurocodes, a permanent monitoring system has been installed to allow for real time evaluation of the reliability level of the structure. The system supplies the necessary information supporting immediate decisions on closure of the stadium in case of an extraordinary snow load. Cost-optimal decisions regarding the closure of the stadium are analysed based on a limit state function, on probabilistic models for the influencing parameters including measurement uncertainty, economic losses related to closure and failure consequences. The results demonstrate the potential of the use of the monitoring systems and probabilistic reliability analysis in order to support decisions and highlight the need for their implementation in future standards.

Keywords: SHM, monitoring, probabilistic methods, reliability analysis, risk analysis, snow load, reliability level.

1 Scope of the fact sheet

The application of risk-based methods to illustrate the potential use of monitored results together with probabilistic reliability analysis is presented. The implementation into standards is outlined.

2 Basis / standards

The development of guidelines in the SHM sector was summarized in a previous fact sheet (Diamantidis et. al., 2015). Thereby the monitoring of the structural behaviour and the associated updating of the real performance and of the reliability of the structure were discussed. Risk and reliability analysis procedures are available and their implementation is shown in (Holicky et. al., 2014). An essential step for the assessment is the specification of target reliability levels (JCSS, 2001; Steenbergen et. al., 2015). The target reliability reflects the acceptable risk since it is associated with consequence classes (ISO 2394; EN 1990).

Guidelines, research publications and experimental data are currently available on inspection and on monitoring and they represent the outcome of the remarkable work done. A significant step thereby is to combine SHM with reliability and risk analysis methods in order to fulfil target safety levels also with support of monitored values. At such a stage alarm threshold for selected damage sensitive feature parameters can be set. More complex damage identification algorithms, including model based and non-model based approaches, can be selected and applied together with reliability analysis techniques.

The implementation of risk-based decisions is illustrated in this fact sheet in a case study dealing with a stadium roof in Northern Italy. Snow loads are important especially in northern and mountainous regions where heavy snowfalls and related accumulation result to considerable loads. The method can be applied to other cases and appropriate decision support systems can be developed.

In recent years, multiple major snowstorms resulted in numerous roof failures. The current reference regarding snow loads in Europe is EN 1991-1-3 that accounts for roof slope, thermal characteristics of the structure, and exposure to wind to quantify the amount of snow that may be present on a roof over the course of a winter season.

3 Application study

The numerical example is focused on a stadium erected at the beginning of the 1990s and located in Trento, Northern Italy, at an altitude of 190 m. The roof of the stadium consists of a cantilever steel beam IPE 500 (Fig. 1). The lengths of the first span and of the cantilever are 4 m and 8 m, respectively. The spacing between adjacent beams is 5 m. The inclination of the steel beam is negligible ($\alpha \approx 0°$).

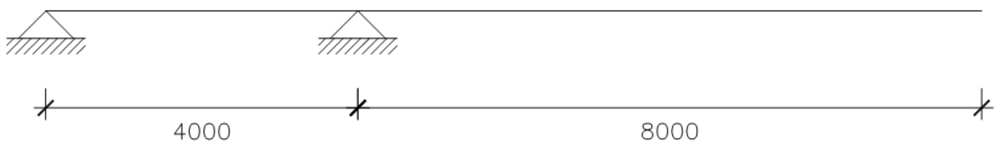

Fig. 1. Scheme of the roof beam

The stadium can accommodate up to 4000 people and it is widely used in order to host sport events, concerts and shows. As the structure is located in the Alpine region and it is subjected to high snow loads, after the recent roof collapses and the related studies, it has been decided to investigate its actual structural reliability. Analysis of past and present prescriptive codes reveals that the design snow load increased significantly over the last decades.

The former Italian standard D.M. 12.02.1982 recommended the following snow loads for zone I (Northern Italy):

$$q_s = 0.9\ kN/m^2 \qquad h_s < 300\ m$$

$$q_s = 0.9 + 1.5(h_s - 300)\ kN/\ m^2 \qquad h_s \geq 300\ m \tag{1}$$

$\alpha \leq 20°$: no reductions

$20° < \alpha < 60°$: 2.5% reduction (linear) for each degree of inclination of the roof

Considering the aforementioned characteristics, a snow load of 0.9 kN/m^2 was assumed. The current code, D.M. 14.01.2008 (NTC 08), for the same site, leads to:

$$q_s = \mu_i \cdot q_{sk} \cdot C_e \cdot C_t = 0.8 \cdot 1.5 \cdot 1 \cdot 1 = 1.2\ kN/m^2 \tag{2}$$

with C_e = 1 (exposure coefficient), C_t = 1 (thermal coefficient), μ_i = 0.8 (shape factor for a monopitch roof), q_{sk} = 1.5 kN/m^2 (characteristic ground snow load for the Alpine region and for altitude over 200 m). The calculated values indicate that snow loads currently assumed are higher than in the past and for this reason many existing structures subjected mainly to snow loads do not achieve the same reliability level imposed to the new structures in modern codes. In order to keep the reliability level of the stadium classified in the highest consequence class CC3 (EN1990, 2002) acceptable, it was decided to implement on the roof of the stadium a permanent online monitoring system of the snow depth (Lanzinger and Theel, 2010). The purpose of the implementation of this permanent monitoring system is to close the stadium and forbid the presence of people in it when the snow depth reaches a threshold value d_{lim} and reliability of the roof cantilevers drops below a specified target level.

The probabilistic reliability analysis is based on the limit state function Z(**X**) for the section of the beam subjected to the maximum bending moment:

$$Z(\boldsymbol{X}) = \vartheta_R W_{\mathrm{pl}} f_{\mathrm{y}} - \vartheta_E L^2/2\, [\gamma_{\mathrm{steel}} \cdot A_{\mathrm{s}} + g_{\mathrm{roof}}\, b + \mu_i \times \gamma_{\mathrm{snow}}(d) \times b \times d] \quad (3)$$

Notation and probabilistic models of all the basic variables based on the recommendations of (JCSS, 2001) are provided in Table 1.

Table 1. Models of basic variables

Basic variable	Dist.	Mean	CoV	Note
Resist. model unc., ϑ_R	LN	1.1	0.05	Yielding resistance for bending without loss of stability (Nadolski and Sykora, 2015)
Section modulus IPE500, W_{pl}	DET	0.002194 m^3	-	-
Yield strength S275, f_{y}	LN	308.6 MPa	0.07	-
Load eff. unc., ϑ_E	LN	1	0.05	Reduced variability considered for the structural system with small uncertainties in idealisation of supports and in composite actions of the cantilevers.
Span of cantilever, L	DET	8 m	-	-
Steel density, γ_{steel}	N	77 kN/m^3	0.01	-
Sectional area IPE 500, A_{s}	DET	0.01155 m^2	-	-
Weight of roofing, g_{roof}	N	0.5 kN/m^2	0.05	Variability could be reduced by measurements; additional cost however cannot be justified due to low sensitivity factor of this variable.
Spacing of cantilevers, b	DET	5 m	-	-
Shape factor, μ_i	N	1	0.05 (0.15)	The coefficient covers deviation of the snow load on the roof from a uniform distribution. The reduced value of CoV is based on the judgement of the authors for this case study while the larger estimate is a general recommendation in (JCSS, 2001). Note that the coefficient does not make distinction between the snow loads on the ground and on the roof in case of snow depth monitoring.
Snow density, $\gamma_{\mathrm{snow}}(d + \Delta d)$	LN	$1.09d + 2.4$; in kN/m^3 for d in m	0.2	The mean is an average of estimates obtained by the snow density models provided in (ISO 4355, 2013) for the location of the stadium. CoV is obtained by comparing outcomes of the ISO 4355 models; CoV can be considered independent of snow depth for $d < 1$ m.
Snow depth, d	N	Monitored, in m	See note	Standard deviation of 0.01 m is considered to account for measurement uncertainty; the most unfavourable value from measurements from several locations on the roof is considered.

Considering limit state function (3), the variation of reliability index β (EN 1990, 2002) with snow depth d is displayed in Fig. 2 for the two alternative values of the coefficient of variation of μ_i (Table 1). It is observed that the increased value of CoV of the shape factor leads to significant drop of the reliability level, about 0.5 in terms of β and the probabilistic model of μ_i deserves a careful consideration in a more advanced analysis.

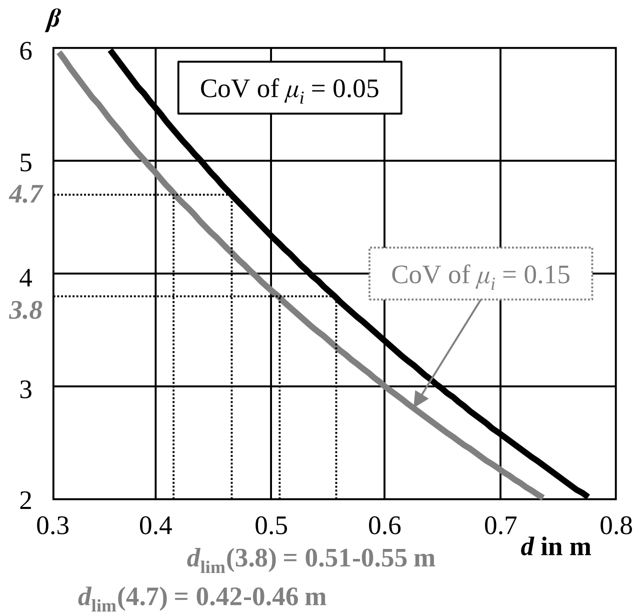

Fig. 2. Variation of reliability index β with snow depth d and limiting values d_{lim} for the selected target levels.

Considering CoV of μ_i = 0.05, the FORM analysis indicates the dominating role of uncertainties in snow density (sensitivity factor -0.84), moderate influence of steel yield strength and model uncertainties (absolute values of sensitivity factors between 0.2-0.35) and minor influence of the other random variables included in limit state function (3). The detailed investigation of uncertainties in snow density is beyond the scope of this fact sheet. However, it is noted that the probabilistic model given in Table 1 is in a good agreement with more elaborated models proposed by (Pistocchi, 2016) for Alpine and Maritime areas, by (Jonas et al., 2009) for Swiss Alps and by (Sturm et al., 2010) for regions in the United States, Canada, and Switzerland.

When a target reliability level is specified, a limit value d_{lim}, above which reliability becomes unacceptable and the stadium must be temporarily closed, can be obtained from Fig. 2. However, target levels for such temporary situations are typically not provided in standards. Recently (Tanner and Hingorani, 2016) proposed a procedure to derive target levels for short-term situations; however a widely accepted approaching is missing and is far to be standardised.

Therefore, a simplified cost-benefit analysis is conducted to decide about the use of the stadium on the basis of the balance between the benefits and the expected failure consequences. The use of the stadium is authorised when the associated benefit B exceeds the probability of failure dependent on snow depth $p_{\mathrm{F}}(d)$, multiplied by the consequences of failure C_f:

$$B \cdot [1 - p_{\mathrm{F}}(d)] \approx B \geq C_{\mathrm{f}} \cdot p_{\mathrm{F}}(d) \tag{4}$$

B and C_f need to be expressed in the same units. Failure costs C_{f} include cost of repair or replacement of the structure, economic losses due to non-availability or malfunction of the structure, societal consequences (costs of injuries and fatalities that can be expressed e.g. in terms of compensations or insurance cost), unfavourable environmental effects and other (loss of reputation, introducing undesirable 'non-optimal' changes of design practice). Usually monetary units can be used to combine the various contributors to failure costs.

Realistically assuming that the benefit is less than the failure costs, $B < C_{\mathrm{f}}$, then the target failure probability based on the economic optimisation, p_T, is obtained from Equation (4):

$$p_F(d) \leq p_T \approx B/C_f \tag{5}$$

The reliability index corresponding to the target probability is (for $B < C_f$):

$$\beta_T = -\Phi^{-1}(p_{F,eco}) \approx -\Phi^{-1}(B/C_f) \tag{6}$$

where Φ^{-1} is the inverse cumulative distribution function of the standardised normal distribution. The target reliability index β_T is related to a short period of a single snowfall, say in terms of one to a few weeks, for which the stadium is to be temporarily closed. Fig. 3 indicates the variation of the target reliability index β_T with the ratio B/C_f reflecting though the sensitivity of the obtained results. The target level is approximately linearly proportional to the order of magnitude of the ratio, which is commonly less than 0.001.

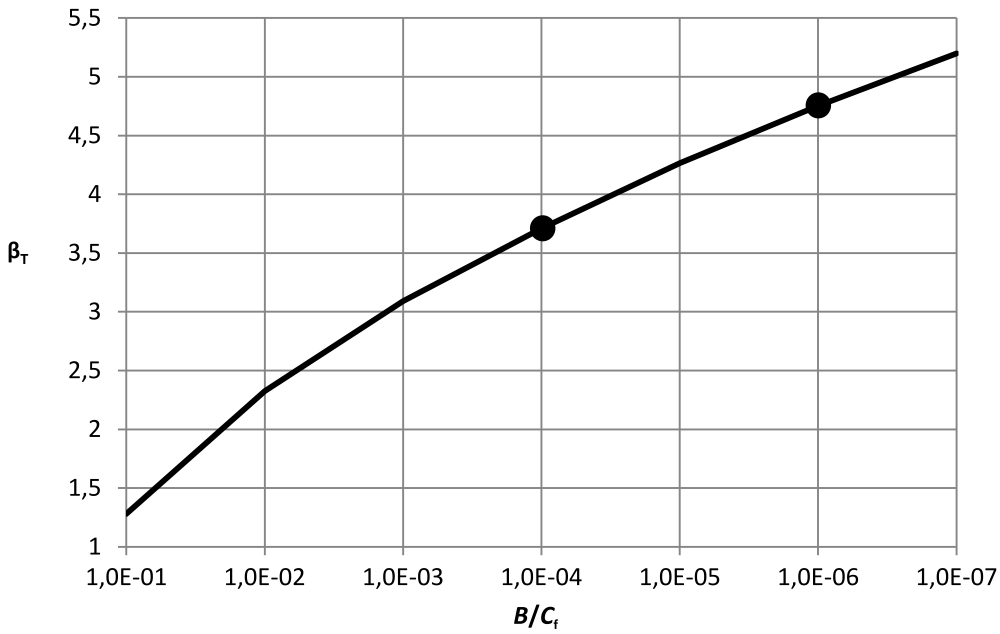

Fig. 3. Variation of the target reliability index β_T with the ratio *B/Cf*

A possible approach is to consider the benefit as the average income deriving from the tickets sold to the public for a certain event and calculated multiplying the ticket cost and the number of tickets sold. Possible values for the ticket cost range from 1€ to 100€. Human failure consequences dominate the total consequences and can be transformed into monetary units by multiplying the expected number of fatalities and the Societal Value of a Statistical Life (SVSL) according to the Life Quality Index approach (ISO 2394, 2015). The order of magnitude of the SVSL is taken here as 1000000 €. It follows that plausible values for the ratio B/C_f are comprised between approximately 10^{-4} and 10^{-6}. The corresponding β_T values are thus ranging between approximately 3.7 and 4.8 (Fig. 3). Based on these estimates, Fig. 2 portrays the limit values of snow depth.

It is noted that the presented cost-benefit analysis assumes that the stadium is operated by a public authority. In the case of a private owner, the SVSL should be replaced by compensation costs related to harm to the users that would need to be specified considering insurance of the company. Further, societal (group) risk criteria for human safety (ISO 2394, 2015; Steenbergen et. al., 2015; Sykora et al., 2016) should be applied in order to verify whether the audience is exposed to excessive safety risks.

It is emphasised that the presented analysis is simplified to illustrate the key steps of decision making and interpreting monitoring results. Future extended studies will investigate also the system aspects of reliability analysis and failure consequence modelling. A simplified approach applied by (Sýkora et al., 2014) for a roof structural system exposed to extreme wind loading is to be utilised.

4 Conclusions

The reliability analysis of the study case reveals that the roof of the stadium is able to sustain snow loads comparable with the snow load currently imposed by the Italian code for new structures.

This system provides a real time evaluation of the reliability level and supports decisions regarding closure of the stadium in case of heavy snowfalls or excessive snow accumulation. This does not prevent possible damages to the structure but avoids fatalities and injuries, thus considerably reduces the total consequences of failure. The analysis can include in more detail the discussed uncertainty contributors.

The simplified case study shows the potential of structural monitoring and provides the methodology on how monitoring and probabilistic reliability analysis can improve decisions about utilisation of structures. The methodology can be extended to a posterior decision analysis implementing the conditional value of sample information (Straub, 2004). Also the consideration of past satisfactory performance can be included in the analyses (Holicky et.al., 2014, Diamantidis et. al., 2015).

Applications of the described procedure can bring considerable societal benefits related to performance of important structures and infrastructures such as stadiums, bridges, congress halls, multi-purpose arenas or structures in energetics. The methods can be applied following the principles of new Eurocodes under development, especially those related to the evaluation of existing structures, to fulfil the imposed target levels of reliability. They allow namely the direct evaluation of reliability and risk and lead to interventions for keeping them below specified acceptable levels.

Acknowledgements

This study was supported by the Regensburg Center of Energy and Resources (RCER) of the Ostbayerische Technische Hochschule Regensburg (OTH Regensburg) and by the Ministry of Education, Youth and Sports of the Czech Republic under Grant LD15037. COST Action TU1402 on Quantifying the Value of Structural Health Monitoring is gratefully acknowledged for networking support.

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5 IABSE WC1 CONTRIBUTIONS

5.1 Indicators for the performance assessment of road bridges through monitoring (paper / presentation)

P. Tanner, M. Prieto

5.2 Bridge scour reliability under changing environmental conditions (paper / presentation)

B. Imam, A. Kallias

5.3 Bridge SMS: Intelligent bridge maintenance and management system (paper / presentation)

D. Bekic, I. Kerin, P. Michalis, E. McKeogh, P. Cahill, V. Pakrashi

5.4 Non Gaussian multi-parameter Bayesian estimation through the mechanical equivalent of logical inference (paper / presentation)

D. Bolognani, A. Verzobio, C. Cappello, D.Tonelli, B. Glisic, D. Zonta

5.5 Integrated Structural Health Monitoring in steel arches bridges using continuous dynamic monitoring: two case studies in China (paper / presentation)

J. Gao, T. Liu, B. Briseghella, G. Carlo Marano

5.6 Optimal sensor placement for Spatial Variability Assessment of Structures (paper / presentation)

F. Schoefs, E. Bastidas-Arteaga

Indicators for the performance assessment of road bridges through monitoring

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.1

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Abstract. Different causes may lead to the non-compliance of a particular requirement related with an existing infrastructure. Often, they may be traced back to deviations from expected actions or resistances. The quantification of parameters related with such influences may provide evidence about the degree of compliance of a given structure with a particular serviceability or safety requirement. Such parameters may therefore be called indicators and associated threshold values can be established on a risk basis, as well as admissible average frequencies for outcrossing.

Indicators can be monitored comparing the measured values continuously to the previously established threshold values. Alarm systems may be installed which are activated in case of outcrossing. Safety measures can therefore be adopted depending on the consequences of the observed non-compliance. Based on such an approach and by using modern information technology, inspections of large infrastructures may be automated and optimised.

A series of indicators for use in road bridge inspection are proposed hereunder, together with their respective threshold values and allowable mean frequency of outcrossing. The paper also includes a practical application in which these criteria are applied to a road bridge with an unknown reliability level.

Keywords: Uncertainties, assessment, monitoring, road bridges, indicators, threshold values

1 Introduction

Structural systems must be engineered, built, operated and maintained to ensure economically sound use throughout their service life, while meeting predefined serviceability and structural safety requirements. Such requirements must be fulfilled with an acceptable level of reliability that depends on a number of parameters, including the reference period considered and possible consequence of non-compliance or failure.

Different influences and circumstances may underlie the non-compliance of a requirement:

- deviations from the assumed values for actions or the effects of environmental influences;
- deviations from values established for other influences, such as construction execution inaccuracies;
- geotechnical actions;
- chemical, physical and biological actions;
- actions or influences not considered;
- dynamic effects such as resonance;
- deviations from the structural or soil strength values assumed;
- strength loss due to decay mechanisms such as corrosion, embrittlement or fatigue;
- structural overloads or strength losses induced by accidental actions.

A structure's compliance with predefined requirements can be determined by quantifying parameters associated with these influences or circumstances, which are consequently called indicators and, generally speaking, refer to system geometry, materials, actions or structural behaviour. The parameters chosen for effective inspection and monitoring should be the ones whose variation has the greatest effect on the reliability of the system studied. Hence, parameter selection depends on the type of technical system involved, its purpose and operation, exposure conditions, constituent materials and the data acquisition resources available.

When inspection planning is based on the adoption of suitable indicators, its scope can be adapted to the condition of the system elements, which can be prioritised in keeping with their significance and the decay mechanisms observed. From this perspective, the use of indicators to define inspection strategy can be interpreted as a risk reduction measure.

The objective of this paper is to define the grounds for the early detection of possible damage or anomalies with a view to adopting risk mitigation measures before an undesired event such as structural collapse occurs. Inspection scope and intensity should be determined on the basis of the characteristics and significance of the structure inspected, as well as of the allowable risk.

2 Scope and assumptions

The present contribution summarizes the principles for the definition of the inspection and maintenance activities for bridges throughout their service life, to be established in detail for each individual case within the framework of a specific bridge project (Tanner and Prieto (2013a, 2013b)). These principles are applicable to all bridges, at all construction and use stages. They are also applicable to temporary load bearing structures, as well as for the assessment of existing bridges, related to refurbishments, changes of use, repair or strengthening.

The specific rules established refer to road bridges, regardless of their constituent materials. The principles defined in this paper should be used in conjunction with a consistent set of standards and recommendations such as the Spanish codes for actions on road bridges, IAP-11 (2011), and the design of steel-, EAE-11 (2011), concrete-, EHE-08 (2008), and composite bridges, RPX-95 (1996). Alternatively, they may also be used together with the respective Eurocode rules.

3 Inspection

Depending on the context in which inspection is conducted and the objectives and resources used, a distinction can be drawn among the following:

- observation;
- periodic inspection;
- control measurements;
- monitoring.

Observation is understood to mean the perception, at pre-defined intervals, of the condition of a structure and its performance. Observations may also be conducted on the occasion of the performance of other tasks, such as maintenance. The qualitative determination and appraisal of the condition of a structure during inspections conducted at pre-defined intervals in accordance with given priorities constitute what is known as *periodic inspection*. *Control measurements* may be taken to quantify certain parameters representative of the structure or its performance. Lastly, *monitoring* consists of determining the condition of a structure through frequent or continuous recording of certain parameters representative of the system or its performance and the comparison of the values recorded to the respective thresholds.

The thresholds to which the data acquired with appropriate techniques and instruments are to be compared must be established depending on the level of reliability associated with each requirement. Information on how to determine thresholds is furnished in section 4 below. The measures that should be adopted when the acquired values for a given indicator exceed one or several of the associated thresholds are also discussed.

The data acquisition instruments used to monitor bridges must be able to record their static (lasting geometric changes) and dynamic (instantaneous geometric changes) behaviour. Moreover, for the data recorded to be useful in the long run, the instruments used must be able, at any given time, to establish the ratio between the existing and initial values. Thanks to recent progress in data acquisition, transmission and processing technologies, most parameters relevant to structural performance, particularly geometric change, can be continuously monitored. The effectiveness of inspection and maintenance measures can be considerably enhanced by systematically and suitably installing modern measuring systems on large infrastructures. One example of such a system can be found in the optical fibre sensors, which deliver resolution on the order of thousandths of a millimetre and feature stability over time.

Data acquisition on the established indicators forms part of programmed bridge inspections and should not be confounded with data collection for structural assessment. The need to assess structural reliability may be the outcome of inspection findings, where, for instance, the indicators quantified exceed the respective thresholds (section 4).

4 Assessment of threshold values associated with serviceability and structural safety

4.1 General

As in the determination of action effects in serviceability or structural safety calculations, the threshold values for the parameters quantified during road bridge inspections, known as reliability indicators, are normally established by structural analysis. Such analysis are based on the principles laid down in a consistent set of standards and recommendations such as the aforementioned national codes IAP-11 (2011), EHE-08 (2008), EAE-11 (2011) and RPX-95 (1996) or the respective Eurocodes. Where existing bridges are involved, the updated values for the relevant variables should be used for the analysis, and where no updated information is available, the nominal values can often be applied.

Structural analysis should deploy parameters able to predict a bearing structure’s performance sufficiently accurately for the control situations studied. The methods used to these ends must be backed by substantiated theory. The analytical model must integrate actions and other influences, geometric data and the properties of the materials constituting the structure and the terrain.

4.2 Requirements

4.2.1 Overview

Table 1 lists the requirements for road bridges, irrespective of the constituent materials, in increasing order of the consequences of possible non-compliance. Indicators are established and the respective threshold values are also given, along with the allowable mean frequency of transgression of the associated requirements. Taking a 1 year reference period as a basis, both the threshold values and the admissible frequencies of transgression are established in such a way to represent an acceptable level of structural performance per unit of time, equivalent to that associated with permanent structures according to the applied code requirements (see also item 4.2.3). The measures to be adopted in the event of non-compliance of any of these requirements are mentioned in item 4.3 below.

The choice of indicators is conditioned by the availability of suitable data acquisition or measuring facilities. In the aforementioned study (Tanner and Prieto (2013a, 2013b)), monitoring was usually, but not exclusively, conducted with fibre optic sensors. As a general rule, when threshold values are determined on the grounds of structural analysis, monitoring can only detect the effects of actions applied after a new or existing bridge is instrumented.

Table 1. Requirements, indicators and thresholds values associated with road bridge serviceability (SLS) and structural safety (ULS).

	Demand	Consequences	Requirement	Indicator	Threshold: Value $E_{ser,lim}$; $C_{ser,lim}$; $E_{d,lim}$		Threshold: Mean frequency $\omega_{ser;}$ $\omega_{d;}$
SLS	Appearance	Reversible	Deformations	Deflection	L/700 1)		50 % of time
SLS	Appearance	Reversible	Deformations	Strain	$E_{ser,lim,2}$		50 % of time
SLS	Comfort	Reversible	Deformations	Deflection	L/1000 2)		Weekly
SLS	Comfort	Reversible	Deformations	Strain	$E_{ser,lim,1}$		Weekly
SLS	Comfort – Maximum – Medium – Minimum	Reversible	Vibrations	Acceleration	a_v 3)	a_h 3)	
					0.5	0.1	-
					1.0	0.3	-
					2.5	0.8	-
ULS	Structural reliability	Reversible	Safety of structure and facilities	Traffic loads 4)	$E_{d,lim,1}$		Weekly
ULS	Structural reliability	Reversible	Safety of structure and facilities	Strain	$E_{d,lim,1}$		Weekly
ULS	Structural reliability	Irreversible	Safety of people	Traffic loads 4)	$E_{d,lim,0}$		Yearly
ULS	Structural reliability	Irreversible	Safety of people	Strain	$E_{d,lim,0}$		Yearly

L: span length
1) Deflection after subtracting possible precamber, bearing in mind long-term effects due to creep, shrinkage and relaxation
2) Deflection due to traffic loads; higher values, up to L/500, acceptable for existing bridges on low traffic capacity roads
3) a_v: vertical acceleration [m/s^2]; a_h: horizontal acceleration [m/s^2]
4) Support reactions are recorded using the bridge as a weighing scale

4.2.2 Threshold values associated with serviceability

A bridge is regarded as serviceably sound if the values of the parameters measured during inspection are non-compliant with condition (1) less often than the allowable frequency, ω_{ser}, associated with the respective threshold value:

$$E_{mon} \leq E_{ser,lim} \tag{1}$$

E_{mon} measured value of a given indicator

$E_{ser,lim}$ threshold value for the same indicator, associated with serviceability

For some indicators associated with serviceability requirements such as deflection due to traffic loads or acceleration induced by dynamic actions in bridges with pedestrian access, standard IAP-11 (2011) lists indicative thresholds for in-service structural performance parameters. Given that these values may be adopted as serviceability thresholds ($C_{ser,lim}$) with no need to conduct structural analysis, bridge performance may be regarded as suitable where the values measured for the respective parameter are non-compliant with condition (2) less often than the allowable frequency, ω_{ser}:

$$E_{\text{mon}} \leq C_{\text{ser,lim}} \tag{2}$$

The threshold value for action effects associated with the relevant control situations that should not be exceeded more than once a week can be determined from the following expression (symbols defined as in standard IAP-11 (2011)):

$$E_{\text{ser,lim,1}} = E\left(\sum_{j\geq1} G_{k,j} \text{"+"} P \text{"+"} \psi_{1,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \psi_{2,i} \cdot Q_{k,i}\right) \tag{3}$$

The threshold value for action effects associated with the relevant control situations that should not be exceeded more than 50% of the time can be determined from the following expression (symbols defined as in standard IAP-11 (2011)):

$$E_{\text{ser,lim,2}} = E\left(\sum_{j\geq1} G_{k,j} \text{"+"} P \text{"+"} \sum_{i\geq1} \psi_{2,i} \cdot Q_{k,i}\right) \tag{4}$$

In the absence of more detailed analysis, the values of the factors for combination values of actions ψ_1 and ψ_2 can be taken from standard IAP-11 (2011) or the respective Eurocode EN 1990:2002/A1:2005 (2005).

4.2.3 Threshold values associated with structural safety

A bridge is regarded as structurally safe if the values of the parameters measured during inspection are non-compliant with condition (5) less often than the allowable frequency, ω_d, associated with the respective threshold value:

$$E_{\text{mon}} \leq E_{d,\text{lim}} \tag{5}$$

E_{mon} measured value of a given indicator

$E_{d,lim}$ threshold value for the same indicator, associated with structural safety

The threshold value for action effects associated with the relevant control situations that should not be exceeded more than once a year can be determined from the following expression, in which the symbols are defined as in standard IAP-11 (2011):

$$E_{d,\text{lim},0} = E\left(\sum_{j\geq1} \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_P \cdot P \text{"+"} \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \cdot \psi_{1,i} \cdot Q_{k,i}\right) \tag{6}$$

The threshold value for action effects associated with the relevant control situations that should not be exceeded more than once a week can be determined from the following expression (symbols defined as in standard IAP-11 (2011)):

$$E_{d,\text{lim},1} = E\left(\sum_{j\geq1} \gamma_{G,j} \cdot G_{k,j} \text{"+"} \gamma_P \cdot P \text{"+"} \gamma_{Q,1} \cdot \psi_{1,1} \cdot Q_{k,1} \text{"+"} \sum_{i>1} \gamma_{Q,i} \cdot \psi_{2,i} \cdot Q_{k,i}\right) \tag{7}$$

In cases where non-compliance of safety criteria may involve risks to persons, the partial factors for actions to be used for the establishment of the threshold values should be based on life-safety risk-related acceptance criteria associated with current best practice. Using a life safety risk metric proposed in Faber et al. (2015), which relates risk exposure due to different activities and applied technologies, the general approach for the definition of the time-dependent safety requirements is to maintain the acceptable life safety risk per time unit during the reference period considered for the use of a structure at the same level as for structures under permanent use conditions.

In the absence of more detailed analysis, and assuming that the investigated bridge is correctly designed to consistent code specifications, the values of the partial factors for actions γ_G, γ_P and γ_Q as well as of the factors for combination values of actions ψ_0, ψ_1 and ψ_2 can be taken from standard IAP-11 (2011) or the respective Eurocodes, EN 1990 (2002), EN 1990:2002/A1:2005 (2005).

4.3 Planning and adoption of measures

If any of the requirements set out in Table 1 is not met, suitable corrective measures must be taken to mitigate the possibility of personal harm or environmental or economic damage. Such measures must be carefully planned, which necessarily entails an analysis of both, the cause or causes of the non-compliance observed and of structural reliability. In certain cases, such as where fluctuations in the mean or extreme values of a given indicator are observed to accelerate or follow a trend, it may be advisable to analyse the values recorded even where no non-conformity has been formally established.

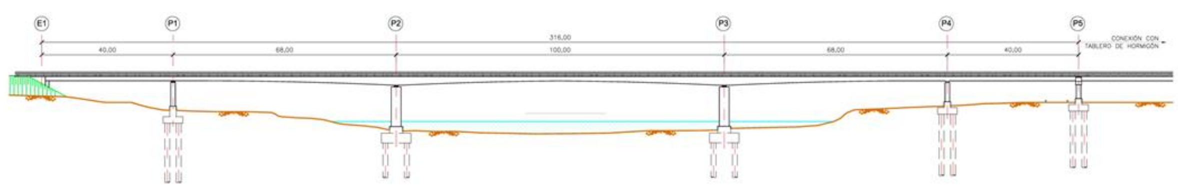
5 Bridge monitoring application

5.1 Introduction

The proposed methodology was applied to a bridge of unknown reliability, located in the province of Seville, Spain. The bridge chosen runs in the east-west direction and crosses the River Guadalquivir. This five-span (40 + 68 + 100 + 68 + 40 m), 316 m long composite bridge features a variable depth (Figure 1a). Its approach viaduct consists of a six-span (4 x 27 + 30 + 22 m), 160 m long continuous, post-tensioned slab which lies outside of the scope of the study.

The bridge deck is 30.1 m wide. Its triple-cell composite box girder varies in plan and elevation view dimensions. The width of the bottom flange of the steel box ranges from 9.2 to 15.4 m while its depth gradually rises from 2.25 at the mid-point in the 100 m span to 4.55 m over piers P-2 and P-3. It is fitted with longitudinal and transverse stiffeners, as well as diaphragms placed at 4 m intervals, in addition to the support diaphragms. The concrete slab is 0.22 m thick with passive (two layers ϕ20 at 0.15) and active (48 tendons 7ϕ0.6" in the sections on piers P-1 and P-4 and 96 tendons in the ones on piers P-2 and P-3) reinforcements. It has pile foundations, confined (Pot-type) neoprene bearings at abutment 1 (E-1), P-1, P-2, P-3 and P-5 and hooped neoprene bearings at P-4.

a)



b) c) d)

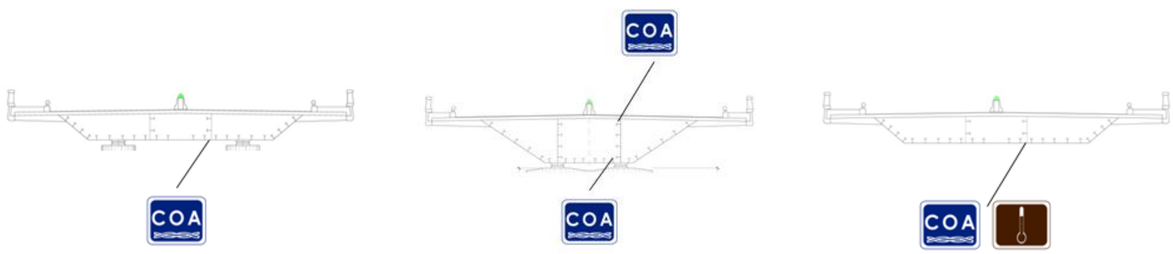


Fig. 1. Investigated bridge and location of sensors: a) longitudinal section; b) cross section P-1; c) cross section P-2; d) main span cross section

5.2 Monitoring system and analytical model

The indicator used to monitor the bridge was unit strain. Given bridge symmetry and taking into account the resolution of the data acquisition system used, only one half of the structure was instrumented. The system chosen to monitor unit strain consisted of 2 m long fibre optic sensors inside steel protective sheaths. These sensors were able to log changes in shape and position to a precision of 0.01 mm for static readings (1 every 10 minutes) and 0.001 mm for dynamic readings (50 per second). The static values logged were the mean of the readings taken in each 10-minute interval.

Three sections were chosen to measure unit strain in the longitudinal direction of the bridge (Figure 1): the two sections with maximum hogging moments over piers P-1 and P-2, which, moreover, concurred with the sections actively reinforced, and the one with maximum sagging moments (mid-point in the 100 m span). Two optical sensors were positioned in the steel cross-section over pier P-2, one on the lower flange and the other on the upper flange (Figure 1c). Pier P-1 (Figure 1b) and the centre of the 100 m span (Figure 1d) were instrumented only on the upper face of the lower flange. The temperature inside the box was recorded in the central cross-section of the main span.

Structural calculations were performed with Civilcad2000 (2000) software. Since the structure rests on deep foundations consisting of piles, analysis of the composite deck alone, i.e., separate from the sub-structure, was deemed sufficient. The construction stages addressed in the analysis of the composite structure included positioning of the steel structure, concrete casting in 5 stages and subsequent cable prestressing in the upper slab, concrete casting on five non-prestressed stretches of the upper slab, concrete casting of the lower slab and prestressing of external post-tensioning. Taking into account that the bridge has been in service for several years when the aforementioned data acquisition system has been installed, the actions studied to assess the structure were self-weight, traffic loads, pedestrians, wind, as well as imposed deformations due to settlement and temperature.

From the stress values obtained in the extreme fibres of the sections studied with the numerical model for the relevant combinations of actions, unit strains were found assuming elastic structural behaviour. As the structure's self-weight, dead loads, prestressing forces and imposed deformations were applied before the installation of the sensors, the established threshold values did not include the respective unit strains (item 4.2.1).

5.3 Comparisons of the findings to threshold values

Data acquisition was available only for 2 months due to budget restrictions. Figure 2 compares the data logged during that time by the sensor positioned on the upper face of the bottom flange in the cross-section over pier P-1 to the respective thresholds. The convention used here was to show compression as negative and tensile strains as positive. Serviceability threshold $E_{\mathrm{ser,lim,1}}$ was not exceeded and $E_{\mathrm{ser,lim,2}}$ was exceeded less than 50% of the time. The readings were also well within structural safety thresholds $\mathrm{E}_{d,\mathrm{lim,1}}$ and $\mathrm{E}_{d,\mathrm{lim,0}}$, which is consistent with the fact that the structure was designed to bear traffic loads characteristic of expressway bridges, whereas its traffic is primarily urban.

6 Final remarks

This paper defines a procedure for monitoring road bridges with a view to early detection of possible damage or anomalies to be able to adopt suitable measures before an undesired event such as structural collapse occurs. It contains proposals for indicators or quantifiable parameters that can provide information on the degree of compliance with serviceability and structural safety requirements. It also discusses recommended threshold values and the mean frequency with which they may be exceeded before risk mitigation measures are required.

The procedure proposed was applied to a road bridge with an unknown reliability level. The monitoring was in place for a relatively short time, but the findings suggest that long-term analysis of the data so acquired could be an improvement for the effectiveness of road bridge inspection and maintenance.

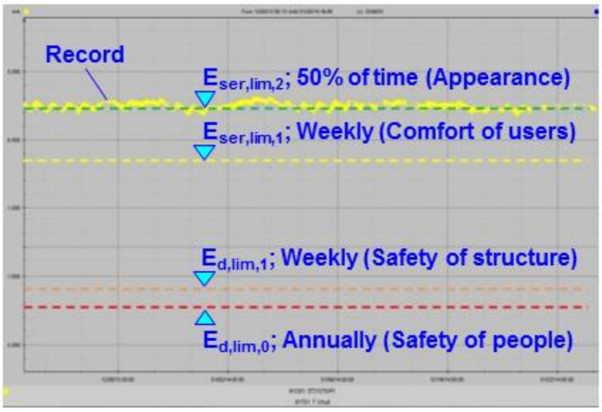


Fig. 2. Results from monitoring with fiber optic sensors on bottom flange of the cross-section over pier P-1 and comparison with threshold values associated with serviceability and structural safety requirements

Acknowledgements

This study was conducted under the SEGUSTRUC project entitled "Investigación y desarrollo en el control de la seguridad estructural con nuevos sensores basados en tecnología de fibra óptica", funded by the Centre for Industrial Technological Development (Spanish initials, CDTI) and awarded to Euroconsult S.A. The collaboration of Álvaro Navareño Rojo, technical adviser to the Deputy Directorate for Conservation of the Spanish Ministry of Internal Development's Directorate General of Roads, and Emilio Asensio García, Chief Officer of Conservation and Operations of the Western Andalusian Department of Roads, enabling the monitoring of the bridges investigated under the SEGUSTRUC project is gratefully acknowledged.

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Bridge scour reliability under changing environmental conditions

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.2

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Abstract. In this paper, a framework is presented for the probabilistic assessment of scour prone bridges considering the potential effects of changing flow characteristics due to climate change. In this methodology, statistical analysis of the expected maximum annual flow is combined with Monte Carlo simulation (MCS) to estimate the probability of scour failure. The uncertainties associated with the different factors which influence scour performance are taken into account using suitable distributions. The influence of climate change on the flow characteristics is considered through changes in the mean and variability (i.e. standard deviation) of the expected maximum annual flow distribution. The methodology is demonstrated through a case study using a bridge in the UK. The results from this investigation revealed that a time-dependent increase in the mean of the expected maximum annual flow can have an adverse effect on local scour performance which is greater in magnitude compared to an increase of its variability (i.e. standard deviation) alone. Amongst the cases examined, however, the most adverse effect on scour performance is observed from the simultaneous increase in both mean and variability of the expected maximum annual flow. The results also highlighted the significance of the uncertainty in foundation depth, commonly present in old bridges, and local scour modelling when estimating scour reliability.

Keywords: bridges, local scour, probabilistic analysis, Monte Carlo simulation, climate change

1 Introduction and Background

Scour is identified as one of most widespread causes of structural failure in bridges spanning over rivers (Wardhana & Hadipriono, 2003). Scour is characterised by the erosion/removal of (underwater) river bed material in the vicinity of the piers which can lead to structural instability if the foundation depth is reached. A number of factors are associated with the scour depths which may potentially develop at the pier and/or abutments of a bridge including, among others, the geometrical characteristics of the pier/abutment, the river characteristics including bed material and angle of attack as well as the flow magnitude at the bridge location. The scour phenomenon has been extensively investigated and a number of – mainly empirical – models are available which allow the quantification of scour depths considering the most significant influencing factors such as pier geometry, river and flow characteristics (Melville & Coleman, 2000). The prediction of scour depths in practice involves significant uncertainty caused by the variability associated with the above parameters as well as the scour prediction models themselves, which have been developed empirically through small-scale laboratory experiments.

A source of uncertainty which is becoming increasingly relevant when predicting future scour performance of bridges is the potential influence of climate change. The increased risk of scour of bridges due to climate change has been recognised worldwide (Transportation Research Board, 2008; DEFRA, 2012). The potential consequences of climate change on bridge scour performance are currently under-researched, on a quantitative basis, and need to be investigated to assist the development and implementation of adaptation strategies (Meyer & Weigel, 2011). Hence, capturing the aforementioned uncertainties in the analysis would allow a more reliable performance assessment of scour prone bridges and assist towards more efficient decision-making under conditions of uncertainty.

An important parameter which directly affects the depth of the developing scour hole is the magnitude of the flow which may potentially be encountered in a river at the bridge location. The flow magnitude in a river is governed by several factors such as catchment characteristics, precipitation patterns, etc. Climate change is predicted to cause changes in the river flow characteristics due to changes in the precipitation patterns and catchment characteristics (Robson, 2002). A number of methods have been devised to predict the potential flow in a particular river, for instance through statistical analysis of historic flow records or alternatively using the rainfall-runoff method which requires knowledge of the precipitation patterns and the catchment descriptors. The statistical analysis of historic flow records through the widely-used WINFAP-FEH 3 software (Robson & Reed, 1999) allows the estimation of the statistical properties of the expected maximum annual flow for any river in the UK.

Alterations in the climatic and weather conditions due to climate change can potentially increase the uncertainty associated with the magnitude as well as the prediction of extreme weather events including extreme precipitation and river flows. The available evidence suggests that these changes may prevail as temporal changes in the statistical properties and distribution of key climatic parameters such as temperature and precipitation (Katz, 1993). Increasing flood frequency and magnitude due to increasing precipitation and/or changes to the catchment characteristics can have a significant effect on the scour performance of bridges. At present, it is difficult to precisely quantify the effect of climate change in terms of precipitation and temperature changes on fluvial flood frequency and magnitude. However, the results of several studies suggest that in some areas flood frequency and magnitude will increase in the future (i.e. occurrence of extreme events will become more frequent).

This paper aims to provide a framework for the scour reliability assessment under changing uncertain flow characteristics due to climate change. In view of the above limitations, an alternative probabilistic approach is suggested in this paper to quantify the effect of potential increase of flood frequency and magnitude due to climate change on the probability of local scour failure. Firstly, the distribution of the expected maximum annual flow is obtained using statistical analysis of existing flow records. Thereafter, gradual changes are introduced to the statistical properties of the flow to account for climate change and Monte Carlo simulation (MCS) is used to estimate the probability of scour failure over time. A case study of a UK bridge is carried out to investigate the potential effects of changing mean and variability of the expected maximum annual flow on the probability of local scour failure in bridge piers to demonstrate the applicability of the proposed framework.

2 Probabilistic Framework

In this paper, local scour is estimated using the HEC-18 design equation (Arneson et al., 2012), given by Eq. (1), which considers scour as a time-independent process, i.e. temporal effects of local scour development are not modelled

$$y_{\max} = 2 y_0 K_1 K_2 K_3 K_4 \left(\frac{D}{y_0} \right)^{0.65} F_0^{0.43} \tag{1}$$

where y_{max} is the maximum scour depth (m), y_0 is the depth (m) of the flow upstream of the bridge pier, K_1, K_2, K_3 and K_4 are coefficients which allow for pier shape, angle of attack, streambed conditions and the river bed material size, D is the pier diameter and F_0 is the Froude number given by

$$F_0 = \frac{V}{(g\, y_0)^{0.5}} \tag{2}$$

where V is flow velocity given by

$$V = \frac{Q}{B y_0} \tag{3}$$

and the flow depth y_0 is given by (BD, 2012)

$$y_0 = \left(\frac{nQ}{B s^{1/2}} \right)^{3/5} \tag{4}$$

where Q is the flow (m^3/s), B is the river width (m), n is the Manning's coefficient and s is the longitudinal slope of the channel. Eq. (4) can be used for wide rivers with ratios B/y_0 exceeding about 10, giving conservative predictions for cases where this is less than about 10 (BD, 2012). In this paper, the flow Q is estimated using the statistical analysis procedures implemented in the software WINFAP-FEH 3 (Robson & Reed, 1999).

In terms of the probabilistic framework for the scour analyses, the performance function for the limit state for local scour of a bridge pier is given by

$$G(t) = D_F - y_{\max}(t) = D_F - \left[2y_0(t) K_1 K_2 K_3 K_4 \left(\frac{D}{y_0(t)} \right)^{0.65} F_0(t)^{0.43} \right] \tag{5}$$

where D_F is the foundation depth. $G(t) \leq 0$ indicates the failure realization of the limit state. Using the statistical properties, including distribution type, for the random variables of Eq. (5) and assuming that the resistance D_F and the load effects $y_{max}(t)$ are statistically independent, the instantaneous (annual) probability of failure, $p_f(t)$, is given by

$$p_f(t) = P[G(t) \leq 0] \tag{6}$$

Eq. (6) is evaluated using Monte Carlo simulation using a sample size of $n = 2\times10^6$ per year. Table 1 shows the random variables and their characteristics. Flow events in different years are assumed to be independent. The cumulative (time-dependent) probability of failure, at any point within a time period, is given by Eq. (7) below, provided that the failures are statistically independent.

$$p_f(0, t_L) = 1 - \prod_{i=1}^{k} \left\{ 1 - P\left[G_{t_i}(t) \leq 0 \right] \right\} \tag{7}$$

The expected maximum annual flow is modelled by fitting a suitable distribution to flood data using the statistical procedures implemented in WINFAP-FEH 3 (Robson & Reed, 1999). This approach is based on the creation and analysis of a pooling group of several catchments of similar hydrological characteristics, with the available years of flow records for each station (in the different catchments) contributing to the total number of station years.

Table 1. Statistical properties of random variables.

	Variable	Mean	CoV	Distr.	Ref.
River	B (m)	65	0.05	Normal	Assumed
	K_3	1.1	0.05	Uniform	NCHRP (2003)
	K_4	1.0	-	-	-
	s	0.0032	0.05	Lognormal	Assumed
	n	0.035	0.28	Lognormal	NCHRP (2003)
Bridge piers	D_F (m)	4.5	-	-	Assumed
	K_1	1.0	-	-	Assumed
	K_2	1.0	-	-	Assumed
	D (m)	2	0.05	Normal	Assumed

3 Bridge Case Study

The bridge considered in this case study is assumed to be located on river Earn in Scotland, UK, assuming alluvial riverbed conditions. WINFAP-FEH 3 provides a number of options for estimating QMED, which is the maximum annual flow with a return period T = 2 years), for instance using the catchment descriptors or annual maxima (AM) series (for more details see Robson and Reed 1999). In this paper, a QMED of 250.2 m^3/s is estimated from annual maxima series of the station. Analysis of pooling group flood data (i.e. annual maximum series) using the WINFAP-FEH 3 software indicates that the generalized extreme value (GEV) distribution, given by Eq. (8) (Kottegoda & Rosso, 1997), is the most suitable distribution for modelling the magnitude of the expected maximum annual flow; its cumulative distribution function is expressed as follows

$$F_{X\,\max} = \exp\left\{ -\left[1 - \frac{k(x-\varepsilon)}{\alpha} \right]^{1/k} \right\} \tag{8}$$

where k is the shape parameter, ε is the location parameter and α is the scale parameter. The GEV distribution parameters obtained from the statistical analysis of stations in similar catchments (i.e. pooling group) in WINFAP are $\alpha = 0.222$, $\varepsilon = 0.919$ and $k = 0.002$. The scale and location parameters of the GEV distribution are given by Eqs. (9) and (10), respectively (Kottegoda & Rosso, 1997)

$$\alpha = \sqrt{\frac{k^2\sigma^2}{\Gamma(1+2k)-\Gamma^2(1+k)}} \tag{9}$$

$$\varepsilon = \mu - \frac{\alpha}{k}\left[1-\Gamma(1+k)\right] \tag{10}$$

where μ is the sample mean, σ is the sample standard deviation and k, ε and α are the shape, location and scale parameters of the generalized extreme value distribution, respectively (Kottegoda & Rosso, 1997). Γ is the gamma function.

The potential effects of climate change on scour are examined through a parametric study in which the scale and location parameters are gradually changed for increasing values in the mean μ and variability (i.e. standard deviation σ) of the flood magnitude in Eqs. (9) and (10). The different analysis cases examined in relation to changing flow conditions are follows: 20%, 40% and 60% increase in the mean value; 20%, 40% and 60% increase in the standard deviation (variability) and 20%, 40% and 60% simultaneous increase in both. These changes are assumed to evolve linearly with time over a 60 year period. In this way, the effects of climate change on the precipitation patterns and catchment descriptors and hence on the flood frequency and magnitude are implicitly considered in the analysis of the different scenarios examined. A foundation depth (FD) of 4.5m is assumed to facilitate the estimation of the scour failure probability.

4 Results and Discussion

Fig. 1 (left) shows the effect of increasing the mean in the expected maximum annual flow on the annual and time-dependent probabilities of failure for the scenarios with mean increases of 20% (45M2000), 40% (45M4000) and 60% (45M6000) over a 60 year period. The results in this figure indicate that the effect of increasing mean is relatively small for the initial 10 years and it gradually becomes noticeable between 10 and 20 years and significant beyond the initial 20 year period. At the end of the examined period the time-dependent probabilities of failure for the scenarios with 20%, 40% and 60% increase in mean are predicted to be approximately 60%, 175% and 340%, respectively, higher than the base line scenario (45M0000) which assumes no changes in the statistical properties of the expected maximum annual flow over time.

Fig. 1 (right) shows the effect of increasing variability (standard deviation) in the expected maximum annual flow on the annual and time-dependent probabilities of failure for the scenarios with variability increases of 20% (45M0020), 40% (45M0040) and 60% (45M0060) over a 60 year period. The results in this figure indicate that the effect of increasing variability is relatively small for the initial 15 years and it gradually becomes significant beyond the initial 20 year period. For the analysis cases examined, the effect of increasing variability in the expected maximum annual flow is predicted to have a relatively smaller effect on the failure probabilities compared to the previous case of assuming a gradually increasing mean. The time-dependent failure probabilities at the end of the 60 year period are predicted to increase by approximately 40%, 90% and 150% for cases 45M0020, 45M4040 and 45M0060, respectively relative to the base line case (45M0000).

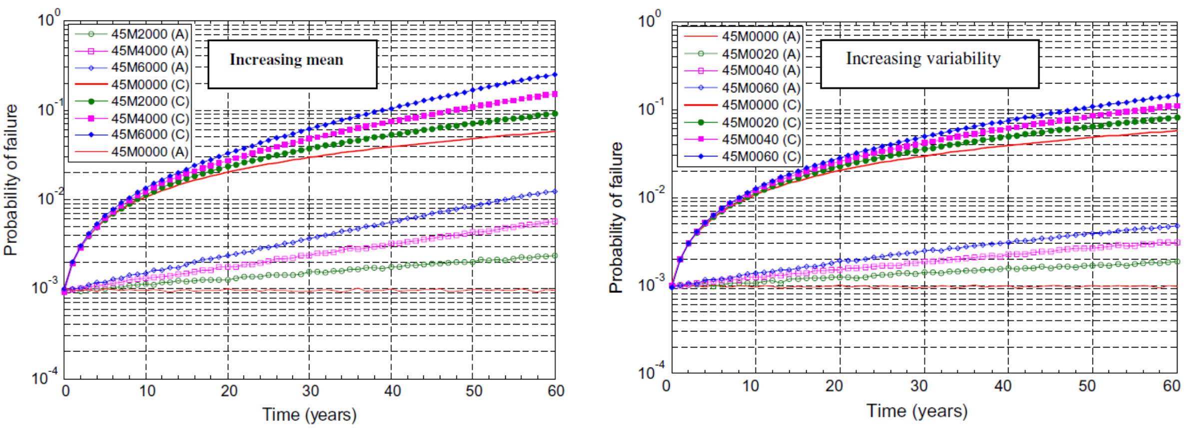

Fig. 1. Effect of increasing mean (left) and variability (standard deviation) (right) in the expected maximum annual flow on the annual (A) and cumulative time-dependent (C) probabilities of scour failure.

Fig. 2(left) shows the cumulative probabilities of scour failure for the previous analysis cases for different foundation depths, i.e. 3.5m, 4m and 5m. As expected, the results in this figure clearly show the influence of foundation depth (D_F) on the predicted time-dependent p_f; with smaller foundation depths having higher probabilities of failure. For the case of FD = 3.5m the effect of changes in the statistical properties of the flow (i.e. increasing mean or/and variability) have an insignificant effect on the time-dependent probabilities of scour failure. The results further indicate that the effect of increasing mean and/or variability is not constant when different foundations depths are considered. For example, for the case of FD = 5m, the increasing variability of the expected maximum annual flow has a greater effect than the increasing mean. Conversely, the increasing mean of the expected maximum annual flow distribution has a greater effect compared to the effect of increasing variability for decreasing foundation depths. As shown in Fig. 2(left), the simultaneous increase of mean and variability of the flow distribution produces the highest probabilities of failure in all cases examined. These results indicate that foundation depth has a significant effect on the predictions. In practice, this variable is deemed with high uncertainty while in many cases no data is available on the actual foundation type and depth of a particular bridge (JBA, 2004). In such cases, conservative values of FD are recommended in assessing scour performance (JBA, 2004). To this end, the systematic collection of actual foundation depth measurements of piers in scour prone bridges would reduce the uncertainty and hence improve the accuracy of the scour failure predictions during assessments.

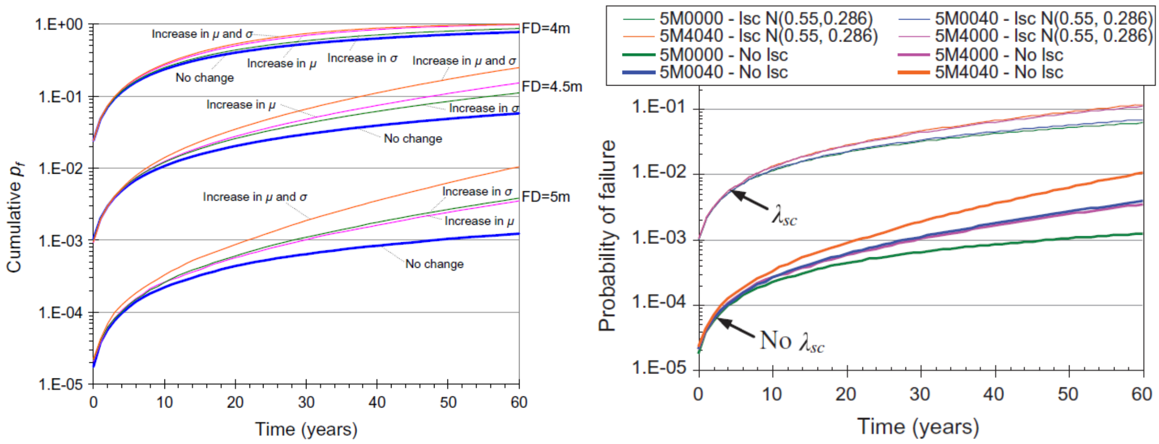

Fig. 2. (left) Effect of foundation depth on time-dependent probability of scour failure; (right) effect of model parameter λ_{sc} on the time-dependent probabilities of local scour failure for up to 40% increase in the mean or/and variability of the expected maximum annual flow distribution for a foundation depth = 5 m.

It has been shown that the scour equation (Eq. (1)) of HEC-18 leads to conservative predictions of local scour (NCHRP, 2003). This is due to the fact that the scour prediction models used in codes of practice have been developed through small-scale laboratory experiments. Comparisons of these prediction models with field measurements of scour depths on real bridges have shown that there is a discrepancy between them. It has been suggested that a model parameter λ_{sc} can be introduced in Eq. (1) to reduce its conservatism (NCHRP, 2003) inherently taking into account epistemic uncertainty. The local scour model is now given by Eq. (11):

$$y_{\max}(t) = 2\lambda_{sc} y_0(t) K_1 K_2 K_3 K_4 \left(\frac{D}{y_0(t)} \right)^{0.65} F_0(t)^{0.43} \tag{11}$$

Several values have been proposed for the statistical properties of λ_{sc}; for a summary see (NCHRP, 2003). To investigate the influence of this parameter a number of scenarios are considered using values for the statistical properties of λ_{sc} suggested in (NCHRP, 2003). The analysis cases in this section assume a 40% gradual increase in mean, standard deviation and the simultaneous increase of both for the expected maximum annual flow considering a foundation depths of 5m. The model parameter λ_{sc} is modelled using a normal distribution with mean = 0.55 and COV = 52% (NCHRP, 2003). In this way comparisons can be made with results presented in previous sections.

Fig. 2(right) shows the predicted time-dependent failure probabilities for the analysis case with foundation depth of 5m. The results in these figures indicate that although the mean value of λ_{sc} is less than 1 (i.e. mean value of

λ_{sc} = 0.55), the introduction of this model parameter causes an increase of the predicted time-dependent failure probabilities due to the high variability associated with model parameter λ_{sc} resulting in an increase of the scour depth values exceeding 5m (generated using MCS in MATLAB). As expected, the mean value of the predicted scour depths is lower than the predicted scour depths when λ_{sc} = 1. The results in Fig. 2(right) also indicate that the very high variability of the model parameter λ_{sc} (COV = 0.52) overshadows the effect of the assumed increasing variability in the expected maximum annual flow on the predicted time-dependent probabilities of failure (i.e. the increasing variability of flow is predicted to have an insignificant effect on cumulative p_f).

5 Conclusions

In this paper a probabilistic methodology was presented for the reliability analysis of local scour in bridge piers, considering the potential effects of climate change through changes in the distribution of the expected maximum annual flow. The procedure was demonstrated though a case study using a bridge located in the UK in which a number of analysis cases were considered to investigate the potential effect of changing flow characteristics on the probability of scour failure. It was found that the effects of gradually increasing mean or variability of the expected maximum annual flow on the predicted probabilities of scour failure were found to be relatively small for the initial 10-15 years. Beyond this initial period their effect on the predicted probabilities of failure becomes significant, with the case of simultaneous increase in mean and variability of the flow having the greatest impact on the predictions. The foundation depth and scour modelling uncertainty was found to have a significant effect on scour failure probability. The results of the case study presented in this paper indicate that the effects of changing flow characteristics on the scour failure probabilities are predicted to reduce with reducing foundation depths. More specifically, when considering the effect of foundation depth in conjunction with changing flow characteristics on the scour failure probability, the results showed that the effects of increasing mean and/or variability of the maximum expected annual flow was more significant in the cases with deeper foundation depths are assumed in the analysis.

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BRIDGE SMS: Intelligent bridge maintenance and management system

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.3

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Abstract. An intelligent system for bridge inspection and management requires a knowledge and appreciation of structural engineering, geotechnics, hydraulics, hydrology, materials and transport management. This study introduces BRIDGE SMS, an EU/FP7 project, which couples state-of-the art scientific knowledge in hydrology, river and structural engineering with industrial knowledge in infrastructure management and web-based bridge management. This involves the application of monitoring systems for the assessment and management of the structural and hydraulic vulnerability of infrastructure assets over waterways in an effort to develop an open-source cloud-based intelligent Decision Support System. BRIDGE SMS aims to deliver procedures for complete bridge inspections, through scour and structural inspections, and to develop a reliable decision support tool which would efficiently manage bridge failure risks in a cost-effective way.

Keywords: bridge monitoring, risk assessment, flooding, scour risk, infrastructure management

1 Introduction

Government agencies, the public and private sectors and professional engineering organizations across Europe need to come together and proactively meet the challenge of creating climate resilient infrastructures (Engineering the Future, 2011). However, water-related hazards associated with extreme precipitation events pose a major threat for the resilience of civil infrastructure over watercourses. Of such hazards, scour is the most complex and challenging water flow and erosion phenomena, leading to structural instability and ultimately catastrophic failures. Scour action is the main cause of bridge failures worldwide (Wardhana, 2003) and has also been identified as highly disruptive and critical in the UK due to its destructive consequences on civil infrastructure systems (Highways Agency, 2009). Such issues are not only confined to onshore locations, as scour and erosion are also considered one of the main complications in the design and operation of offshore infrastructure (Michalis et al., 2013). Despite more resources being invested in protecting structures from water-related hazards, future projections indicate that the frequency of extreme flooding across Europe is anticipated to double by 2050 with severe consequences on infrastructure assets (Jongman et al., 2015).

The evaluation of the hydraulic and structural vulnerability (Znidaric et al., 2011) of civil assets over watercourses is a critical issue taking into account the deterioration of the structure as well as the cost and technical and logistical issues arising from rehabilitation or possible replacement of bridges (Michalis et al., 2012; Weninger-Vycudil et al., 2015). One of the main disadvantages of current standards and inspection practices is that, in general, they do not reveal actual riverbed level variations around the foundations of structures due to several issues associated mainly with the actual procedure and frequency of inspections (Michalis et al., 2015). Even though water and flow characteristics around bridge piers have been thoroughly investigated both experimentally and numerically, there is still a lack of a tool that can offer fast and reliable predictions (Valyrakis et al., 2015). Therefore, the implementation of a real-time decision support tool and condition monitoring platforms is a highly attractive option required to efficiently manage bridge infrastructure. The importance of a multidisciplinary approach to achieve an effective bridge management system (BMS) through the continual inspection, assessment and maintenance of bridges cannot be overstated. The nature of current BMS is that, most often, emphasis is placed on a single discipline, e.g. structural, and other elements such as hydrologic and scour effects do not receive the attention that is warranted, which can have catastrophic implications on the health of bridge stock, costs and public health.

BRIDGE SMS[1] addressed such issues by coupling state-of-the art scientific knowledge in hydrology, river and structural engineering with industrial knowledge in infrastructure management and web-based bridge management systems. The project proposes to develop an open-source cloud-based intelligent Decision Support System focusing on, but not limited to, the assessment and management structural and hydraulic vulnerability of bridges over waterways.

The BRIDGE SMS project is built on industry-academia collaborations between University College Cork (UCC), Faculty of Civil Engineering, University of Zagreb (UNIZAG), Cork County Council (CCC), Infraestruturas de Portugal (INFPO) and ARCTIS, a software development company. The BRIDGE SMS project has also established collaboration with the University of Minho (UNIMIN) in Portugal, South Dublin City Council (SDCC), Dublin City Council (DCC) and University College Dublin (UCD) in Ireland, all of which have pledged their support for this project. Collaboration between UCC and UNIZAG evolved as a result of a major railway bridge collapse at Malahide viaduct, the main railway line from Dublin to Belfast, in August 2009 with a passenger service train involved in the incident (McKeogh & Bekic, 2010a, 2010b, 2010c). UCC and UNIZAG were also involved in the inspection and assessment of scour risk on more than 100 railway bridges in Ireland (Bekic et al., 2012). Recently, a collaboration of some of the authors in UCD, UCC and UMinho, as a part of COST Action TU1406, has analysed thousands of bridge inspections in Cork County, Ireland and of Portugal at a network level (Hanley et al., 2016).

2 Some considerations on the existing BMS systems

2.1 Existing Bridge Management Systems

Historically, the majority of bridge management systems focuses mainly on structural issues without adequate emphasis on bridge scour risk. An overview of existing bridge management systems, published by the IABMAS Bridge Management Committee assessed a total of 25 Bridge Management Systems (BMS), in operation across 18 countries that are being used to manage 1,000,000 assets (Mirzaei et al., 2014). The report indicated that all BMS show a strong focus on structural health monitoring of bridge structures, managing this aspect of bridge stability to varying degrees.

The IABMAS report (Mirzaei et al., 2014) also highlighted that while the BMS are strikingly similar in their overall approach and operation, there was a lack of standardisation, which meant that systems could not be easily adopted by other agencies. For instance, all BMS, except PONTIS (now AASHTOWare) are used only within the country in which they were developed. The report (p.46) concluded that *"a certain level of standardisation could potentially enhance the exchange of knowledge and experience between managing agents, and improve the usefulness of management systems."*

One of the key BRIDGE SMS objectives is to provide standardised methods for the assessment of bridge scour risk, which would enable transferability of the bridge management system to different regions and countries. One example of simplifying the standardisation is the introduction of common input data sources (e.g. forecasts, soil moisture, etc.). As a partner of the European Flood Awareness System (EFAS), BRIDGE SMS gives an opportunity to implement common procedures among different organisations.

2.2 Existing standards for assessment of infrastructure

Extensive research has been carried out into existing standards worldwide, such as the US DOT, 2001a; US DOT, 2001b; US DOT, 2009; US DOT, 1988; US DOA, 1998; British Railway Board, 1993; The Highways Agency, 2012 in addition to the consideration of other proposed scour risk assessment methods. Such methods include:

a) an assessment based on stability for a stable reference reach and then the departure from stable conditions on an unstable reach of the same stream type (Rogsen, 2001),

[1] 'Intelligent Bridge Assessment Maintenance and Management System', http://www.bridgesms.eu/

b) a diagnostic approach in which the system and its variables are defined, and an evaluation is carried out to assess the causal mechanisms producing the current condition (Montgomery & MacDonald, 2002), or
c) a simple and brief stability assessment based on sound indicators, supported by photographs and by walking a certain distance upstream and downstream of the project reach (Johnson, 1999).

The existing standards and policies for the evaluation of bridge scour risk have been developed mainly by the US and UK agencies. In the US, three documents on the assessment and management of bridge scour risk have been published (US DOT, 2001a; US DOT, 2001b; US DOT, 2009) together with its own scour assessment programme by the US Department of Agriculture, Forest Service (1998). In the UK, two standards for the assessment of bridge scour risk were published by two agencies: the Railtrack method by British Rail (British Railway Board, 1993) and the Design Manual BA 74/06 by the UK Highways Agency (Highways Agency, 2006) which was replaced by BD 97/12 (Highways Agency, 2012). Other published standards and manuals of EU countries mainly consider the analysis of scour process, the stability of structures in the water and the scour protection measures.

In the study by Bekic et al. (2012) two methods for assessment of bridge scour risk were compared for an identical bridge network, namely the Colorado method (U.S. Department of Agriculture, 1998) and modified BA 74/06 method (Highways Agency, 2006). The comparison of two methods on 100 railway bridges showed that the resulting scour rankings differ for 20% of bridges with a significant difference of ranking on 10% of cases. The study also highlighted some drawbacks of the existing evaluation of bridge scour risk. It was identified in several studies (Johnson, 2005; Yanmaz et al., 2007; Bekic et al., 2012) that improvements of the bridge scour assessment methodology could lead to more confident ratings of scour risk and its management.

3 BRIDGE SMS Methodology and Approach

BRIDGE SMS will deliver bridge inspection and bridge management methodologies, guidelines and a decision support tool supported by a software platform that empowers engineers and key personnel to predict, identify and prepare for potentially destructive flood events. This will lead to lower maintenance/planning costs and more secured bridge management/operation improving public safety and reducing costs for bridge owners, insurers and maintainers. The proposed system should provide efficient infrastructure management which includes the following:

- decision making, maintenance and mitigation;
- multi-level prioritization list of all structures (bridges);
- inventory about the single structure (name and ID, road / railway line, location, structure type, year of construction, directions on bridge location);
- priority and current status (recommendations based on real-time data from monitoring systems);
- proposed short and medium term works and maintenance;
- easy access to all documents about the structure history and future plans (bridge inspections, comments, reports, pictures, maintenance, construction works, etc.).

The BRIDGE SMS key goals are as follows:

1. Development of standardised methods for bridge scour and structural inspections and bridge management customised to industrial needs.
2. Risk assessment for efficient management of the potential effects of flood events.
3. Development of a database framework which is designed for intuitive use, encouraging participation by personnel at all levels within management authorities.
4. Development and application of low-cost structural health monitoring systems which can deliver real-time key information regarding the impact of water-related hazards at hydraulic structures.
5. Development of a system that:
    a. Collects, integrates and processes real-time data at regular intervals from weather and hydrologic sources, meters and gauges, and other sensing devices.
    b. will rapidly notify based on in-built intelligence and decision-making processes, relevant personnel of possible maintenance and failure issues.
    c. will advise in relation to current Scour Risk at bridge structures and prompt an appropriate Plan of Action (PoA) which may involve various levels of maintenance and repair.

   d. which will be capable of prioritizing and optimizing the operational and maintenance budget costs for infrastructure companies.

6. Maximise the use of new Information and Communications Technology (ICT) hardware such as tablets and cloud-based systems for on-site rapid communications, etc.

As part of the overall development process, a BRIDGE SMS pilot study is underway on the Bandon catchment area in Cork County (Ireland) where bridge scour and flooding issues have been reported over the past decades. It is planned that methodologies and tools developed within BRIDGE SMS project will applied to streams in different geographic regions: (1) region with a history of scour problems based on information from on-site engineers (CCC and INFPO) and a (2) region where the waterways are stable and without scour issues but with potential structural issues.

3.1 Risk assessment to efficiently manage the potential effects of hydraulic events

The vulnerability of bridges to failure is generally influenced by two basic factors, the degree of stress or degradation that a bridge can safely withstand and the corresponding severity of the hazardous event required to induce this degree of stress or degradation. Components of the risk determination will involve the product of the estimated probability of failure (including hydrological, hydraulic and geomorphological factors) and the total cost of failure (bridge replacement, loss of life).

The current EU practices on the selection of scour risk management measures include the deterministic and the probabilistic approaches. Deterministic approaches are developed around the risk matrix (Federal Office for Water Management, Switzerland) or the fault tree method (Hoffmans & Verheij, 1997; Pilarczyk, 1995; Pilarczyk, 1998). In the probabilistic approaches, the risk is evaluated by the probability of bridge failure due an extreme flood event.

BRIDGE SMS aims to effectively assess risks and direct personnel in a more efficient manner with improved inspection standards and the application of technology to accelerate inspections using an intelligent database system to prioritise maintenance tasks. An intuitive, accessible database for cataloguing all available bridge data from multiple experts and sources will make pertinent information easily retrievable as BRIDGE SMS software will provide an automated way of assessing the individual and cumulative risks to the bridge structure. This will also enable timely manner interventions at vulnerable structures and an improved bridge safety and reliability.

The system will allow the integration of external data for efficient decision-making while at the same time adding value to existing data collection services, such as meteorological stations and water level gauges. Collaborations with public and private organisations in Ireland was established (Office of Public Works, Met Éireann, EPA, Waterways Ireland and local authorities) to incorporate meteorological data and hydrological data. Prototype instrumentation viz. a Weather Information Logging Device (WILD) and a Bridge Information Recording Device (BIRD) were developed by the project which will incorporate various sensing technologies applied to pilot bridges to assess their condition (Michalis et al, 2017).

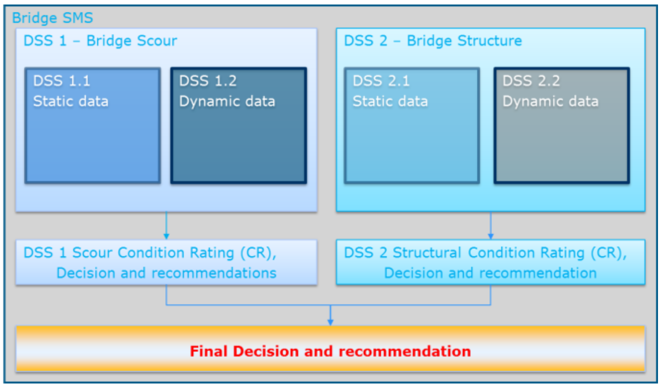

Fig. 1. BRIDGE SMS Decision Support System (DSS)

The Risk Analysis method will form an integral part of the BRIDGE SMS system. The risk of failure will be evaluated through a probabilistic approach. The continuous feedback nature of the BRIDGE SMS system will allow optimisation of risk indices based on real-time monitoring complemented by the catalogued data on assets and historic events.

In order to effectively manage the bridge assets, the project also focuses on preparing a robust cloud-based system consisting of two Decision Support System modules (Structural DSS and Scour DSS). Each module operates independently providing an output based on risk indices with the final decision and recommendation deriving from both modules (Fig. 1).

3.2 Real-time platform for effective bridge management

Developing the system as a cloud-based platform with mobile and tablet applications reduces platform limitations frequently associated with engineering software. The interface would provide GIS data on the bridges in the BRIDGE SMS database delivering critical real-time information obtained from the BMS, in addition to decisions and recommendations about the condition of the bridge (Fig. 2).

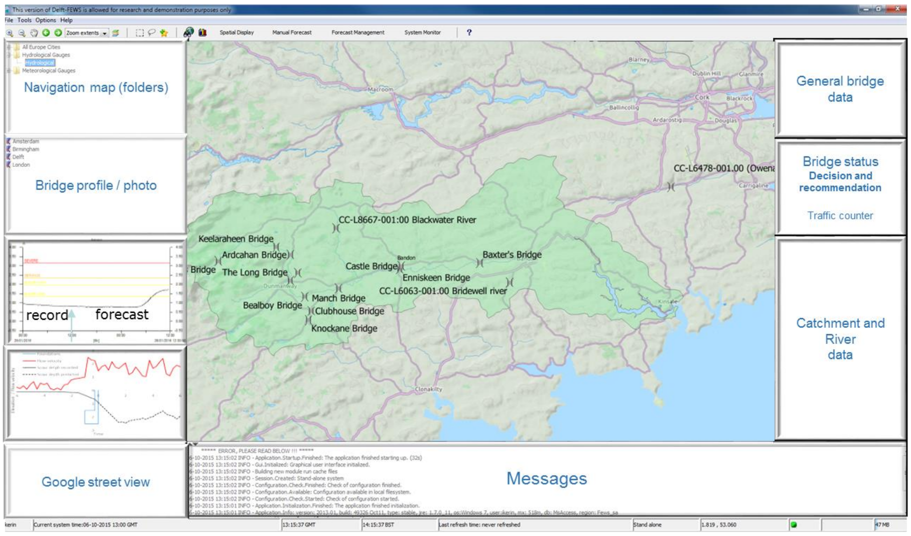

Fig. 2. BRIDGE SMS software interface

The BRIDGE SMS will develop standardised methods for both bridge structural and scour assessment. The BRIDGE SMS platform will also encourage knowledge sharing between agencies, and foster research beyond the specific functionality of a bridge management system. The system will be tested on several networks in collaboration with CCC/INFPO personnel ensuring their relevance and practicality focusing on:

- reaction to events in specific catchment;
- notifications of personnel and improvement of scour management issues.

4 Conclusions

Transportation assets represent a critical component of society's infrastructure systems. Flood induced scour is the leading cause of bridge failures worldwide and one of the main climate change impacts on highway and railway infrastructure. The scour process is considered as one of the most complex and challenging water flow and erosion phenomena, leading to a drastic reduction in the safe capacity and stability of structures over water. A large number of bridge structures under a single management unit highlight the need for self-informing system for efficient management of bridge structural, scour and flooding risks. The bridge inspection and management systems are required to integrate knowledge and understanding from multiple fields: hydraulics, hydrology, structural engineering, geotechnics and infrastructure management. However, an absence of similar methodologies for many aspects of bridge inspections and maintenance is identified as one of the major issue of the existing bridge management systems. Limited possibilities for inter-comparison of methods and results also limit the potential for further development and testing of various standards in Europe and worldwide.

The BRIDGE SMS EU/FP7 project aims to couple state-of-the art scientific expertise in multidisciplinary engineering sectors with industrial knowledge in infrastructure management. The project will deliver guidelines for complete inspections of bridges over waterways and develop a decision support tool to efficiently manage and prevent flood-related structural failures. This is assisted by the development of standards that could be easily adopted by asset managing organisations, the application of low-cost structural monitoring systems and the development of a software platform that aims to provide key information for cost-effective and timely decisions about the condition of infrastructure.

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Acknowledgements

The authors wish to acknowledge the financial support of the European Commission, through the Marie-Curie Marie Curie Industry-Academia Partnership and Pathways Network BRIDGE SMS – “Intelligent Bridge Assessment Maintenance and Management System” (BRIDGE SMS) (FP7-People-2013-IAPP- 612517).

Non Gaussian multi-parameter Bayesian estimation through the mechanical equivalent of logical inference

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.4

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Abstract. Structural Health Monitoring requires engineers to understand the state of a structure from its observed response. When this information is uncertain, Bayesian probability theory provides a consistent framework for making inferences. However, structural engineers are often unenthusiastic about Bayesian logic, finding its application complicated and onerous, and prefer to make inference using heuristics. Here, we propose a quantitative method for logical inference based on a formal analogy between linear elastic mechanics and Bayesian inference with linear Gaussian variables. To start, we investigate the case of single parameter estimation, where the analogy is stated as follows: the value of the parameter is represented by the position of a cursor bar with one degree of freedom; uncertain pieces of information on the parameter are modelled as linear elastic springs in series or parallel, connected to the bar and each with stiffness equal to its accuracy; the posterior mean value and the accuracy of the parameter correspond respectively to the position of the bar in equilibrium and to the resulting stiffness of the mechanical system composed of the bar and the set of springs. Similarly, a multi-parameter estimation problem is reproduced by a mechanical system with as many degrees of freedom as the number of unknown parameters. In this case, the inverse covariance matrix of the parameters corresponds to the Hessian of the potential energy, while the posterior mean values of the parameters coincide with the equilibrium – or minimum potential energy – position of the mechanical system. We use the mechanical analogy to estimate, in the Bayesian sense, the drift of elongation of a bridge cable-stay undergoing continuous monitoring. We demonstrate how we can solve this in the same way as any other linear Bayesian inference problem, by simply expressing the potential energy of the equivalent mechanical system, with a few trivial algebraic steps and with the same methods of structural mechanics. We finally discuss the extension of the method to non-Gaussian estimation problems.

Keywords: decision making, value information, mechanical equivalent, Bayes' theorem, decision parameter estimation

1 Introduction

Structural engineers usually have a solid background in mechanics, yet not always a good relationship with probability theory. In most cases, this is not that critical because code-based design is practically probability-free, with serious probabilistic analysis typically being confined to the most recondite annexes of the codes (EN 1990:2002). It is different for those engineers who grapple with structural health monitoring (SHM), an activity where the objective is to estimate the state of a structure from an uncertain batch of observations provided by different kind of sensors, such as strain gauge (Zonta, Lanaro & Zanon, 2003), fiber optic sensor (Inaudi & Glisic, 2006) and accelerometers (Zonta, Wu, Pozzi et al., 2010). A consistent framework for making inferences from uncertain information is Bayesian probability theory (Sohn & Law, 2000) (Bruschetta, Zonta, Cappello et al., 2013). Yet structural engineers are often unenthusiastic about Bayesian formal logic, finding its application complicated and burdensome, and they prefer to make inference by using heuristics. In this contribution, we wish to help structural engineers reconcile with probabilistic logic (Taynes, 2003) by

suggesting a quantitative method for logical inference based on a formal analogy between mechanics and Bayesian probability. We will state the fundamentals of the analogy in the next section.
To start, we will limit the analogy to the case of linear Gaussian single-parameter estimation, which corresponds in the mechanical counterpart to mere linear elastic single-degree-of-freedom analysis: a cakewalk for structural engineers. In section 3, we apply this formal analogy to a classical inference problem: the estimation of the deformation of a cable belonging to a cable-stayed bridge, characterized by two independent parameters. We will carry out the simple problem of linear regression by solving the equivalent mechanical system of springs.

2 Formulation of the analogy for a single parameter

In this section, we refer to the problem of logical inference of a single parameter based on uncertain information (Cappello, Bolognani & Zonta, 2015). The goal is to estimate a parameter θ based on a set of uncertain information y_i. Further assumptions are that all the uncertain quantities have Gaussian distribution, and that the relationship between information and parameter is linear. When the problem is linear and Gaussian, in principle we can solve any logical inference problem using the following two fundamental rules.

First inference rule or inverse-variance weighting rule (Ku, 1966). Given a set of n observations y_i of variance σ_i^2, the inverse of the variance σ_θ^2 of the parameter is the sum of the inverse-variances of the observations, and the expected value of the parameter μ_θ is the inverse-variance weighted sum of the observations:

$$\frac{1}{\sigma_\theta^2}=\sum_{i=1}^{n}\frac{1}{\sigma_i^2}\ ,\ \ \mu_\theta=\frac{\sum_{i=1}^{n}\frac{y_i}{\sigma_i^2}}{\sum_{i=1}^{n}\frac{1}{\sigma_i^2}}\ . \tag{1a,b}$$

Second inference rule or linear propagation of uncertainties (Rabinovich, 2005) (Kirkup & Frenkel, 2006). The indirect measurement $y = x_1 + \ldots + x_m$, being the sum of m different arguments x_j of variance σ_j^2, the variance of the observations is the sum of the variance of the arguments and the mean value of the indirect observation is the sum of the arguments:

$$\sigma_y^2=\sum_{j=1}^{m}\sigma_j^2\ ,\ \ \mu_y=\sum_{j=1}^{m}x_j\ . \tag{2a,b}$$

Before proceeding it is also convenient, primarily to lighten notation, to introduce the quantity

$$w=\sigma^{-2}=\frac{1}{\sigma^2}\ . \tag{3}$$

The quantity w is compatible with the official definition of accuracy (ISO5725-6:1994, 2012) and the word itself intuitively connects to the practical meaning of w: the higher the accuracy w of an observation is, the more accurate our knowledge about the parameter becomes. Therefore, in the rest of the paper we will refer to the inverse-variance w simply as accuracy. Based on that, we can reword and reformulate the two basic inference rules.

First inference rule. Given a set of n observations y_i with accuracy w_i, the accuracy w_θ of the parameter estimation is the sum of the accuracy of the observations, and the mean value of the parameter μ_θ is the sum of the observations weighted with their accuracy:

$$w_\theta=\sum_{i=1}^{n}w_i\ ,\ \ \mu_\theta=\frac{\sum_{i=1}^{n}y_i w_i}{w_\theta}\ . \tag{4a,b}$$

Second inference rule. The indirect measurement $y = x_1 + \ldots + x_m$ being the sum of m different arguments x_j of accuracy w_j, the inverse-accuracy of the observation is the sum of the inverse-accuracy of the arguments and the mean value of the indirect observation is the sum of the arguments:

$$\frac{1}{w_y} = \sum_{j=1}^{m} \frac{1}{w_j}, \quad \mu_y = \sum_{j=1}^{m} x_j . \tag{5a,b}$$

At this point, it is not difficult for a structural engineer to spot in (4a) the same form of the expression that provides the stiffness of a set of springs in parallel; and similarly, (5a) reminds of the stiffness expression of a set of springs in series. This opens a door to set an analogy between the world of logic and the world of mechanics. Particularly, the analogy statements (Cappello, Bolognani & Zonta, 2015) are summarized in Table 1, while Figure 1 shows the mechanical representation of simple linear Gaussian inference problems.

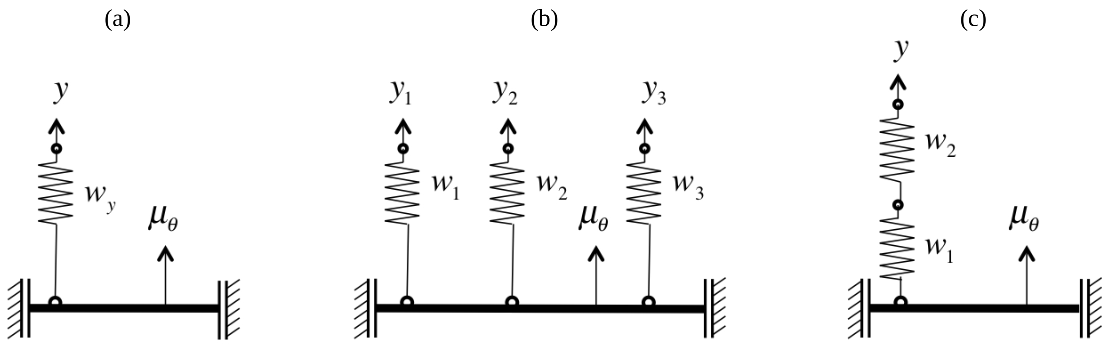

Fig. 1. Mechanical analogy of simple linear Gaussian inference problems: parameter estimation based on one observation (a), three uncorrelated observations (b), one observation affected by two uncorrelated sources of uncertainty (c).

Table 1. Analogy between inference and mechanical models

Symbol	Logical meaning	Mechanical meaning
w, σ^{-2}	accuracy, inverse-variance	stiffness
σ^2	variance	flexibility
y	observation	pre-stretch
μ	expected value	equilibrium displacement

3 Extension of the analogy to N parameters

Now, we analyse a generic inference problem with N unknown parameters to estimate, represented by the vector $\boldsymbol{\theta} = (\theta_1,....,\theta_N)^{\mathrm{T}}$. We imagine that each parameter is characterized by a prior mean value μ_{θ_i} and a prior standard deviation σ_{θ_i}; the latter is linked by the equation $w_{\theta_i} = \sigma_{\theta_i}^{-2}$ to the ith accuracy, which in our mechanical analogy represents the stiffness of the spring associated to each single parameter. The multivariate Gaussian distribution (Bishop, 2006), linked to the N-dimensional vector θ, takes the form:

$$\mathrm{N}(\boldsymbol{\mu}, \Sigma; \boldsymbol{\theta}) = \frac{1}{(2\pi)^{\frac{N}{2}}} \frac{1}{|\Sigma|^{\frac{1}{2}}} \mathrm{e}^{\left\{-\frac{1}{2}(\boldsymbol{\theta}-\boldsymbol{\mu})^T \Sigma^{-1} (\boldsymbol{\theta}-\boldsymbol{\mu})\right\}}, \tag{6}$$

where $\boldsymbol{\mu}$ is the N-dimensional mean vector, containing the N values μ_{θ_i} associated to each parameter, Σ is the NxN covariance matrix, and $|\Sigma|$ denotes the determinant of Σ.

We can notice that the exponent is characterized by a quadratic form that corresponds to the potential energy $\mathrm{E_p}(\boldsymbol{\theta})$ of a mechanical system with N degrees of freedom, related to the inference problem in question. It takes the following mathematical form:

$$\mathrm{E_p}(\boldsymbol{\theta}) = -\ln(\mathrm{N}(\boldsymbol{\mu},\Sigma;\boldsymbol{\theta})) = \frac{1}{2}(\boldsymbol{\mu}-\boldsymbol{\theta})^{\mathrm{T}}\Sigma^{-1}(\boldsymbol{\mu}-\boldsymbol{\theta}). \tag{7}$$

Here, we name the inverse of the covariance matrix $\boldsymbol{\Lambda} = \Sigma^{-1}$; this is also known as accuracy matrix (Bishop, 2006). Its diagonal terms represent the posterior stiffness $w_{\theta_i|\mathbf{y}}$ of each single parameter θ_i.

Now, to obtain the N diagonal elements to $\boldsymbol{\Lambda}$ we must get the second derivative of $\mathrm{E_p}(\boldsymbol{\theta})$ with respect to each of the parameters θ_i; the elements out of diagonal are instead obtained by calculating the mixed derivatives of each parameter with respect to all other parameters. To obtain the covariance matrix we simply make the inverse of $\boldsymbol{\Lambda}$. The diagonal elements of Σ represent the posterior variance $\sigma^2_{\theta_i|\mathbf{y}}$ of each single parameter θ_i. The posterior mean values $\mu_{\theta_i|\mathbf{y}}$ of each parameter θ_i correspond to those values that minimize the potential energy of our mechanical system. Therefore, to discover them, we have to resolve an algebraic system with N variables in which there are the partial derivatives of
$\mathrm{E_p}(\boldsymbol{\theta})$, each with respect to each parameter θ_i, set equal to zero.

4 A case study: elongation of a cable belonging to Adige Bridge

Structural monitoring has been recognized as a powerful information tool, especially with regard to bridges management (Pozzi, Zonta, Wang et al., 2010), and request a deep knowledge of Bayesian rules. For this reason, we apply our method to the Adige Bridge (Cappello, Zonta, Pozzi et al., 2015) (Bruschetta, Zonta, Cappello et al., 2013), a two-span cable-stayed bridge located ten kilometres north of the city of Trento, Italy (Figure 2). The composite deck is made from 4 “I”-section steel girders and a 25 cm cast-on-site concrete slab. The deck is also supported by 12 stay cables, 6 on each side, which have a diameter of 116 mm and 128 mm. Their operational design load varies from 5,000 kN to 8,000 kN. The cables are anchored to the bridge tower, consisting of four pylons and located in the middle of the bridge. When the construction was completed, the Italian Autonomous Province of Trento, which owns and manages the bridge, decided to install a monitoring system to continuously record force and elongation of the stay cables. Elongations are recorded by 1 m long gauge sensors, placed on each of the 12 cables. These fiber-optical sensors (FOS) (Glisic, Posenato & Inaudi, 2007) are based on fiber Bragg gratings (FBG) which rely on a principle similar to that of photonic crystals (Zonta, Chappini, Chiasera et al., 2009) but provide better precision. These sensors also record local temperature for thermal compensation.

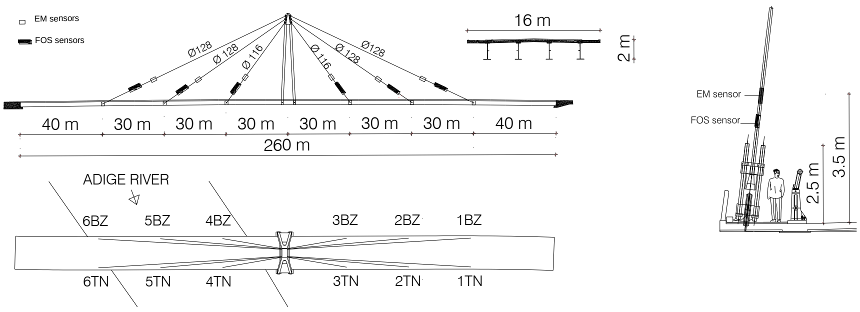

Fig. 2. Longitudinal section of the bridge and sensor layout (upper left); plan view of the bridge (lower left); cross-section of the bridge (right).

4.1 Two parameters to estimate

As an example, we use data acquired from October 12, 2011, to November 25, 2012, for cable 1TN, purified of the effect of temperature. We consider only one sample a day, recorded between 4 AM and 6 AM, as we assume the temperature in this period to be constant. We have discarded those days in which no samples were found in the time interval. Figure 3 shows the data acquired, expressed in terms of difference of deformation and time:

$$\Delta y = y_i - y_1, \quad \Delta t = t_i - t_1. \tag{8a,b}$$

During the analysis, 411 deformation measurements were recorded with an uncertainty for each measurement equal to $w_y = 0.0016\,\mu\varepsilon^{-2}$, i.e. $\sigma_y = 25\,\mu\varepsilon$. This is clearly a classical problem of linear regression. We have to estimate the two parameters that best characterize the straight line fitting our time-dependent data set. The function employed is:

$$y = y_0 + \varphi \cdot t, \tag{9}$$

where y_0 is the intercept and φ the slope of the straight line fitting our dataset. As we said before, the goal is to estimate the vector of the parameters $\boldsymbol{\theta} = (y_0, \varphi)^T$ that characterizes the parametric model resulting in the observations $\mathbf{y} = (y_1, y_2, ..., y_N)^T$, linearly dependent on the time *t*, as shown in Figure 4. We can represent the problem as a bar with two degrees of freedom: vertical translation and rotation. According to the parametric model defined in (9), we consider the slope of the bar linked to the parameter φ, its length to the time t and its distance from the ground floor to the parameter y_0.

Based on our experience, we assign to the two parameters φ and t two prior Gaussian distributions that give us the initial information about the state of the bar. We connect the left-hand end of the rigid bar to a vertical linear elastic spring with flexibility equal to the standard deviation of the prior distribution associated to the parameter y_0 and pre-stretch equal to its mean value. We connect the same end to a torsion spring with flexibility and imposed rotation equal respectively to the standard deviation and the mean value of the prior distribution associated to the parameter φ, as shown in Figure 4. Finally, we introduce the measurements as a system of linear springs, each with flexibility and pre-stretch equal respectively to the standard deviation and value associated to a single measurement. Each spring is placed at a distance from the torsion spring equal to the corresponding interval of time t_i. The elastic potential of the mechanical system of Figure 4 becomes:

$$E_p(y_0,\varphi) = \frac{1}{2} w_y (y_0 - \mu_{y_0})^2 + \frac{1}{2} w_\varphi (\varphi - \mu_\varphi)^2 + \frac{1}{2} w_y \sum_{i=1}^{N} [(y_0 + \varphi t_i) - y_i]^2, \tag{10}$$

where $\Delta y_i = y_0 + \varphi \cdot t_i - y_i$ represents the elongation suffered by the *N* springs linked to the observations, due to a generic translation y_0 and a generic rotation φ imposed on the system. The accuracy matrix is simply the Hessian matrix of (10):

$$\boldsymbol{\Lambda} = \begin{bmatrix} \dfrac{\delta^2 E_p(y_0,\varphi)}{\delta y_0^{\,2}} & \dfrac{\delta^2 E_p(y_0,\varphi)}{\delta y_0 \delta\varphi} \\ \dfrac{\delta^2 E_p(y_0,\varphi)}{\delta\varphi \delta y_0} & \dfrac{\delta^2 E_p(y_0,\varphi)}{\delta\varphi^2} \end{bmatrix}. \tag{11}$$

The inverse of the matrix (11) represents the covariance matrix Σ: the first term of its diagonal is the posterior variance associated to the parameter y_0 while the second term on the same diagonal is the posterior variance associated to the parameter φ. To identify instead the values $\mu_{y_0|\mathbf{y}}$ and $\mu_{\varphi|\mathbf{y}}$ that represent the posterior mean values associated respectively to the parameters y_0 and φ, we must solve the system formed by the first derivative of (10) with respect to the parameter y_0 and the parameter φ, set equal to zero.

$$\begin{cases} \dfrac{\partial E_p(\boldsymbol{\theta})}{\partial y_0} = w_{y_0}(y_0 - \mu_{y_0}) + w_y \sum_{i=1}^{N} [(y_0 + \varphi t_i) - y_i] = 0 \\ \dfrac{\partial E_p(\boldsymbol{\theta})}{\partial \varphi} = w_\varphi(\varphi - \mu_\varphi) + w_y \sum_{i=1}^{N} t_i[(y_0 + \varphi t_i) - y_i] = 0 \end{cases}. \tag{12}$$

The solutions of the system (12) give us the values of $\mu_{y_0|\mathbf{y}}$ and $\mu_{\varphi|\mathbf{y}}$, that represent the posterior mean values associated respectively to the parameters y_0 and φ and that minimize the potential $E_p(y_0,\varphi)$ of our mechanical system. Now we can substitute the numerical values into the equations formulated above, and we obtain the final outcomes reported in Table 2, compared with the prior values of the parameters. Figure 3 reports the two straight lines interpolating our dataset. We obtain the same results as applying the flexibility method to the same mechanical system (Cappello, Bolognani & Zonta, 2015), although, with the potential energy, we considerably reduce the computational cost of the algebra.

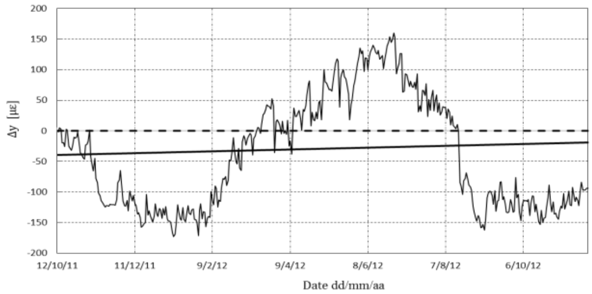

Fig. 3. Relative strain of cable 1TN and interpolating lines.

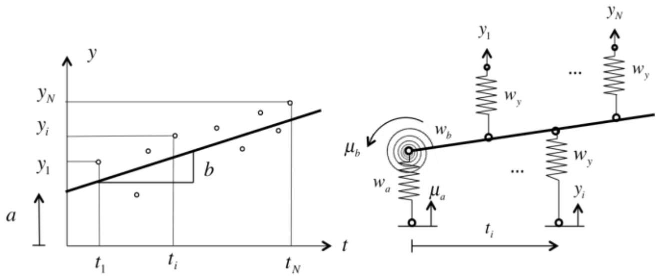

Fig. 4. Linear regression problem in the world of Mechanics.

Table 2. Prior and posterior values of the parameters to estimate

Prior distributions			
Parameter y_0		Parameter φ	
w_{y_0} $[\mu\varepsilon^{-2}]$	0.0025	w_φ $[\mu\varepsilon^{-2} day^2]$	1
σ_{y_0} $[\mu\varepsilon]$	20.00	σ_φ $[\mu\varepsilon\ day^{-1}]$	1.0000
μ_{y_0} $[\mu\varepsilon]$	0.00	μ_φ $[\mu\varepsilon\ day^{-1}]$	0.0000
Posterior distributions			
Parameter y_0		Parameter φ	
w_{y_0} $[\mu\varepsilon^{-2}]$	0.6601	w_φ $[\mu\varepsilon^{-2} day^2]$	36893
σ_{y_0} $[\mu\varepsilon]$	2.44	σ_φ $[\mu\varepsilon\ day^{-1}]$	0.0103
μ_{y_0} $[\mu\varepsilon]$	-49.07	μ_φ $[\mu\varepsilon\ day^{-1}]$	0.0473

4.2 Three parameters to estimate

We now extend the case of Adige Bridge, presented in the previous Section, by introducing the effect of temperature $\Delta\hat{T}$. Thus, we must estimate an additional parameter α and the model that fits our time dependent dataset becomes the following:

$$\Delta\hat{y} = y_0 + \alpha \cdot \Delta\hat{T} + \varphi \cdot \Delta\hat{t}\ . \tag{13}$$

In Figure 5, we can note the N translation springs linked to the different measurements with stiffness $w_{LH} = \sigma_{LH}^{-2} = 0.0016\mu\varepsilon^{-2}$ and the springs linked to the prior distribution: a translation spring associated to the parameter y_0, a rotational spring associated to α and a rotational spring associated to φ, whose numerical values are the same as the case in the previous Section. To determine the posterior standard deviation of the three parameters to estimate (y_0, α, φ) we have to express the potential energy $E_p(y_0, \alpha, \varphi)$ of the mechanical system represented in Figure 5, as a function of the three

unknown parameters. We can now obtain the accuracy matrix Λ simply by calculating the Hessian Matrix associated to $E_p(y_0,\alpha,\varphi)$, and the covariance matrix from the inverse of Λ.
To discover the values $\mu_{y_0|\mathbf{y}}$, $\mu_{\alpha|\mathbf{y}}$ and $\mu_{\varphi|\mathbf{y}}$, which represent the posterior mean values associated respectively to the parameters y_0, α and φ, we must solve the system formed by the first derivative of the potential energy with respect to the three parameters, set equal to zero. Figure 6 shows the graphical representation, using the software Matlab, of the two surfaces fitting our data set. Finally, Table 3 reports the numerical values obtained from the posterior distribution of the parameters.

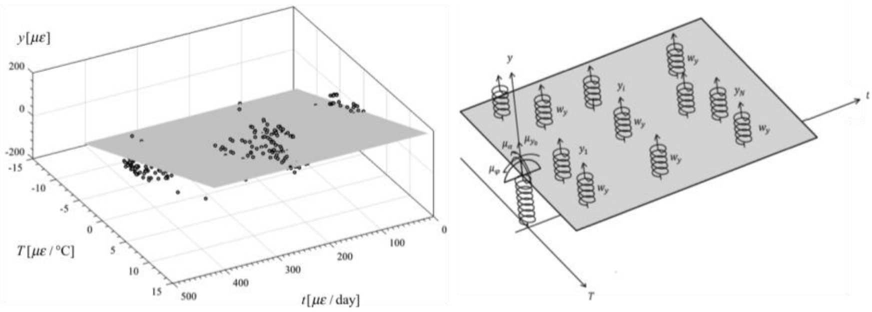

Fig. 5. Representation of a linear regression problem with three parameters to estimate, in the world of Mechanics.

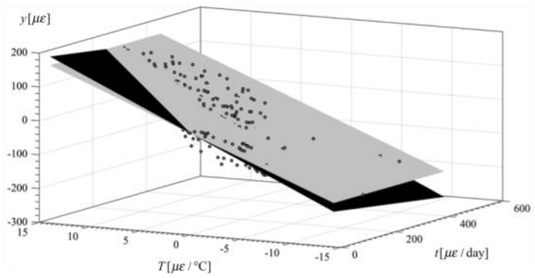

Fig. 6. Representation of the two fitting surfaces associated to the prior parameters (grey) and to the posterior parameters (black).

Table 3. Posterior values of the three parameters to estimate

Posterior distributions					
Parameter y_0		Parameter φ		Parameter α	
w_{y_0} $[\mu\varepsilon^{-2}]$	0.6601	w_φ $[\mu\varepsilon^{-2}\ day^2]$	36893	w_α $[\mu\varepsilon^{-2}\ °C^2]$	27.88
σ_{y_0} $[\mu\varepsilon]$	2.54	σ_φ $[\mu\varepsilon\ day^{-1}]$	0.0106	σ_α $[\mu\varepsilon\ °C^{-1}]$	0.20
μ_{y_0} $[\mu\varepsilon]$	0.48	μ_φ $[\mu\varepsilon\ day^{-1}]$	-0.1209	μ_α $[\mu\varepsilon\ °C^{-1}]$	13.80

5 Non Gaussian variables: single parameter estimation

How does theory of the mechanical equivalent change if we decide to involve non-Gaussian variable? As it is logical, we will obtain non-linear springs, whose constitutive laws vary according to the probability distributions that characterize them. To extend the mechanical analogy to distribution other than the Gaussian one is very simple thanks to the three basic rules that will presented in section 5.2.

5.1 The Gaussian case

To better understand how to proceed, we must first start to work with a Gaussian distribution. We consider a univariate Gaussian distribution with deviation standard σ_x and mean value μ_x, where x

represents the unknown parameter to estimate. As we obtained in Section 3, the elastic potential associated to this is:

$$\mathrm{N}(\mu,\sigma;x) = \frac{1}{x\sqrt{2\pi}}\,\mathrm{e}^{-\frac{1}{2\sigma^2}(x-\mu)^2}\ ,\ \ \mathrm{E_p}(x) = \frac{1}{2}\,w\left(x-\mu\right)^2 . \tag{14}$$

We calculate now the natural logarithm of the probability function defined before:

$$\mathrm{In}(\mathrm{N}(\mu,\sigma;x)) = \mathrm{In}(\frac{1}{\sigma\sqrt{2\pi}}) + \mathrm{In}(\mathrm{e}^{-\frac{1}{2\sigma^2}(x-\mu\ )^2}) = -\frac{1}{2\sigma^2}\left(x-\mu\right)^2 + a, \tag{15}$$

where a is an additive constant, which we can omit.

By focusing our attention on (15), it is not so difficult discover that the relation between the Gaussian distribution and the elastic potential is the following:

$$\mathrm{E_p}(x) = -\mathrm{In}(\mathrm{N}(\mu,\sigma;x)) = \frac{1}{2\sigma^2}\left(x-\mu\right)^2 . \tag{16}$$

As we already know, we can obtain the stiffness of the spring w that characterizes the Gaussian distribution in this way:

$$\mathrm{k}(x) = \frac{\mathrm{d}^2\mathrm{E_p}(x)}{\mathrm{d}x^2} = \frac{1}{\sigma^2} = w, \tag{17}$$

and, according to the laws of the Mechanics (Ward, 1992), the elastic force F, i.e. the return force, which correlates the stiffness of the spring with its pre-stretch, is conservative. This means that we can define a scalar potential (potential energy) that the force will be the gradient or, in the case of a dimension, the first derivative.

$$\mathrm{F}(x) = \frac{\mathrm{dE_p}(x)}{\mathrm{d}x} = \mathrm{k}(x)\cdot x. \tag{18}$$

With these premises, it is clear that a Gaussian distribution, in the mechanical equivalent analogy is represented as a spring with a parabolic potential, i.e. proportional to the square of x, and with linear constitutive law, expressed by the relation with the return force F and the parameter x. It is evident that, in the case of a Gaussian distribution, the relation between the elastic force and the unknown parameter, which represents the pre-stretch of the spring in the mechanical analogy, is linear: this is the reason why, in the preceding sections, we have schematized inference problems as mechanical systems composed by linear elastic springs in series or in parallel. The mechanical properties linked to a Gaussian distribution with the statistical quantities reported in Table 4, are shown in Table 19.

Table 4. Statistical properties of the probability distribution $\mathrm{N}(1,1;x)$

$\boldsymbol{\mu}$	1
σ	1
mean	$\mu=1$
variance	$\sigma^2=1$
median	$\mu=1$
mode	$\mu=1$

5.2 Non-linear cases

Now, we denote with $f(x;a,b)$ a generic probability distribution, where x is the unknown parameter to estimate, a and b the parameters that characterize the probability distribution under exam. According to Eq. (16-18), the potential energy, the elastic force and the stiffness linked to the spring representing the generic distribution f are the following:

$$\mathrm{E}_p(x) = -\mathrm{In}(f),\ \mathrm{F}(x) = \frac{\partial \mathrm{E}_p(x)}{\partial x},\ k(x) = \frac{\partial^2 \mathrm{E}_p(x)}{\partial^2 x}. \tag{19a,b,c}$$

In the following sections we will report some examples, regarding the main probability distributions used in the world of logic, and we will try to define for each the mechanical features of the springs that represent them.

5.2.1 Lognormal distribution

The lognormal distribution (Forbes et al., 2011) is applicable to random variables that are constrained by zero but have a few very large values. The resulting distribution is asymmetrical and positively skewed. In particular, in engineering field, the lognormal distribution is often used to describe the fatigue behaviour of many mechanical components and the mechanical resistance of structural materials, as the steel.

The application of a logarithmic transformation to the data can allow the data to be approximated by the symmetrical normal distribution, although the absence of negative values may limit the validity of this procedure. In other words, it is the probability distribution of a random variable x whose logarithm *In*(x) follows a normal distribution, and it takes the following form:

$$\mathrm{l}(\lambda,\varepsilon;x) = \frac{1}{x\varepsilon\sqrt{2\pi}}\mathrm{e}^{-\frac{1}{2\varepsilon^2}\left(\mathrm{In}(x)-\lambda\right)^2} \quad \text{with } 0 < x < +\infty, \tag{20}$$

where λ is the mean of $\mathrm{In}(x)$ and ε the standard deviation of $\mathrm{In}(x)$, which are both dimensionless. Table 5 shows the statistical properties of the lognormal distribution.

Table 5. Statistical properties of the probability distribution $\mathrm{l}(\lambda,\varepsilon;x)$

median	e^{λ}
mean	$e^{\lambda+\frac{1}{2}\varepsilon^2}$
variance	$e^{2\lambda+\varepsilon^2}\cdot(e^{\varepsilon^2}-1)$
mode	$e^{\lambda-\varepsilon^2}$

How can we model a spring representing $\mathrm{l}(\lambda,\varepsilon;x)$? The answer is simple: we must use the three aforementioned expressions (19a,b,c), to obtain the trend of the potential, of the elastic force and of the stiffness of the spring linked to the lognormal distribution.

$$\mathrm{E}_p(x) = -\mathrm{In}(\mathrm{l}(\lambda,\varepsilon;x)) = \frac{1}{2\varepsilon^2}\left(\mathrm{In}(x)-\lambda\right)^2 + \mathrm{In}(x) + a\,, \tag{21}$$

where a is an additive constant that we can neglect.

$$\mathrm{F}(x) = \frac{\partial \mathrm{E}_p(x)}{\partial x} = \frac{1}{x\varepsilon^2}\left(\mathrm{In}(x) - \lambda + \varepsilon^2\right), \tag{22}$$

$$k(x) = \frac{\partial^2 \mathrm{E}_p(x)}{\partial^2 x} = \frac{1}{x^2\varepsilon^2}\left(1 - \mathrm{In}(x) + \lambda - \varepsilon^2\right). \tag{23}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 6, are shown in Table 19.

Table 6. Statistical properties of the probability distribution $l(1,1;x)$

λ	1
ε	1
median	$e^{\lambda} \simeq 2.72$
mean	$e^{\lambda+\frac{1}{2}\varepsilon^2} \simeq 4.48$
variance	$e^{2\lambda+\varepsilon^2} \cdot (e^{\varepsilon^2}-1) \simeq 34.51$
mode	$e^{\lambda-\varepsilon^2} = 1$

5.2.2 Extreme value (Gumbel) distribution

The extreme value distribution (Gao & Sun) was developed as the distribution of the largest of a number of values and was originally applied to the estimation of flood levels. It has since been applied to the estimation of the magnitude of earthquakes. The distribution may also be applied to the study of athletic and other records. We consider the distribution of the largest extreme. Reversal of the sign of *x* gives the distribution of the smallest extreme. This is the Type I, the most common of three extreme value distributions, known as the Gumbel distribution. Its probability density function takes the following form:

$$\mathrm{g}(a,b;x) = \frac{1}{b} e^{\frac{-(x-a)}{b} - e^{\frac{-(x-a)}{b}}} \quad \text{with } -\infty < x < +\infty, \tag{24}$$

where *a* and *b* are the parameters that characterize the distribution.

The main statistic quantities of the Gumbel distribution are reported in Table 7.

Table 7. Main statistical quantities of a random variable *x*, belonging to a Gumbel distribution

median	$a - b \cdot \mathrm{In}(\mathrm{In}(2))$
mean	$a - b \cdot \Gamma'(1)$
variance	$b^2\pi^2/6$
mode	a

where $\Gamma'(1) = -0.57722$ is the first derivative of the Gamma function $\Gamma(n)$ with respect to $n = 1$.

Extreme value variates correspond to the limit, as *n* tends to infinity, of the maximum value of *n*-independent random variates with the same continuous distribution. Logarithmic transformations of extreme value variates of Type II (Frèchet) and Type III (Weibull) correspond to Type I Gumbel variates. After introducing the main properties of the distribution, we are able, with the same Eq. (19a,b,c) of the previous Section, to spot the potential, the elastic force and the stiffness function linked to a Gumbel distribution $\mathrm{g}(a,b;x)$.

$$\mathrm{E_p}(x) = -\mathrm{In}(\mathrm{g}(a,b;x)) = \mathrm{In}(b) + \frac{(x-a)}{b} + e^{\frac{-(x-a)}{b}} = \frac{(x-a)}{b} + e^{\frac{-(x-a)}{b}} + c, \tag{25}$$

where *c* is an additive constant that we can neglect.

$$\mathrm{F}(x) = \frac{\partial \mathrm{E_p}(x)}{\partial x} = \frac{1}{b} - \frac{1}{b} e^{\frac{-(x-a)}{b}} = \frac{1}{b}\left(1 - e^{\frac{-(x-a)}{b}}\right), \tag{26}$$

$$\mathrm{k}(x) = \frac{\partial^2 \mathrm{E_p}(x)}{\partial x^2} = \frac{1}{b^2} e^{\frac{-(x-a)}{b}}. \tag{27}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 8, are shown in Table 19.

Table 8. Statistical properties of the probability distribution $g(1,1;x)$

a	1
b	1
median	$a-b\cdot\ln(\ln(2))\simeq 1.37$
mean	$a-b\cdot\Gamma'(1)\simeq 1.58$
variance	$b^2\pi^2/6\simeq 1.64$
mode	$a=1$

5.2.3 Cauchy distribution

The Cauchy distribution (Forbes et al., 2011) is of mathematical interest due to the absence of defined moments. Its probability density function takes the following form:

$$c(a,b;x)=\left\{\pi b\left[1+\left(\frac{x-a}{b}\right)^2\right]\right\}^{-1} \quad \text{with } -\infty<x<+\infty, \tag{28}$$

where a and b are the parameters that characterize the distribution. The main statistic quantities of the Cauchy distribution are reported in Table 9.

Table 9. Main statistical quantities of a random variable x, belonging to a Cauchy distribution

median	a
mean	Does not exist
variance	Does not exist
mode	a

The Cauchy distribution is unimodal and symmetric, with much heavier tails than the normal. The probability density function is symmetric about a, with upper and lower quartiles, $a\pm b$. The potential, the elastic force and the stiffness function linked to the Gumbel distribution $c(a,b;x)$ are:

$$E_p(x) = -\ln\left(\left\{\pi b\left[1+\left(\frac{x-a}{b}\right)^2\right]\right\}^{-1}\right)=\ln\left(1+\left(\frac{x-a}{b}\right)^2\right)+c, \tag{29}$$

where c is an additive constant that we can neglect.

$$F(x)=\frac{\partial E_p(x)}{\partial x}=\frac{1}{1+\left(\frac{x-a}{b}\right)^2}\cdot\frac{2}{b}\left(\frac{x-a}{b}\right)=\frac{2\left(x-a\right)}{b^2+\left(x-a\right)^2}, \tag{30}$$

$$k(x)=\frac{\partial^2 E_p(x)}{\partial x^2}=\frac{2\left(x-a\right)-2}{\left(b^2+\left(x-a\right)^2\right)^2}. \tag{31}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 10, are shown in Table 19.

Table 10. Statistical properties of the probability distribution $c(1,1;x)$

a	1
b	1
median	$a=1$
mean	Do not exist
variance	Do not exist
mode	1

5.2.4 Beta distribution

Applications include modeling random variables that have a finite range, *a* to *b*. An example is the distribution of activity times in project networks. The beta distribution is frequently used as a prior distribution for binomial proportions in Bayesian analysis. Its probability density function takes the following form (Forbes et al., 2011):

$$\beta(\nu,\omega;x) = \frac{x^{\nu-1}(1-x)^{\omega-1}}{B(\nu,\omega)} \text{ with } 0 \le x \le 1, \tag{32}$$

where $\nu > 0$ *a* and $\omega > 0$ are the parameters that characterize the distribution, and $B(\nu,\omega)$ is the Beta function with arguments ν,ω given by $B(\nu,\omega) = \int_0^1 u^{\nu-1}(1-u)^{\omega-1}du$.

The main statistic quantities of the Cauchy distribution are reported in Table 11.

Table 11. Main statistical quantities of a random variable x, belonging to a Beta distribution

mean	$\nu/(\nu+\omega)$
variance	$\nu\omega/\left[(\nu+\omega)^2(\nu+\omega+1)\right]$
mode	$(\nu-1)/(\nu+\omega-2)$, $\nu>1, \omega>1$

The potential, the elastic force and the stiffness function linked to the Gumbel distribution $\beta(\nu,\omega;x)$ are:

$$E_p(x) = -\text{In}\left(\frac{x^{\nu-1}(1-x)^{\omega-1}}{B(\nu,\omega)}\right) = (1-\nu)\,\text{In}\left(x\right) + (1-\omega)\,\text{In}\left(1-x\right) + \text{In}\left(B(\nu,\omega)\right), \tag{33}$$

where $\text{In}\left(B(\nu,\omega)\right)$ can be considered as an additive constant that we can neglect, because it does not depend on the variable x.

$$F(x) = \frac{\partial E_p(x)}{\partial x} = \frac{(1-\nu)}{x} - \frac{(1-\omega)}{1-x}, \tag{34}$$

$$k(x) = \frac{\partial^2 E_p(x)}{\partial x^2} = -\frac{(1-\nu)}{x^2} - \frac{(1-\omega)}{(1-x)^2}. \tag{35}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 12, are shown in Table 19.

Table 12. Statistical properties of the probability distribution $\beta(10,2;x)$

ν	10
ω	2
mean	$\nu/(\nu+\omega) \simeq 0.83$
variance	$\nu\omega/\left[(\nu+\omega)^2(\nu+\omega+1)\right] \simeq 0.011$
mode	$(\nu-1)/(\nu+\omega-2) = 0.9$

5.2.5 Trapezoidal distribution

In probability theory and statistics, the trapezoidal distribution is a continuous probability distribution with lower limit *a*, upper limit *d* and modes *b* and *c*, where $a < d$ and $a \le b \le c \le d$. Between *b* and *c*, the probability density is constant; otherwise one has the generalized trapezoidal distribution. Special cases of the trapezoidal distribution include the uniform distribution (with $a = b$ and $c = d$) and the triangular distribution (with $b = c$). Trapezoidal distributions (Gao & Sun) seem to be appropriate for modeling the duration and the form of a phenomenon which may be represented by three stages. The first stage can be viewed as a growth-stage, the second corresponds to a relative stability and the third represents a decline (decay). These distributions however are restricted since the growth and decay (in the first and third stages) are limited in the trapezoidal case to linear forms and the second stage represents complete stability rather than a possible mild incline or decline. The trapezoidal probability density function is of the form:

$$\mathrm{f}(a,b,c,d;x) = \begin{cases} u \cdot \left(\dfrac{x-a}{b-a}\right) & a \le x < b \\ u & b \le x < c \\ u \cdot \left(\dfrac{d-x}{d-c}\right) & c \le x < d \end{cases}, \text{ on } [a,d] \subset \mathbb{R} \tag{36}$$

where a<b<c<d and $u = \dfrac{2}{d+c-b-a}$.

The main statistic quantities of the trapezoidal distribution are reported in Table 13.

Table 13. Main statistical quantities of a random variable x, belonging to a trapezoidal distribution

mean	$(a+b+c+d)/4$
variance	$\dfrac{(d-a)^2(1+\beta)^2}{24}$
mode	Any value in [b,c]

where $0 \le \beta < 1$ represents the ratio between the smaller and the larger base of the trapezoidal function.

The potential, the elastic force and the stiffness function linked to the symmetric trapezoidal distribution $\mathrm{f}(a,b,c,d;x)$ are:

$$\mathrm{E_p}(x) = -\mathrm{In}(\mathrm{f}(a,b,c,d;x)) = \begin{cases} -\mathrm{In}(u) + \mathrm{In}(b-a) - \mathrm{In}(x-a) & a \le x < b \\ -\mathrm{In}(u) & b \le x < c \\ -\mathrm{In}(u) + \mathrm{In}(d-c) - \mathrm{In}(d-x) & c \le x < d \end{cases}, \tag{37}$$

$$\mathrm{F}(x) = \frac{\mathrm{dE_p}(x)}{\mathrm{d}x} = \begin{cases} -\dfrac{1}{x-a} & a \le x < b \\ 0 & b \le x < c \\ \dfrac{1}{d-x} & c \le x < d \end{cases}, \tag{38}$$

$$\mathrm{k}(x) = \frac{\mathrm{d}^2\mathrm{E_p}(x)}{\mathrm{d}x^2} = \begin{cases} \dfrac{1}{(x-a)^2} & a \le x < b \\ 0 & b \le x < c \\ \dfrac{1}{(d-x)^2} & c \le x < d \end{cases} . \tag{39}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 14, are shown in Table 19.

Table 14. Statistical properties of the probability distribution $\mathrm{f}(2,4,8,10;x)$

a	2
b	4
c	8
d	10
mean	$(a+b+c+d)/4=6$
variance	$\dfrac{(d-a)^2(1+\beta)^2}{24}=6$
mode	$(a+b+c+d)/4=6$

5.2.6 Triangular distribution

In the theory of probability the triangular is a probability distribution continues whose probability density function describes a triangle (Forbes et al., 2011), or that it is nothing on the two extreme values and is linear between these and an intermediate value (the mode). In statistics is used as a model when the sample available is very limited. Its density probability function is the following:

$$t(a,b,c;x) = \begin{cases} \dfrac{2}{c-a}\dfrac{x-a}{b-a} & a \le x < b \\ \dfrac{2}{c-a} & x = b \\ \dfrac{2}{c-a}\dfrac{c-x}{c-b} & b < x \le c \end{cases} \quad \text{on } [a,c] \subset \mathbb{R}. \tag{40}$$

The main statistic quantities of the Cauchy distribution are reported in Table 15.

Table 15. Main statistical quantities of a random variable x, belonging to a triangular distribution

mean	$(a+b+c)/3$
variance	$\left(a^2+b^2+c^2-(ac+cb+ab)\right)/18$
median	$a+\sqrt{(c-a)(b-a)/2}$ se $b \ge \dfrac{a+c}{2}$
mode	b

The potential, the elastic force and the stiffness function linked to the triangular distribution $\mathrm{t}(a,b,c;x)$, defined on an interval $[a,c] \subset \mathbb{R}$ are:

$$\mathrm{E_p}(x)=\begin{cases} -\mathrm{In}\left(\dfrac{2}{c-a}\dfrac{x-a}{b-a}\right)=-\mathrm{In}(x-a)+\mathrm{In}(c-a)+\mathrm{In}(b-a)+k & a\le x<b\\ -\mathrm{In}\left(\dfrac{2}{c-a}\right)=\mathrm{In}(c-a)+k & x=b\\ -\mathrm{In}\left(\dfrac{2}{c-a}\dfrac{c-x}{c-b}\right)=-\mathrm{In}(c-x)+\mathrm{In}(c-a)+\mathrm{In}(c-b)+k & b<x\le c \end{cases}, \qquad (41)$$

$$\mathrm{F}(x)=\frac{\partial \mathrm{E_p}(x)}{\partial x}=\begin{cases} -\dfrac{1}{x-a} & a\le x<b\\ 0 & x=b\\ \dfrac{1}{c-x} & b<x\le c \end{cases} \text{ on } [a,c]\subset\mathbb{R}, \qquad (42)$$

$$\mathrm{k}(x)=\frac{\partial^2 \mathrm{E_p}(x)}{\partial x^2}=\begin{cases} \dfrac{1}{(x-a)^2} & a\le x<b\\ 0 & x=b\\ \dfrac{1}{(c-x)^2} & b<x\le c \end{cases} \text{ on } [a,c]\subset\mathbb{R}. \qquad (43)$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 16, are shown in Table 19.

Table 16. Statistical properties of the Probability distribution $\mathrm{t}(2,6,10;x)$

a	2
b	6
c	10
mean	$(a+b+c)/3=6$
variance	$\left(a^2+b^2+c^2-(ac+bc+ab)\right)/18\simeq 2.67$
median	$a+\sqrt{(c-a)(b-a)/2}=6$ se $b\ge\dfrac{a+c}{2}$
mode	b =6

5.2.7 Uniform distribution

As we have seen before, if we analyzed a Beta distribution with the parameter that characterize it equal to one, $\beta(1,1;x)$, we obtain a uniform distribution (Forbes et al., 2011). In probability theory the uniform distribution is a probability distribution continues which is uniform on a set, or that attaches the same probability to all the points belonging to a given interval [a, b] contained in the set. The uniform distribution is usually defined on a continuous range $S=[a,b]\subset\mathbb{R}$; in this case it is indicated $U(a,b)=U([a,b])$. Its probability density is:

$$u(a,b;x)=\frac{1}{b-a} \text{ on } [a,b]. \qquad (44)$$

The main statistic quantities of the Cauchy distribution are reported in Table 17.

Table 17. Main statistical quantities of a random variable x, belonging to a uniform distribution

mean	$(a+b)/2$
variance	$\left(b^2-a^2\right)/12$
median	$(a+b)/2$
mode	any value in $[a,b]\subset\mathbb{R}$

In this case it is not so difficult to understand that the potential of the associated spring will be constant, while the elastic force and the stiffness will result equal to zero in their entire domain.

$$\mathrm{E_p}(x) = -\mathrm{In}\left(\frac{1}{b-a}\right) = \mathrm{In}(b-a), \tag{45}$$

$$\mathrm{F}(x) = \frac{\partial \mathrm{E_p}(x)}{\partial x} = 0, \tag{46}$$

$$\mathrm{k}(x) = \frac{\partial^2 \mathrm{E_p}(x)}{\partial x^2} = 0. \tag{47}$$

The mechanical properties linked to a lognormal distribution with the statistical quantities reported in Table 18, are shown in Table 19.

Table 18. Statistical properties of the probability distribution $\mathrm{u}(2,10;x)$

a	2
b	10
mean	$(a+b)/2=6$
variance	$\left(b^2-a^2\right)/12=8$
median	$(a+b)/2=6$
mode	any value in $[a,b]\subset\mathbb{R}$

5.2.8 Discussion of the results

Table 19 shows the constitutive lows of the springs belonging to all the non-linear distributions presented in the previous sections. Particularly, it is interesting to note that the potential energy has a minimum in correspondence to the mode of the probability distribution and not in correspondence to the mean. In addition, the potential energy is not symmetric respect to its minimum value when the starting distribution is also not symmetric. In some cases, we notice that, when the displacements become remarkable, the elastic force becomes constant, by tending to a value little greater than zero, and in consequence the stiffness, which is the first derivative of the elastic force tends to zero.

6 Conclusions

We have stated an analogy between the world of logic and the world of mechanics, allowing us to solve, using the methods of classical structural engineering, any complex inference parameter estimation problem, in which the values of the parameters have to be estimated based on multiple Gaussian-distributed uncertain observations. By simply expressing the potential energy of the mechanical system associated to our inference scheme, we are able, with a few trivial algebraic steps, to determine the posterior mean values and standard deviations of the parameters to estimate. With the aid of real-life structural health monitoring cases, we have showed how our approach allows structural engineers to solve simply general problems of linear regression. Although the examples shown in this paper are incidentally all structural engineering cases, the scope of application of the method is evidently the most general, and we seek to demonstrate in the future its applicability to inference problem arising from various disciplinary fields, including cognitive science, economics and law.

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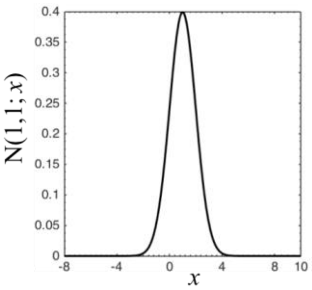
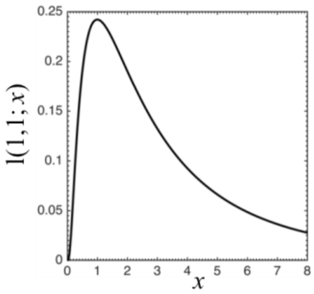
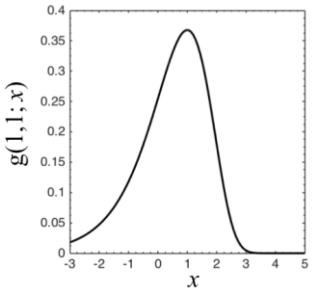
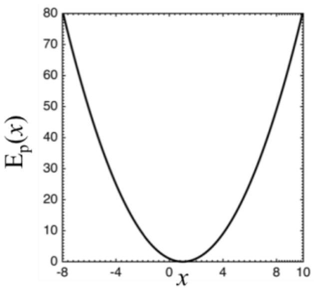
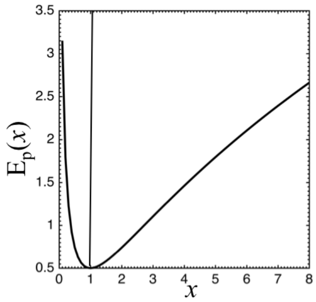
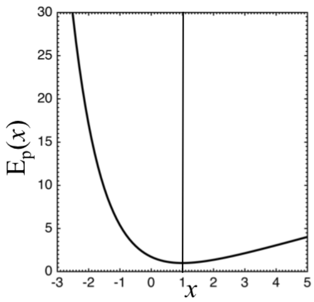
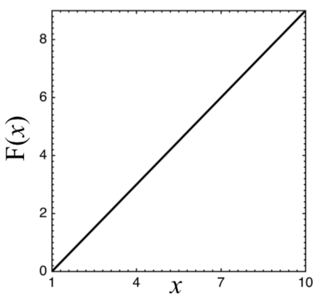
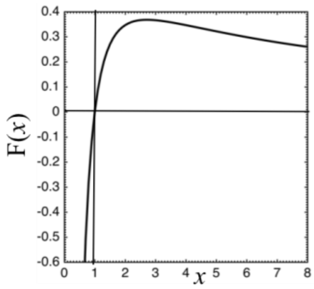
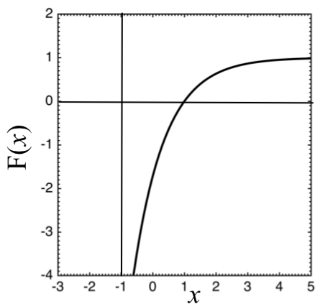
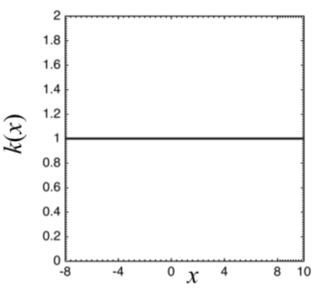
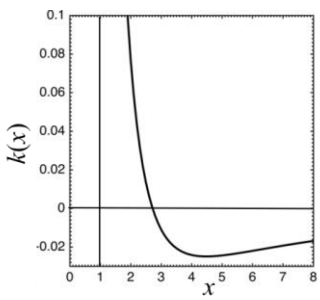
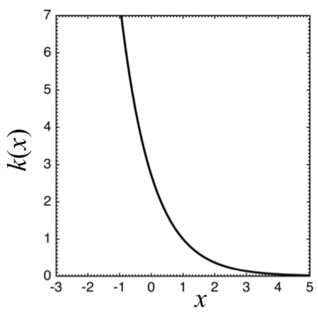
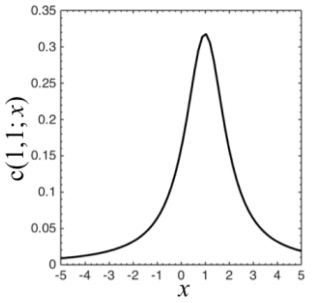
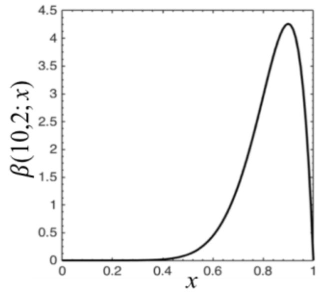
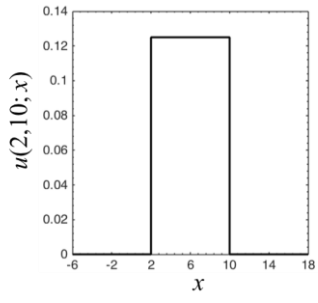
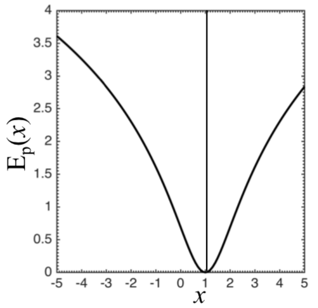
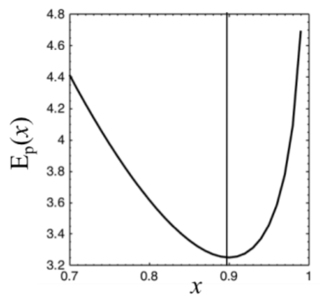
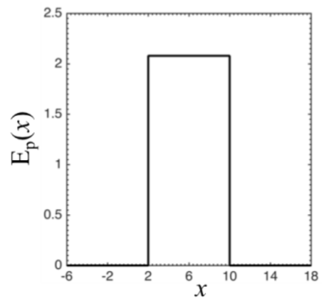
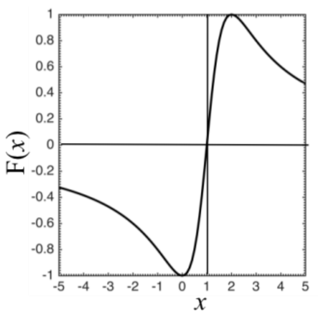
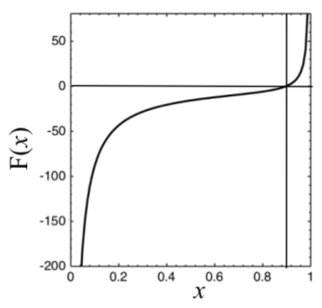
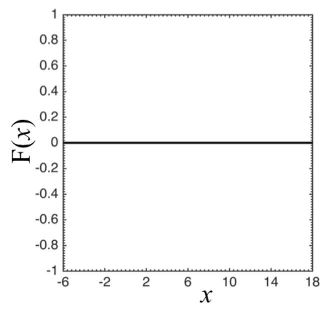
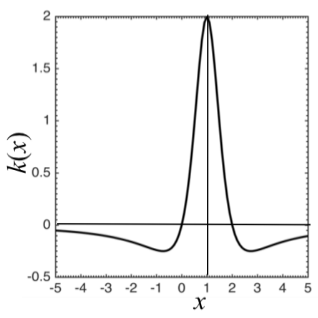
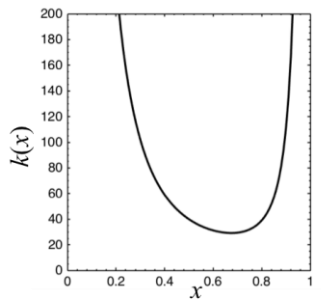
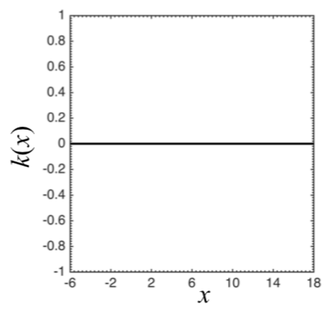
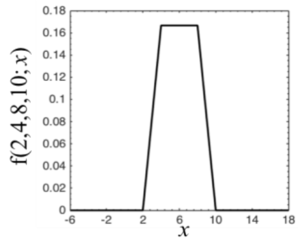
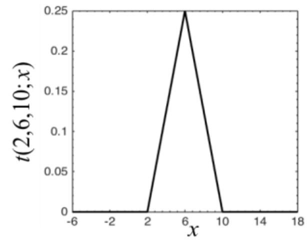
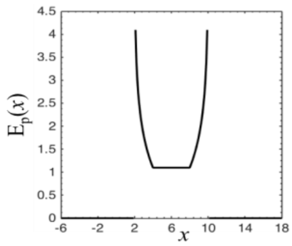
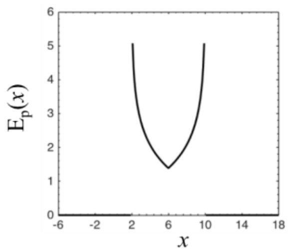
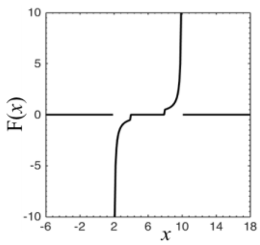
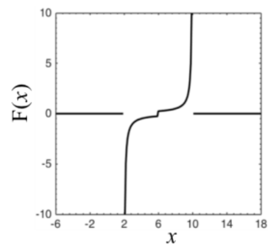
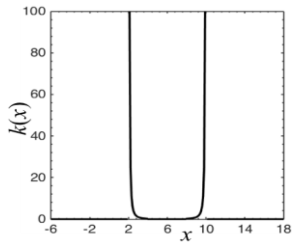
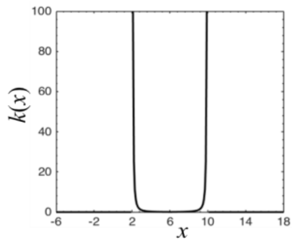
Table 19. Constitutive lows of the springs belonging to different kinds of non-linear distributions

	Gaussian $(\mu, \sigma = 1)$	**Lognormal** $(\lambda, \varepsilon = 1)$	**Gumbel** $(a, b = 1)$
Distribution $f(x)$	$N(\mu,\sigma;x)=\frac{1}{\sqrt{2\pi\sigma^2}}e^{-\frac{1}{2\sigma^2}(x-\mu)^2}$ 	$l(\lambda,\varepsilon;x)=\frac{1}{x\varepsilon\sqrt{2\pi}}e^{-\frac{1}{2\varepsilon^2}(In(x)-\lambda)^2}$ 	$g(a,b;x)=\frac{1}{b}e^{-\frac{-(x-a)}{b}-e^{\frac{-(x-a)}{b}}}$ 
Potential energy $E_p(x) = -In(f(x))$	$E_p(x)=\frac{1}{2}w(x-\mu)^2$ 	$E_p(x)=\frac{1}{2\varepsilon^2}(In(x)-\lambda)^2+In(x)$ 	$E_p(x)=\frac{(x-a)}{b}+e^{\frac{-(x-a)}{b}}$ 
Elastic force $F(x) = \frac{dE_p(x)}{dx}$	$F(x)=w(x-\mu)$ 	$F(x)=\frac{1}{x\varepsilon^2}(In(x)-\lambda+\varepsilon^2)$ 	$F(x)=\frac{1}{b}\left(1-e^{\frac{-(x-a)}{b}}\right)$ 
Stiffness $k(x) = \frac{d^2E_p(x)}{dx^2}$	$k(x)=w$ 	$k(x)=\frac{1}{x^2\varepsilon^2}(1-In(x)+\lambda-\varepsilon^2)$ 	$k(x)=\frac{1}{b^2}e^{\frac{-(x-a)}{b}}$ 

	Cauchy $(a=1, b=1)$	**Beta** $(\nu=10, \omega=2)$	**Uniform** $(a=2, b=10)$
Distribution $f(x)$	$c(a,b;x)=\left\{\pi b\left[1+\left(\frac{x-a}{b}\right)^2\right]\right\}^{-1}$ 	$\beta(\nu,\omega;x)=\frac{x^{\nu-1}(1-x)^{\omega-1}}{B(\nu,\omega)}$ 	$u(a,b;x)=\frac{1}{b-a}$ on $[a,b]$ 
Potential energy $E_p(x)=$ $-In(f(x))$	$E_p(x)=In\left(1+\left(\frac{x-a}{b}\right)^2\right)$ 	$E_p(x)=In[(x^{1-\nu})((1-x)^{(1-w)})]$ 	$E_p(x)=In(b-a)$ 
Elastic force $F(x)=$ $\frac{dE_p(x)}{dx}$	$F(x)=\frac{2(x-a)}{b^2+(x-a)^2}$ 	$F(x)=\frac{(1-\nu)}{x}-\frac{(1-w)}{1-x}$ 	$F(x)=0$ 
Stiffness $k(x)=$ $\frac{d^2E_p(x)}{dx^2}$	$k(x)=\frac{2(x-a)-2}{(b^2+(x-a)^2)^2}$ 	$k(x)=-\frac{(1-\nu)}{x^2}-\frac{(1-w)}{(1-x)^2}$ 	$k(x)=0$ 

	Trapezoidal $(a = 2, b = 4, c = 8, d = 10)$	**Triangular** $(\text{a}= 2, b = 6, c = 10)$
Distribution $f(x)$	$f(a,b,c,d;x) = \begin{cases} \frac{2}{d+c-b-a}\cdot\frac{x-a}{b-a} & a \le x < b \\ \frac{2}{d+c-b-a} & b \le x < c \\ \frac{2}{d+c-b-a}\cdot\frac{d-x}{d-c} & c \le x < d \end{cases}$ 	$t(a,b,c;x) = \begin{cases} \frac{2}{c-a}\cdot\frac{x-a}{b-a} & a \le x < b \\ \frac{2}{c-a} & x = b \\ \frac{2}{c-a}\cdot\frac{c-x}{c-b} & b < x \le c \end{cases}$ 
Potential energy $\text{E}_p(x) = -In(f(x))$	$\text{E}_p(x) = \begin{cases} -In\left(\frac{2}{d+c-b-a}\cdot\frac{x-a}{b-a}\right) & a \le x < b \\ -In\left(\frac{2}{d+c-b-a}\right) & b \le x < c \\ -In\left(\frac{2}{d+c-b-a}\cdot\frac{d-x}{d-c}\right) & c \le x < d \end{cases}$ 	$\text{E}_p(x) = \begin{cases} -In\left(\frac{2}{c-a}\cdot\frac{x-a}{b-a}\right) & a \le x < b \\ -In\left(\frac{2}{c-a}\right) & x = b \\ -In\left(\frac{2}{c-a}\cdot\frac{c-x}{c-b}\right) & b < x \le c \end{cases}$ 
Elastic force $\text{F}(x) = \frac{d\text{E}_p(x)}{dx}$	$\text{F}(x) = \begin{cases} -\frac{1}{x-a} & a \le x < b \\ 0 & b \le x < c \\ \frac{1}{d-x} & c \le x < d \end{cases}$ 	$\text{F}(x) = \begin{cases} -\frac{1}{x-a} & a \le x < b \\ 0 & x = b \\ \frac{1}{c-x} & b < x \le c \end{cases}$ 
Stiffness $k(x) = \frac{d^2\text{E}_p(x)}{dx^2}$	$k(x) = \begin{cases} \frac{1}{(x-a)^2} & a \le x < b \\ 0 & b \le x < c \\ \frac{1}{(d-x)^2} & c \le x < d \end{cases}$ 	$k(x) = \begin{cases} \frac{1}{(x-a)^2} & a \le x < b \\ 0 & x = b \\ \frac{1}{(c-x)^2} & b < x \le c \end{cases}$ 

Integrated Structural Health Monitoring in steel arches bridges using continuous dynamic monitoring: two case studies in China

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.5

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Abstract: Structural health monitoring (SHM) is going to be a standard tool for bridge management as a decision support for maintenance. Different strategies have been proposed in literature, using different devices and data analysis tools. This paper shows how dynamic monitoring system should be an interesting support for maintenance in steel arch bridges subject to marine environmental condition. The case study deals with two steel arch bridges that are fundamental lifeline connections in the island of Xiamen, China. The monitoring system consists in an integrated data analysis from different sources, such as vibrations from accelerometers, strain from linear strain gauge (located on the rigid suspenders, on the vault and on the cross-section of the arch rib of the bridges), and finally environmental conditions considering temperature and humidity. The paper presents results obtained using data stored in a long period under different climate and load conditions, and how they should be used for a reliable bridge management.

Keywords: structural health monitoring, bridge maintenance, data analysis, decision support

1 Introduction

In the field of structural engineering, there is a recognized need to monitor the health of civil infra-structures. In fact, in the last years the development of heavy traffic on bridges, often structurally deficient or functionally obsolete, has determined an increase of dynamic effects. The control of safety and comfort of bridges needs dynamic testing, an effective technique that can provide useful information for calibrating numerical models, evaluating simulation strategies, retrofitting and maintenance.

Structural Health Monitoring (SHM) is a non-destructive direct technique that allows a continuous or regular measurement of structural and ambient parameters, which includes input tests and ambient tests [1,2,3]. In general, tests with measured inputs are conducted on small bridges, while in other cases output-only tests, based on ambient vibrations (generated by wind or traffic), are the only practical and economic way for exciting the structures without disturbing their normal operation. The main goal of SHM is often (a) the experimental evaluation of modification in structural features, checking the evolution as consequence of a progressive damage; (b) identification of structural parameters to check and update numerical models; (c) finally to evaluate the safety or the serviceability of existing structures, quantifying entity and location of damage [4].

The bridge management presents several critical issues that hamper the effective planning and scheduling of the necessary strategies to the preservation of service (traffic flow). Among these problems, both environmental (wind, earthquake) and anthropogenic vibrations caused by daily human activities (building construction, vehicular and rail traffic) should be taken into consideration.

For this reason, decision support by structural monitoring is a fundamental tool for the correct managing of bridge maintenance. The correct meaning of structural monitoring is to measure a set of physical parameters at specific time intervals and allow you to follow the evolution of structural conditions over time.

One of the main parameters to be analyzed in bridge structural monitoring is the vibratory phenomenon, which can be characterized by having a large or a small amplitude. While the large amplitude vibrations can cause extensive damage due to the severe dynamic loads, the small amplitude does not represent an imminent danger, but can affect the structural strength, especially in the bridges that have already been subjected to important dynamic actions. These issues need to be studied so as to evaluate the effects of vibratory phenomena on the bridges.

Moreover, as an index of structural performance, vibration response of the bridges is strongly influenced by environmental conditions. So, an integrated monitoring system should use other sensors except accelerometers to deal with environmental parameters, such as temperature and wind velocity.

According to integrated analysis of those data, two types of approaches can be followed to evaluate the effects of vibrations on bridges: one refers to mathematical modeling that can analyze various load scenarios and the other is structural monitoring based on experimental methods. Briefly, the necessary tools for structural monitoring consist of a distributed sensing subsystem of facilities and a data acquisition subsystem able to collect data from sensors.

Regarding mathematical modeling, it presents critical issues related to its effectiveness because it strongly depends on the assumptions on which it is founded. . Thus, the approach based on experimental methods is more attractive because they can be used as a support for the calibration of numerical models, as well as to provide direct information about the negative effects of vibration.

Different from the above approaches, in the present study output-only SHM is adopted for the evaluation of vertical traffic-induced vibrations on two very peculiar arch bridges.

Finally, it is worth noting that the effects of moving loads on bridge structures can be investigated by means of powerful computers and advanced numerical methods [5,6]. This approach is based on numerical simulation of bridge-vehicle interaction and also because its complexity can unlikely be adopted for practical purposes. In this context, Structural Health Monitoring can become a fundamental tool to assess the serviceability conditions of existing bridges [7,8,9,10,11,12,13]. Output-only test, based on ambient vibrations (generated by wind or traffic), are in fact the only practical and economic means for exciting the structure without disturbing their normal operation.

This paper describes real applications of an integrated bridge monitoring mixing dynamic and non dynamic sensors to support the management of maintenance in two relative important bridges in sea environmental conditions, which connect the island of Xiamen to the mainland in China. The monitoring system involves integrated data analysis from different sources, such as vibrations from accelerometers, strain from linear strain gauge (located on the rigid suspenders, on the vault and on the cross-section of the arch rib of the bridges), and finally environmental conditions considering temperature and humidity. Preliminary results obtained using data stored in a long period under different climate and load conditions are presented as well as how they should be used for a reliable bridge management.

2 Description of monitoring systems

Tianyuan Bridge and Wuyuan Bridge are located in the island of Xiamen, China, crossing the Wuyuan Bay. So, they are located over sea and close to the coast, which constitute conditions suitable for corrosion and thus cause section losses. Also, wind forces are significantly higher in this coastal region. To improve their maintenance and predict possible problems ahead of time, continuous monitoring of these structures can be considered as a promising approach.

2.1 Tianyuan Bridge

Tianyuan Bridge is a half-through steel arch bridge with a span of 120m and width of 32m, as shown in Fig. 1. The arch axis is parabolic. The single box arch rib is located in the middle of the deck girder. The deck is connected with the arch rib by 14 rigid suspenders with a spacing of 6m. At the intersection, the deck girder is seated at two short spandrel piers over the arch rib. Another two piers support the deck girder at the two ends.

Figure 1 : Tianyuan Bridge (behind is Wuyuan Bridge)

In this real-life bridge monitoring application, the instrumentation plan is designed to monitor the most critical structural components. The current installation consists of more than 100 sensors and summary of the sensors used in SHM system is shown in Table 1. Fig. 2 shows the positions of these sensors.

Table 1. Sensors deployed for SHM of Tianyuan Bridge

No.	Parameter		Sensor type	Amount
1	Environmental effect	Temperature	Thermometer	4
		Humidity	Hygrometer	4
2	Loading sources	Weigh-in-motion system		6
		Traffic condition	Video Camera	2
3	Structural responses	Tension in hanger	Strain sensors	20
4		Deflection at arch crown	GPS	2
5		Vibration in deck girder	Acceleration sensor	30
6		Strain in arch rib	Dynamic strain sensor	16
7		Strain in deck girder	Dynamic strain sensor	42
Total				126

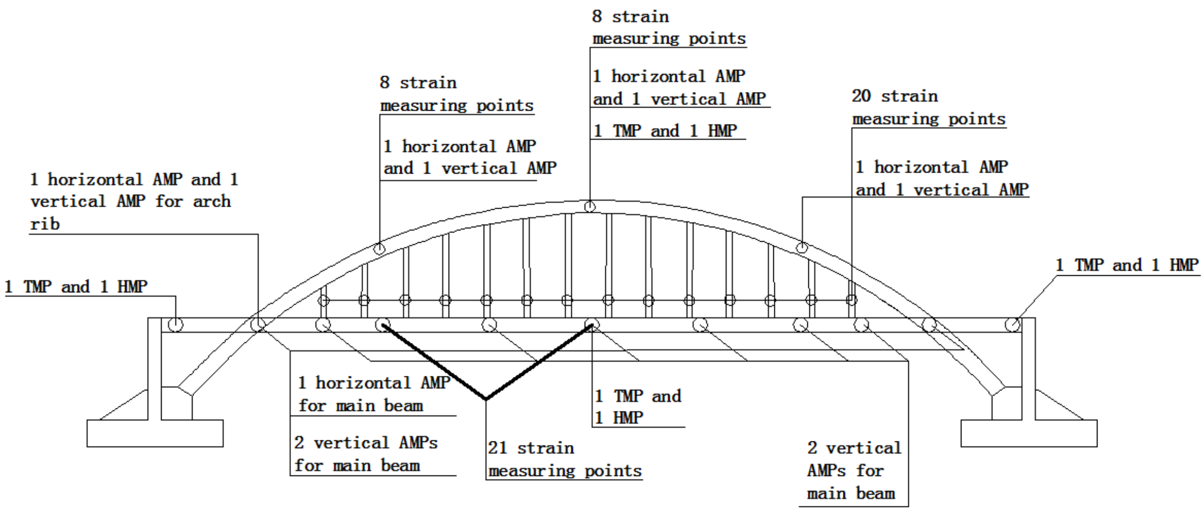


Fig. 2 Arrangement of sensors.

(TMP - temperature measuring point; HMP - humidity measuring point; AMP - acceleration measuring point)

2.2 Wuyuan Bridge

Wuyuan Bridge is a half-through steel tied arch with a main span of 208 m and two side spans of 58 m, as shown in Fig. 3. The deck system is steel-concrete composite girder. The SHM system devised for Wuyuan Bridge consists of four modules, namely, Module 1 - Sensory System (SS), Module 2 - Data Acquisition and Transmission System (DATS), Module 3 - Data Processing and Control System (DPCS), Module 4 - Structural Health Data Management System (SHDMS). The SS and DATS modules are distributed inside the structure, while the DPCS and SHDMS modules are placed in the monitoring and control room.

Fig. 3 Wuyuan Bridge

The SS module is composed of about 134 sensors in six main types, as listed in Table 2. These sensors are deployed for monitoring of three categories of parameters: (i) loading sources, (ii) structural responses (strain, acceleration, displacement and geometric configuration), and (iii) environmental effects (temperature and humidity). The arrangement of the sensors are shown in Fig. 4.

Table 2. Sensors deployed for SHM of Wuyuan Bridge

No.	Parameter		Sensor type	Amount
1	Environmental effects	Wind speed and direction	Ultrasonic anemometer	1
		Temperature	Thermometer	4
		Humidity	Hygrometer	4
2	Loading sources	Traffic condition	Camera	2
4	Structural responses	Displacement at arch crown	GPS	1
5		Tension in hanger	Acceleration sensor	20
6		Vibration in arch rib and deck girder	Acceleration sensor	30
7		Strain in arch rib	Dynamic strain sensor	22
8		Strain in deck girder	Dynamic strain sensor	50
Total				134

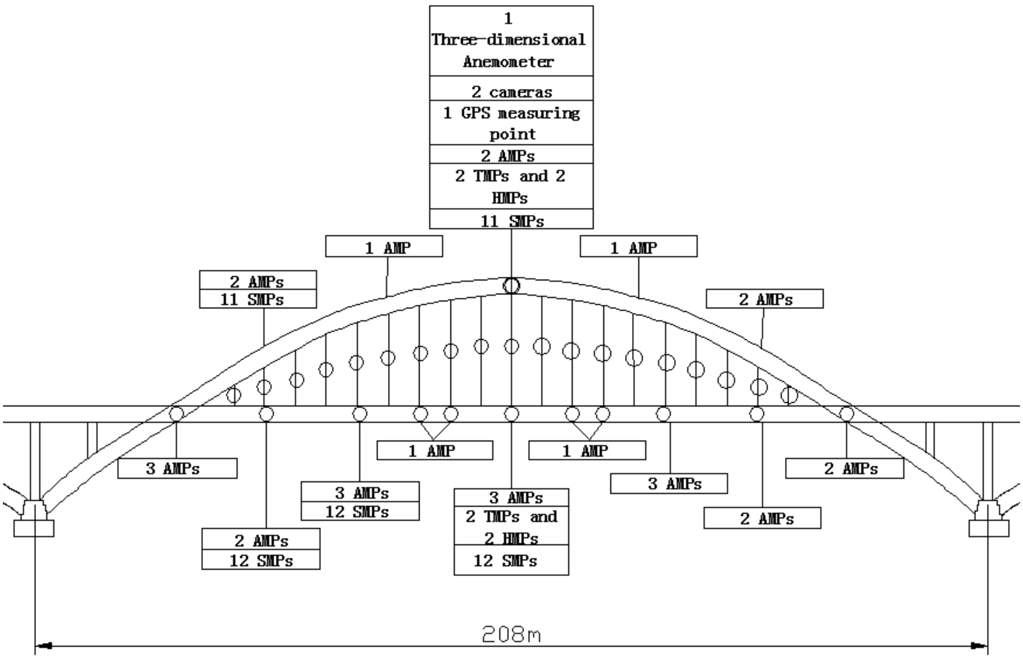


Fig. 4 Sensor arrangement for Wuyuan Bridge

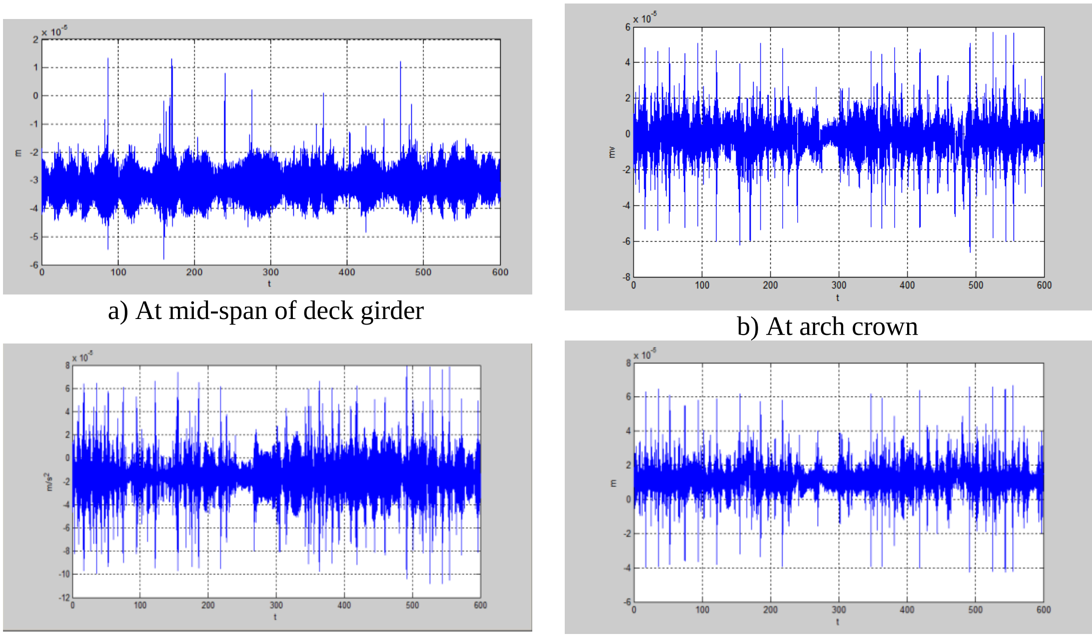
(TMP - temperature measuring point; HMP - humidity measuring point; AMP - acceleration measuring point; SMP - strain measuring point)

3 Preliminary results from dynamic monitoring

3.1 Strain

The basic principle of strain-based structural health monitoring is that changes in the physical properties of a structure will cause changes in the amplitudes of strain measurements. One significant advantage is that this approach is able to detect and localize damage by analyzing time-domain strain measurements from bridges. Also, the strain-based SHM for damage detection allows the ease of data collection and the flexibility in data analysis when ambient traffic crosses a bridge. The system, having a network of fiber optic sensors, is able to autonomously and continuously collect and manage strain data through wireless communications. The strain data collected from ambient traffic on the bridge are used to extract the quasi-static live-load response.

Based on the finite element analysis of Tianyuan Bridge, the critical sections in the arch rib and deck girder are taken as the concerning section, and totally 78 positions are installed with strain sensors, as shown in Fig. 4. Fig. 5 shows the strain time history at some measuring points. It is demonstrated that within the 10 min time range, the maximum strain reached 125.6uε at deck girder, 57.13uε at arch crown, 79.13uε at the longest suspender and 66.5uε at the shortest suspender.



a) At mid-span of deck girder

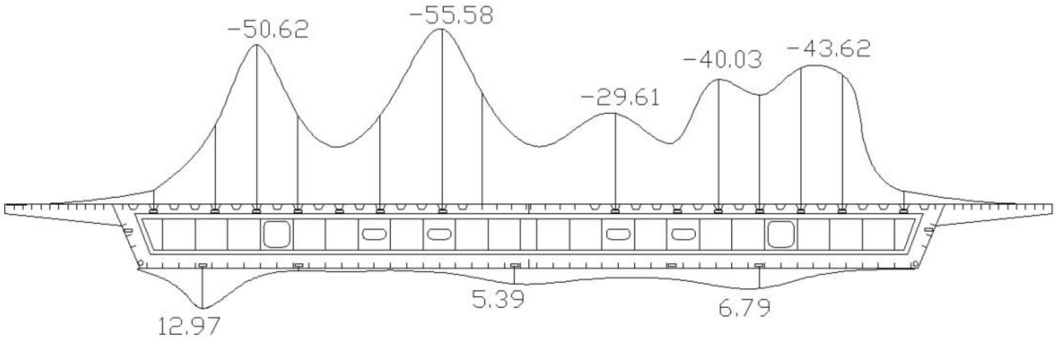
b) At arch crown

c) At the longest suspender

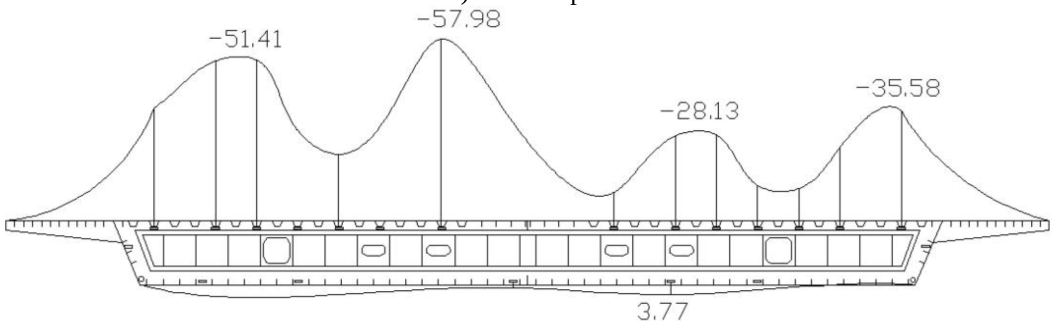
d) At the shortest suspender

Fig. 5 Strain time history at some measuring points

At the mid-span and quarter span of the deck girder, there are totally 21 measuring points for each section. When all the monitoring data at these points are extracted within a time range, the average values can be calculated. Then, the distribution of the strain along the section can be obtained, as shown in Fig. 6. Fig. 7 shows the strain distribution along the mid-span and quarter span section of the arch rib.

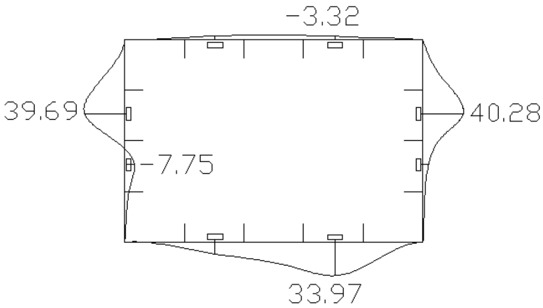


a) At mid-span

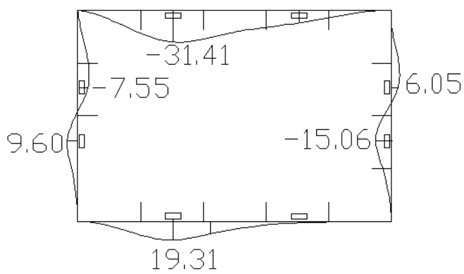


b) At quarter-span

Fig. 6 Strain distribution along the section



a) At arch crown

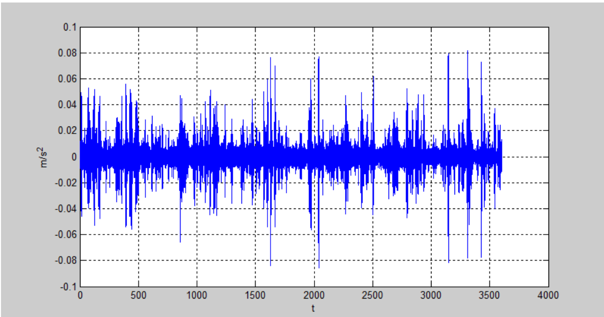


b) At quarter span

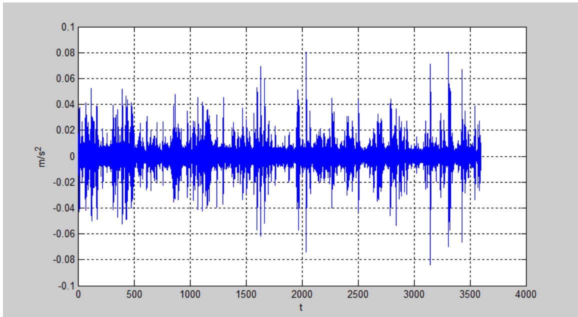
Fig. 7 Strain distribution along the section of arch rib

3.2 Acceleration

Fig. 8 shows the acceleration time history within one hour at mid-span of deck girder.



a) Vertical



b) Transverse

Fig. 8 Acceleration time history at mid-span of deck girder

4 Conclusions

As well known, the aims of adopting a structural health monitoring (SHM) system for bridges are related to damage identification, structural safety evaluation, and maintenance decision making. This paper presents the set up and the implementation of an integrated structural monitoring system in Tianyuan Bridge and Wuyuan Bridge in Xiamen, China. Due to severe marine environmental conditions, an integrated SHM is necessary for bridge management and maintenance. The monitoring system consists in an integrated data analysis from different sources, such as vibrations from accelerometers, strain from linear strain gauge and environmental conditions considering temperature and humidity. Through a long period of structural monitoring under different climate and load conditions, the analyzed SHM systems generate time-specific status information such as bridge vibrations, strain distribution in several cross-sections, displacements, stresses in the hangers and so providing data support for bridge maintenance and decision making, which reduces the maintenance cost and improves the technical level of long-term management.

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Optimal sensor placement for Spatial Variability Assessment of Structures

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DOI: https://doi.org/10.5592/CO/BSHM2017.5.6

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Abstract. Structural reliability assessment is largely influenced by the spatial variability of material properties or defaults; however, there are still various challenges for their characterization and modeling. Structural Health Monitoring (SHM) could provide useful information in space and time for spatial variability characterization of material properties and mechanical solicitations; nevertheless, this challenge is arduous because of the large number of potential sensor positions of local disruptions/failures. This paper proposes a methodology to optimize the spatial distribution of embedded sensors used for spatial variability assessment of stationary random fields. The optimization criterion relies on the width of the confidence interval of statistics for the characteristics to identify. For sake of simplicity, the paper illustrates the method for one-dimensional problems. The proposed method is applied firstly to a numerical example were several hypothetical structural configurations that could be found in practice are studied. It is finally applied to two case studies (a reinforced concrete beam and a steel wharf) where water content and loss of steel thickness are respectively measured. The results show that the stationary property is useful to deduce the minimum quantity of sensors and their position for a given quality requirement. They also allow us to propose a criterion for defining if regular or non-regular spacing of sensors along the inspection zone is more appropriate depending on the component length and autocorrelation structure of the random field.

Keywords: spatial variability; confidence interval; inspection optimization; stationary field; sensor spacing; Structural Health Monitoring

1 Introduction

Structural serviceability and safety are influenced by the different sources of uncertainty involved during their whole lifetime: material properties, loading, measures, model, deterioration, etc. A probabilistic structural analysis that includes the more influential uncertainties is therefore paramount to minimize both failure risks and design and maintenance costs. Nowadays, there are significant advances in probabilistic modeling at the scale of a single section of the structure. However, various works have demonstrated that the reliability assessment for a given component is largely influenced by the spatial variability of material properties or defaults [Stewart, 2004; Li et al., 2014; Li, 2004; Srivastava, 2012; O'Connor & Kenshel, 2013; Griffiths & Fenton, 2000; Pasqualini et al., 2013]. Although the consideration of spatial variability is essential for proper reliability assessment, there are still various challenges for their characterization and modeling.

Non-Destructive Testing (NDT) and Structural Health Monitoring (SHM) could provide useful information in space and time for spatial variability characterization of material properties and mechanical solicitations. Several studies focused on the use of NDTs for spatial variability characterization at a given time. For example, Nguyen et al [Nguyen et al., 2013, 2014] combined several NDT techniques, kriging and variograms to assess the spatial variability of concrete at different scales (point, local and global). Gomez-Cardenas et al. [Gomez-Cardenas et al., 2015] proposed a two-step approach to optimize the number and position of ultrasound measures required to localize critical zones. More recently, Schoefs et al. (2016) proposed a methodology to find an optimal inspection configuration (number and localization of NDT measures) that minimizes the error of identification of probability distributions for a given quantity of interest (resistance, porosity, water content, etc.) with spatial dependency. SHM could be more useful for characterizing the evolution in time of this spatial variability. Most part of research efforts in SHM have focused on the spatial localization of defects or damage of structural components [Hu et al., 2015; Kuprapha & Warnitchai, 2012]. However, spatial variability characterization of loading or material properties from SHM data is still a challenge because of the finite number of sensors and the large number of potential positions of local failures or disruptions. Numerical algorithms and specific multi-sensor systems should be developed towards this aim.

Within this framework, the main objective of this paper is to propose a methodology to optimize the spatial distribution of embedded sensors used for spatial variability assessment of stationary random fields. Stationary random fields have a stochastic structure and probabilistic properties that could be used to provide rational aid tools for optimizing the number and location of sensors. The assessment of the shape of the Auto-Correlation Function (ACF) is paramount for this spatial variability characterization.

The paper starts in section 2 with a review of key concepts of spatial random field modeling with a focus on stationary random fields. Section 3 describes the proposed method for optimal sensors positioning in order to characterize the spatial correlation structure, illustrated with numerical examples and real measurements on a reinforced concrete (RC) beam (section 4).

2 Main assumptions for the stochastic modeling

In order to simplify the presentation of the proposed methodology, we consider the following main assumptions about the sensors and the random field modeling:

- The stochastic field is considered as: Gaussian, second order stationary, statistically homogeneous and its marginal distribution is known. This assumption implies that less information is required for its characterization.
- Each realization (single trajectory) represents the probabilistic information of all trajectories: mean, variance, spatial correlation. A single trajectory is then sufficient to describe the spatial variability, i.e. the stationary field is ergodic.
- A larger number of discrete sensors can be placed over the same component to characterize both randomness and spatial variability (e.g., from 20 to 60). We consider in this paper long structures with a significant number of sensors (around 200).
- Sensor measurements are considered as 'perfect' according to the definition provided in [Schoefs et al., 2009].
- Damage is not affected by the loading or that the effect of loading on the spatial variability can be modelled.

  Extensions are developed in Schoefs et al. (*under review*).

Several approaches can be used to describe a stochastic field $Z(x,\theta)$ where x denotes the position and θ the hazard: Karhunen-Loève expansion, approximation by Fourier series, EOLE approximation, etc.. This paper uses a Karhunen-Loève expansion to model the stochastic field $Z(x,\theta)$. This expansion represents a random field as a combination of orthogonal functions on a bounded interval.

3 Sensor placement strategy and goals

3.1 Definition of the Spatial Correlation Threshold

The paper focuses on the assessment of the ACF (exponential for example in Eq. (1)) of a stationary field. An optimal geo-positioning of sensors along a trajectory (sampling of the random field) should provide an accurate assessment of the ACF parameters (i.e. b in Eq. (1)) with a limited number of sensors.

$$\rho(\Delta x) = exp\left(\frac{-\Delta x}{b}\right), 0 < b \quad \text{and} \quad \Delta x = x_1 - x_2 \tag{1}$$

This exponential ACF will serve r numerical simulations in the following. When looking for the usual shape of a correlation function a regular spacing of sensors could not be optimal. If the distance between two sensors L_b is large, the decay of autocorrelation for short distances cannot be assessed. On the contrary, if L_b is small, there is some information provided for many sensors that will not be useful for the assessment of b. Figure 2 shows that it is possible to install different number of sensors for high and low autocorrelation zones to obtain a good assessment of the autocorrelation parameter by reducing the total number of sensors Ns. The objective is to get a spacing of sensors providing a larger amount of data in the zone of high correlation. However, there is a limited feedback on the autocorrelation function (and consequently the value of b) for defining clearly the high autocorrelation zones. Section 4 presents a sensitivity study about the influence of the a priori knowledge of b.

Let us consider a one-dimensional spatial field. The methodology could be applied on a set of trajectories representing: (i) a set of 1D components (beams), or (ii) a very long 1D-component subdivided artificially or physically (expansion joint or construction joints) in a set of short components, or belonging to a wall structure (steel sheet pile or concrete wall).

In order to limit monitoring costs (number of sensors), we propose to monitor some zones of a trajectory with sensors separated by "sufficiently short distance L_b" allowing us to assess the shape of the ACF (Eq. (1)) that is controlled by the parameter b. This “sufficiently short distance” can be seen as an Inspection Distance Threshold (IDT). Thus the non-regular distances of sensors spacing L_b^i should satisfy: $L_b^i \in]0, \text{IDT}[$ in the highly correlated zones.

The IDT is defined by assuming that, after a given distance, the events measured from an inspection can be assumed as weakly correlated. A Spatial Correlation Threshold (SCT) defines this weak correlation. For instance, Schoefs et al. (2016) proposed a value SCT = 0.3 to get fairly correlated events and SCT = 0.5 to get high correlated events. For an exponential ACF, the SCT is linked with IDT by:

$$\text{IDT} = -b \cdot ln(\text{SCT}) \tag{2}$$

This paper considers the value SCT = 0.4 to determine IDT. For example, for this value of SCT and b = 1.0 m, IDT = 0.67 m. The effect of this choice is discussed in [Schoefs et al. 2016].

3.2 Parametrization of non regular spacing

In view to reduce the set of potential solutions and simplify the design of the network of sensors, we propose a parameterization. It is based on a division of the trajectory (structural component) into n_p pieces of same size L_m and then a subdivision of each piece into a decreasing N_c^i number of equidistant sensors, with distance L_b^i, following a series according to the octree approach. This approach has the advantage to get more information (more sensors) for small distances between points where the slope of the auto-correlation function must be fitted accurately. The number of sensors in the first piece is computed by:

$$N_c^1 = \text{Round}\left(\frac{N_s}{1 + \sum_{i=2}^{n_p} \frac{1}{2(i-1)}}\right) \tag{3}$$

The number of sensors for the pieces $N_c^2, \dots, N_c^{n_p-1}$ (i.e., N_c^i with $i \in [2;n_p–1]$) is estimated from:

$$N_c^i = \text{Round}\left(\frac{N_c^1}{2(i-1)}\right) \tag{4}$$

The number of sensors for the last piece, $N_c^{n_p}$, is the remaining number of sensors. Knowing the number of sensors in each piece, the distance between sensors in each piece is deduced. To satisfy the condition of sufficient correlation between measurements, we should avoid a distance larger than IDT for the pieces located in the high correlation zone. The length of this zone L_{hc} depends on the autocorrelation parameter b and could be estimated from Eq. (1) by considering a low value of autocorrelation. For example, for b=1m $L_{hc} \approx -b \ln(0.01) \approx$ 4.6m. However, if $L_b^i > IDT$ for the pieces located in this high correlation zone, the total number of sensors, N_s, should be increased until ensuring this condition for a given number of pieces.

3.3 Parameter estimation, sensitivity analysis and optimization

Stationary stochastic fields are simulated by the Karhunen-Loève expansion assuming an exponential ACF (Eq. (1)), whose parameter b has to be identified by knowing the two first statistic moments (μ_Z, σ_Z). Based on a continuous trajectory, for fixed values of N_s and n_p, we obtain a sample of discrete realizations from the sensor measurements $\hat{Z} = \{z_1, z_2, \dots, z_{N_s}\}$ corresponding to the sensors positions $X = \{x_1, x_2, \dots, x_{N_s}\}$ following the discretization procedure presented in previous section. We assess the value of b by using the Maximum Likelihood Estimate method (MLE), reported by Li (2004). To account for the effect of random shape of trajectories, the analysis is carried out over a database containing 10,000 trajectories generated by Monte-Carlo simulations. This allows estimating 10,000 values $\hat{b}$ for each distribution of the sensor – i.e. one set of the couple (N_s, n_p). We select in this paper a confidence interval of the mean $\mu_{\hat{b}}$ expressed as a percentage Δ of the theoretical (true) value b^{th} to evaluate the quality of the SHM. From the 10,000 Monte-Carlo simulations we estimate the bounds of the confidence interval and the probability $P_{I,b}$ to get values inside the confidence interval, from the monitoring data. In a reliability study, $P_{I,b}$ will be discussed according to the requirements on the accuracy of the probability of failure assessment [Stewart, 2006]. Thus we focus on the quality estimator:

$$P_{I,b} = P(\mu_{\hat{b}} \in [(1-\Delta)b^{th}, (1+\Delta)b^{th}]) \tag{5}$$

where $\mu_{\hat{b}}$ is the mean value of $\hat{b}$ computed from 10,000 Monte-Carlo simulations. We define another estimate ε_b, the normalized quadratic error of the parameter $\hat{b}$:

$$\varepsilon_b = \left(\frac{\hat{b} - b^{th}}{b^{th}}\right)^2 \tag{6}$$

Finally, the optimal position and number n_p^{opt} of sensors is obtained by:

$$n_p^{opt} = \underset{n_p}{Arg\max}\{P_{I,b}(N_s)\} \tag{7}$$

4 Numerical simulations and real study case

4.1 Application to a numerical study case

For illustrating the methodology and generalization purposes, it is considered in the following sections a set of 1D-components (beams) with a very large total length L>>b. The case of components with a limited size is discussed in (Schoefs et al., *under review*) excepting those where L < Lb for which it is theoretically impossible to identify fully the stochastic field. The Gaussian stationary stochastic field is characterized by: b_{th}=1m, IDT=0.91m from Eq. (2), μ_Z = 100 and σ_Z = 20. The objective is to optimize the position of sensors in view to reach a good assessment of b for an error Δ = 10%. We first analyze the effect of the number of pieces (n_p) on the quality of assessment defined according to Eq. (5) for a large number of sensors N_s and large length L; namely N_s=200 and L=100m. We vary the number of pieces from 1 (200 sensors equally separated by the distance L_b = IDT) to 20 (72, 36, 18, 12, 9, 7, 6, 5, 5, 4, 4, 3, 3, 3, 3, 2, 2, 2, 2, and 2).

Figure 1a presents the evolution of the quality estimator ($P_{I,b}$) with n_p for 10,000 simulated trajectories.

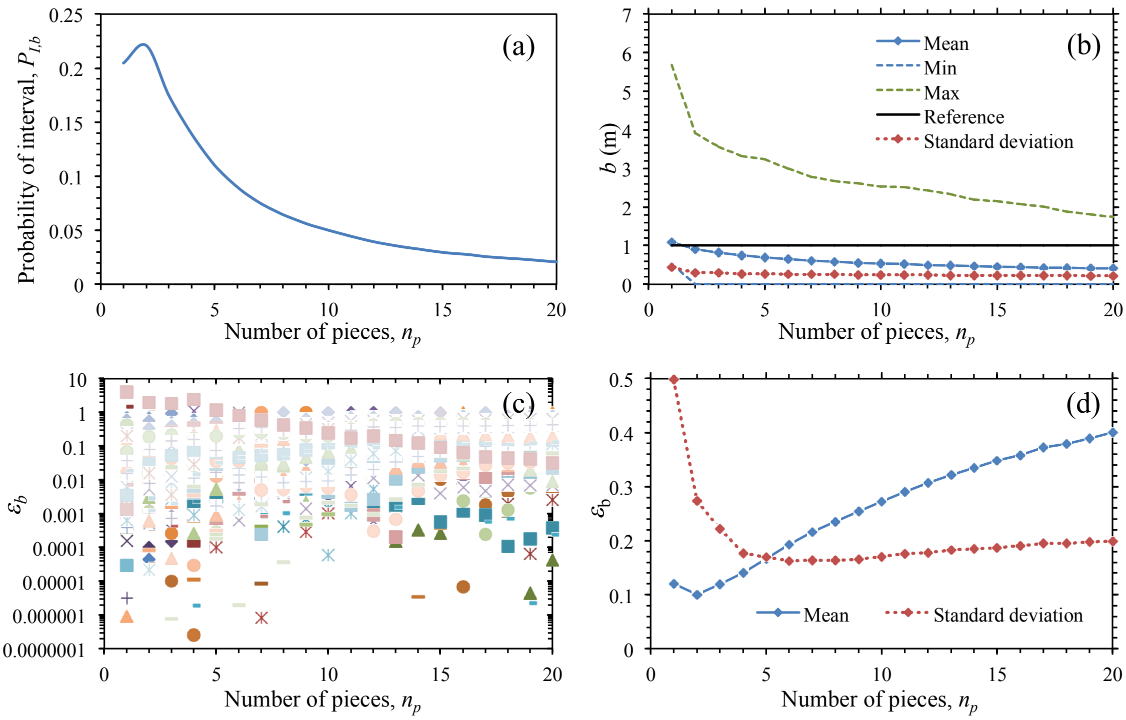

Fig. 1. Effect of number of pieces n_p on:
a - the probability of interval $P_{I,b}$; b - the estimated values of b
c - the distribution of ε_b; d - the mean and standard deviation of ε_b (N_s=200, L=100m and Δ=10%)

The regular spacing obtained for n_p=1 is shown to be not optimal whereas the optimum is found for n_p=2 with 133 sensors spaced 37.8 cm in the first piece of 50m and 67 sensors with spacing equal to 74.6cm in the second piece. Figure 1 presents other results to improve the understanding of the causes of this trend. Figure 1b plots the evolution with n_p of the two first statistics (mean and standard deviation) and the minimum and maximum values obtained for b from a sample of size 10,000. It is observed that the mean value decreases slightly with n_p and becomes stable with a significant bias in comparison to the reference theoretical value (1m). This means that identification algorithm underestimates the value of b. Thus, even if the standard deviation decreases with n_p, $P_{I,b}$ is not optimal for high values of n_p. Note that the maximum and minimum bounds are not symmetrical to the mean; that is due to the non-symmetrical distribution of b for a fixed sensor distribution. Figure 1c presents the potential relative error ε_b that can reach 4.8 (near 500%) for one realization upon 10,000. The results on Figure 1d show the mean and standard deviation of ε_b and confirm that the error on the mean governs the level of the quality estimator $P_{I,b}$ where the minimum value of the mean error is obtained for n_p=2. Figure 1d also indicates that there is a significant reduction of the standard deviation of the error from n_p=1 to 2.

We focus now on the effect of small perturbations around the value of b on the optimal solution. This is a key issue for studying the robustness of the solution obtained from an *a priori* value of b. This sensitivity analysis studies the effect of b on the error ε_b by assuming that b takes the following values around the reference one (i.e., b=1m): 0.8, 0.9, 1, 1.1, and 1.2 m (Figure 2).

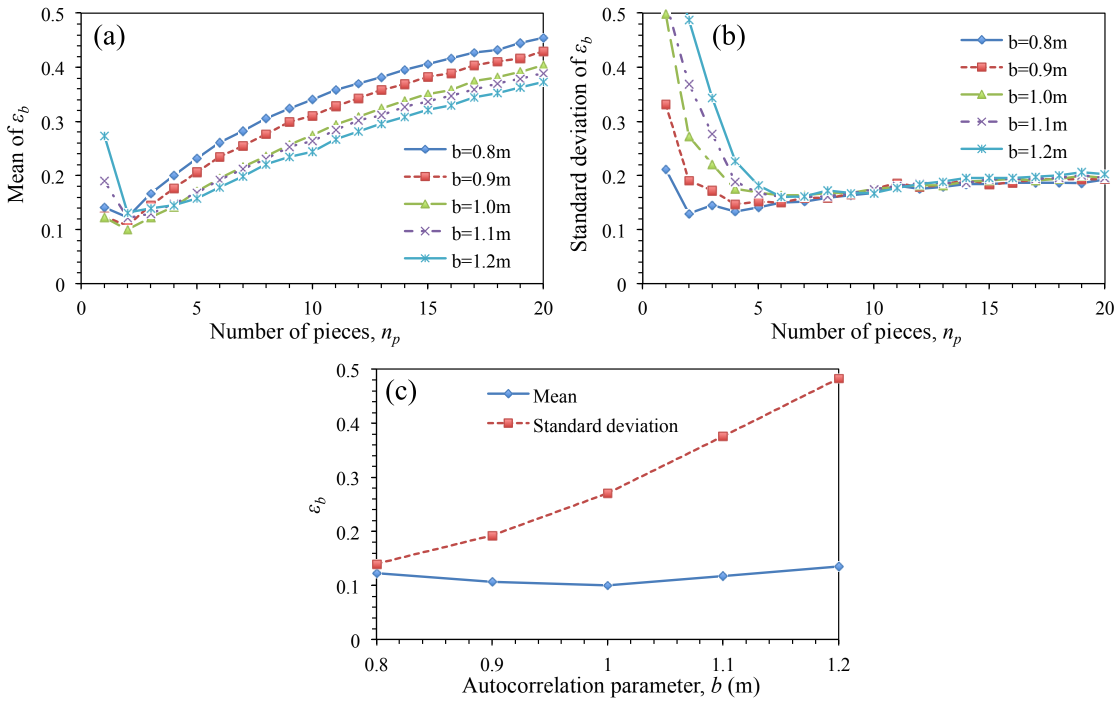

Fig. 2. Sensitivity of ε_b for:
a - the mean of ε_b; b - the standard deviation of ε_b
c - the mean and standard deviation of ε_b for n_p=2 (N_s=200, L=100m)

Figure 2a plots the mean of the error ε_b for various values of n_p and b. It is found that the minimum error corresponds to $n_p = 2$ for all values of b. Figure 2b shows that the standard deviation of the error is sensitive to b for $n_p = 2$ and that leads to a given value for $n_p > 5$. Figure 2c presents the mean and standard deviation of the error for $n_p = 2$. It is noted that the error on the mean is almost constant with a minimum for b=1m but the error on the standard deviation increases with b. It is possible to conclude from this trend that under-estimating slightly the value of b reduces the error.

The cases of small structures or small number of sensors are available in Schoefs et al. (*under review*).

4.2 Illustration for a real study case

The authors applied the proposed methodology and previous findings to two study cases for which real spatially distributed data are available (Schoefs et al., *under review*). We consider in this paper only one of them, the inspection of the water content along a 16m length reinforced concrete beam placed on the site of IFSTTAR Laboratory, Nantes, France [Schoefs et al., 2016]. The measurements were carried out by using a capacitive NDT tool.

Figure 3a presents the spatial measurements (trajectory) of the water content (RC beam). Mean and standard deviation are: $\mu_W = 6.3\%$, $\sigma_W = 0.67\%$ for the RC beam (computed from a sample of 80 measures every 20 cm).

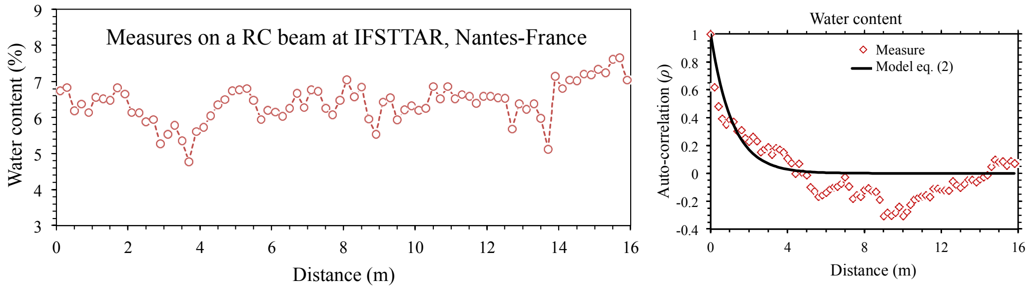

Fig. 3. a- Experimental trajectories of water content; b- auto-correlation data and fitted ACFs for water content in a RC beam (IFSTTAR, Nantes, France)

Figure 3b presents the computed autocorrelation values. We obtain a classical shape including negative values [henari & Dodaran, 2010]. Applying the procedure described previously it was found that the exponential correlation functions are appropriate to model the empirical values. The following parameter of the autocorrelation function (Eq. (1)) were estimated: $b_W = 0.42$m assumed in the following results and discussions as the theoretical value. Based on the fitted auto-correlation functions, the Inspection Distance Thresholds (IDT) is: $IDT_W = 0.5$m. Taking into account the length of the structural components and the findings of Schoefs et al. (*under review*), we propose a regular spacing of measurements for the RC beam because the ratio $IDT_W/L = 0.4/16 > 1/40$. In the following we compare real and numerical estimations to determine the appropriateness of the proposed sensor spacing in each case.

We estimate the errors on the identification of ε_b for both real data and simulations for the two study cases. Numerical simulations are based on: (i) the procedures to generate trajectories, and (ii) the values of mean, standard deviation and autocorrelation parameter identified upon. The main goal of this section is to validate the proposed numerical approach as well as to verify if the practical recommendations of the previous numerical findings could be applied to real measures.

Figure 4a compares the evolution of the error of the assessment of b by considering various n_p for results obtained from simulations and those computed from real data measured on the RC beam. The numerical mean as well as the minimum and maximum values were computed from 10,000 simulations.

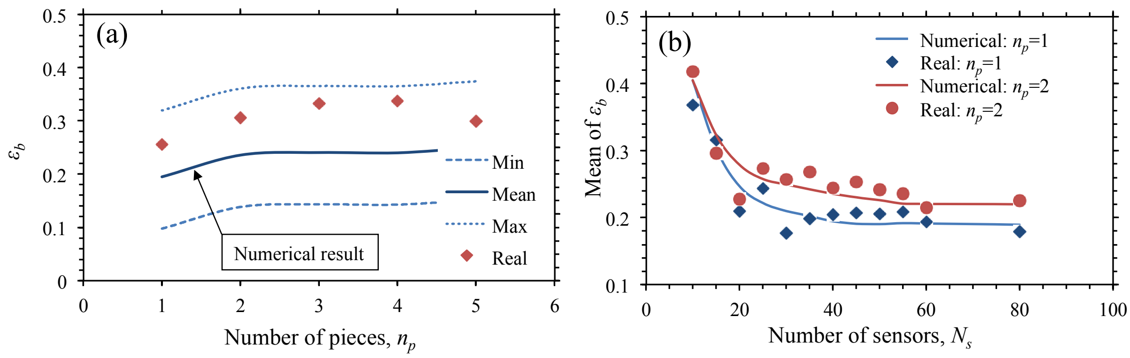

Fig. 4. Comparison between simulated results and real inspection values in the case of water content (RC beam, L =16 m, IDT=0.4m):
a - effect of n_p on ε_b, for N_s=40 sensors; b - effect of N_s on the mean of ε_b

The results show that the numerical and mean values are close and that ε_b is minimum for $n_p = 1$. This behavior confirms the recommendation based on the numerical findings for the case of $L <$ 40IDT where the regular spacing is suggested in such a case. Figure 4b compares the evolution of the mean of ε_b with N_s for n_p=1 and n_p=2. It is observed that the mean of ε_b decreases when more information from additional sensors is considered.

There is a convergence in the error that is faster when n_p=1; in such a case it is reached for N_s > 30 sensors. The results also show that a regular spacing with n_p=1 leads to lower error for both simulated and real data.

5 Conclusion

This paper proposed an original method for defining a non-regular spacing of sensors devoted to the assessment of the autocorrelation function parameter of stationary fields. The method is based on the probabilistic identification of the autocorrelation function parameter and aims at reducing the error on its estimation. Numerical simulations of Gaussian stationary stochastic fields illustrate the potential of the method by providing a decision aid tool when a limited number of sensors is available. Based on these numerical results, it was found that the position of sensors is a key factor for estimating the autocorrelation function parameter.

The paper shows also the important role of the position of sensors in the estimation of the autocorrelation function parameter on a real study case (concrete beam).

Acknowledgements

The authors would like to acknowledge the Pays de la Loire Region for supporting the projects ECND-PdL and SI3M as well as the European commission for funding the DuratiNet EC Interreg project (http://www.duratinet.org).

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6 TECHNICAL VISIT OVERVIEW

Bridge building across Sava River in Zagreb started in the end of the 19th century and up until now 13 bridges have been built on 11 different locations in Zagreb area, with more planned.

During this technical visit, participants have visited 5 bridges on 3 different locations – old composite Sava Bridge, railway steel Green Bridge, arch steel Liberty Bridge, prestressed girder Adriatic Bridge and extrados prestressed Homeland Bridge. These 5 bridges gave the most relevant overview of all the bridge building eras in Zagreb, each specific with its material, structural and architectural identity.

The Old Sava Bridge (1938) is the world first composite structure. Today, it is protected as a cultural heritage monument and serves as pedestrian bridge. The Sava Bridge was built on the piers of the old bridge which was moved to a new location. Superstructure is a continuous composite girder comprising to steel I sections distanced 5,8 m and a reinforced concrete deck. With the increase in traffic, Sava bridge was considered for widening and strengthening, but this idea was dismissed in favor of the new Adriatic bridge. Sava Bridge was however kept for exclusive pedestrian traffic and in 1975 placed under heritage protection program to deny any attempts in adapting or changing it. Today, it is in desperate need of rehabilitation.

The Green Bridge (1939) is the Langer beam, a through tied arch bridge with rigid deck and light arch. It is interesting that the City of Zagreb built the piers first, and afterwards announced the competition

Sava River Bridges in Zagreb included in technical visit: Old Sava Bridge, Green Bridge, Adriatic Bridge, Liberty Bridge and Homeland Bridge.

for the bridge design. Rehabilitation of the bridge was carried out in 2013-2014 in order to increase load capacity of the bridge. The column with fixed bearing was strengthened by concrete layer with depth of 60-105 cm and connected by steel dowels to older part of the column. The column foundation was strengthened by additional 24 pilots (Φ=1000 mm, L=25 m).

The Adriatic Bridge (1981) has been built as prestressed girder bridge with separated areas for vehicles traffic and streetcar railroad in order to meet increased traffic demands and replace the Sava Bridge. Most of the superstructure comprises prefabricated girders 39 m long, a concrete deck and cross girders. The largest span above the river was achieved by utilizing 12 m pier cantilevers comprising box cross sections. Together with the for mentioned prestressed girders placed in between these cantilevers, a 63 m span was erected.

Famous Liberty Bridge (1951) with steel arch serves as central Sava River crossing. It was designed and built as urban bridge including many originally designed details, but also plan of wider urban and transport interventions. The central 100 m long span above the river is achieved by twin steel box arch, extremely slender (section height of 100 cm) with the arch rise of only 7,4 m. Superstructure is a composite grillage comprising longitudinal and transverse girders 85 cm high and concrete deck 25 cm thick. Columns are steel sections filled with concrete.

The Homeland Bridge (2006) is contemporary externally prestressed extrados bridge. The bridge is built for road, street car, bicycle and pedestrian traffic, and infrastructurally for water supply and wastewater lines. Its location is strategically positioned as the eastern entry into the town and connection to Zagreb airport. Due to external prestressing of the main span, box height was maintained constant through the whole bridge. The height of the main deviator for external prestressing above the deck plate is only 16 m, which was preferred over cable stayed solution due to the vicinity of the airport.

Anđelko Vlašić & Marija Kušter Marić

References

A. Vlašić, J. Radić. Zagreb City and Sava River. In *Proceedings of the 4th CCC Scientific Symposium - Concrete Engineering in Urban Development*, pages 213-220, Zagreb, SECON HDGK, 2008.

J. Radić, J. Bleiziffer, M. Kušter. Liberty Bridge – Central Sava River Crossing in Zagreb. In *Proceedings of the 4th CCC Scientific Symposium - Concrete Engineering in Urban Development*, pages 207-212, Zagreb, SECON HDGK, 2008.

7 WORKSHOP CONCLUSION & FUTURE PROGRESS

Zagreb Joint Workshop gathered researchers and practitioners having different areas of expertise and initiated discussions towards the same long-term objective - improvement of the bridge management leading to satisfied users and bridge operators, and sustainable development of European road network.

This e-Proceedings collects their experience and research activities in the area of the structural health monitoring, performance assessment, quality control, maintenance and management of bridges.

Exchange of experience and ideas, vivid discussions and different suggestions were elaborated through the event, particularly during the Closing Session of the workshop and Joint Steering Committee of both COST Actions TU1406 & TU1402 and IABSE WC1.

The following topics were identified in terms of possible further cooperation for joint progress:

– establish the unique terminology to have the same interpretation of terms and definitions within the comprehensive Glossary;
– application of the Value of Information methodology (available at www.cost-tu1402.eu) on Quality Control plan;
– sharing the COST Action TU1406 WG1 Performance Indicators database (available at www.tu1406.eu) and completion with data concerning the measurement type for each indicator and corresponding costs;
– identification and application of both Quality Control and Value of Information methodologies to a common case study (e.g. common bridge example with adequate collection of data);
– possibility of cross-sharing few interactive Short term Scientific Missions;
– shaping guidelines for end-users by developing clear procedures for application of background theories;
– revealing crucial research topics and initiation of joint HORIZON project proposals.

The first joint action of all three associations is already under progress. The special issue of the IABSE Journal Structural Engineering International was organised and announced during preparation of the workshop: *The Value of Health Monitoring in Structural Performance Assessment*. The main objective of this SEI issue will be to obtain an overview of the latest scientific advancements and professional activities in the area of health monitoring techniques and methods useful and valuable to be implemented in structural performance assessment strategies and further on in management systems of infrastructures. At the time of writing this *Conclusion*, 20 promising abstracts are accepted, which makes a very good start for a collection of manuscripts with innovative ideas and findings.

ANNEX A
PRESENTATIONS

TU1406
COST ACTION

IABSE

TU1402
COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Bayesian uncertainty quantification and sparse Bayesian learning for model updating in structural health monitoring

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02^{nd} – 03^{rd} March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
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Faculty of Civil Engineering

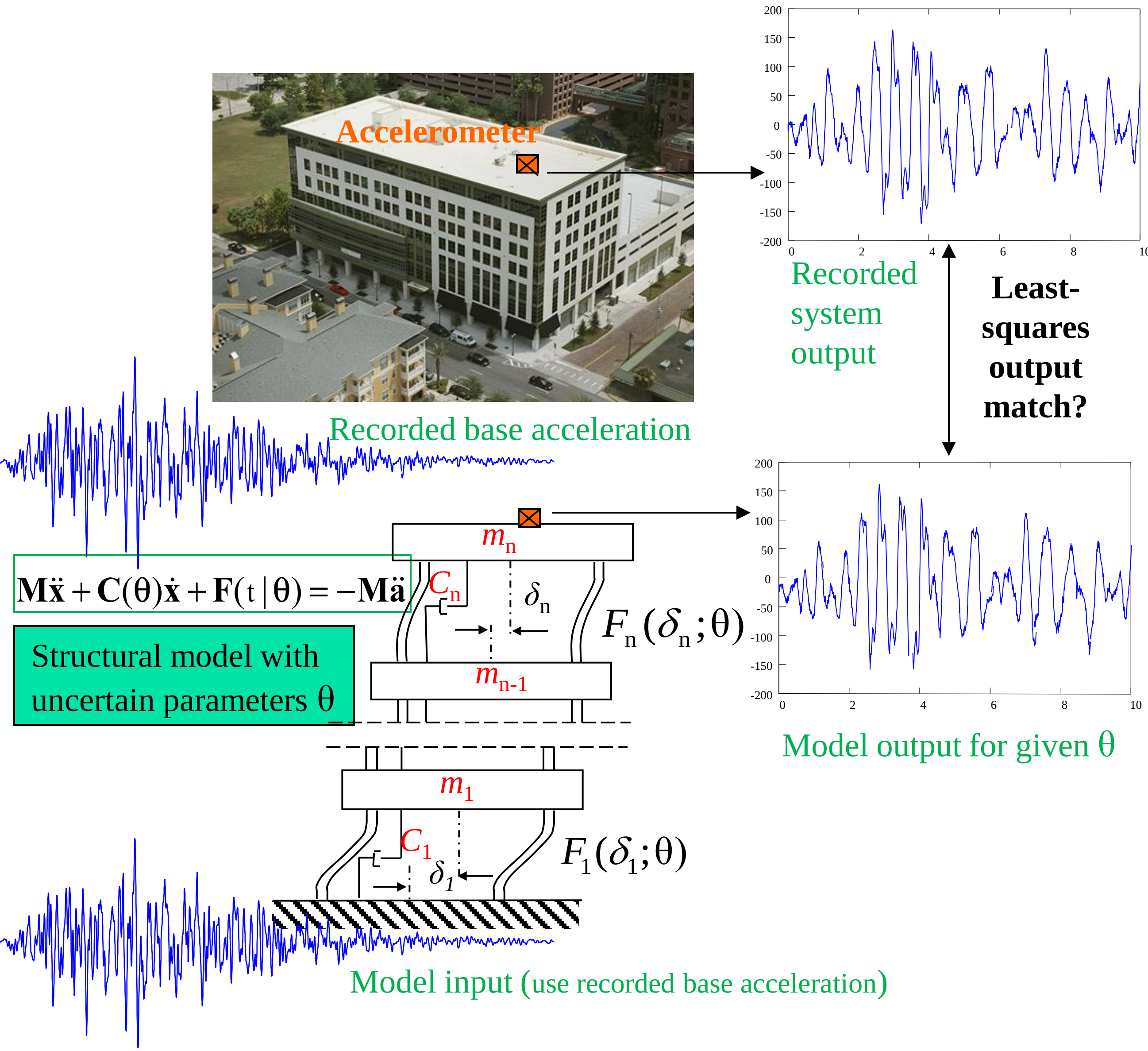
Focus of Presentation

- **Part I: Quantifying modeling uncertainty –** Overview of Bayesian probability approach where the probability of a model is a meaningful concept, with a special focus on the Bayesian Ockham Razor

  - Allows analysis that is robust to modeling uncertainties, both prior (or pre-posterior) (e.g. design based on reliability or life-cycle cost optimization), and posterior (e.g. system ID, structural health monitoring, state &/or parameter estimation, robust control)

- **Part II: Exploiting sparsity -** Use of prior information implying sparsity in function/vector expansions, especially SBL (sparse Bayesian learning) due to Tipping (2001)
  - Has been applied recently by us to Bayesian Compressive Sensing for wireless signals in SHM (Structural Health Monitoring) in CACAIE 2014, Prob. Eng. Mech. 2016
  - Will present our recent work applying SBL to improve ill-conditioned inverse problem of determining damage from changes in identified modal parameters of a monitored structure

System identification in SHM: Typical approach



Goal: Use system I/O data $\mathcal{D}$ to update models for system behavior to predict its response and current health status

Typical Approach: Propose a deterministic model with uncertain parameter vector $\theta \in \Theta$ and then **estimate** its value by using data $\mathcal{D}$ e.g. least-squares output-error, maximum likelihood or maximum a posteriori (MAP) estimates

System identification in SHM: Parameter estimation

- **Problem #1**: No model is expected to exactly represent a system's behavior – so **no true parameter values** to estimate!

- **Problem #2**: Parameter estimates are often **non-unique** (model is **unidentifiable** based on data $\mathcal{D}$) – fixing some parameter values to make unique estimates for the others may produce biased predictions

- **Problem #3**: Every model will have uncertain **prediction errors** (e.g. "unmodeled dynamics") – how can we quantify this uncertainty?

- **Resolution:** Use **Bayesian system identification*** for inference about **plausible system models** based on data, going beyond simple parameter estimation in order to make probabilistic predictions that are robust to modeling uncertainty

*J.L. Beck. Bayesian system identification based on probability logic. *Struct. Control & Health Monitoring* , 2010

Two prevailing interpretations of probability: Frequentist & Bayesian identification

Frequentist

Defn: Probability is the relative frequency of occurrence of an "inherently random" event in the "long run"

1) Looks rigorous but implied limit cannot be done

2) Probability distributions are inherent properties of "random" phenomena

3) Limited scope, e.g. no meaning for the probability of a model

4) "Inherent randomness" is assumed but cannot be proved (it may be just "hidden" information)

Bayesian

Defn: Probability is a measure of the plausibility of a statement based on specified information

1) Has a rigorous foundation as a multi-valued logic for quantitative plausible reasoning [R.T. Cox, 1946, 1961]

2) Probability distributions represent states of plausible knowledge about systems and phenomena, not their inherent properties

3) Probability of a model is a measure of its plausibility relative to other models in a set

4) Pragmatically quantifies uncertainty due to missing information; no claim that this is due to nature's "inherent randomness"

Probability (as a) logic: Rigorous foundation for Bayesian probability

- **Extends binary Boolean logic** to a multi-valued logic for quantification of plausible reasoning under incomplete information

- **Key idea**: **Probability** $\mathbf{P}[b|a]$ = measure of **plausibility** of statement b based on the information in statement a

  [$\mathbf{P}[b|a]$=1 if a is true implies b is true; =0 if a implies b is false]

- **Seminal work on foundations by R.T. Cox:**

  "Probability, Frequency and Reasonable Expectation", Amer. J. Physics 1946

  The Algebra of Probable Inference, Johns Hopkins Press 1961

- **Treatise on theory and applications by E.T. Jaynes:**

  Probability Theory – The Logic of Science, Cambridge U. Press 2003

Fundamental concept for Bayesian approach: Stochastic model class for a system

- **Model class** $\mathcal{M}$ defined by fundamental probability models:

  (1) Chosen **set of I/O probability models for prediction**:

  $$\{ p(Y_n \mid U_n, \theta, \mathcal{M}) : \theta \in \Theta \subset R^{N_p} \}$$

  U_n, Y_n= input, output time histories; θ=uncertain model parameters
  e.g. use **stochastic embedding** of a set of deterministic models

  (2) Chosen PDF $p(\theta|\mathcal{M})$ (**prior**) over this set to express the initial **relative plausibility** of each probability model in (1) specified by a value of θ

- If system data $\mathcal{D}_N = \{\hat{Y}_N, \hat{U}_N\}$ is available, then **Bayes' Theorem** using (1) and (2) gives updated PDF (**posterior**):

  $$p(\theta \mid \mathcal{D}_N, \mathcal{M}) \propto \underbrace{p(\hat{Y}_N \mid \hat{U}_N, \theta, \mathcal{M})}^{\text{Likelihood function}} \overbrace{p(\theta \mid \mathcal{M})}^{\text{Prior PDF}}$$

  - quantifies the relative plausibility of each probability model in (1) after incorporating the information in $\mathcal{D}_N$

Posterior probabilities of multiple candidate model classes

- **Ingredients:**

  (1) $\mathbf{M}$: specifies a set of candidate stochastic model classes for a system, $\{\mathcal{M}_1,\ldots,\mathcal{M}_J\}$, and a prior $P[\mathcal{M}_j|\mathbf{M}]$ over this set

  (2) Input and output data $\mathcal{D}$ from system

- **Posterior probability** $P[\mathcal{M}_j|\mathcal{D},\mathbf{M}]$, $j = 1,\ldots,J$, from Bayes' Theorem at model-class level:

Evidence for model class $\mathcal{M}_j$

$$P[\mathcal{M}_j\,|\,\mathcal{D},\mathbf{M}] = \frac{p(\mathcal{D}\,|\,\mathcal{M}_j)\,P[\mathcal{M}_j\,|\,\mathbf{M}]}{p(\mathcal{D}\,|\,\mathbf{M})} \propto \underbrace{p(\mathcal{D}\,|\,\mathcal{M}_j)}_{\text{Evidence}} \quad \text{if } P[\mathcal{M}_j\,|\,\mathbf{M}] = 1/J$$

- Use for Bayesian model-class selection/comparison/assessment

Calculation of evidence for model class $\mathcal{M}_j$ based on $\mathcal{D}$

- **Evidence** (or marginal likelihood) for $\mathcal{M}_j$ based on data $\mathcal{D}$:

$$p(\mathcal{D}\,|\,\mathcal{M}_j) = \int \overbrace{p(\mathcal{D}\,|\,\theta_j, \mathcal{M}_j)}^{\text{Likelihood}}\, \overbrace{p(\theta_j\,|\,\mathcal{M}_j)}^{\text{Prior}}\, d\theta_j$$

- Calculate using <u>Laplace's asymptotic method</u> if $\mathcal{M}_j$ is <u>globally identifiable</u> based on data $\mathcal{D}$ [Beck & Yuen: J. Eng. Mech. 2004]

  or by TMCMC [Ching & Chen: J. Eng. Mech. 2007]

  or using <u>posterior</u> (not prior) <u>samples</u> for $\mathcal{M}_j$; e.g. stationarity method [Cheung & Beck: CACAIE 2010]

  or employing Approximate Bayesian Computation [Chiachio et al: SIAM J Sci. Comp. 2014; Vakilzadeh et al: Mech. Sys. Signal Proc. 2017]

Information-theoretic interpretation of evidence for model class $\mathcal{M}_j$

- Bayes' Theorem at model-class level automatically gives a quantitative Ockham's Razor or Principle of Model Parsimony that avoids over-fitting of data [Gull 1988; Mackay 1992] ["Entities should not be multiplied unnecessarily" - William of Ockham, 1285-1349]

- Shown rigorously in Ching, Muto & Beck 2005; Muto & Beck 2008:

Log evidence = Mean data fit of $\mathcal{M}_j$ [posterior mean of log likelihood]

– Expected information gain about model parameters θ from data $\mathcal{D}$ [relative entropy/Kullback-Leibler info]

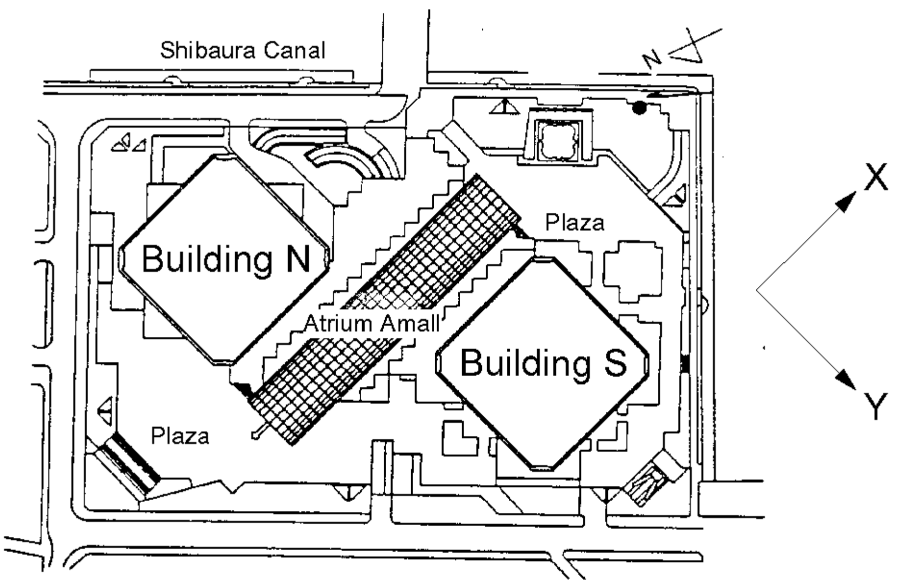
= Measure of consistency of model class with the data

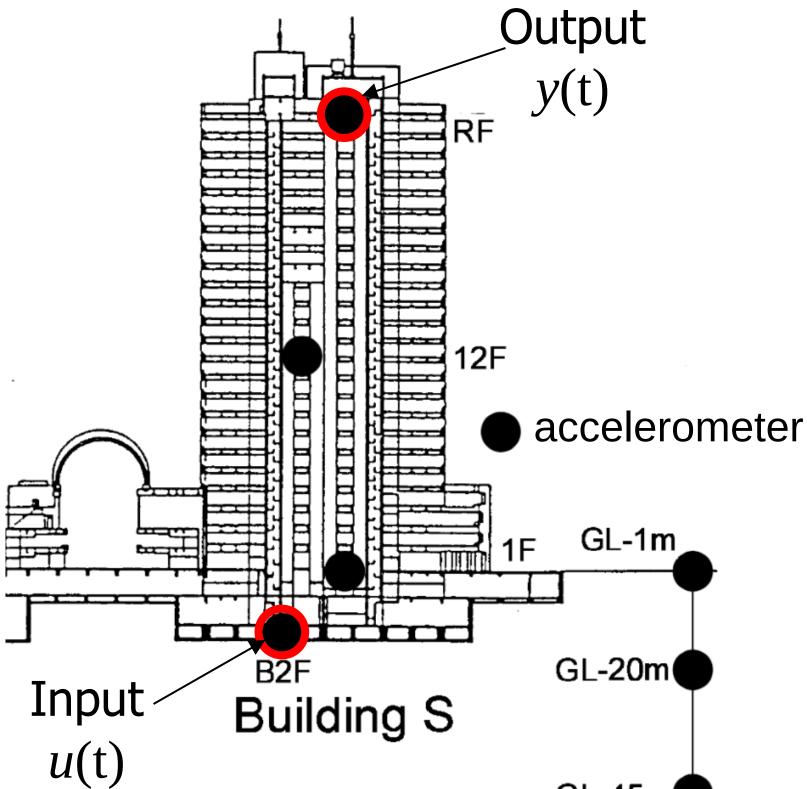
– Penalty for more complex model classes that extract more information from the data

- This quantitative tradeoff of the data fit and model complexity has been called the Bayesian Ockham Razor

Example: Bayesian modal identification for SHM of 24-story building in Tokyo, Japan

- Steel framed and 97.7m high
- 24 and 2 stories above and below the ground

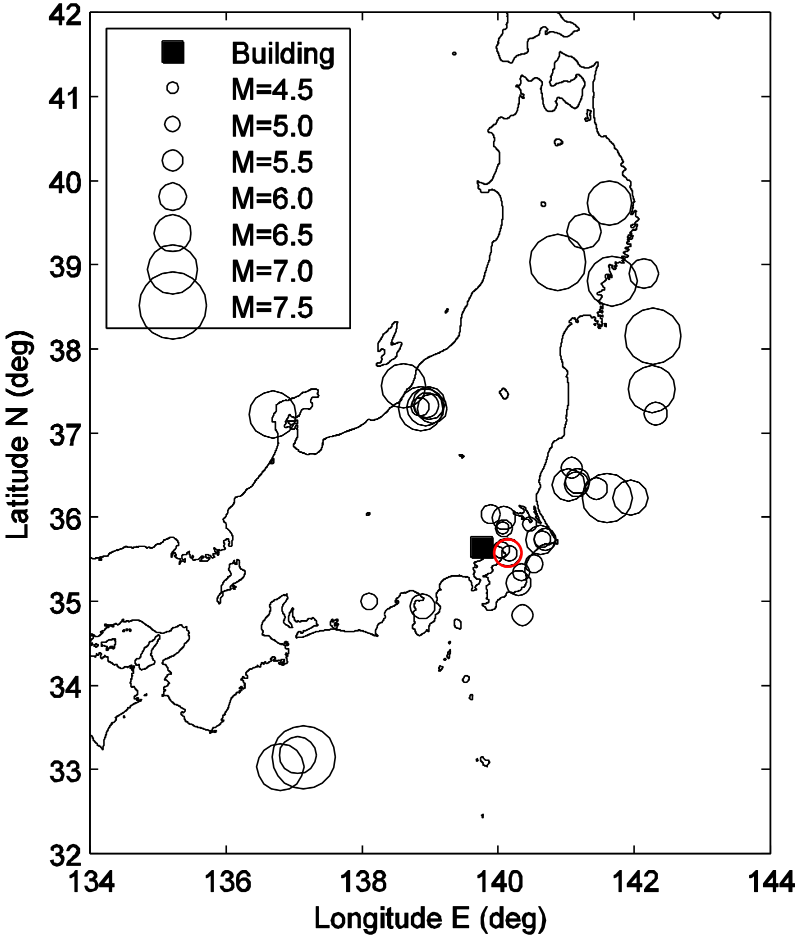




Saito & Beck: Eq. Eng. Struc. Dyn. 2010

43 earthquake records over 9 years

No.	Date (y/m/d)	Time (GMT+9h)	Location N (deg)	Location E (deg)	Epicentral distance (km)	Depth (km)	M	JMA seismic intensity
1	1999/07/15	07:56	35.917	140.467	71	56	4.9	1.0
2	1999/09/13	07:56	35.567	140.167	38	77	5.0	1.9
3	2001/04/03	23:57	34.995	138.108	166	33	5.1	1.0
4	2001/12/02	22:01	39.395	141.267	436	122	6.4	0.8
5	2002/02/12	22:44	36.585	141.085	158	48	5.5	1.2
6	2002/07/24	05:05	37.228	142.318	289	30	5.7	0.9
7	2002/11/03	12:37	38.893	142.142	417	46	6.1	0.9
8	2003/05/12	00:57	35.865	140.088	38	47	5.2	2.6
9	2003/05/12	00:59	35.872	140.072	38	50	4.6	1.7
10	2003/05/17	23:33	35.735	140.653	82	47	5.1	0.9
11	2003/05/26	18:24	38.805	141.682	389	71	7.0	2.3
12	2003/09/20	12:54	35.215	140.303	69	70	5.8	2.3
13	2003/10/15	16:30	35.610	140.052	27	74	5.1	2.7
14	2003/11/12	17:26	33.170	137.057	370	398	6.5	1.6
15	2003/11/15	03:43	36.428	141.168	154	48	5.8	1.2
16	2004/04/04	08:02	36.387	141.157	150	49	5.8	1.3
17	2004/07/17	15:10	34.833	140.358	106	69	5.5	1.5
18	2004/09/05	19:07	33.028	136.800	398	38	6.9	1.0
19	2004/09/05	23:57	33.143	137.142	367	44	7.4	1.5
20	2004/10/06	23:40	35.988	140.088	48	66	5.7	2.8
21	2004/10/23	17:56	37.292	138.867	199	13	6.8	2.0
22	2004/10/23	17:59	37.312	138.855	201	16	5.3	0.8
23	2004/10/23	18:03	37.353	138.983	201	9	6.3	1.4
24	2004/10/23	18:34	37.305	138.930	198	14	6.5	2.1
25	2004/10/25	06:04	37.330	138.947	200	15	5.8	0.9
26	2004/10/27	10:40	37.292	139.033	193	12	6.1	1.6
27	2005/02/16	04:46	36.035	139.895	45	45	5.4	2.5
28	2005/04/11	07:22	35.727	140.620	79	52	6.1	2.1
29	2005/06/20	01:15	35.733	140.693	85	51	5.6	1.3
30	2005/07/23	16:34	35.582	140.138	35	73	6.0	3.5
31	2005/08/08	00:06	36.338	141.445	170	46	5.6	0.7
32	2005/08/16	11:46	38.150	142.278	357	42	7.2	2.9
33	2005/10/19	20:44	36.382	141.042	141	48	6.3	1.8
34	2007/01/16	03:17	34.937	138.892	111	175	5.8	1.4
35	2007/03/25	09:41	37.220	136.685	326	11	6.9	0.9
36	2007/07/16	10:13	37.557	138.608	235	17	6.8	2.0
37	2007/08/16	04:15	35.443	140.530	74	31	5.3	1.5
38	2007/08/18	16:55	35.342	140.345	63	20	5.2	1.0
39	2008/05/08	01:02	36.230	141.948	208	60	6.4	1.2
40	2008/05/08	01:45	36.227	141.607	179	51	7.0	2.4
41	2008/06/14	08:43	39.028	140.880	388	8	7.2	1.7
42	2008/07/19	11:39	37.520	142.263	305	32	6.9	1.6
43	2008/07/24	00:26	39.732	141.635	482	108	6.8	1.6



ARX model class $\mathcal{M}_d$ of order d

- **Auto-Regressive eXogenous model:**

Prediction error

$$\mathrm{y}_n = -\sum_{j=1}^{d} a_j \mathrm{y}_{n-j} + \sum_{j=0}^{d} b_j \mathrm{u}_{n-j} + \mathrm{e}_n$$

Output

Input

$$y_n = y(n\Delta t) \qquad u_n = u(n\Delta t)$$

$$e_n = e(n\Delta t) \sim \text{i.i.d. } \mathcal{N}(0, \sigma^2)$$

  - Gaussian likelihood based on I/O data $\mathcal{D}_N = \{\hat{Y}_N, \hat{U}_N\}$

- **Model parameters:** $\theta = \left[a_1, \ldots, a_d, b_0, \ldots, b_d, \sigma^2\right]^{\mathrm{T}} \in R^{2d+2}$

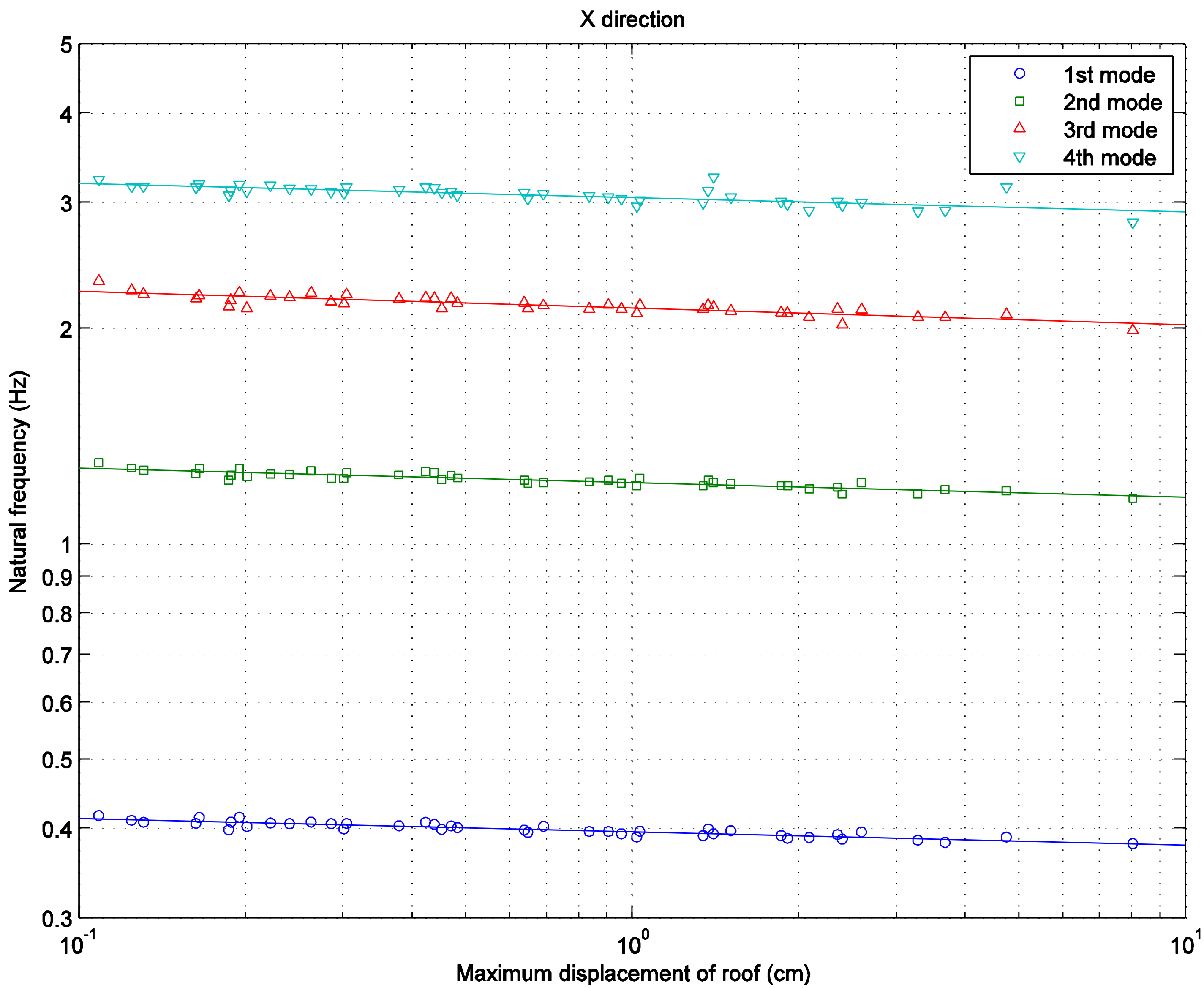
  - Gaussian priors on coefficients, lognormal prior on σ^2

Probability of each model class: Record #30

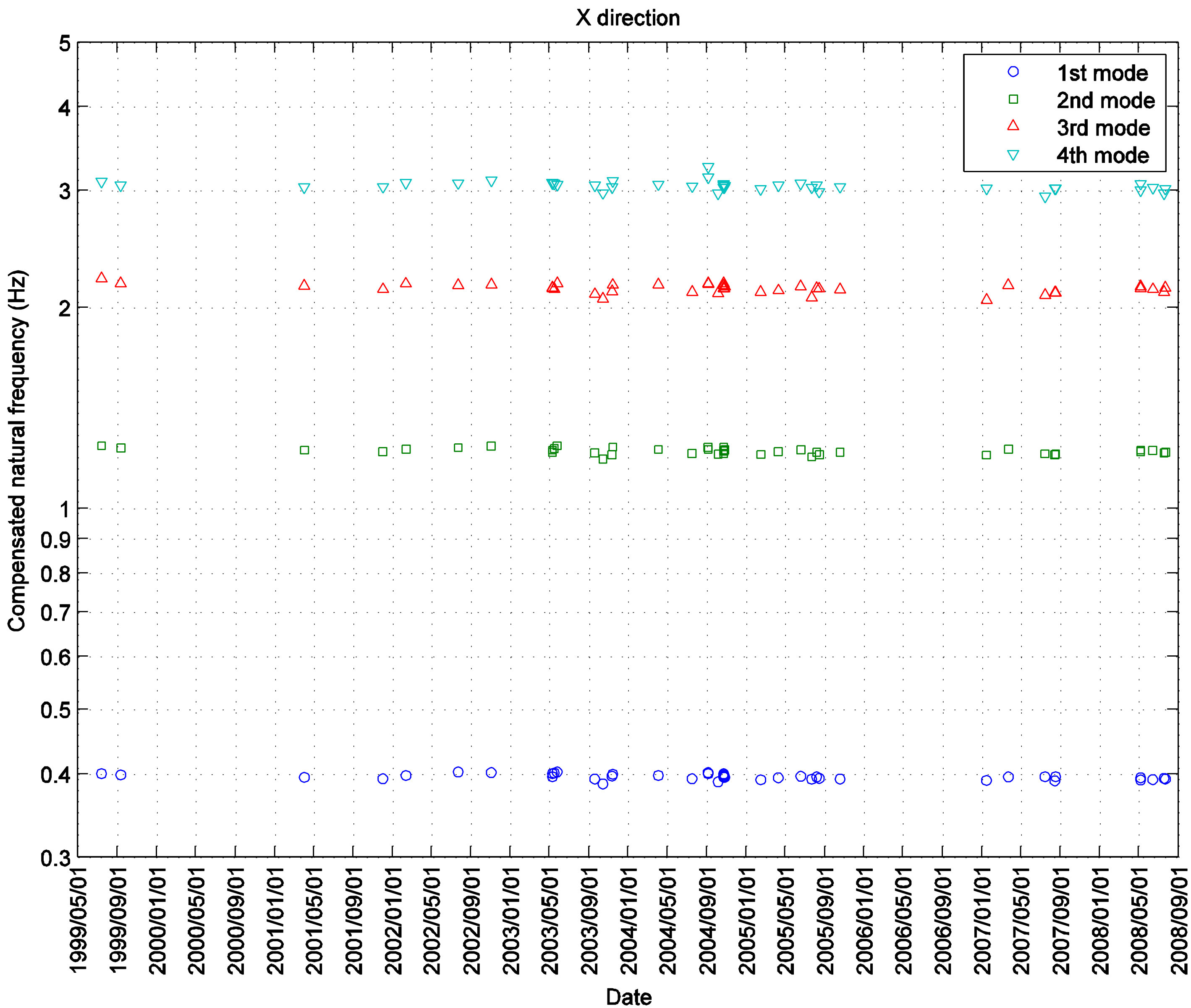
Model order d	$\cdots$	26	28	30	32	$\cdots$
Posterior probability of model class $\mathcal{M}_d$	$\cdots$	0.0	0.93	0.07	0.0	$\cdots$

- Equal prior probabilities are chosen for each $\mathcal{M}_d$
- Most probable a posteriori ARX order is d=28 among all model classes with d=10 to 60

Natural frequencies vs response amplitude



Natural frequencies after amplitude compensation

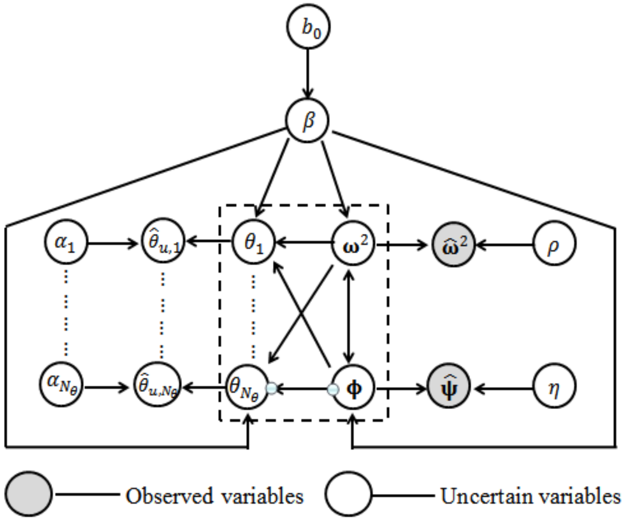


Sparse Bayesian learning from modal data for system identification and structural health monitoring

Yong Huang and James L. Beck

International Journal for Uncertainty Quantification, 2015
Structural Safety, 2016
Computer Methods in Applied Mechanics and Engineering, 2017
Proceedings of TU1402/1406 Zagreb Workshop, 2017

Sparse Bayesian learning for SHM from modal data



Hierarchical Bayesian model

ω^2 = system natural frequencies

ϕ = system mode shapes

θ = stiffness scaling parameters

β = eigenequation-error precision

- Structural stiffness matrix (expanded in terms of chosen substructure contributions; e.g. from FEM)

$$\mathbf{K}(\boldsymbol{\theta}) = \mathbf{K}_0 + \sum_{j=1}^{N_\theta} \theta_j \mathbf{K}_j$$

- Expect $\Delta\boldsymbol{\theta} = \boldsymbol{\theta} - \boldsymbol{\theta}_u$ due to damage to be **sparse**; i.e. stiffness reductions from undamaged state occur in only a few substructures
- Gaussian likelihood functions for $\boldsymbol{\omega}^2, \boldsymbol{\phi}$ and $\boldsymbol{\theta}$:

$$p(\hat{\boldsymbol{\omega}}^2 \,|\, \boldsymbol{\omega}^2, \rho), \quad p(\hat{\boldsymbol{\psi}} \,|\, \boldsymbol{\phi}, \eta)$$

$$p(\hat{\boldsymbol{\theta}}_u \,|\, \boldsymbol{\theta}, \boldsymbol{\alpha}) = \prod_{i=1}^{N_\theta} \mathcal{N}(\hat{\theta}_{u,i} \,|\, \theta_i, \alpha_i^{-1})$$

Pseudo data (MAP)   Learning from data induces sparseness

- For joint prior, use Gaussian eigenequation error:

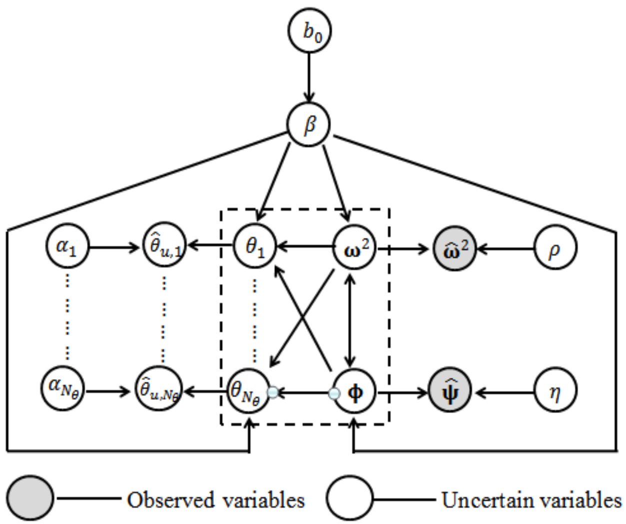
$$\mathbf{e}_i = (\mathbf{K}(\boldsymbol{\theta}) - \omega_i^2 \mathbf{M})\boldsymbol{\phi}_i \sim \mathcal{N}(\mathbf{e}_i \,|\, \mathbf{0}, \beta^{-1}\mathbf{I}_{N_d})$$

for the i th mode (i=1,…,N_m) to get:

$$p(\boldsymbol{\omega}^2, \boldsymbol{\phi}, \boldsymbol{\theta} \,|\, \beta) \propto (2\pi/\beta)^{-N_m N_d/2} \exp\left\{ -\frac{\beta}{2} \sum_{i=1}^{N_m} \left\| (\mathbf{K}(\boldsymbol{\theta}) - \omega_i^2 \mathbf{M})\boldsymbol{\phi}_i \right\|^2 \right\}$$

Gives Gaussian conditional priors for any parameter vector given the other two

Fast SBL algorithm for structural stiffness loss (focus on the uncertainty quantification of $\boldsymbol{\theta}$)



- Collect all parameters except $\boldsymbol{\theta}$ as:

$$\boldsymbol{\delta} = [(\boldsymbol{\omega}^2)^T, \rho, \boldsymbol{\phi}^T, \eta, \boldsymbol{\alpha}^T, \beta]^T$$

- Full posterior PDF for data $\mathcal{D} = \{\hat{\boldsymbol{\psi}}, \hat{\boldsymbol{\omega}}^2, \hat{\boldsymbol{\theta}}_u\}$:

$$p(\boldsymbol{\theta}, \boldsymbol{\delta} \mid \mathcal{D}) = p(\boldsymbol{\theta} \mid \boldsymbol{\delta}, \mathcal{D})\, p(\boldsymbol{\delta} \mid \mathcal{D})$$

Gaussian conditional posterior

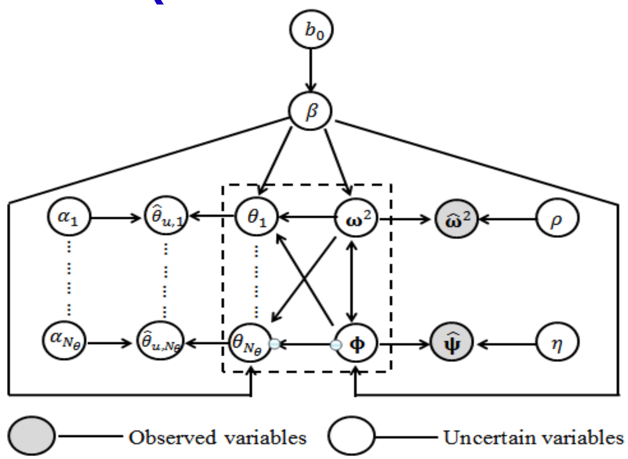
Bayes' Thm: $p(\boldsymbol{\delta} \mid \mathcal{D}) \propto p(\mathcal{D} \mid \boldsymbol{\delta})\, p(\boldsymbol{\delta})$

Analytical evidence for model class $\mathcal{M}(\boldsymbol{\delta})$ from:

$$p(\mathcal{D} \mid \boldsymbol{\delta}) = \int p(\mathcal{D} \mid \boldsymbol{\theta}, \boldsymbol{\delta})\, p(\boldsymbol{\theta} \mid \boldsymbol{\delta})\, \mathrm{d}\boldsymbol{\theta}$$

- Finding maximizing value $\tilde{\boldsymbol{\delta}}$ of posterior $p(\boldsymbol{\delta} \mid \mathcal{D})$ implements Bayesian Ockham Razor, automatically inducing sparseness in the inferred stiffness reductions $\Delta\boldsymbol{\theta}$ and gives: $p(\boldsymbol{\theta} \mid \mathcal{D}) \approx p(\boldsymbol{\theta} \mid \tilde{\boldsymbol{\delta}}, \mathcal{D})$ (Laplace approx.)
- All uncertain parameters are estimated solely from the data, so no user-intervention is needed. No solution of structural eigenequations needed

SBL algorithm using Gibbs sampling (full characterization of the posterior uncertainty)



$$p(\boldsymbol{\phi} \mid \hat{\boldsymbol{\psi}}, \boldsymbol{\omega}^2, \boldsymbol{\theta}, \boldsymbol{\beta}, \eta)$$
$$p(\boldsymbol{\omega}^2 \mid \hat{\boldsymbol{\omega}}^2, \boldsymbol{\phi}, \boldsymbol{\theta}, \boldsymbol{\beta}, \rho)$$
$$p(\boldsymbol{\theta} \mid \hat{\boldsymbol{\theta}}_u, \boldsymbol{\phi}, \boldsymbol{\omega}^2, \boldsymbol{\beta}, \boldsymbol{\alpha})$$

Analytical generic form

$$p(\mathbf{x}_1 \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3, \beta, \boldsymbol{\kappa})$$
$$\boldsymbol{\kappa} = \eta, \rho \text{ or } \boldsymbol{\alpha}$$

Can derive generic conditional posterior of parameters for Gibbs Sampling:

$$p(\mathbf{x}_1 \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3) = \int p(\mathbf{x}_1 \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3, \boldsymbol{\kappa}, \boldsymbol{\beta}) p(\boldsymbol{\kappa} \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3) p(\boldsymbol{\beta} \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3) d\boldsymbol{\kappa} d\boldsymbol{\beta}$$
$$\approx p(\mathbf{x}_1 \mid \hat{\mathbf{y}}, \mathbf{x}_2, \mathbf{x}_3, \tilde{\boldsymbol{\kappa}})$$

- Finding the maximizing value $\tilde{\boldsymbol{\kappa}}$ of the posterior for each $\boldsymbol{\kappa}$ then incorporates the Bayesian Ockham Razor for each of $\boldsymbol{\phi}, \boldsymbol{\omega}^2, \boldsymbol{\theta}$
- By marginalizing out the eigenequation-error precision $\boldsymbol{\beta}$ analytically:

  - more robust to noise and outliers (Gaussian ➔ Student's t-distribution)
  - reduces sample variance and "burn-in" period by avoiding sampling of $\boldsymbol{\beta}$

Summary of the Gibbs sampling algorithm

$\mathcal{D} = \{\hat{\psi}, \hat{\omega}^2, \hat{\theta}_u\}$ from modal ID in current state and $\hat{\theta}_u$ from calibration state

1. Initialize samples $\{\boldsymbol{\phi}^{(0)}, (\boldsymbol{\omega}^2)^{(0)}, \boldsymbol{\theta}^{(0)}\}$
2. $n = 1$
3. Sample system mode shapes $\phi^{(n)} \sim p(\phi \,|\, \mathcal{D}, (\omega^2)^{(n-1)}, \theta^{(n-1)})$
4. Sample natural frequencies $(\omega^2)^{(n)} \sim p(\omega^2 \,|\, \mathcal{D}, \phi^{(n)}, \theta^{(n-1)})$
5. Sample stiffness scaling parameters $\theta^{(n)} \sim p(\theta \,|\, \mathcal{D}, \phi^{(n)}, (\omega^2)^{(n)})$
6. $n = n+1$
7. If $n \leq n_{\max}$ then go to Step 3

All PDFs are known Student t-distributions

- After a "burn-in" period, posterior samples of $\{\phi, \omega^2, \theta\}$ are obtained
- Automatically gives samples from marginal posterior: $\boldsymbol{\theta}^{(n)} \sim p(\boldsymbol{\theta} \,|\, \mathcal{D})$ ("magic" of Monte Carlo simulation)
- No parameter-tuning is required. Also, effective dimension of GS sampling is always 3 because of parameter grouping in 3 vectors

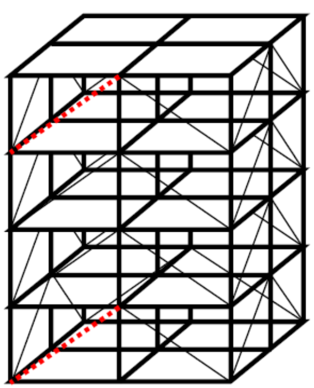
Application of Sparse Bayesian learning in SHM

Config 1

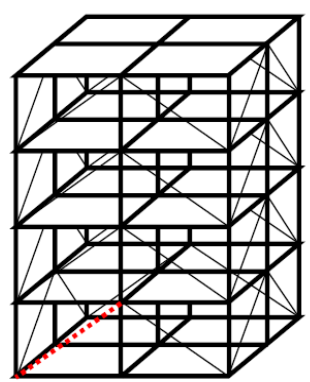
Experimental Phase II Benchmark problem

- ¼-scale steel braced-frame (Dyke et al. 2003)
- 15 accelerometers were placed for sensing structural vibrations under hammer impact
- Five modes (first and second translation models in each direction and the first torsion mode) are identified from each of 3 time-history segments
- **Goal:** Infer stiffness losses relative to undamaged structure from identified modal parameters in each damage configuration

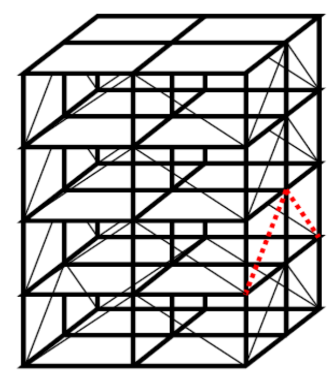
Brace-damage patterns



Config 4

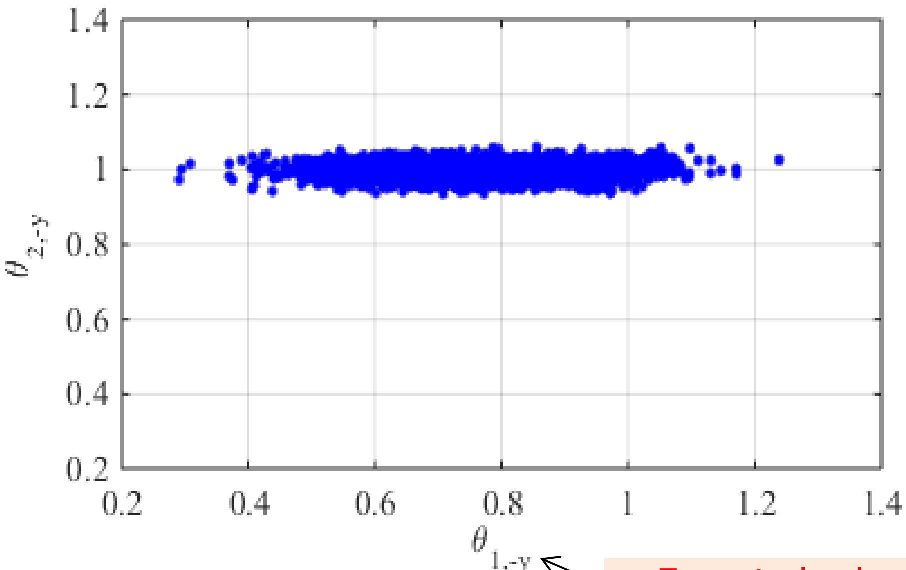


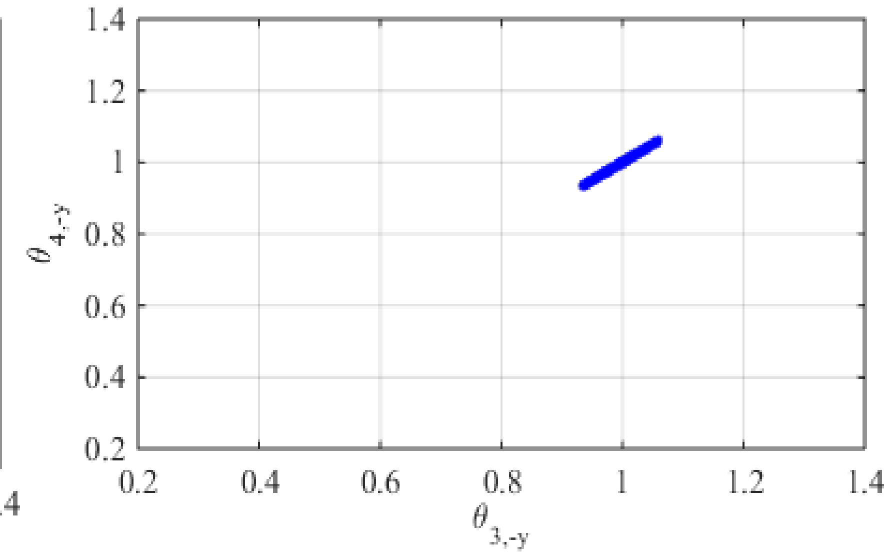
Config 5



Config 6

Posterior Gibbs samples for stiffness scaling parameters for one face of 4 stories: Config. 5





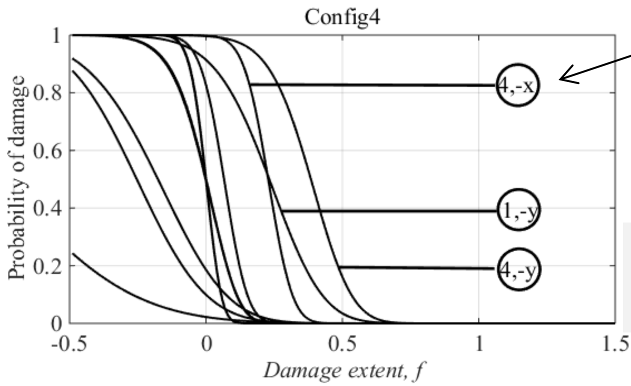
Expected value for damaged substructure is 0.77

- For undamaged substructures, the posterior means of the estimated stiffness changes are close to zero and the corresponding uncertainties are much smaller due to inducing model sparsity
- For damaged substructures, larger uncertainties are probably due to significant modeling error in chosen FEM substructure stiffness matrices

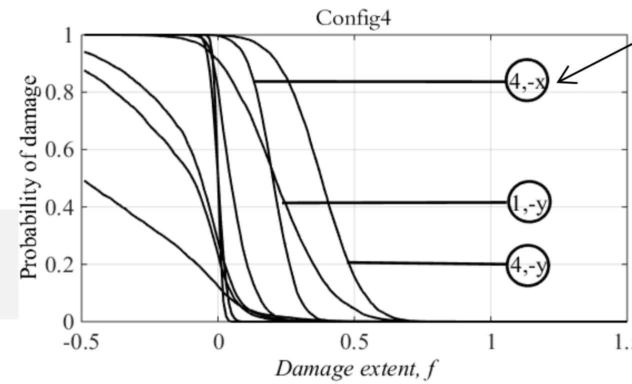
Estimated damage probability for each substructure

- Fast algorithm
- Gibbs Sampling algorithm

False detection

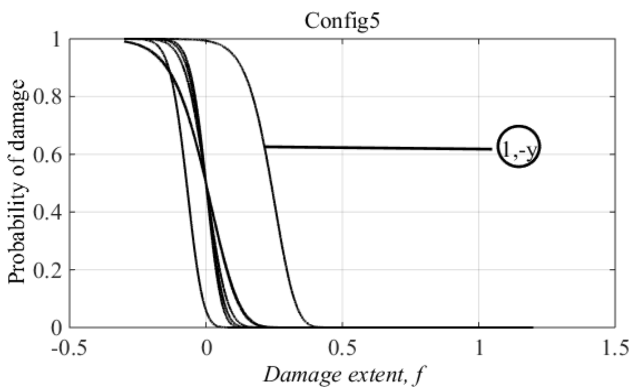


(a) Config. 4



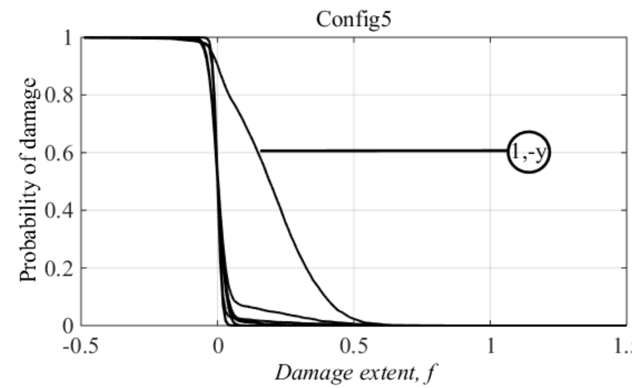
False detection

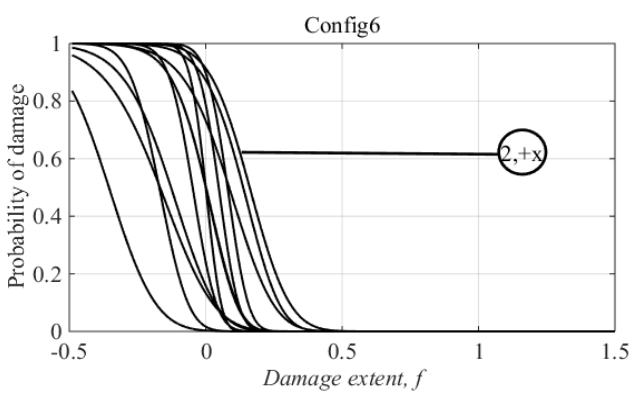
Two algorithms give similar results



(b) Config. 5

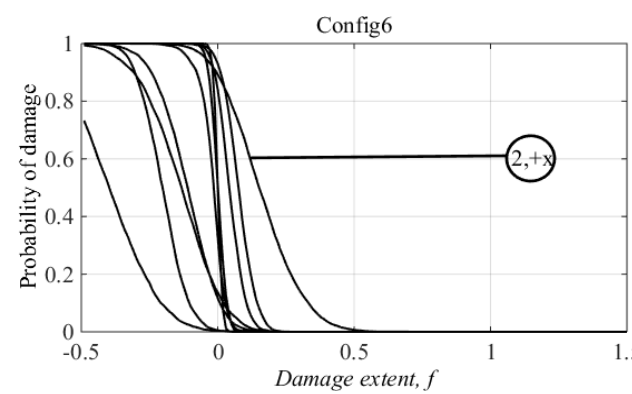
Smaller posterior uncertainty for undamaged substructures





(c) Config. 6

The occurrence of false detections is less likely



Concluding Remarks

- Probability (as a) logic provides a rigorous Bayesian framework to quantify modeling uncertainty in system ID and SHM
  - Treats uncertainty due to missing information (epistemic); the assumption of inherent randomness is not needed (aleatory)
  - Uses only the probability axioms and the probability models defined by a chosen stochastic model class for the system
  - Using posterior probability of model classes automatically quantifies Ockham's Razor, which penalizes models that are too simple ("under-fit" the data) and too complex ("over-fit" the data)

- Sparse Bayesian learning has important applications in structural health monitoring; for example:
  - Inferring stiffness loss from modal data where damage is usually spatially sparse
  - Recovering missing data during wireless signal transmission in an SHM sensor network using Bayesian compressive sensing (Huang et al., Prob. Eng. Mech. 2016)

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Operational Monitoring and Serviceability Assessment of Long-Span Bridges

Ho-Kyung Kim – Seoul National University, Korea

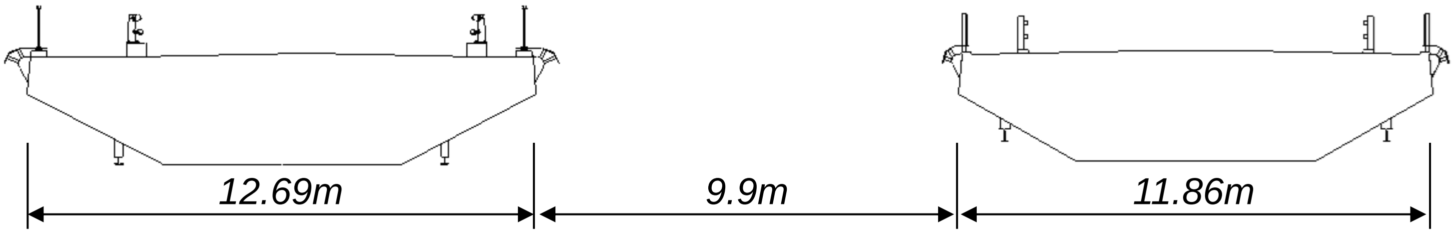
교량설계핵심기술연구단
Korea Bridge Design & Engineering Research Center

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

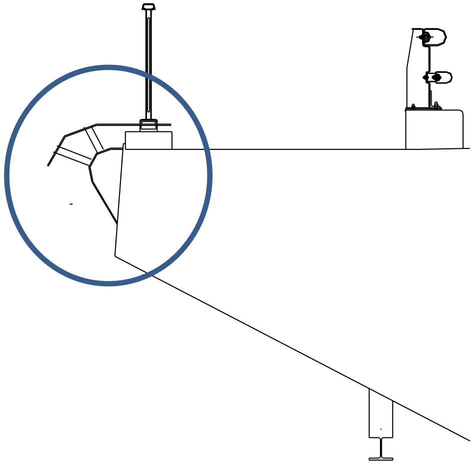
JINDO BRIDGE

- *Parallel cable-stayed bridges (L=70+344m+70)*

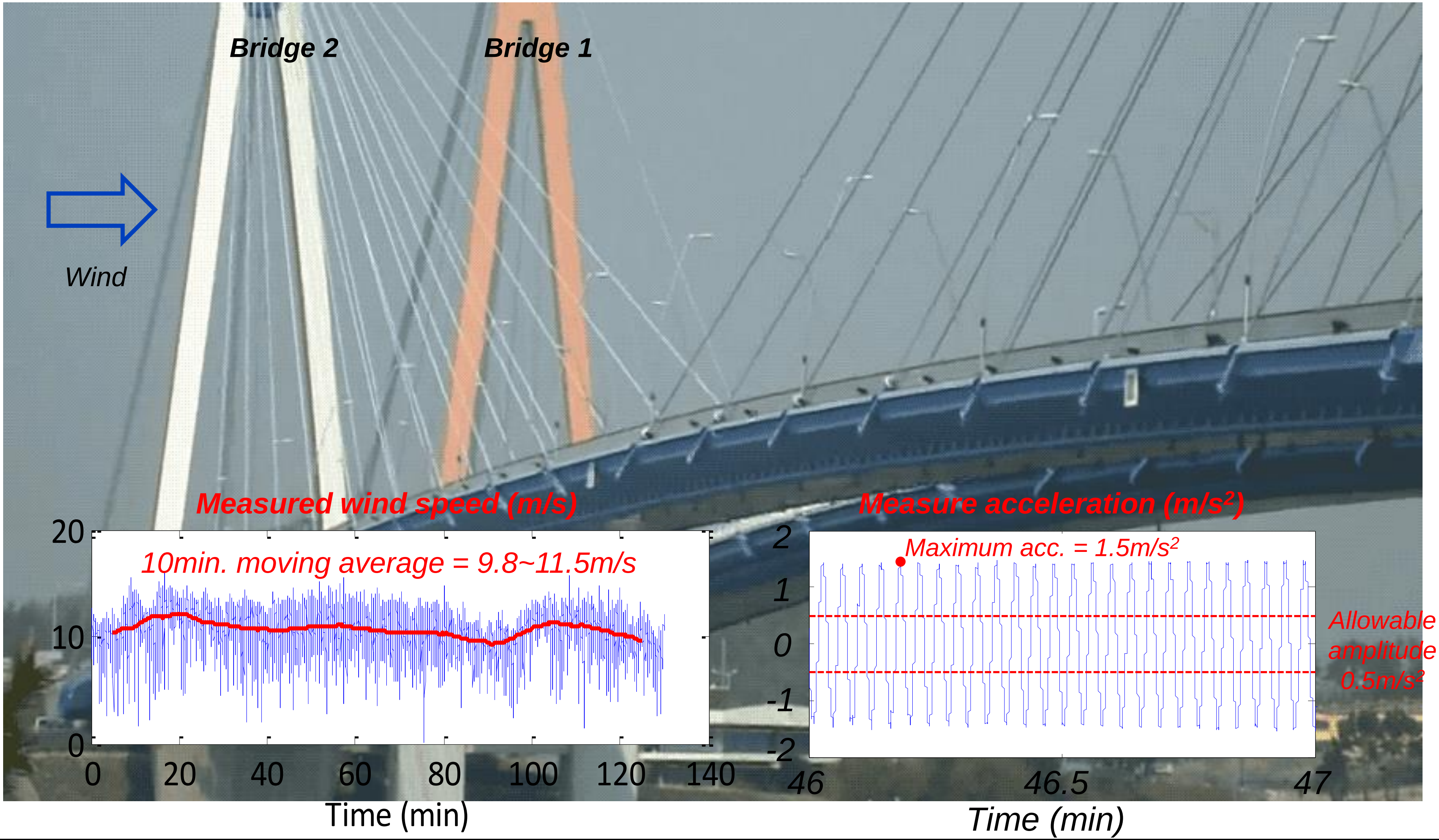


2nd Jindo bridge (2005) *1st Jindo bridge (1984)*

- *Guide vanes on girders to mitigate VIV*



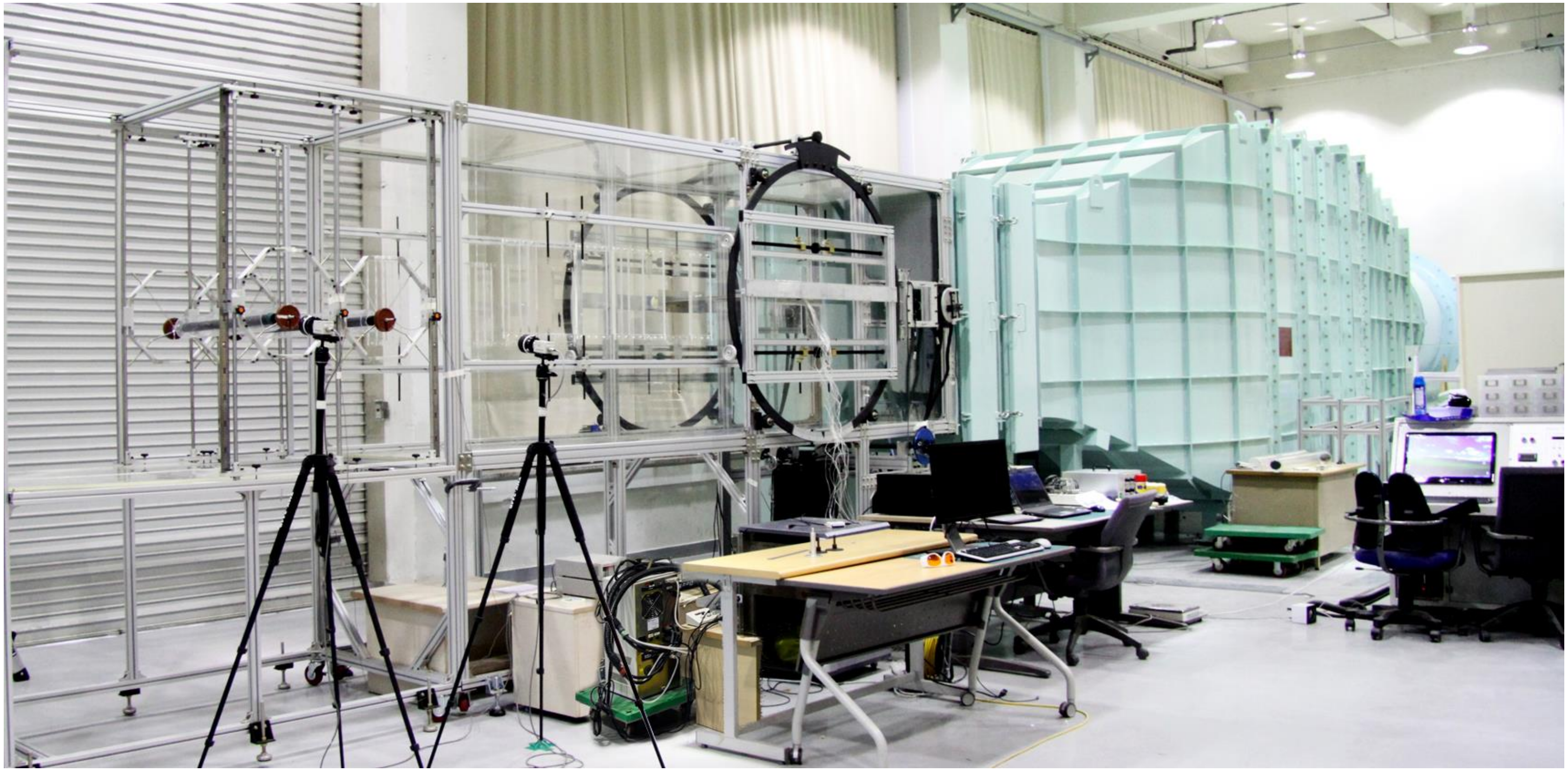
VIV IN 2ND JINDO BRIDGE (APR. 2011)



REPRODUCED IN WIND TUNNEL

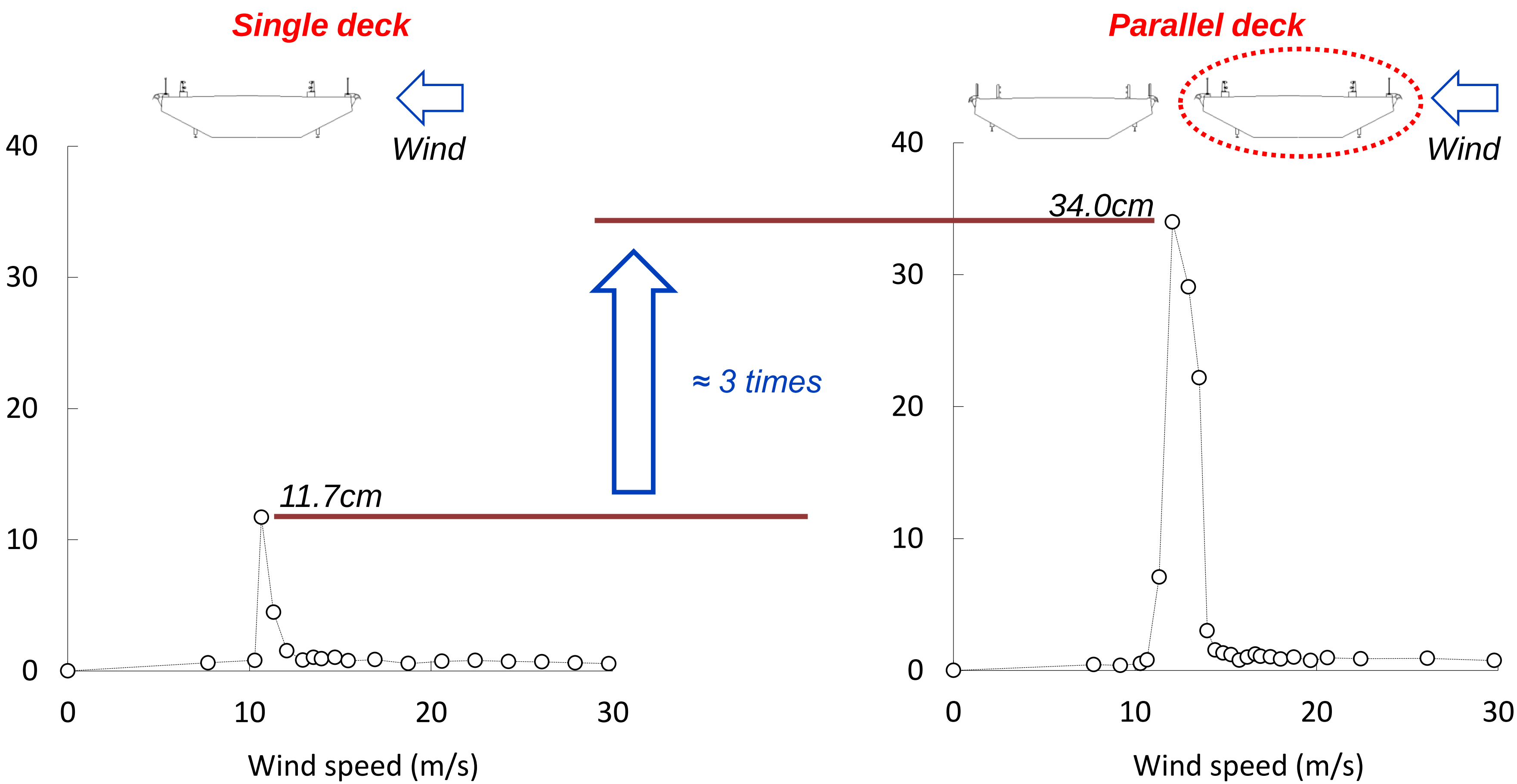
- *Test section: W(1.0m) X H(1.5m) X L(4.0m)*
- *Maximum wind speed: 23m/s*

Le Cachalot Wind Tunnel
Seoul National University

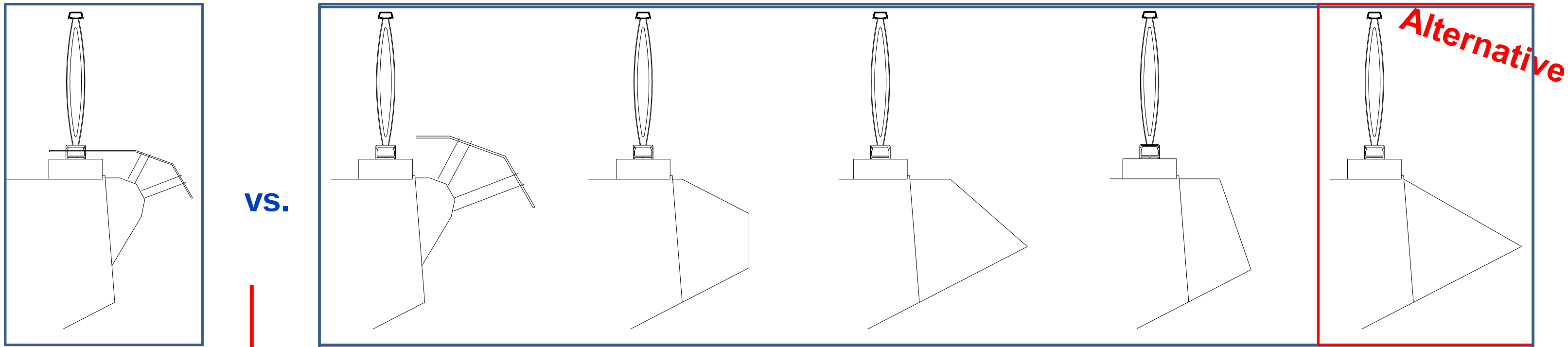


INTERFERENCE EFFECTS DUE TO A PARALLEL DISPOSITION

- *Vertical single amplitude(cm), Upstream deck, Low damping set-up (0.1%)*

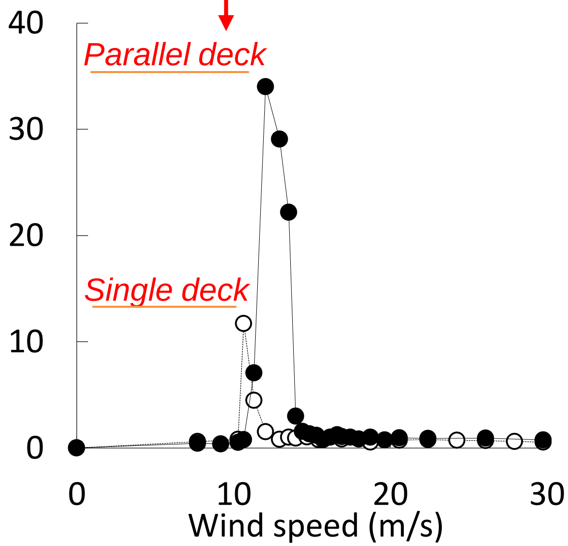


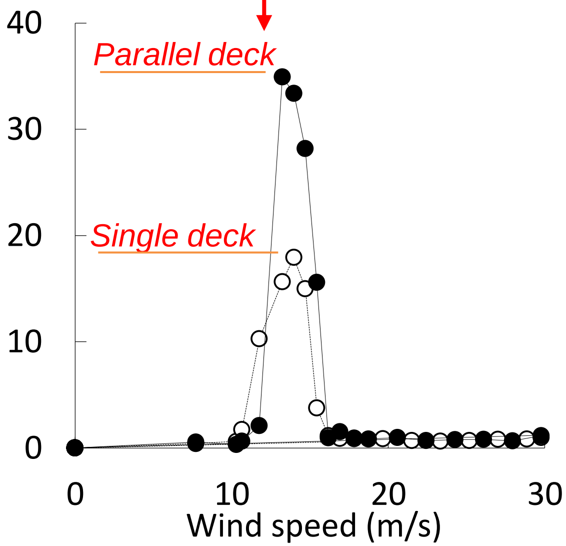
ANY AERODYNAMIC MEASURE?



As-is

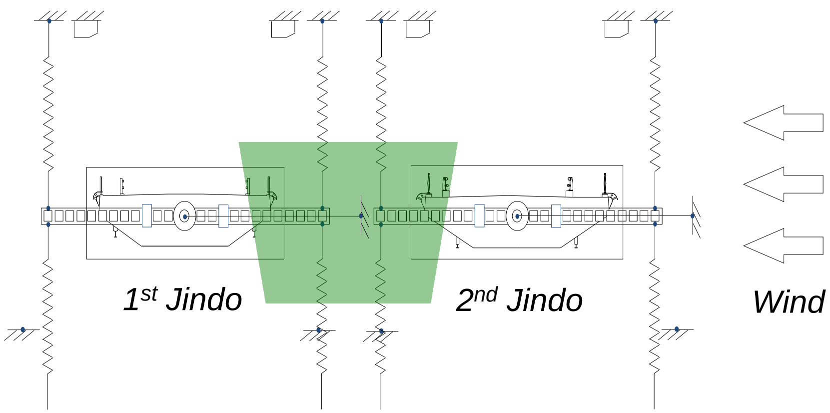
Alternative aerodynamic attachments

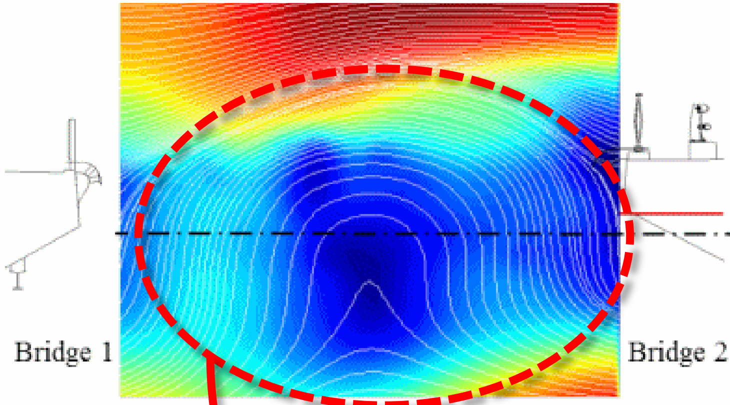


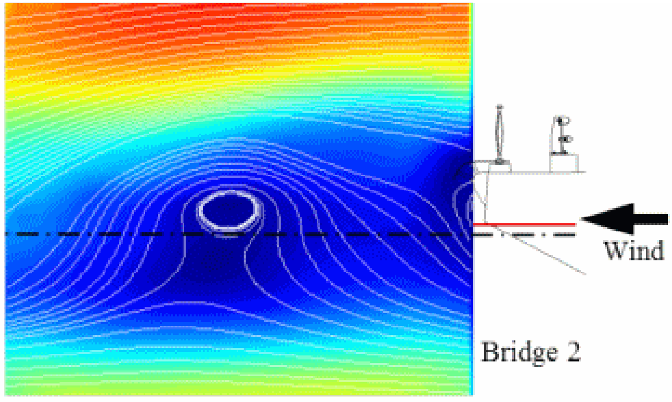


FLOW VISUALIZATION WITH PIV

Seo et al. (2013). Interference effect on vortex-induced vibration in a parallel twin cable-stayed bridge. J. Wind Eng. Ind. Aerodyn. 116, 7-20.







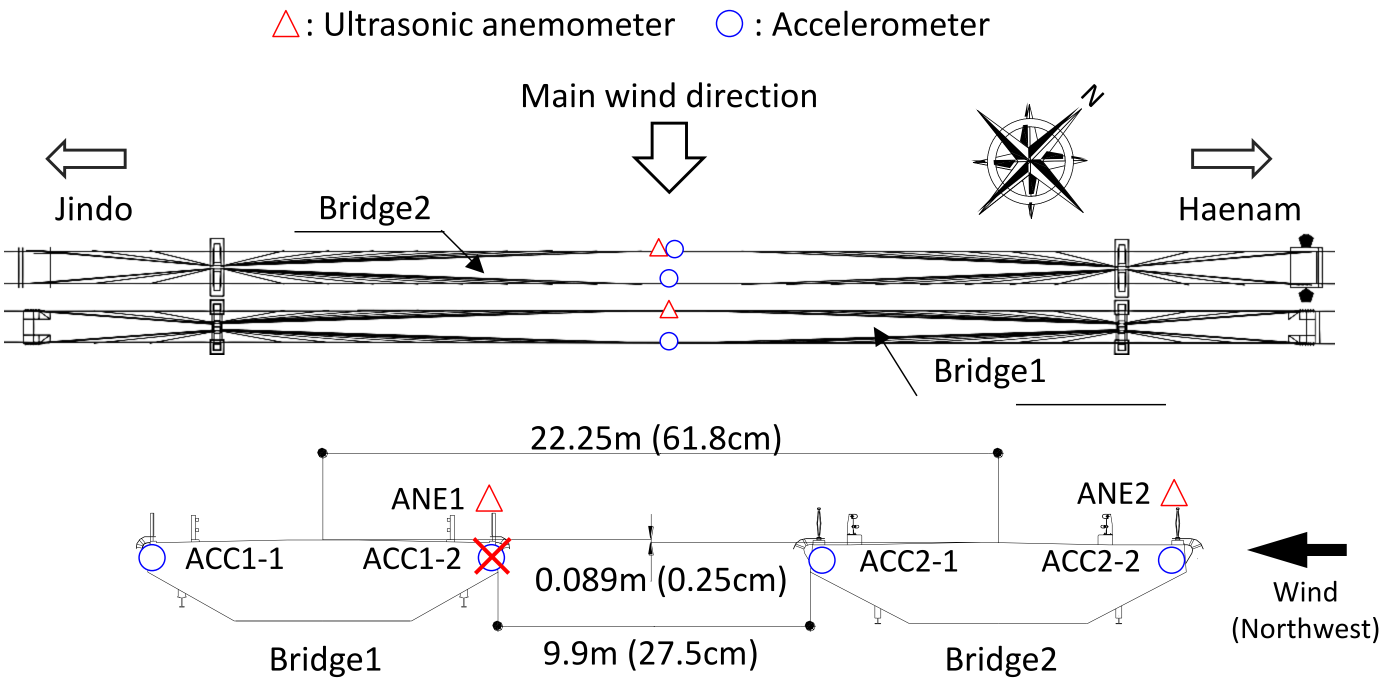
Parallel deck **Single deck**

Alternating "Ω" and "℧" streamlines between parallel decks

OPERATIONAL MODAL ANALYSIS (OMA)

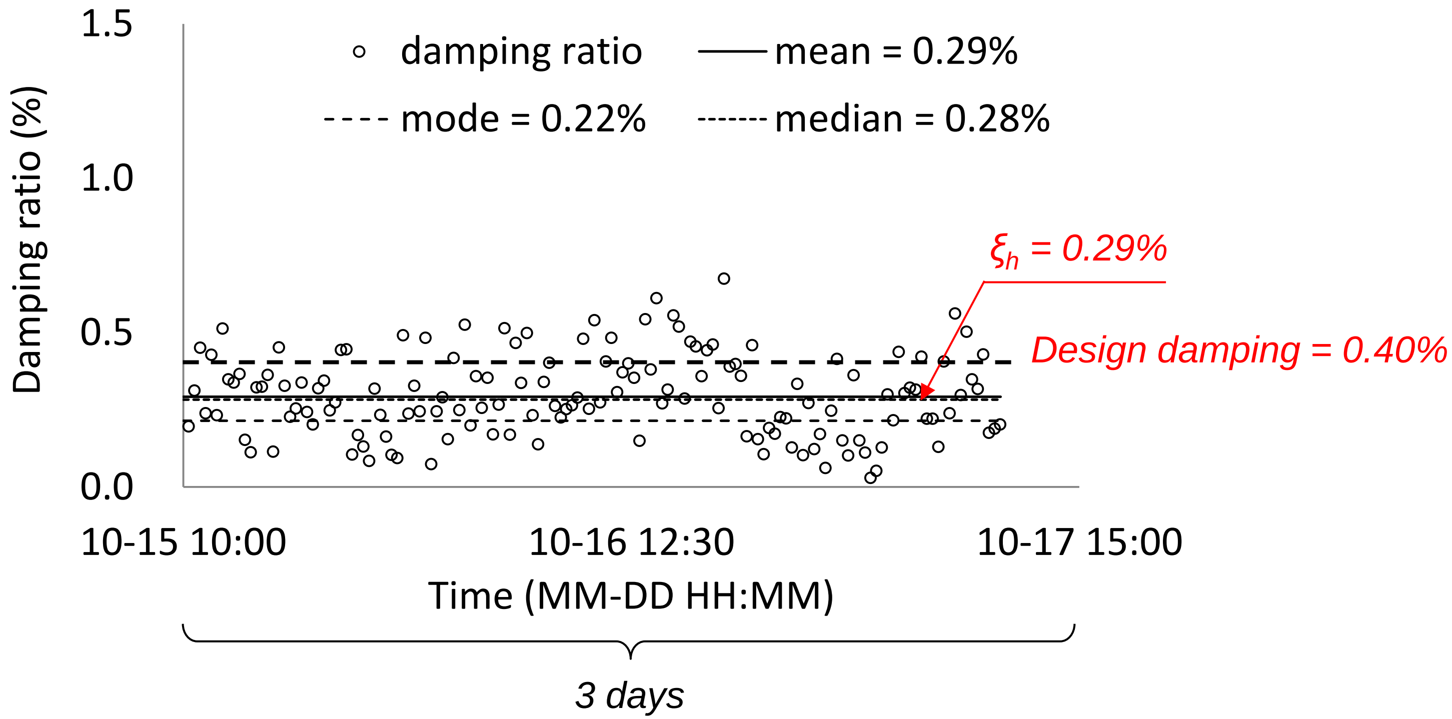
Built-in sensors

- *4 accelerometers at the center of mid span*
- *2 ultrasonic anemometers at center of mid span*



IDENTIFICATION OF DAMPING RATIOS (NEXT-ERA)

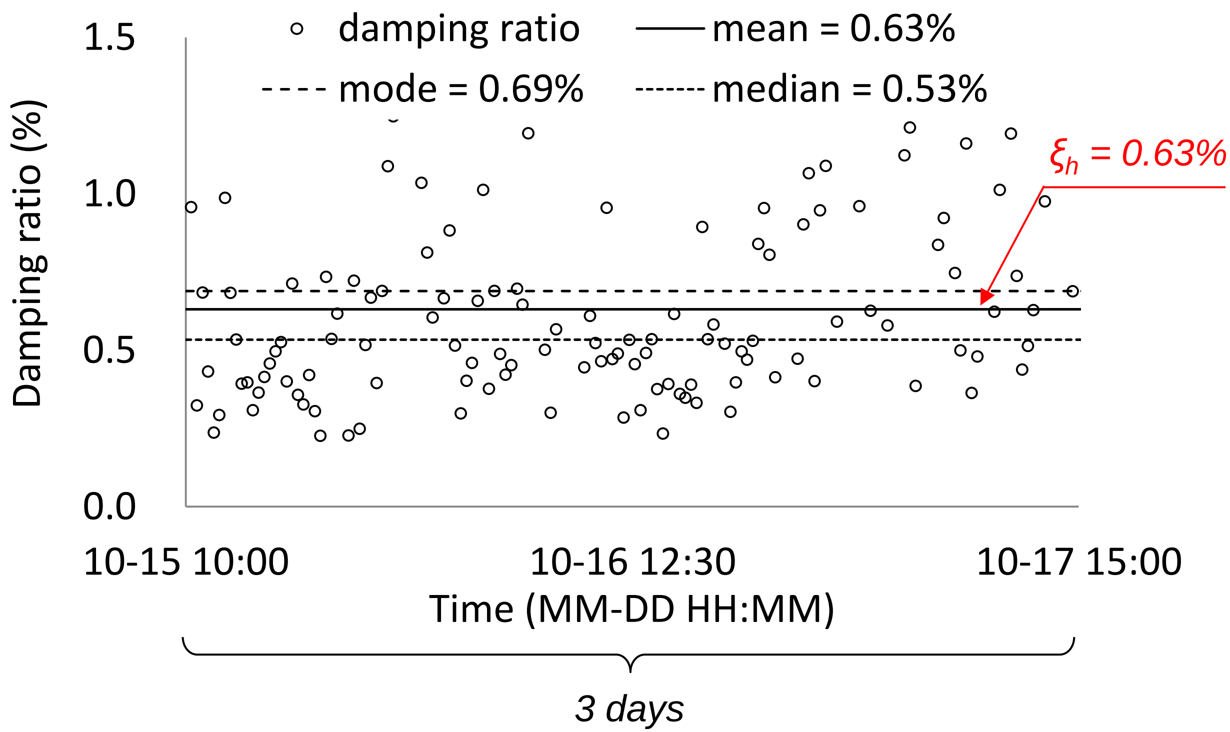
Bridge 2



- *The damping ratio is 0.29% in mean value.*
- *Lower than 0.40%, the recommended value in the design guidelines (KSCE 2006).*

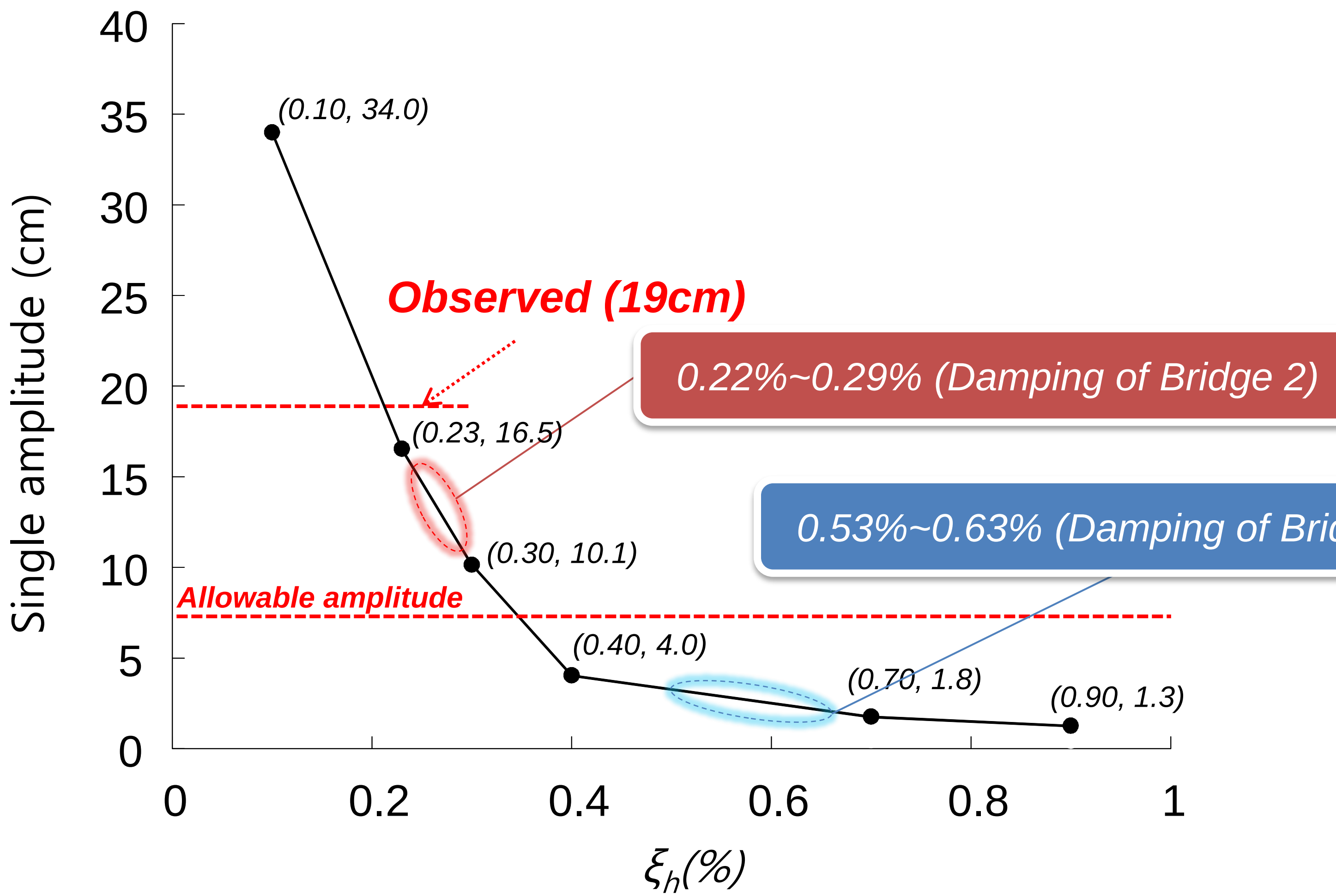
IDENTIFICATION OF DAMPING RATIOS (NEXT-ERA)

Bridge 1



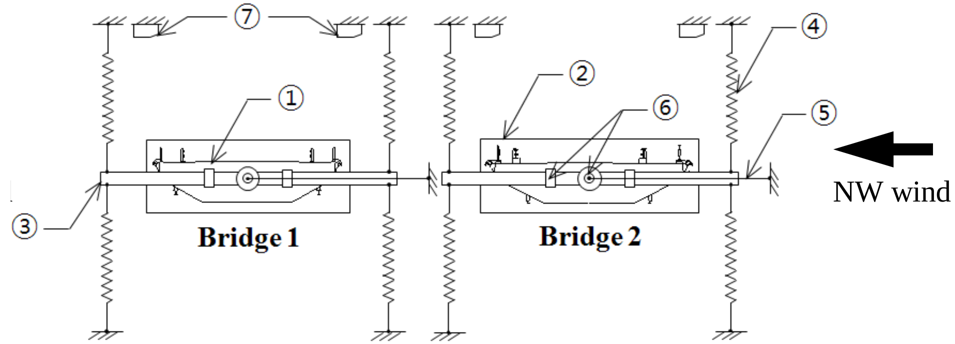
- *The damping ratio is higher than that of Bridge 2 resulting in a mean value of 0.63%.*

AMPLITUDES VS. DAMPING RATIOS (WIND TUNNEL TEST for Bridge 2)



FURTHER INVESTIGATIONS ON INTERACTIVE BEHAVIOR

- *Set-up in wind tunnel considering*
  - *Estimated damping ratios*
  - *Difference in natural frequencies*

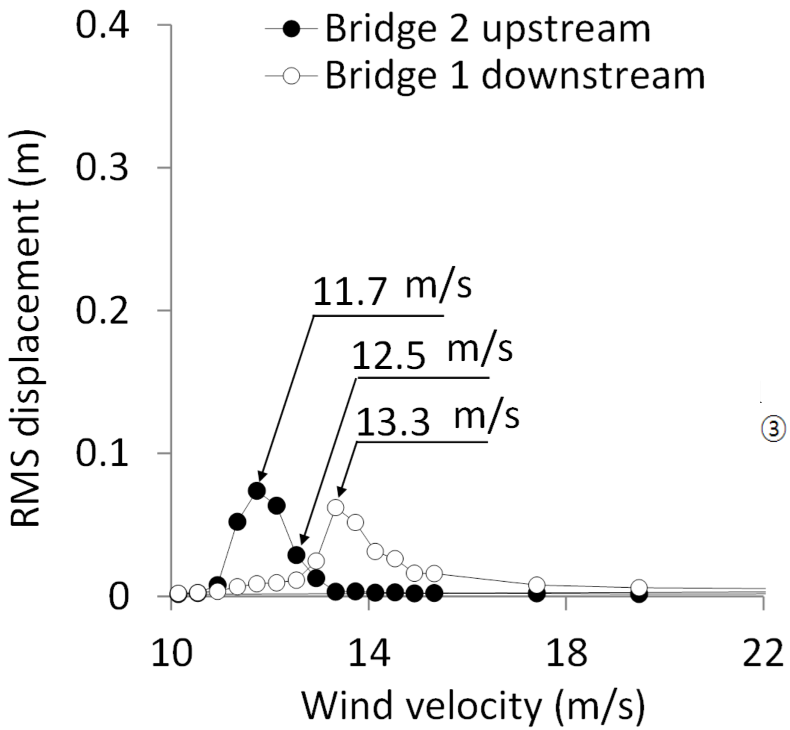


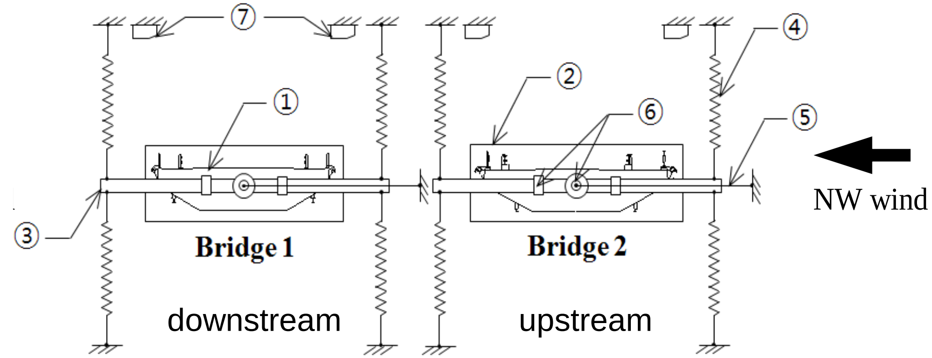
Parameters	Bridge 2			Bridge 1		
	Prototype	Model (target)	Model (measured)	Prototype	Model (target)	Model (measured)
Mass (kg/m)	8978	6.927	6.904	6950	5.363	5.376
Natural Frequency (Hz)	0.436	4.919	4.919	0.513	5.780	5.920
Damping Ratio (%)	0.280	0.280	0.280	0.580	0.580	0.550

OMA-based damping ratios!

INTERACTIVE BEHAVIOR BETWEEN TWO DECKS

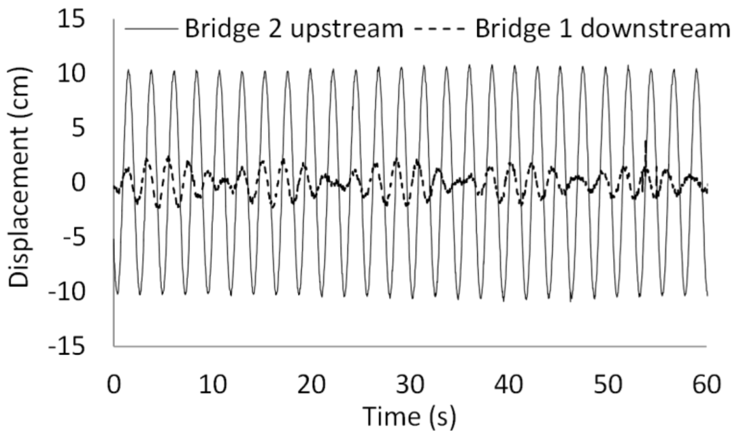
- Natural frequency of Bridge 2 is lower than that of Bridge 1.
  - VIV starts in Bridge 2 first.
  - Further increase of wind velocity leads VIV in Bridge 1.

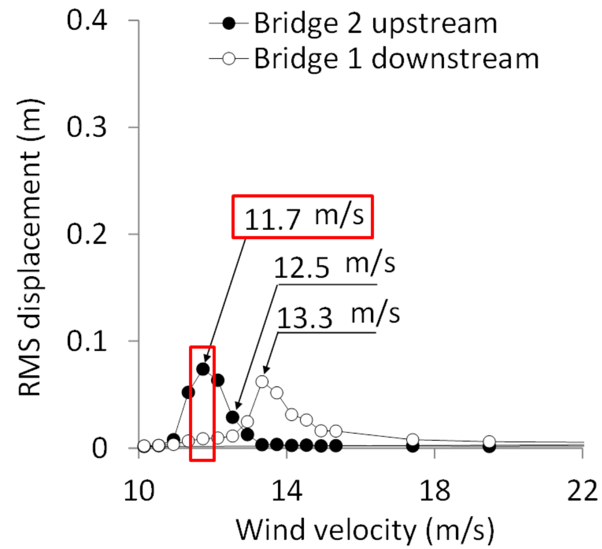




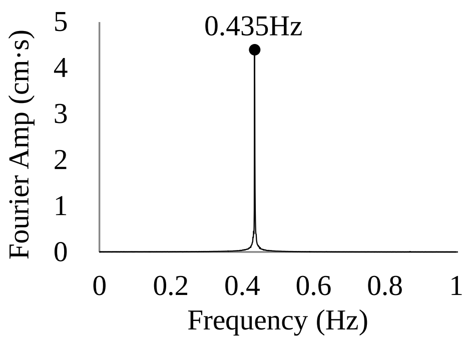
INTERACTIVE BEHAVIOR BETWEEN TWO DECKS

▶ Wind velocity of 11.7m/s

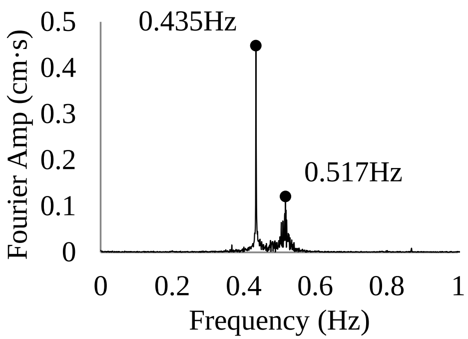




- PSD of Bridge 2

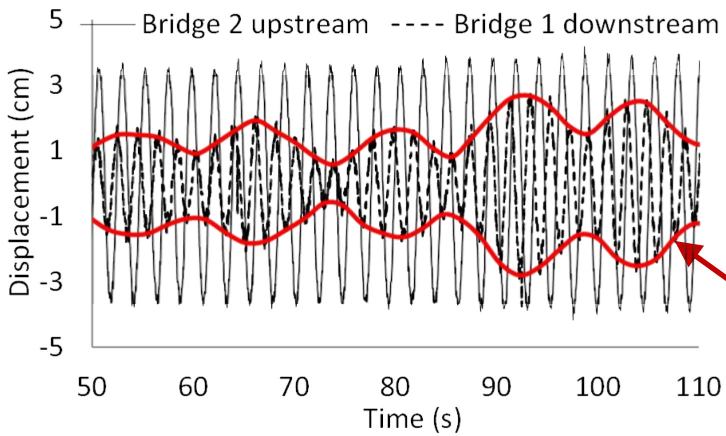


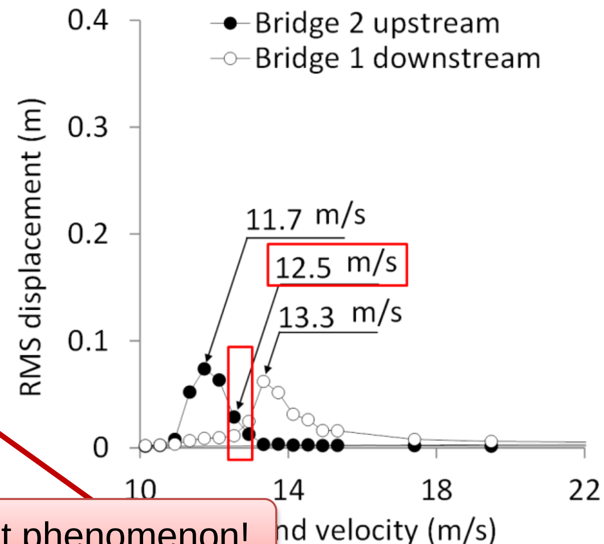
- PSD of Bridge 1



INTERACTIVE BEHAVIOR BETWEEN TWO DECKS

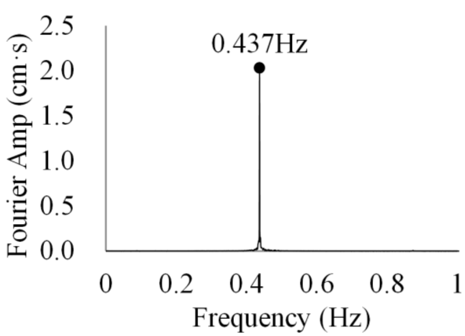
▶ Wind velocity of 12.5m/s





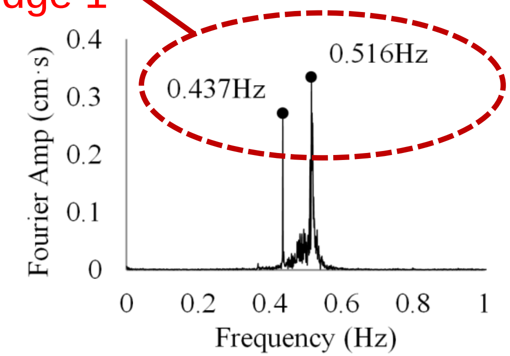
Beat phenomenon!

- PSD of Bridge 2



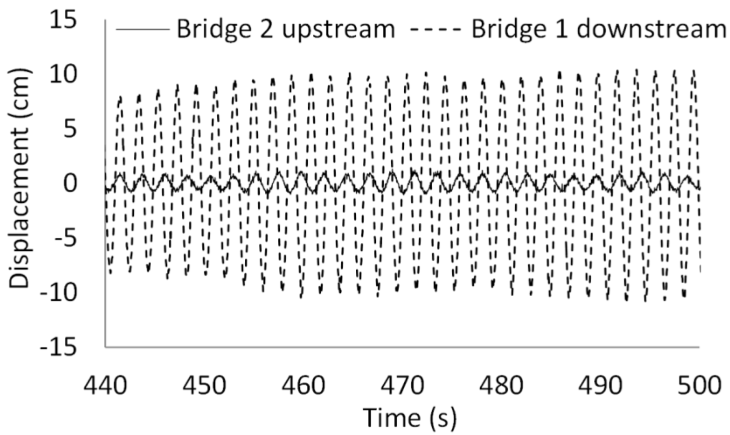
- PSD of Bridge 1

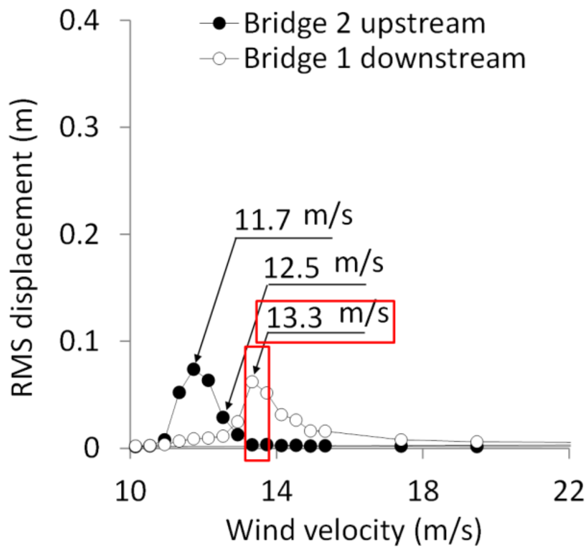
Close frequencies



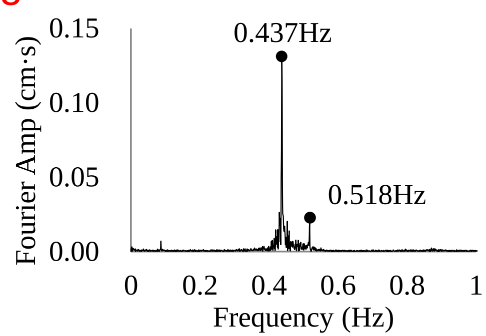
INTERACTIVE BEHAVIOR BETWEEN TWO DECKS

Wind velocity of 13.3m/s

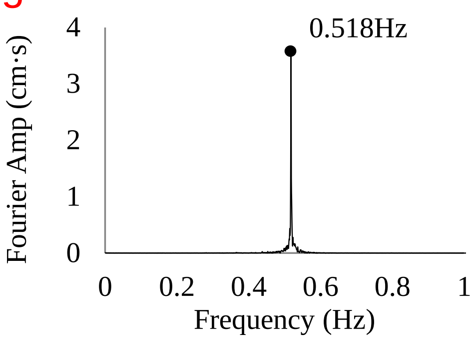




- PSD of Bridge 2

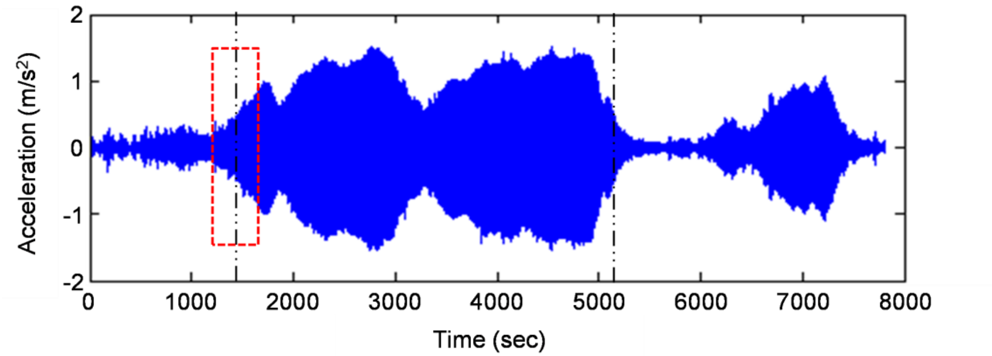


- PSD of Bridge 1

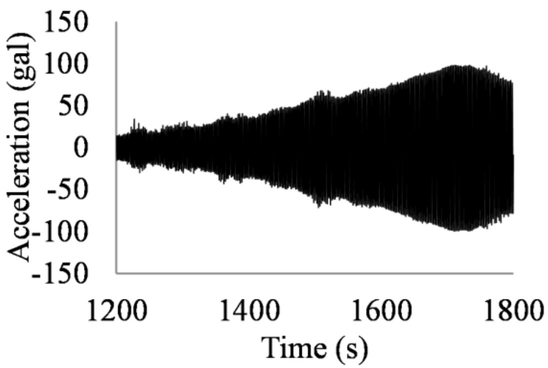


OBSERVATION FROM OPERATIONAL MONITORING DATA

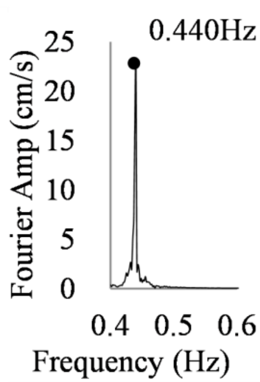
▶ *At the time of VIV starting*



Bridge 2

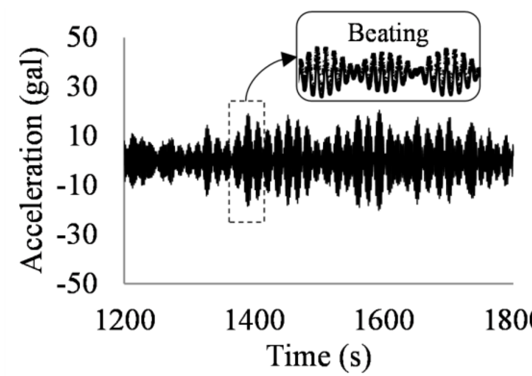


Acceleration

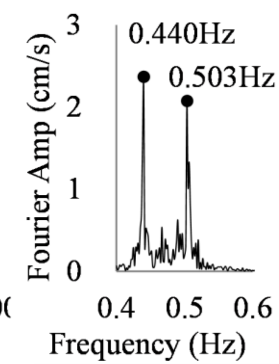


PSD

Bridge 1



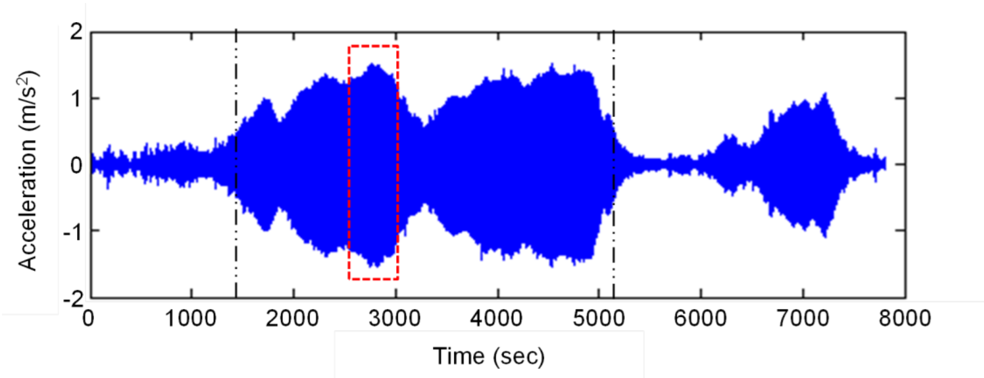
Acceleration



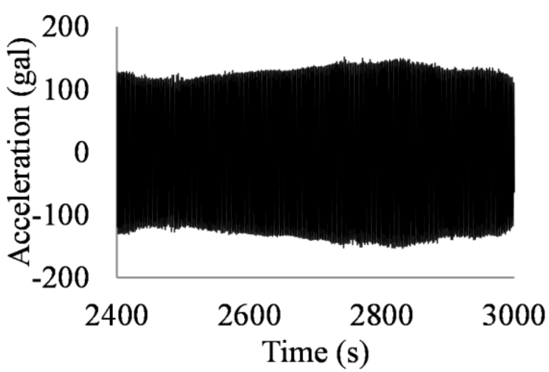
PSD

OBSERVATION FROM OPERATIONAL MONITORING DATA

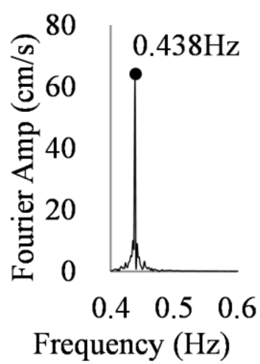
- *At the time of fully-developed VIV*



Bridge 2

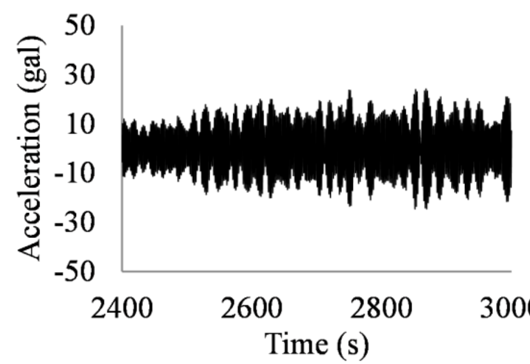


Acceleration

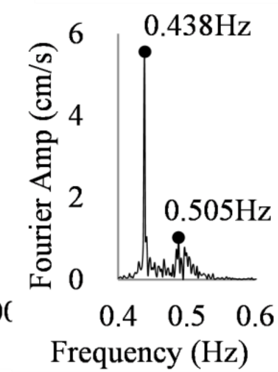


PSD

Bridge 1



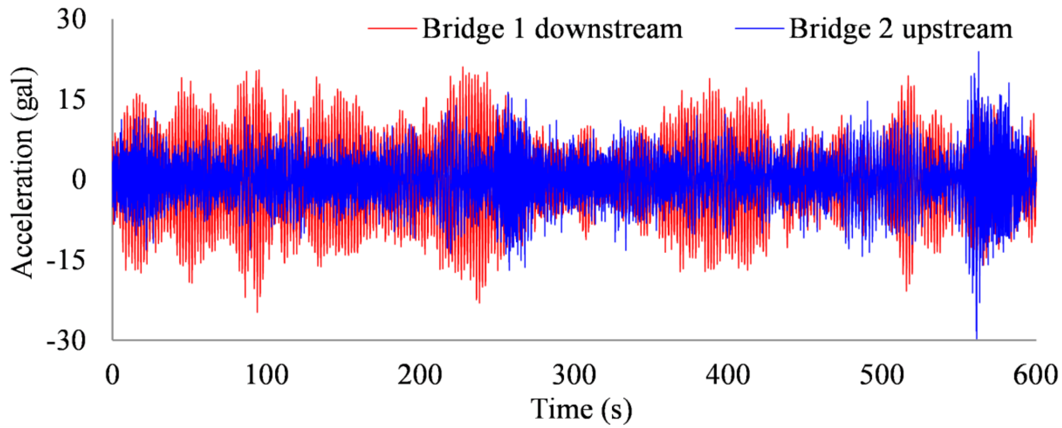
Acceleration



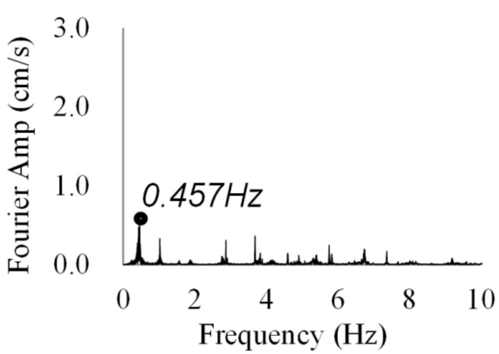
PSD

OBSERVATION FROM OPERATIONAL MONITORING DATA

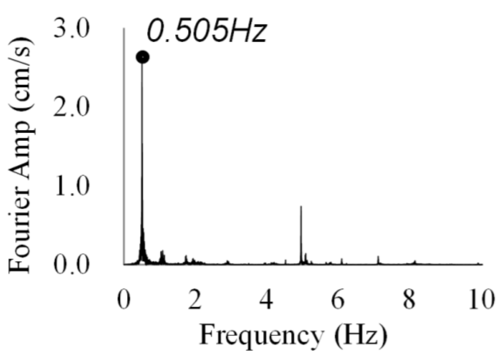
- *VIVs during typhoon Tembin, U = 14.8m/s*
  - *Acceleration of both bridges*



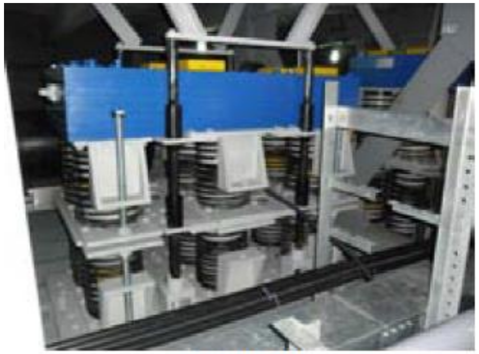
- *PSD of Bridge 2*



- *PSD of Bridge 1*



IMPLEMENTATION OF MULTIPLE TMDS (MTMD)



MTMD #1

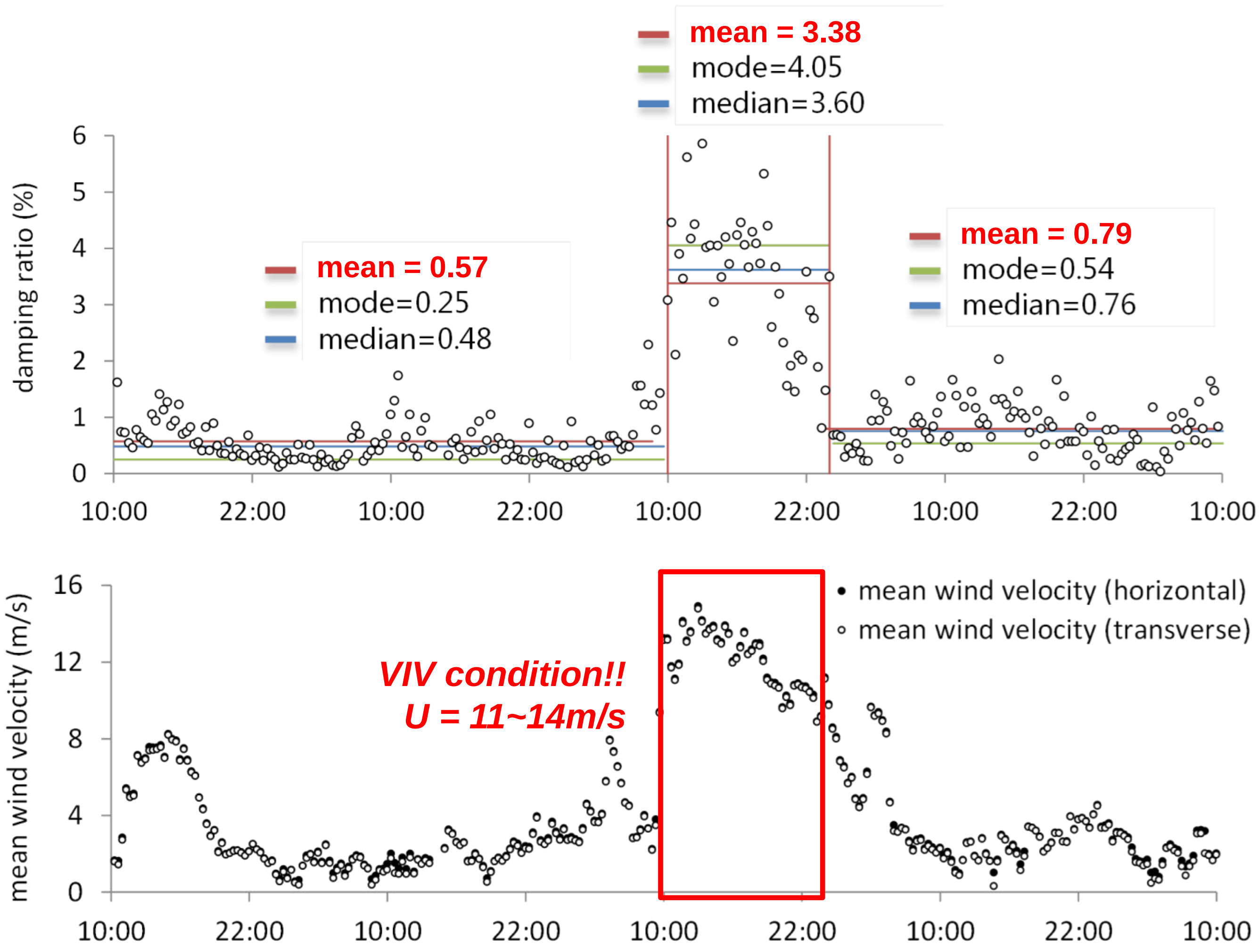
MTMD #2

MTMD #3

MTMD #4

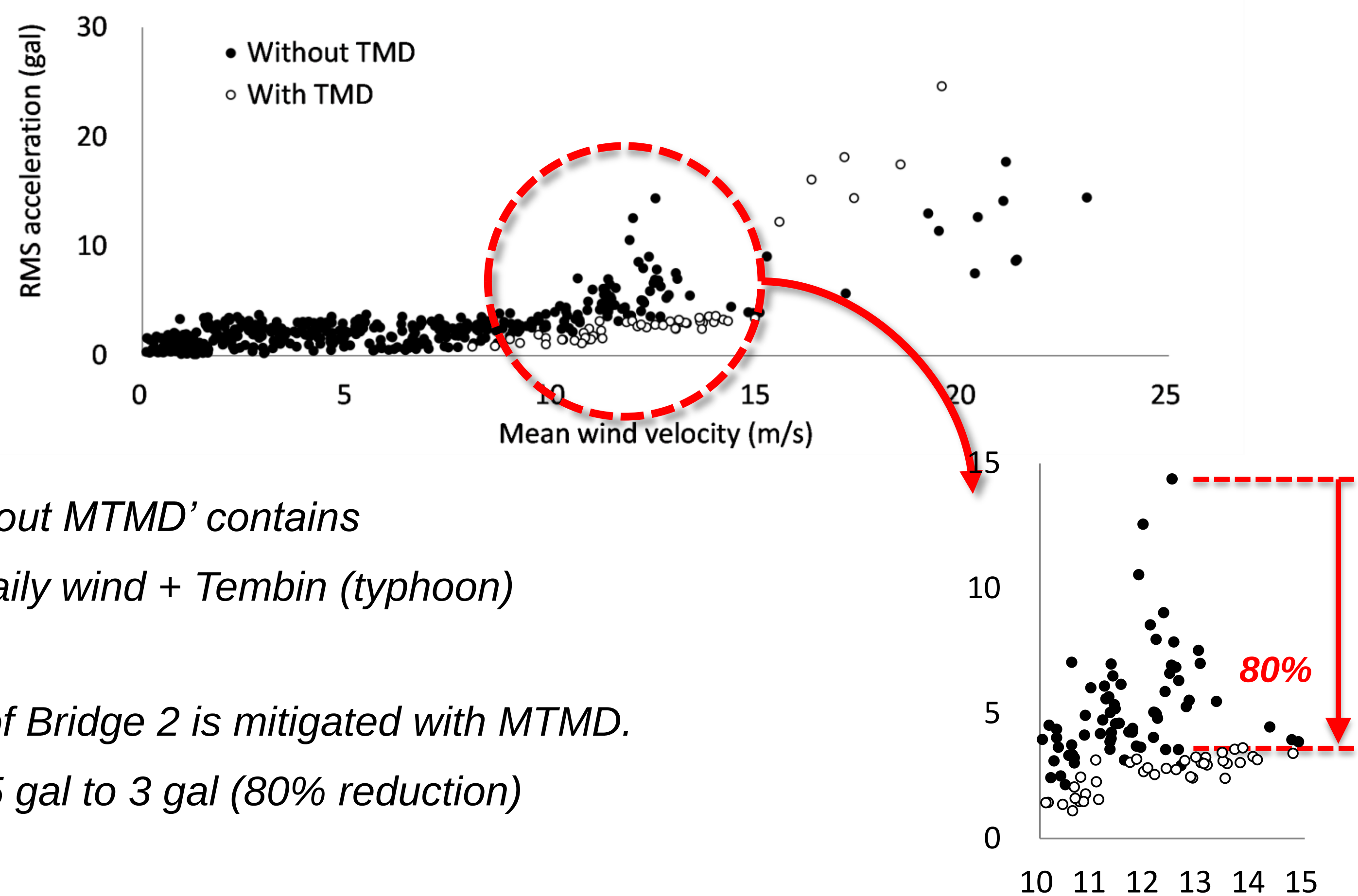
Seo et al. (2015). Mitigation of vortex-induced vibration of twin cable-stayed bridge girder using multiple tuned mass dampers. Mag. KSSC, 27(4), 57–62.

INCREASED DAMPING RATIO WITH MTMD



ENHANCED VIBRATIONAL SERVICEABILITY

Vertical acceleration of center of mid span in Bridge 2



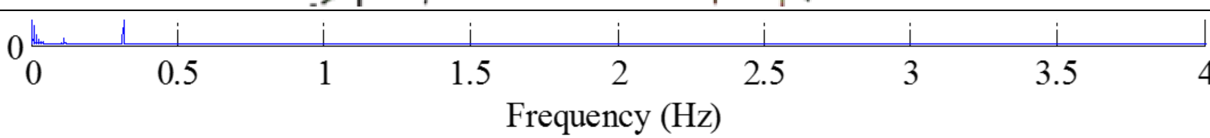
- *'Without MTMD' contains*
  - *Daily wind + Tembin (typhoon)*
- *VIV of Bridge 2 is mitigated with MTMD.*
  - *15 gal to 3 gal (80% reduction)*

VIV IN YI SUN-SHIN BRIDGE (OCT. 2014)

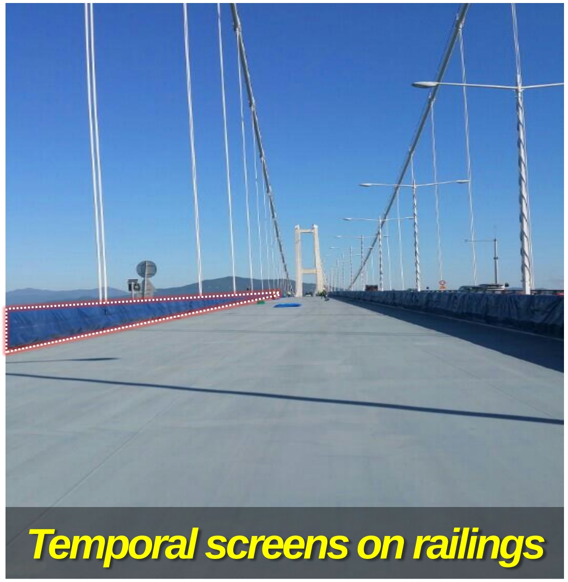
FIELD MEASURED DATA OF VIV

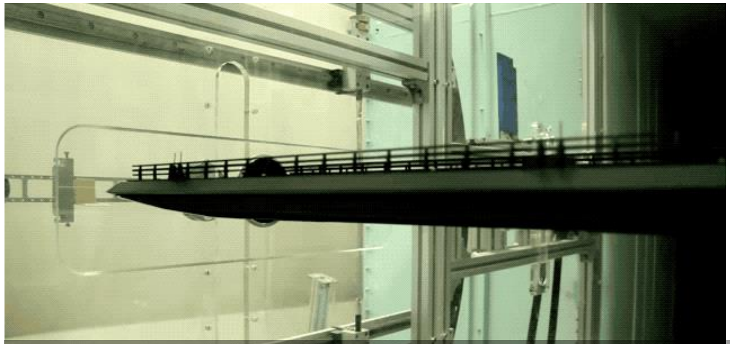
$$d_{allowable} = \frac{0.5\text{m}/s^2}{(2\pi f)^2} \times 2 = \frac{0.5\text{m}/s^2}{(2\pi \cdot 0.3176Hz)^2} \times 2 = 0.25\ (m)$$

Mode	Mode shape	Natural frequency (Hz)
1st Symmetric		0.0812
1st Anti-symmetric		0.1126
2nd Symmetric		0.1459
2nd Anti-symmetric		0.1757
3rd Symmetric		0.2155
		0.2393
3rd Anti-symmetric		0.2647
4th Symmetric		0.3128
4th Anti-symmetric		0.3161
		0.3604



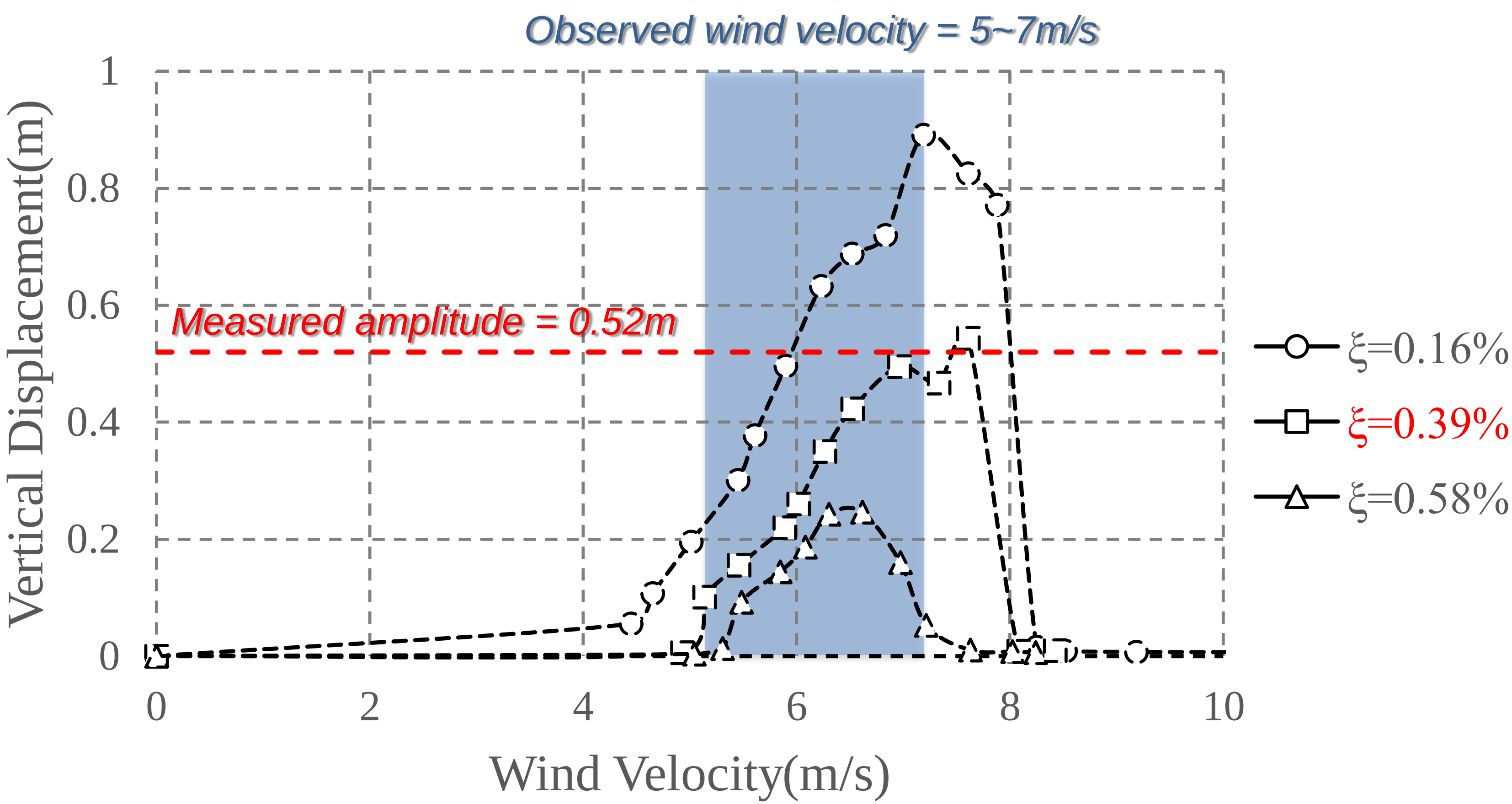
TEMPORAL SCREENS FOR CURING EPOXY ASPHALT PAVEMENT



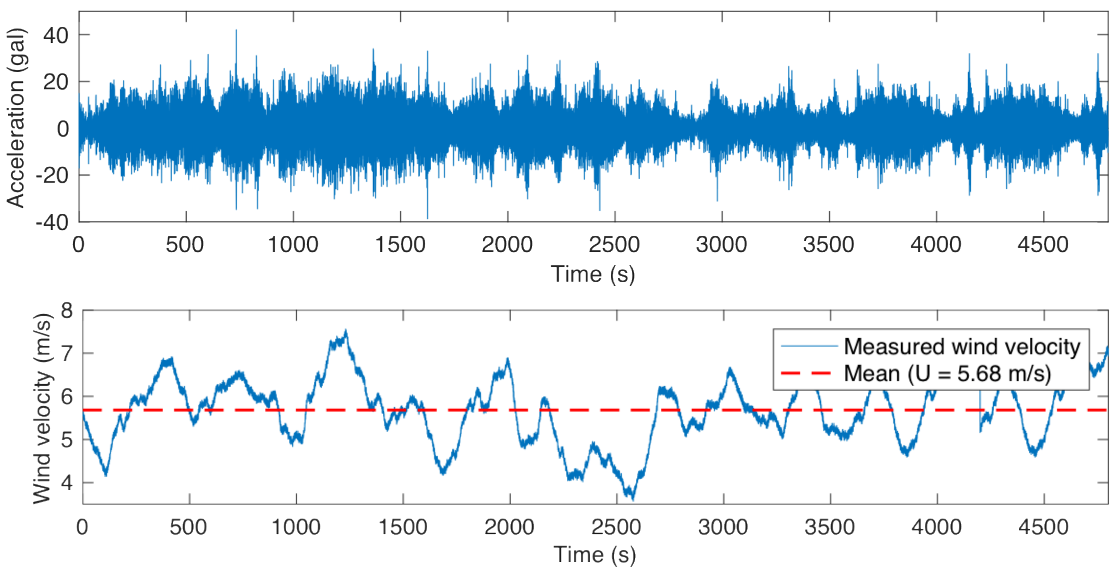


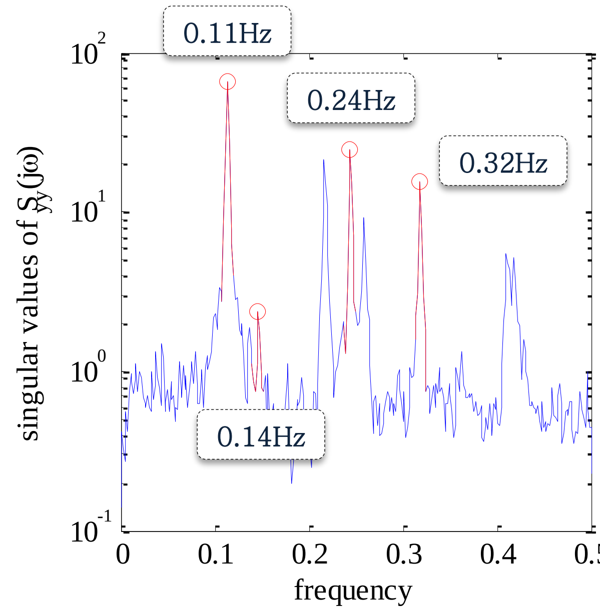
Section model test under low damping ratio (ξ = 0.16%)

A-V CURVES FOR THE SETUP OF 4TH MODE



IDENTIFIED DAMPING RATIOS FROM OMA

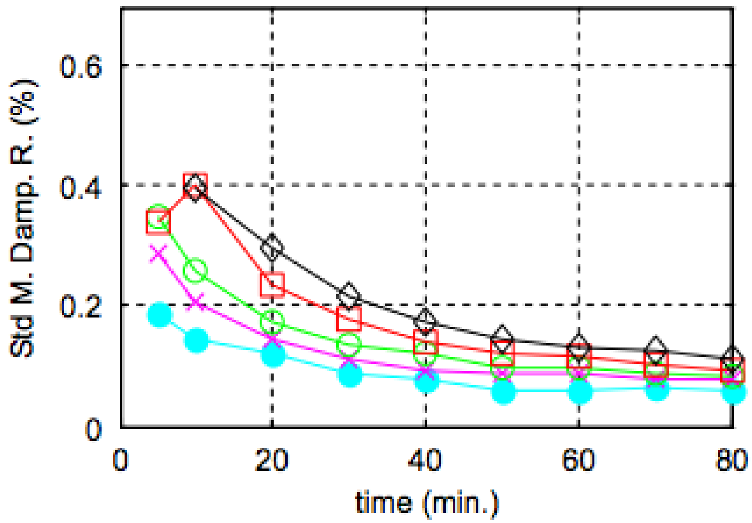




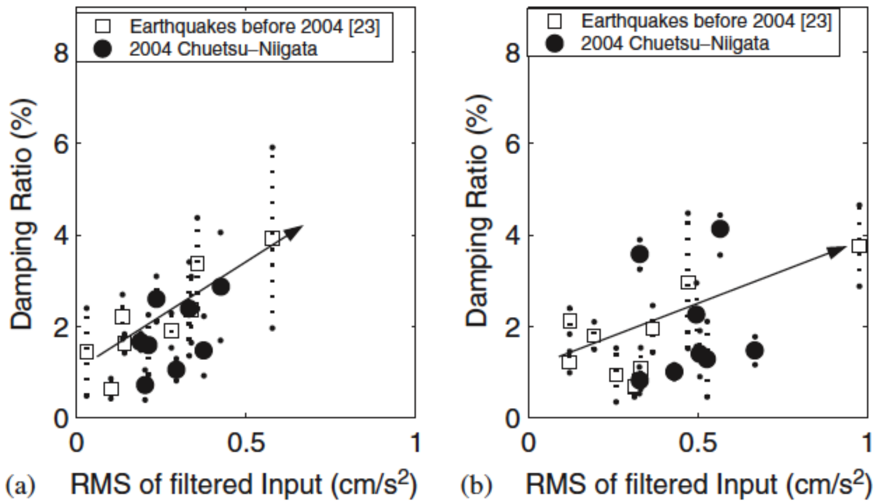
OMA	1st Vertical	2nd Vertical	3rd Vertical	4th Vertical
NExT-ERA	1.17%	2.42%	0.44%	**0.38%**
FDD	1.03%	2.46%	0.45%	**0.43%**

UNCERTAINTIES IN OMA-BASED DAMPING ESTIMATION

- ***Ill-posed problem***: *Sensitive to data length/analysis parameters (Magalhães et al., 2010)*
- ***Nonstationarity in data***: *OMA generally assumes stationary white noise process.*
- ***Nonlinearity***: *Amplitude-dependency (Siringoringo and Fujino, 2008; Chen et al., 2016).*



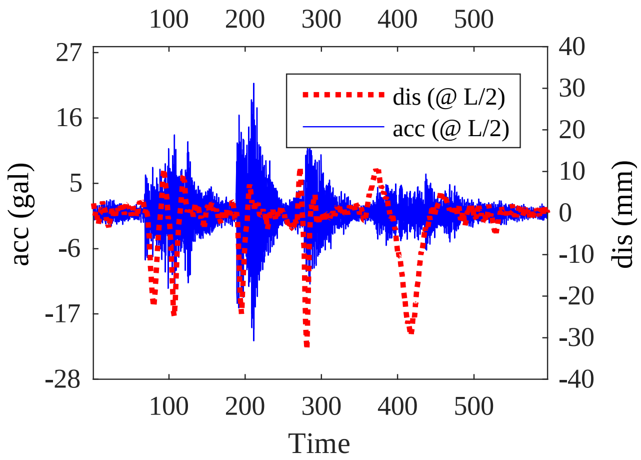
Magalhães et al. (2010). Damping estimation using free decays and ambient vibration tests. Mechanical Systems and Signal Processing, 24(5), 1274-1290.



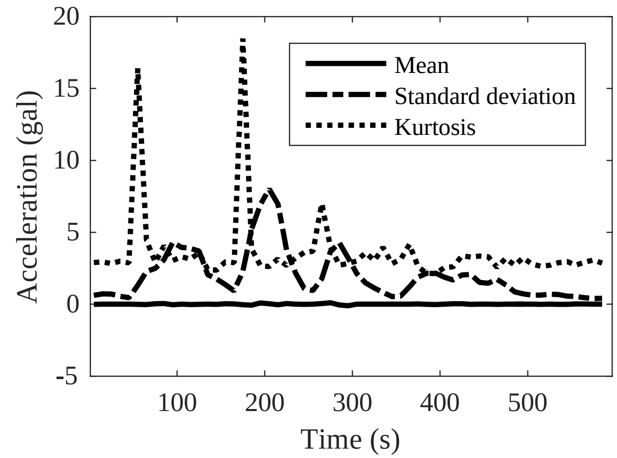
Siringoringo and Fujino (2008). System identification applied to long-span cable-supported bridges using seismic records. Earthquake Engineering & Structural Dynamics, 37(3), 361-386.

NONSTATIONARITY IN VEHICLE-INDUCED RESPONSE

Sparsely loaded vehicles in 2-lane bridges



Measured acc. and displ.

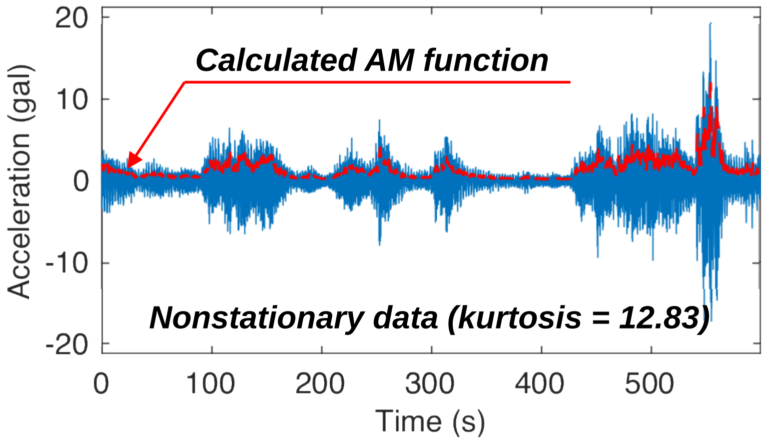


Corresponding statistics

SIGNAL STATIONARIZATION WITH AMPLITUDE-MODULATING FUNCTION

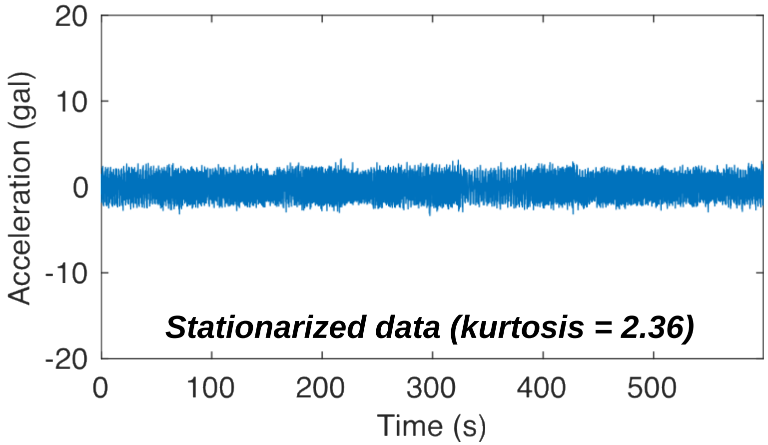
*Calculate **amplitude-modulating (AM)** function by **temporal root mean square** of nonstationary acceleration*

$$\Gamma(t) = C\sqrt{\frac{1}{T}\int_{t-T/2}^{t+T/2} a^2(\tau)d\tau}$$



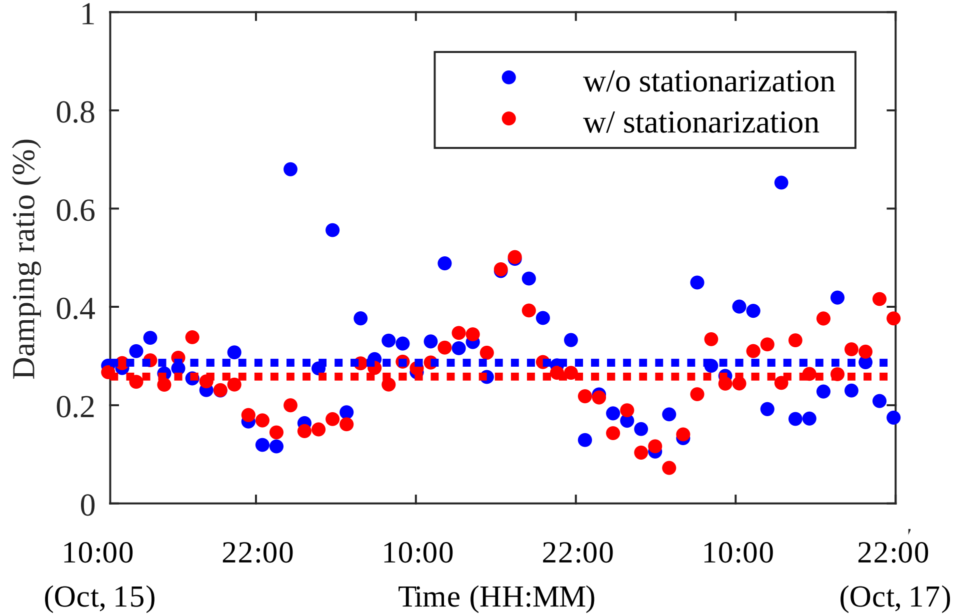
*Modulating of vehicle-induced acceleration for **approximated stationary acceleration***

$$a(t) \simeq \Gamma(t)\hat{a}(t) \rightarrow \boldsymbol{\hat{a}(t) \simeq a(t)/\Gamma(t)}$$



Chiang,and Lin (2008). Identification of modal parameters from nonstationary ambient vibration data using correlation technique. AIAA journal, 46(11), 2752-2759.

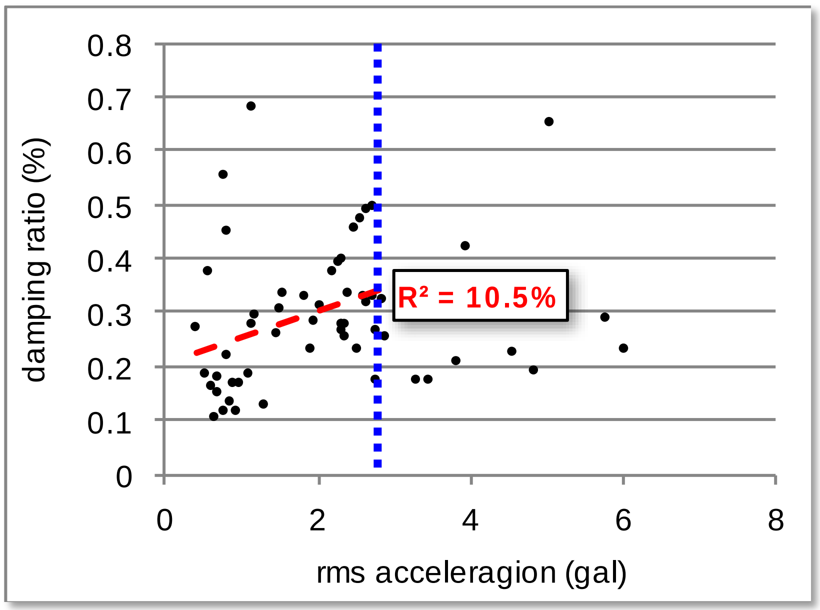
REDUCTION OF SCATTERING IN IDENTIFIED DAMPING RATIOS



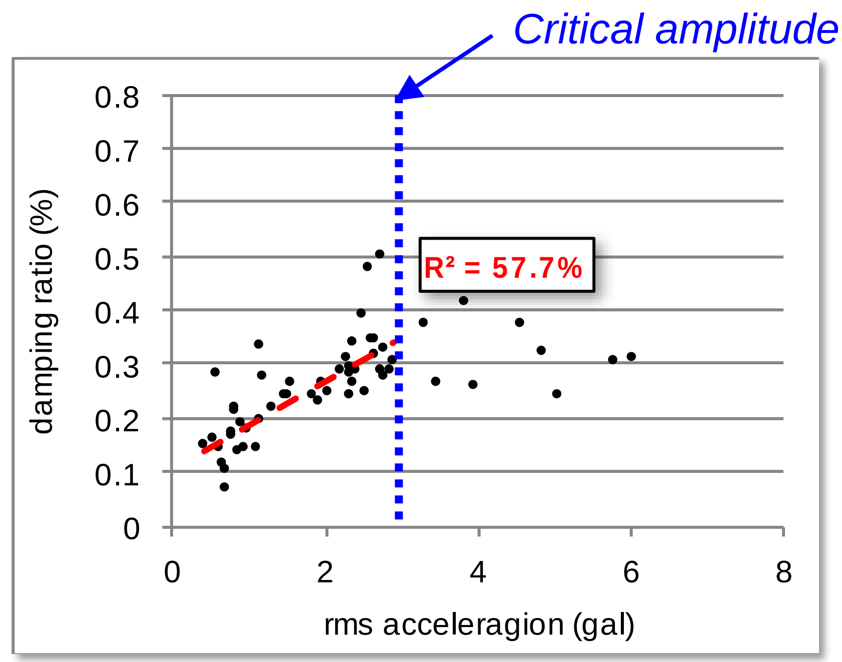
	Simple NExT-ERA	Optimal Parameters	Stationarization
Mean	0.3%	0.3%	0.3%
C.O.V	60.1%	44.6%	33.7%

ENHANCED TENDENCY OF AMPLITUDE-DEPENDENCY

W/O stationarization



W/ stationarization



CONCLUDING REMARKS

- *Several observations of VIV from operational monitoring were somehow related to low damping ratios of as-built bridges.*

- *Recent advances in technology such as use of high-strength materials and adoption of simpler details may result in a potential drop in inherent energy-dissipating capacity of structures, which is unfavorable in the serviceability aspect.*

SUGGESTIONS TO BRIDGE OWNERS

- *The bridge owner should confirm actual level of damping ratios for potential modes vulnerable to VIV, just after the completion of construction. This is a fundamental procedure in serviceability assessment of flexible bridges in the extension of wind tunnel tests performed in the design stage. If necessary, wind tunnel tests can be performed again based on the OMA results.*

THANKS FOR YOUR KIND ATTENTION.

Ho-Kyung Kim – Seoul National University, Korea

Special thanks to
Park, J., Kim, S. and Hwang, Y.
and
Ministry of Land, Infrastructure and Transport,
Regional Province of JeollaNamdo, and TE Solution

교량설계핵심기술연구단
Korea Bridge Design & Engineering Research Center

TU1406
COST ACTION

IABSE

TU1402
COST ACTION

DECISION THEORY
VALUE OF STRUCTURAL HEALTH MONITORING
SHM STRATEGIES
STRUCTURAL PERFORMANCE

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

NEW TECHNOLOGIES FOR CONDITION ASSESSMENT OF EXISTING STRUCTURES IN JAPAN

Hitoshi Furuta– Kansai University, Japan

Kansai University

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

INTRODUCTION -1

In Japan, a new project called “Cross-ministerial Strategic Innovation Promotion Program (SIP)” supported by Japanese government has started three years ago, one of which deals with the innovative technologies for maintenance, renewal and management of infrastructures.

In this program, 60 new technologies have been developed using IT, sensor and material engineerings.

The 60 technologies are classified into five categories:

1) Inspection, monitoring and diagnosis technologies,
2) New structural materials,
3) Information and communication technologies,
4) Robotics
5) Asset management.

The program also covers such infrastructures as bridge, earthwork, river, tunnel and transportation.

INTRODUCTION -2

In this paper, representative technologies developed in this program will be introduced by paying attention to the condition assessment of existing structures.

For steel structures, several non-contact testing methods have been developed by using laser, radar, magnetic, electromagnetic wave, and vibration technologies.

For concrete structures, radar, X-ray, ASEM, image and pattern recognition technologies are employed.

For all infrastructures UAV, sensor, and wireless network technologies are very useful, because they can provide us with available information with ease.

INTRODUCTION -3

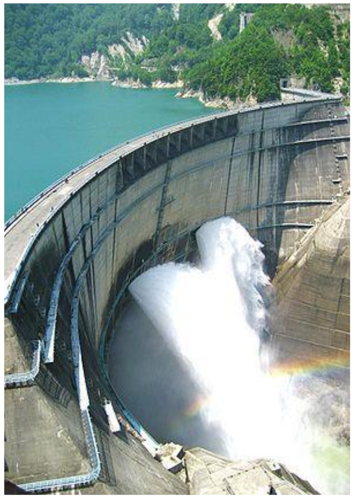
In addition, this paper introduces the application of chaos theory into structural damage assessment in non-destructive inspection and vibration-based health monitoring.

Firstly, a new impact acoustic method based on attractor analysis is described, which can improve the accuracy of conventional methods of investigating the frequency domain.

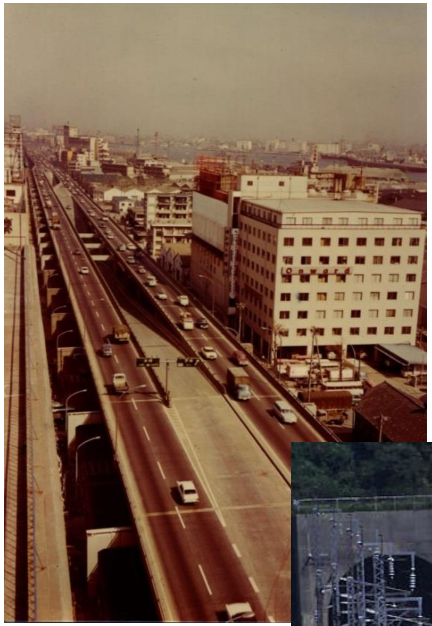
The proposed method detects exfoliations and confirms the filling conditions inside steel-concrete composite slabs by evaluating the difference of convergence processes of attractors reconstructed from acoustic signals.

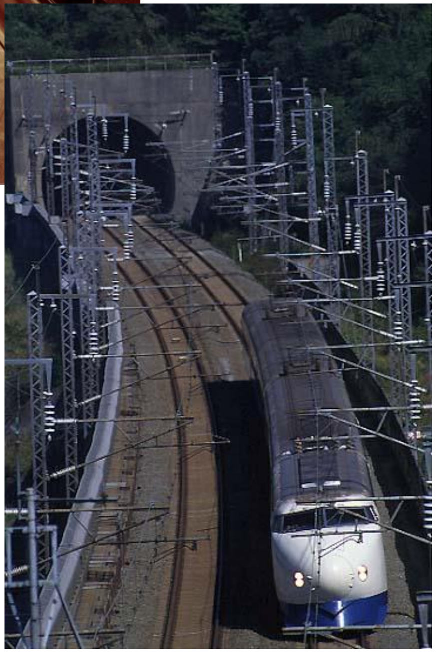
Another attempt is made to develop a baseline-free vibration-based health monitoring method that can detect damage locations without using any baseline data. The effectiveness of applying chaos theory to structural damage assessment is discussed through field experiments and numerical simulations.

What is infrastructure?



High Economic Growth from 1960s to 1980s





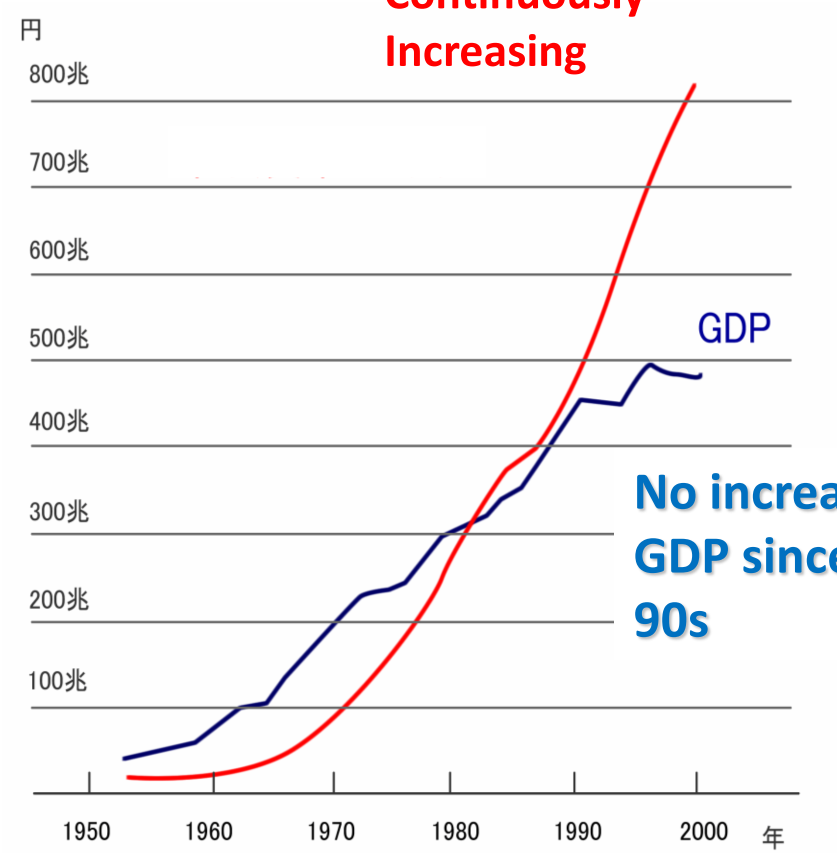
A lot of infrastructure in Japan was developed in a period of high-economic growth from 1960s to 1980s. Year1964 (last Tokyo Olympic)

Tokyo Metropolitan Expressway opened.

Operation of the high speed train, or Shinkansen, started also in 1964.

Japan: Facing a Turning Point

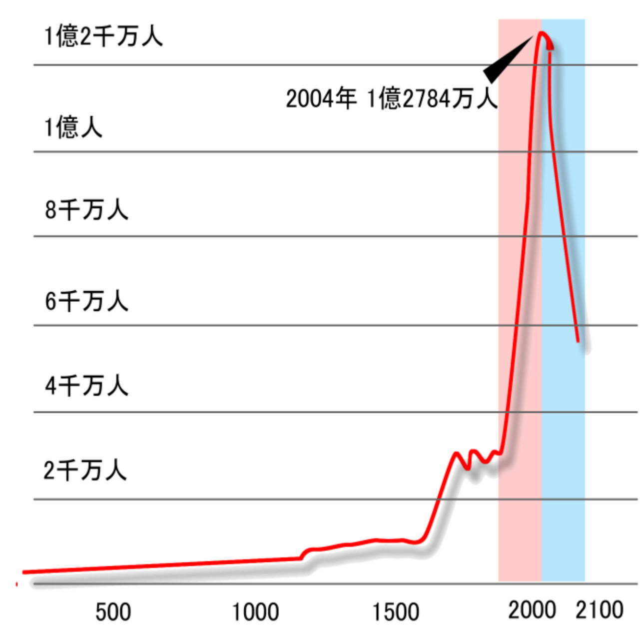
Stocks
Continuously
Increasing



No increase in GDP since late 90s

Depopulation!

The peak:
128 million people in 2004
→ Now 125 mil.
→ less than 90 mil. in 2060



Infrastructure shall be managed under a very limited budget and with limited human resources

Society changes, but infrastructure remains old.

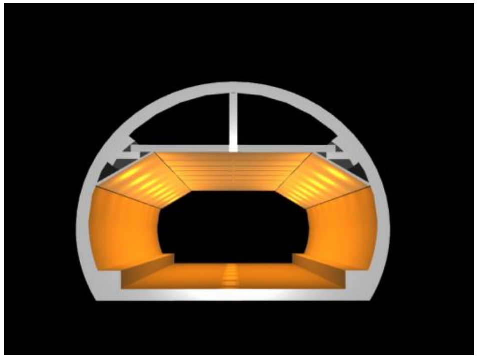
After 50 years, the number of vehicles increased drastically. But, the functional value decreases year by year.

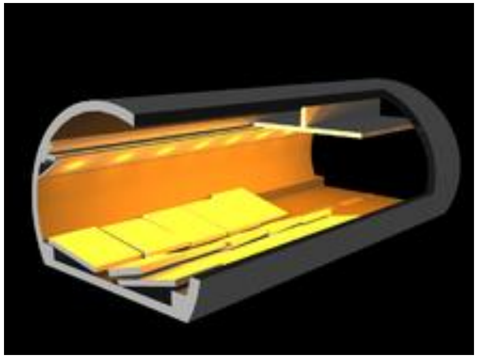
Traffic jam often occurs due to deck repairs. Expressway changes to a "world longest parking lot."

40-year old Highway Tunnel Accident

The ceiling panels suddenly fell down. Several vehicles were crushed and there were 9 fatalities.

The first human-loss accident related to infrastructure maintenance in Japan.



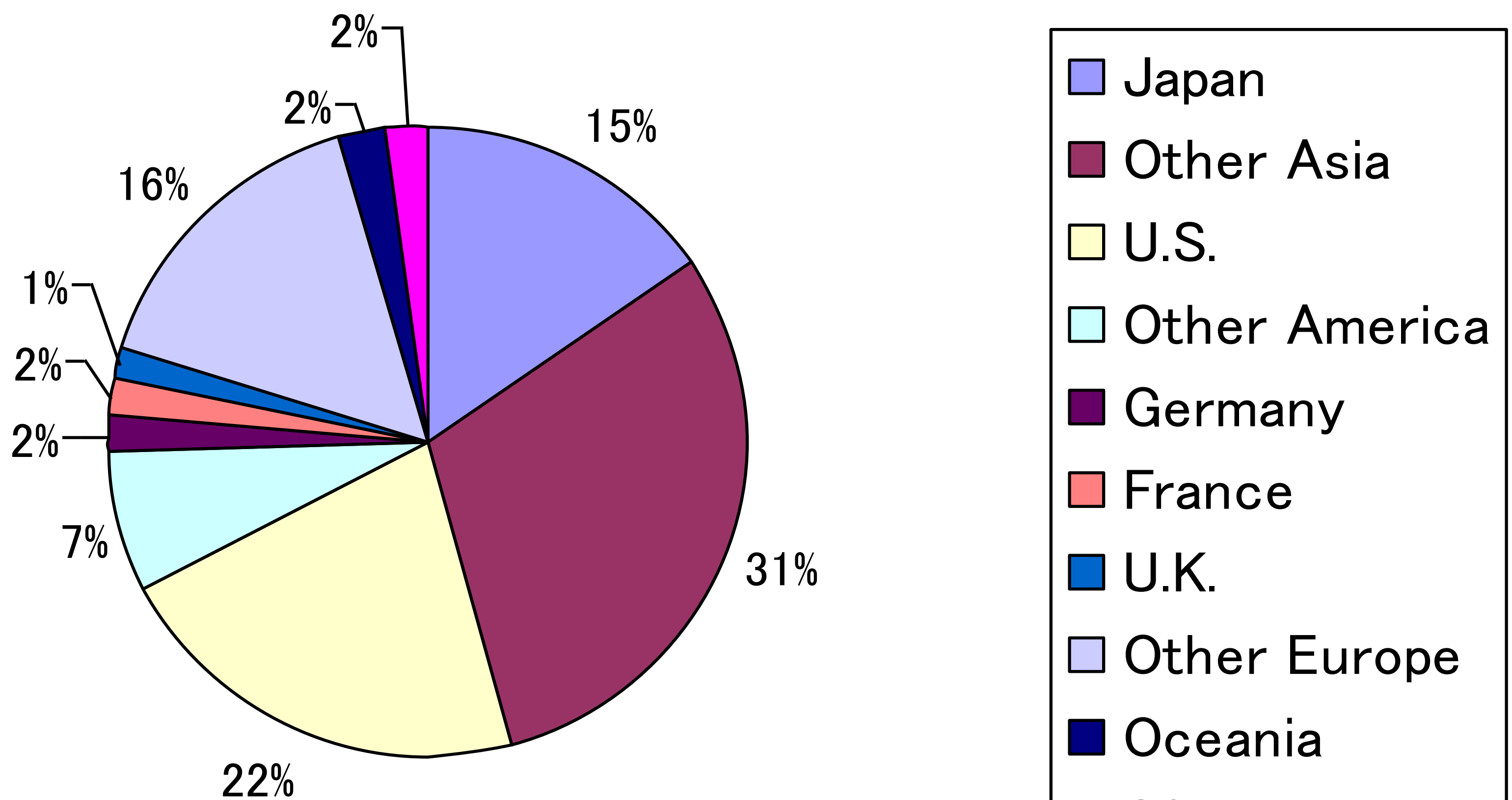


Hands-on visual inspection of infrastructure every 5 years has been compulsory since July 1, 2014

Japan: Living with Natural Disasters

Earthquakes, Tsunamis, Typhoons, Volcano eruptions, Floodings, etc.

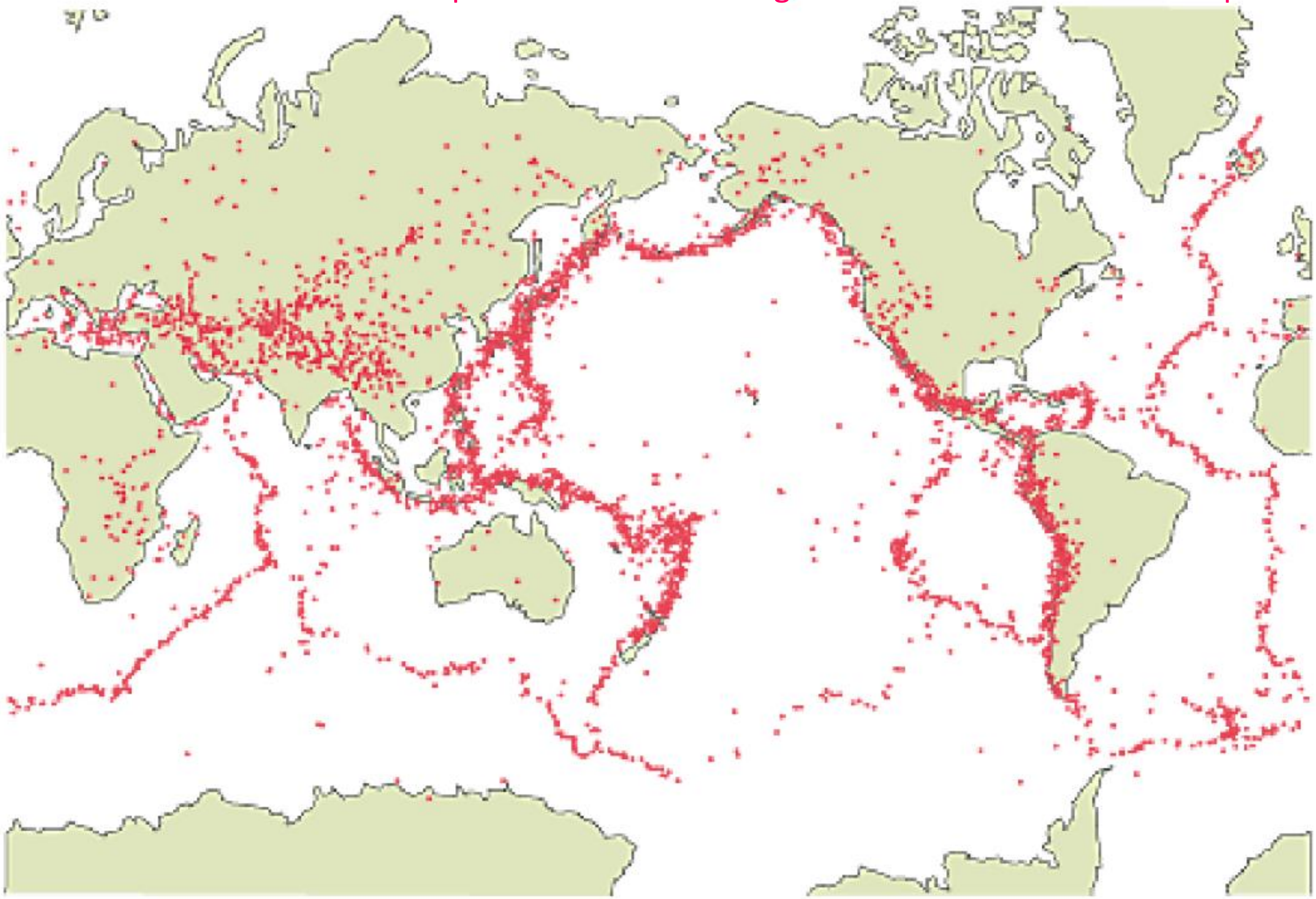
Natural-disaster-related losses (from 1970 to



Japan+other Asian countries+USA=70% vs. European countries=20%

Distribution of Epicenters

Red dots show location of epicenters located along the Pacific rim. Cannot see Japan.



Civil engineering alone is not sufficient for proper management of infrastructure.
A variety of cutting-edge technologies shall be integrated.

I&C Technology

Robotics / AI

Sensing

Chemistry

Materials

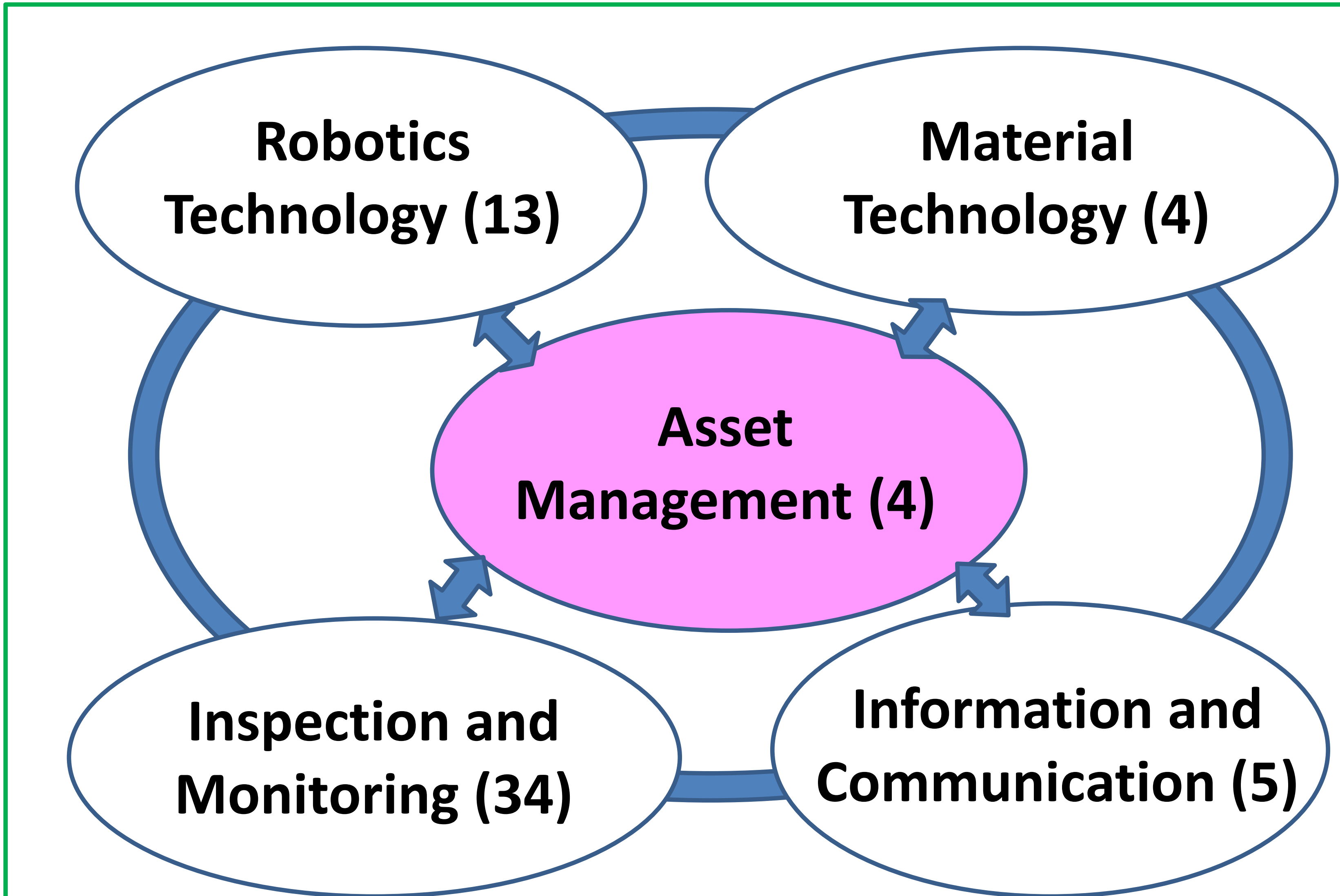
Structures

This is a new and innovative approach to Civil Engineering!

Infrastructure Management System

* Risk Management under abnormal conditions
* Daily Asset Management under normal conditions

Budget: 30Mil. Euro/ year for 60 themes



More than 1,500 researchers & engineers are involved.
Universities, Governmental Institute, Public institute and Industries.

Machine-assisted Inspection of Bridges

Manual inspection using a movable inspection gondola

Hands-on inspection relies on human senses.

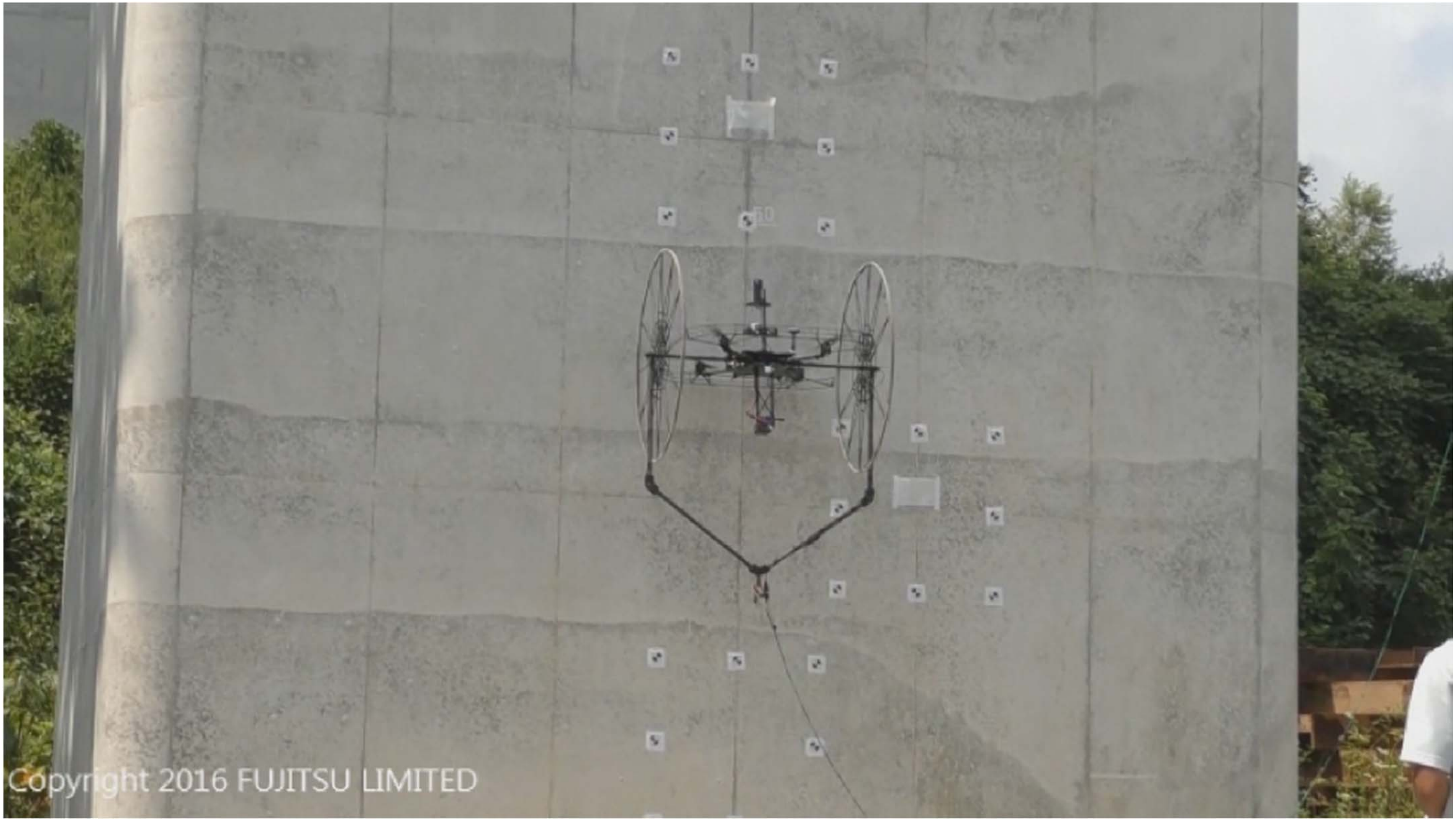
- eyes
- ears
- hands

Analogue to Digital

Manual to Automatic

Flying robot. Drone, UAV, ROV

2輪車を使った橋脚点検ロボット（富士通＋北大＋東大）

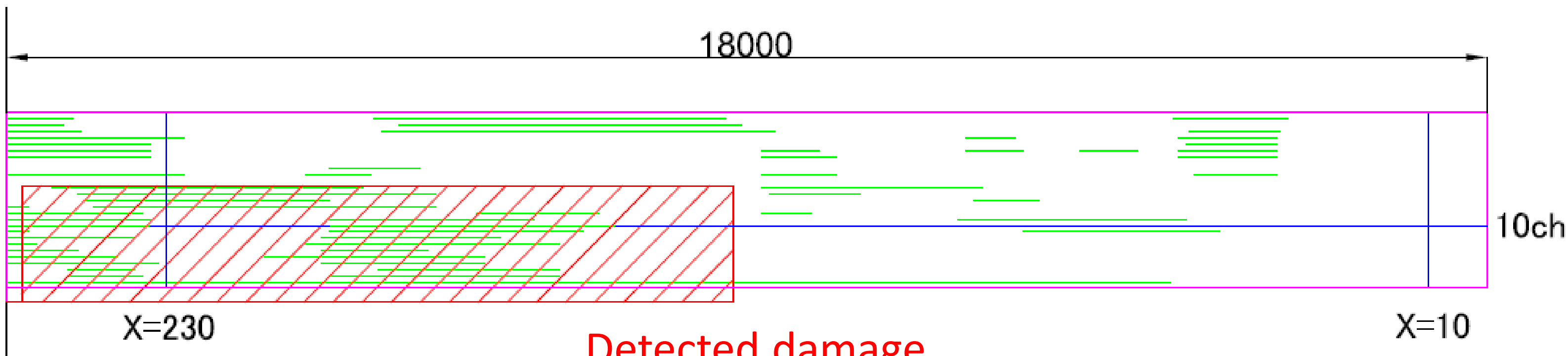


Bridge-Deck Damage Detection using Radar

Radar

80km/hour

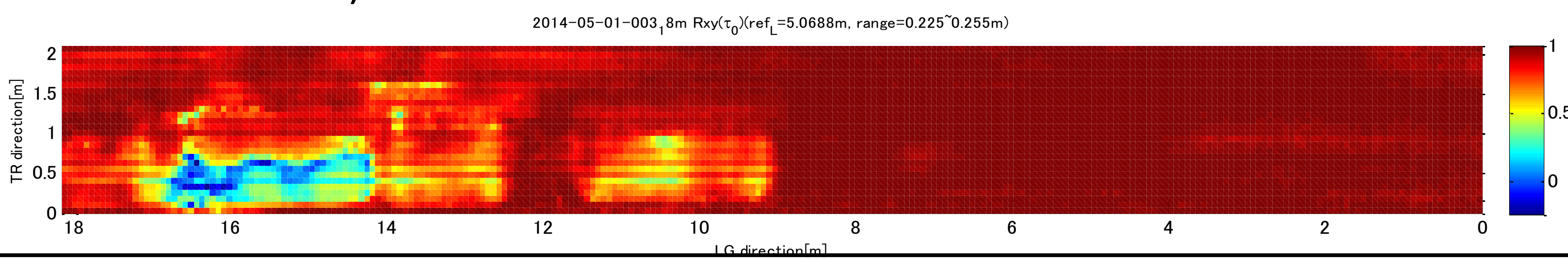
(Fast Scanning and Non-contact detection)



Detected damage

18 m bridge deck

A Result of Signal Processing (Processing Time: Around 1 min.)



Inspection Data Management using 3D-CAD Model

Geotagging technology

Estimate camera's pose using sequential Omni-directional images to add geographical data to proximity images.

Proximity image

Omni-directional image

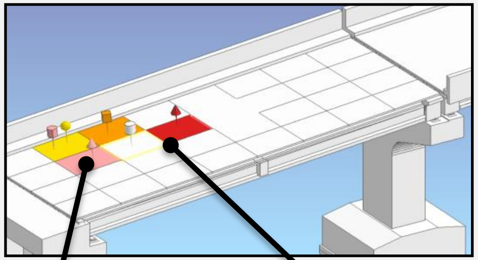
Pose estimator

Geotagged proximity image

Inspection data management using 3D-CAD model

Inspection data

3D-CAD



Geotagged images

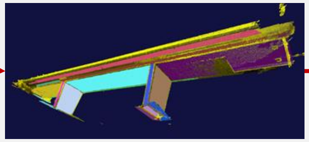
Inspection records

3D-CAD model generation technology

Semi-automated generation of 3D-CAD model from 3D point cloud.

Automatic registration

Automatic principal plane detection



Application software example

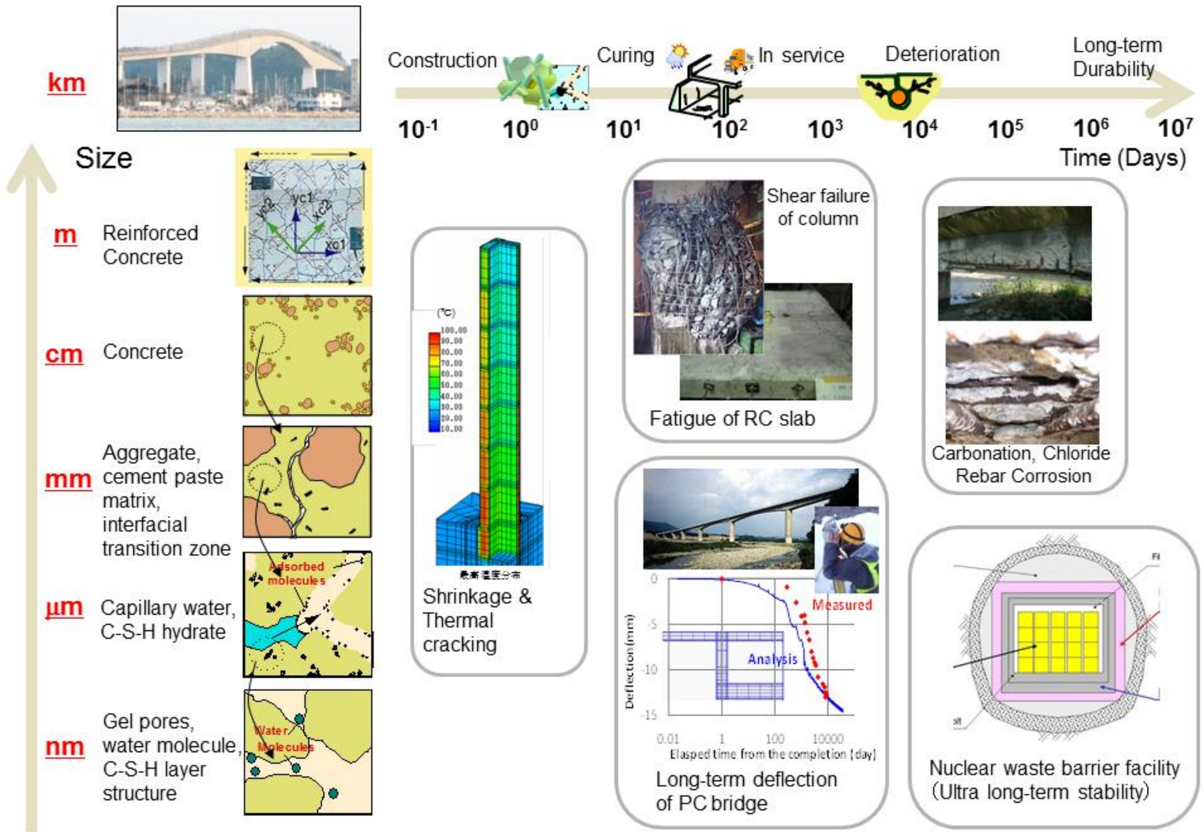
A bridge inspector can refer past inspection data on 3D-CAD model using tablet PC.

R&D for Long-life Concrete Decks

Management of concrete deck deterioration is essential → Assessment of remaining life and rational bridge management for PDCA cycle

Base technology: **Multi-scale, multi-chemo-physics modeling**

Prof. Koichi Maekawa
Univ. of Tokyo



Non destructive testing

Repair and strengthening

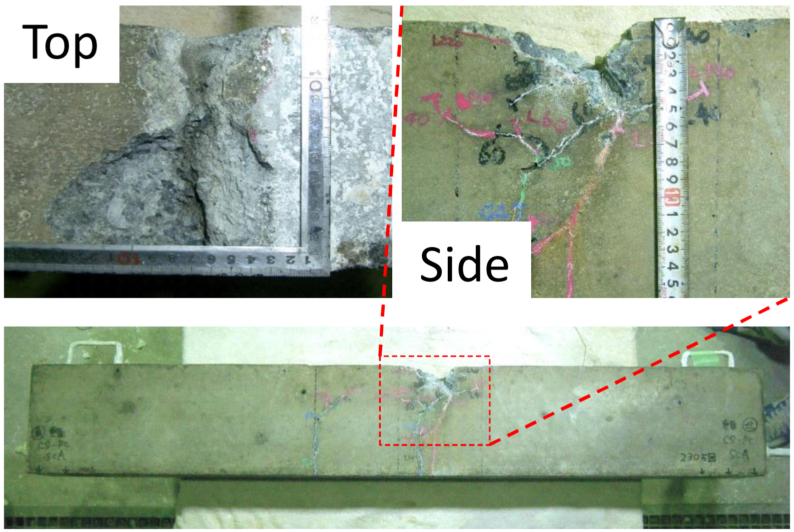
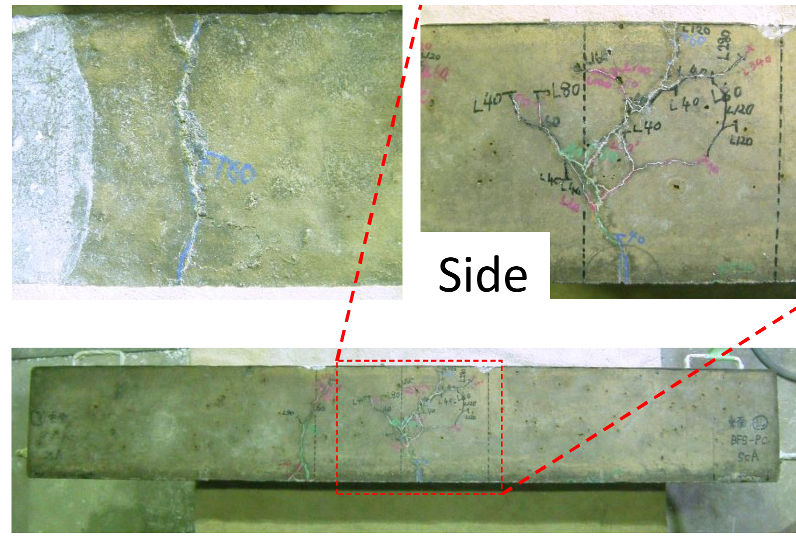
Assessment information

Asset management data base

action

Super-high Durable Concrete

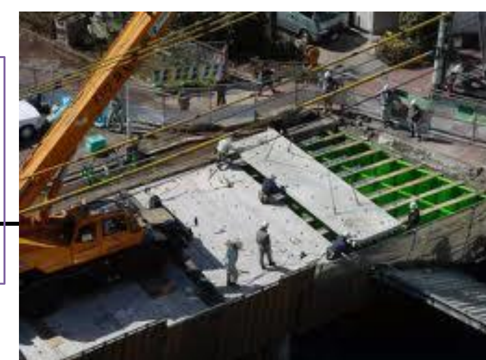
【Investigation of fatigue loading effects on freeze-thaw resistance in concrete】

Crushed sand	Blast furnace slag fine aggregate
	
Freeze-thaw 360 cycles, Fatigue test 2.3 mil.	Freeze-thaw 600 cycles, Fatigue test 4mil.

・Development of concrete strong against cold weather & cyclic loading with blast furnace slag

・Development of quality management system for blast furnace slag fine aggregate

To prolong life of concrete deck slab



Keys of the R&D

1) Must be user-oriented

→ Must be attractive to users

2) Accurate, high speed and inexpensive

3) Reliable long-term prediction

4) Durable high-quality material

5) Wide-scale database management

After couple of years, fruitful outcomes are expected.
The work will continue even after SIP ends in 2019,

Structural damage assessment based on chaotic Theory

This study aims to introduce chaos theory into structural damage assessment using non-destructive inspection and vibration-based health monitoring.

First attempt is made to develop a new impact acoustic method using attractor analysis.

Second attempt is made to develop a baseline-free damage detection method using chaotic excitation and recurrence quantification analysis.

Attractor-based impact acoustic method for identification of exfoliation of concrete structure

Objectives

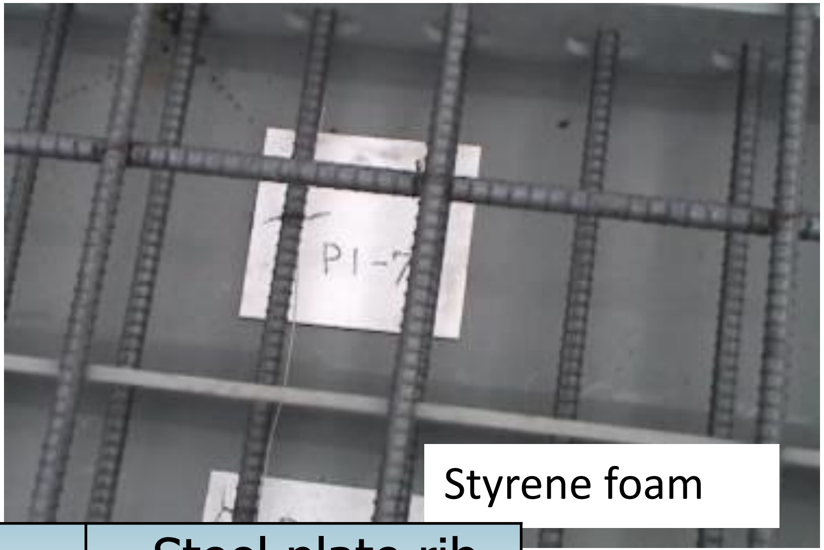
Clarify the problems of conventional impact acoustic method investigating frequency domain.

Propose a new impact acoustic method using attractor analysis.

- Detect the difference between intact part and damaged part by evaluating convergence process of attractor made from acoustic signals.

Demonstrate the applicability and efficiency of the proposed method through field experiments.

Fields experiments



Material	Size Unit:mm	Steel plate rib form
Styrene foam Thickness: 5mm	50×50	1
	100×100	1
	200×200	1
Styrene foam Thickness: 1mm	50×50	2
	100×100	3
	200×200	6
	300×300	2

Attractor-based evaluation of impact acoustics

Attractor-based evaluation has been successfully applied to many engineering areas as one of advanced health monitoring system.

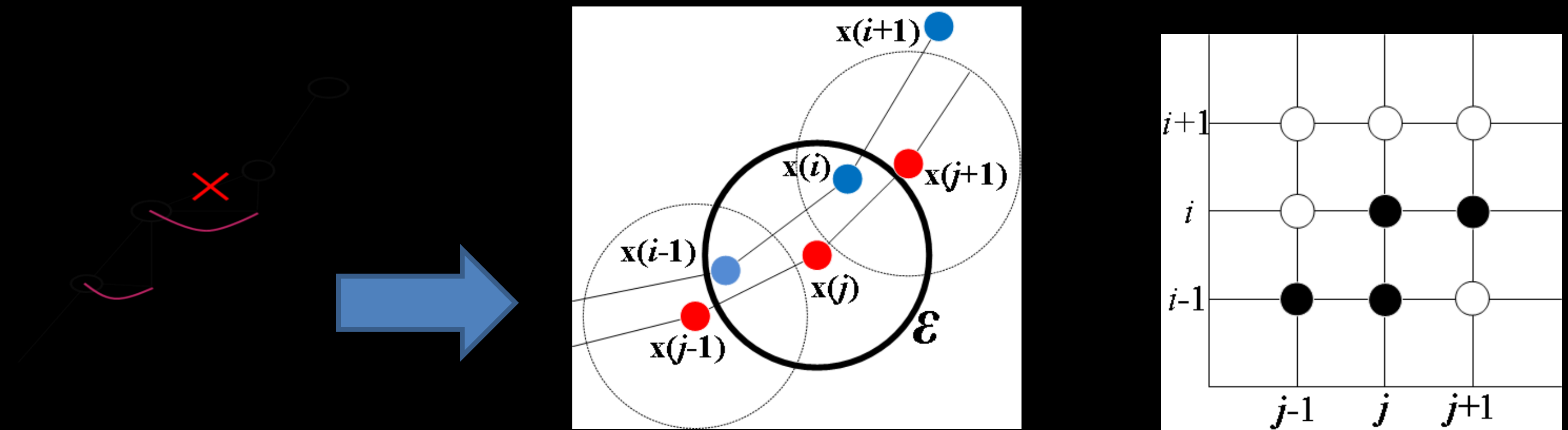
The proposed method assesses the integrity of the observation structure by evaluating the convergence process of reconstructed attractors toward a fixed point from the results of Lyapunov exponent analysis.

Vibration-based monitoring using chaotic excitation and recurrence quantification analysis

Evaluation scheme for damage detection

Amplitude of response data observed from damage element is large.

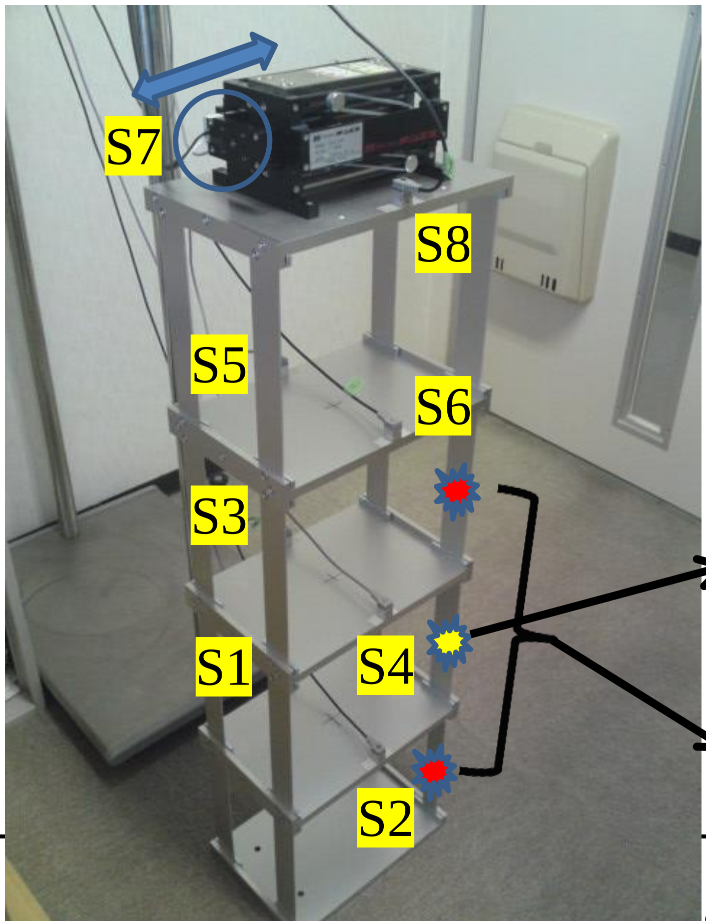
ε: minimum standard deviation from response data



Damage reduces recurrence points !!

The % recurrence obtained at damage element indicates the smaller value than that obtained at intact element.

Laboratory experiments

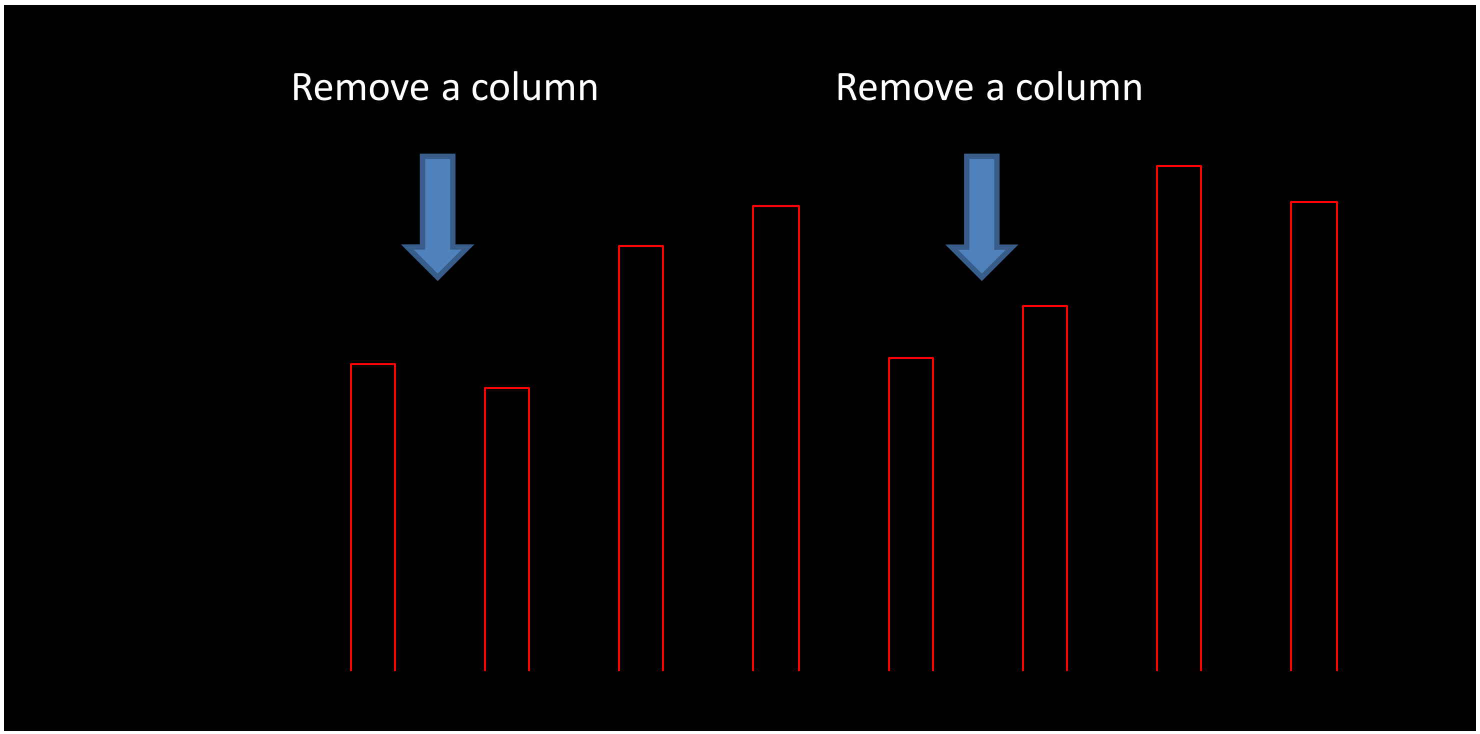


Measurement Points

* 1st story: Sensor1, Sensor2
* 2nd story: Sensor3, Sensor4
* 3rd story: Sensor5, Sensor6
* 4th story: Sensor7, Sensor8

Damage Scenario 1	Remove a column at 2nd story
Damage Scenario 2	Remove columns at 1st and 3rd stories

Results of damage scenario II



From the decreasing of %REC, it is predicted that the 1st and 3rd stories have something abnormal.

CONCLUSION

New technologies developed in Japan were introduced, in which consists of 60 innovative technologies. They are related to inspection, materials, IT, robotics, and asset management.

All the technologies have been investigated paying attention to their advantages and availability. Through the field testing, a part of them will be utilized in practical maintenance and management of existing structures.

While new technologies developed in SIP were described here, there are more new technologies has been developed in Japan. For instance, Hanshin express corporation has developed so called “Mitsukeru-kun K” which is used for inspecting steel bridge orthotropic deck plate by Eddie current sensor.

CONCLUSION-2

The new damage detecting method was proposed here. It could detect changes of the phase structure of the attractor reconstructed from the acoustic signals sensitively even for small defect models that were undetected using the conventional method with FFT.

This study attempted to introduce chaos theory into structural damage assessment using non-destructive inspection and vibration-based health monitoring.

A new impact acoustic method using attractor analysis. Through comparisons with the results obtained by the ordinary method evaluating frequency domain, it was found that the proposed method could catch effectively the difference between intact part and damaged parts that could not be recognized by ordinary method.

Acknowledgement

The author is grateful for the support of SIP project. Especially, Prof. Fujino (Project Director) and Dr. Ishizuka are acknowledged for providing information of SIP project and valuable slides.

TAHNK YOU FOR YOUR KIND ATTENTION!!

Hitoshi Furuta – Kansai University, Japan

Kansai University

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

PERFORMANCE INDICATORS FOR ROAD BRIDGES

Overview of findings and future progress

Alfred Strauss – University of Natural Resources and Life Sciences (BOKU), Austria

Ana Mandic Ivankovic – University of Zagreb, Croatia

Jose C. Matos – University Minho, Portugal

Joan R. Casas – UPC – BarcelonaTech, Spain

Sérgio Fernandes – ANSER Lda., Portugal

BOKU

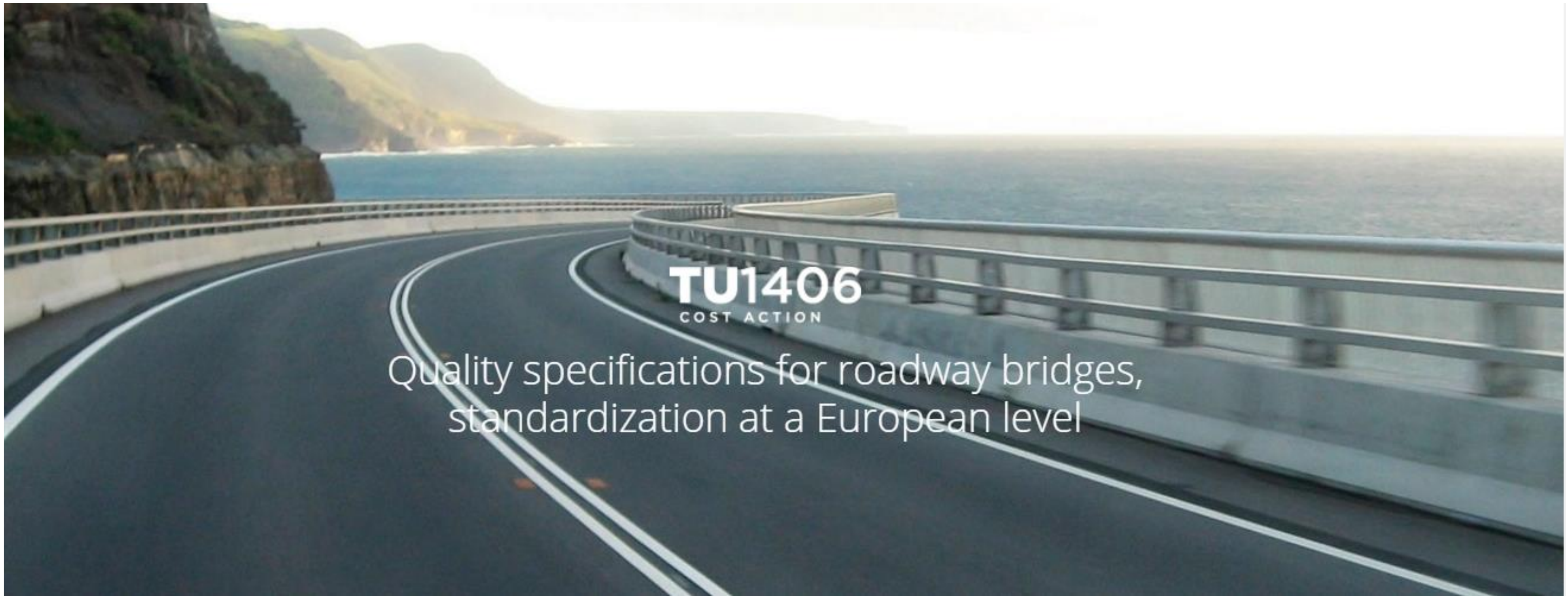
ANSER, Lda.
Arquitectura e Engenharias

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

INTRODUCTION

Main objective of COST ACTION TU1406 is to develop a **guideline** for the establishment of **Quality control** plans in **roadway bridges** at a **European level** and moreover it will also be analyzed the **possibility of incorporation new indicators** related to sustainable performance of roadway bridges.



ORGANIZATION

WG5. Drafting of guidelines/recommendations

- Existing documentation (format and content)
- Document preparation
- Easy to use document

WG4. Implementation in a case study

- Benchmarking
- Validation
- Discussion

WG1. Performance indicators

- Technical indicators
- Environmental indicators
- Others

WG2. Performance goals

- Technical goals
- Environmental goals
- Others

WG3. Establishment of a QC plan

- Bayesian nets
- Procedure to develop a QC plan for a single bridge

1st Survey phase

Questionnaire associated with predefined performance indicator

Performance indicators indicated in selected attached documents

2nd Survey phase

Nomination of MC members for:

- contacting roadway owners and operators
- uploading I-DOC, E-DOC, B-DOC

Nomination of operating persons per country for:

processing national documents according to guidelines

screening I-DOC, E-DOC, B-DOC

Core group WG1

processed documents

transferring to Database

Core group WG1-3

analyzing PI database

Geneva → **January** → **Belgrade**

Publishing official COST e-Book WG1 activities

Preparation of WG1 activity and endreport

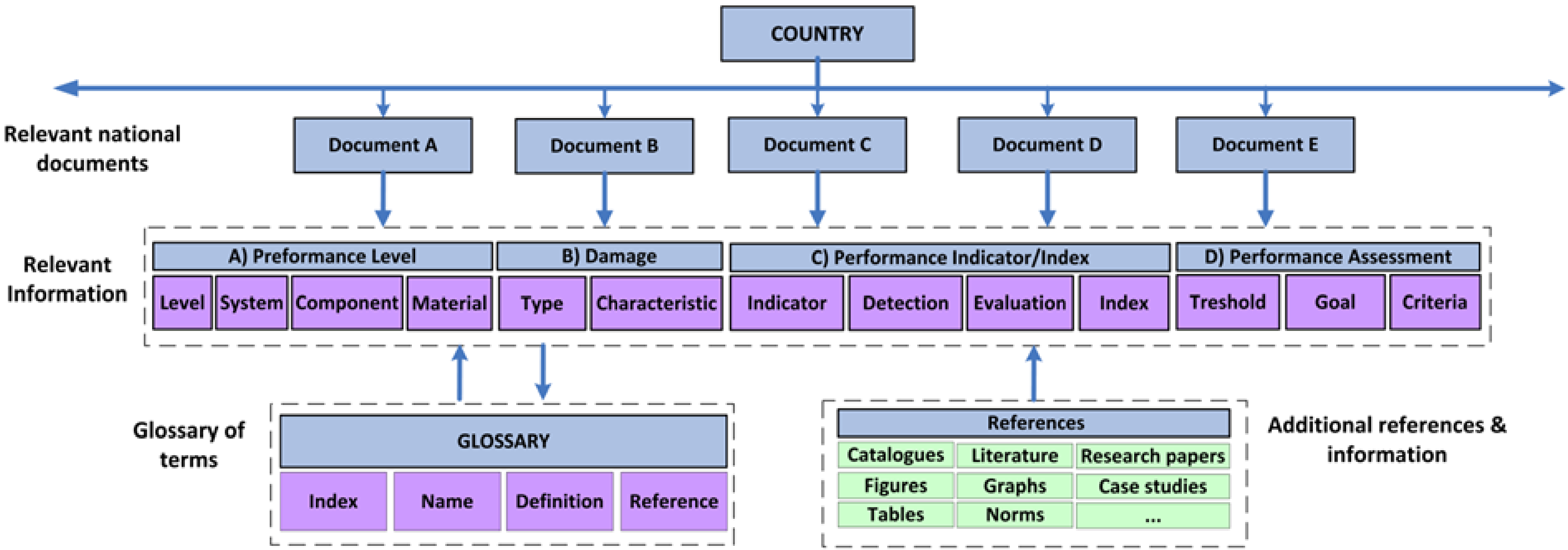
Call to researchers for:

uploading performance associated documents

transferring documents to Database

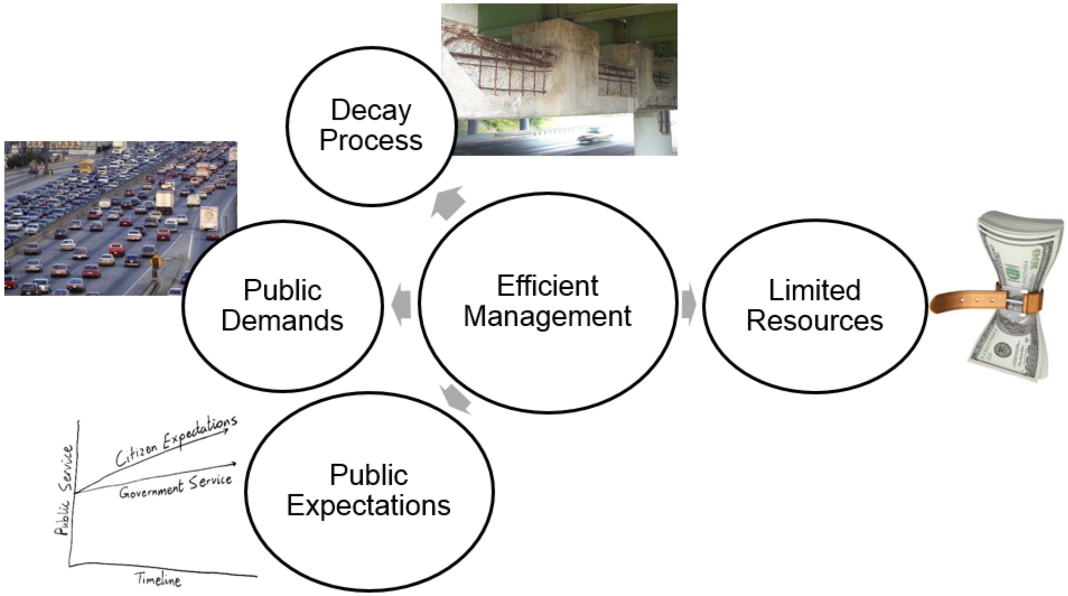
analyzing PI - Database

ELEMENTS AND COARSE STRUCTURE OF DATABASE



DEFINITION OF PERFORMANCE INDICATOR

Parameter **measurable** and **quatifiable** related to the bridge performance that can be compared with a **target measure** of a **performance goal** or can be used for **ranking** purposes among a bridge population in the framework of a Quality Control Plan or life-cycle management (**decisions, action involving economic resources**)



SCREENING RESULTS *OPERATOR* DOCUMENTS / *DATABASE*

List of screened documents

SURVEY OF PERFORMANCE INDICATORS

Country	Croatia	New Document

num	Responsible Person	Document	Doc. Type	Author	Year
1	Ana Mandić Ivanković	*Handbook of damages on bridge elements*	Evaluation	Hrvatske ceste d.o.o., dr.sc. Danijel Tenžera	2014
2	Ana Mandić Ivanković	*Guidelines for bridge inspections*	Inspection	Hrvatske ceste d.o.o.	2014
3	Ana Mandić Ivanković	*HRMOS manual – Bridge management*	Inspection	Hrvatske ceste d.o.o.	1999
4	Dominik Skokandić	*HRMOS manual – Bridge management – General bridge inspection*	Inspection	Hrvatske ceste d.o.o.	1999
5	Dominik Skokandić	*Handbook of damages on bridges*	Inspection/ evaluation	Hrvatske Autocesete d.o.o.	2010
6	Ana Mandić Ivanković	*Guideline for bridge evaluation*	Evaluation	Hrvatske Autocesete d.o.o.	2010
7	Ana Mandić Ivanković	*Bridge Management Planning*	Background document	Hrvatske Autocesete d.o.o.	2008
8					
9					
10					
11					
12					

Blank | Names_Table | GeneralData | Cro_1 | Cro_2 | Cro_3 | Cro_4 | Cro_5 | Cro_6 | Cro_7

READY

Croatian example

Handbook of damages on bridge elements

C) Performance Indicator/Index				D) Performance Assessment		
indicator	detection	evaluation	index	threshold	goal	criteria
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			crack width (	Damage Assessment	

C) Performance Indicator/Index				D) Performance Assessment		
indicator	detection	evaluation	index	threshold	goal	criteria
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			affected dep	Damage Assessment	
Damage degree	Direct_Measurement			sag (cm)	Damage Assessment	
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			affected dep	Damage	

Element	All bridge types	Foundations		Damage_State	Degradation	
Element	All bridge types	Foundations	Concrete	Damage_State	Spalling	
Element	All bridge	Foundations	Concrete	Damage_State	Spalling	

Blank | Names_Table | GeneralData | Cro_1 | Cro_2 | Cro_3 | Cro_4 | Cro_5 | Cro_6 | Cro_7

READY

SCREENING RESULTS *RESEARCH* DOCUMENTS / *DATABASE*

Austrian example

SURVEY OF RESEARCH PERFORMANCE INDICATORS

Article	Performance assessment of concrete s
Author	Strauss, Zan
Year	
Abstract	An efficient evaluation and prediction of fundamental requirement for life-cycle a structures. Important tools and valuable methods. Unfortunately, due to their pra information gathered with inspection and monitoring methods need to be used in the most effective manner possible. The aim of this contribution is to present a framework performance indicators of concrete structures prone to fatigue, A theoretical background with selected indicators is presented t methods including inspection and monitoring information with i
Journal	IABSE Conference – Structural Engineering: Providing Solutions Geneva, Switzerlan
Keywords	life-cycle analysis; performance indicators; probabilistic perf

Performance Indicator	Young modulus
Type of Indicator	Material property
Mathematical Formulation	
Threshold	
Intentions (where to apply)	In order to evaluate the fatigue performance of the critical cross
Level of maturity	Research stage
Case study	STRABAG test foundation in Cuxhaven
Performance Indicator	**Reliability index**
Type of Indicator	Reliability
Mathematical Formulation	
Threshold	
Intentions (where to apply)	In order to evaluate the fatigue performance of the critical cross
Level of maturity	Research stage
Case study	STRABAG test foundation in Cuxhaven

References

[1] Zhao, Y.-G., Zhong, W.-Q., Ang, A.H.-S., 2007. Estimating joint failure probability of series structural systems. J. Eng. Mech. 133, 588–596.

[2] Strauss A, Vidovic A, Zambon I, Grossberger H, Bergmeister K. Monitoring information and probabilistic based prediction models for the

[3] Mark, P., Stangenberg, F., Bergmeister, K., Strauss, A., Ahrens, M.A., 2013. Lebensdauerorientierter Entwurf, Konstruktion, Nachrechnung

SURVEY OF PERFORMANCE INDICATORS

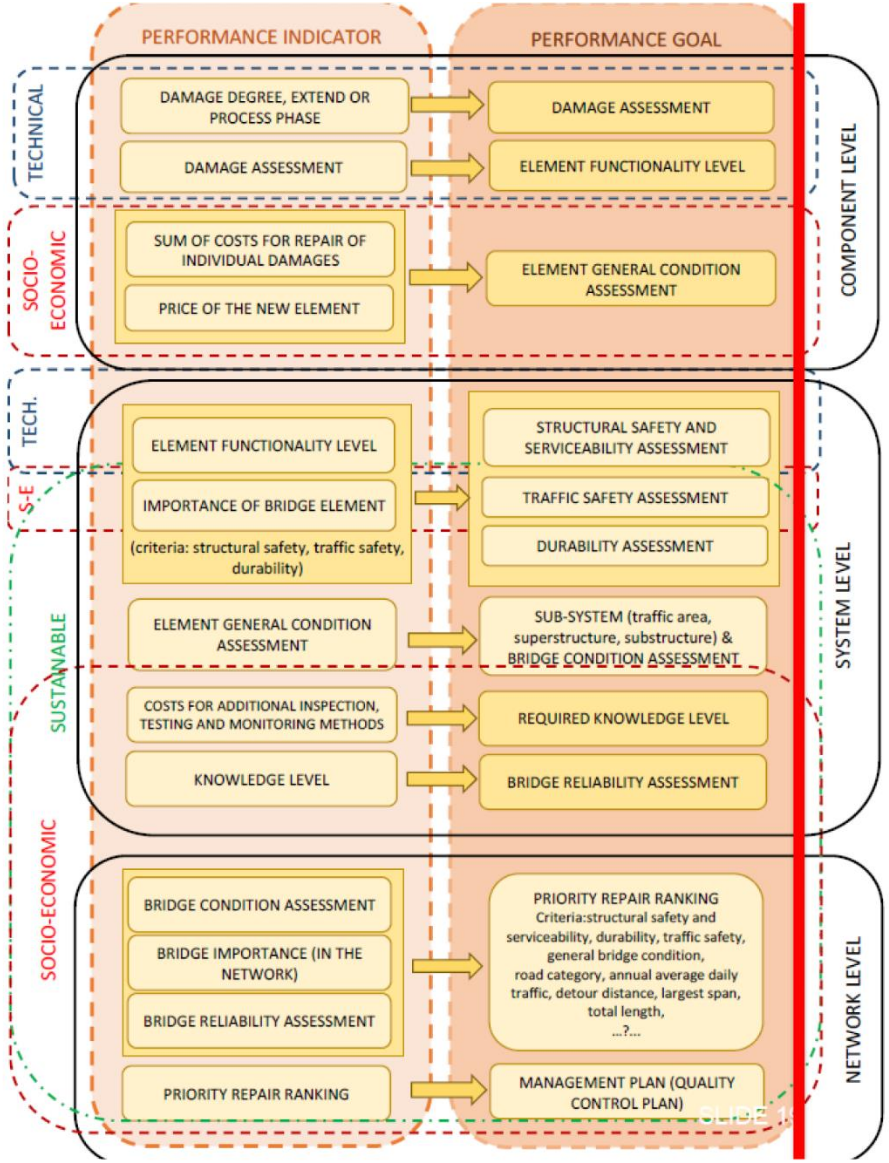
Country	Austria

Add Article

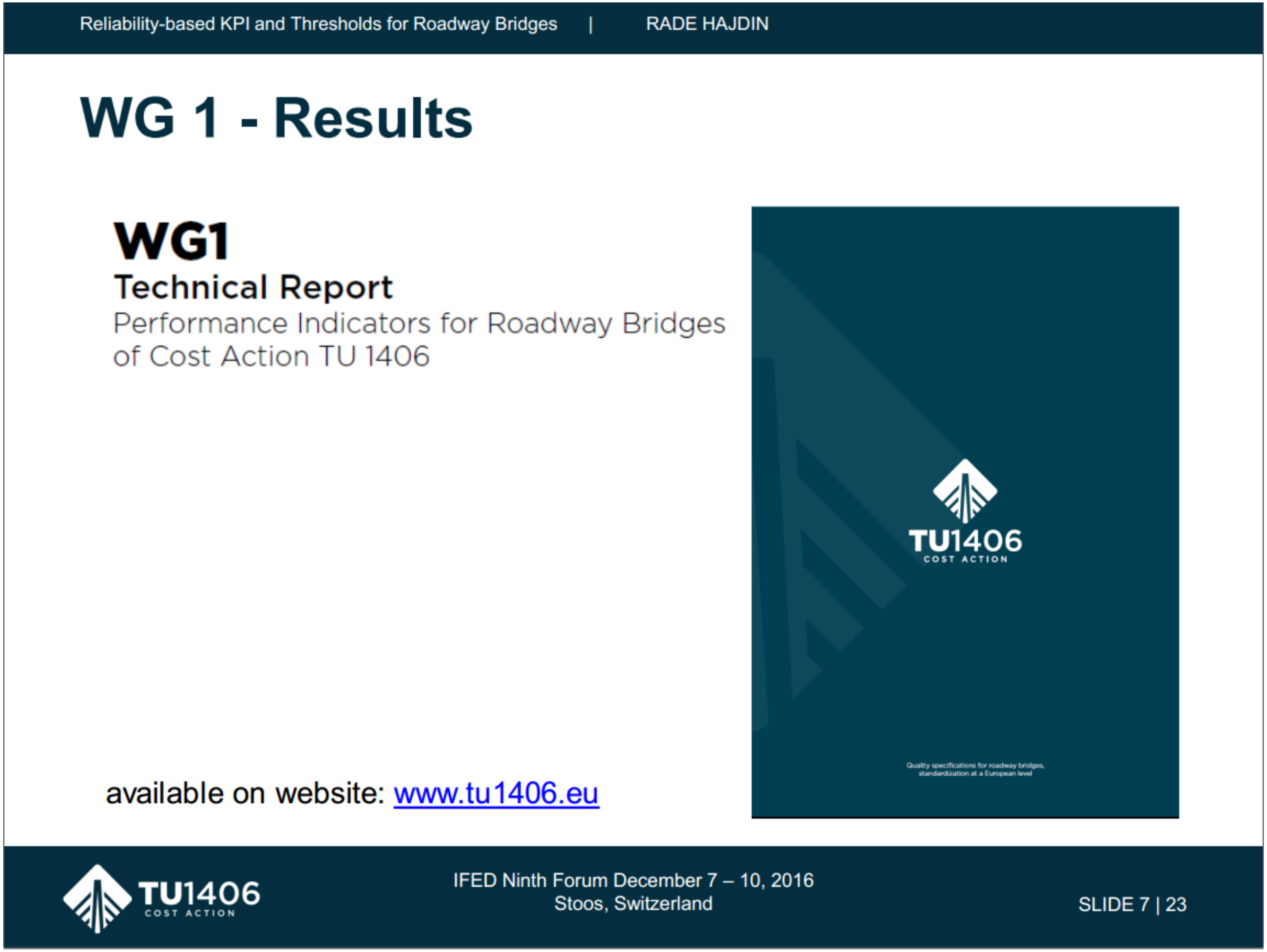
num	Responsible Person	Article	Author	Year
1	Ivan Zambon	*Performance assessment of concrete structures based on probabilistic prediction models and monitoring information*	Strauss, Zambon, Vidovic, Grossberger, Bergmeister	2015
2				
3				
4				

Indicators and goals

- Interactions between KPI and PG are contemplated, as they are crucial for optimal quality control and management of road bridges



TU1406 COST ACTION IABSE


Reliability-based KPI and Thresholds for Roadway Bridges | RADE HAJDIN
WG 1 - Results
WG1
Technical Report
Performance Indicators for Roadway Bridges of Cost Action TU 1406
available on website: www.tu1406.eu
TU1406
COST ACTION
Quality specifications for roadway bridges, standardization at a European level
TU1406
COST ACTION
IFED Ninth Forum December 7 – 10, 2016
Stoos, Switzerland
SLIDE 7 | 23

WG 1 REPORT

TU1406
COST ACTION

Quality specifications for roadway bridges,
standardization at a European level

BASE

Estonia
- List of documents
- Homogenized database

Finland
- List of documents
- Non-homogenized database

France
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Greece
- List of documents
- Homogenized database

CONTENT LIST

01
General

General

Performance Indicators terms after surveying

02
Operators

Operators

Operators list of documents and database per country

03
Research

Research

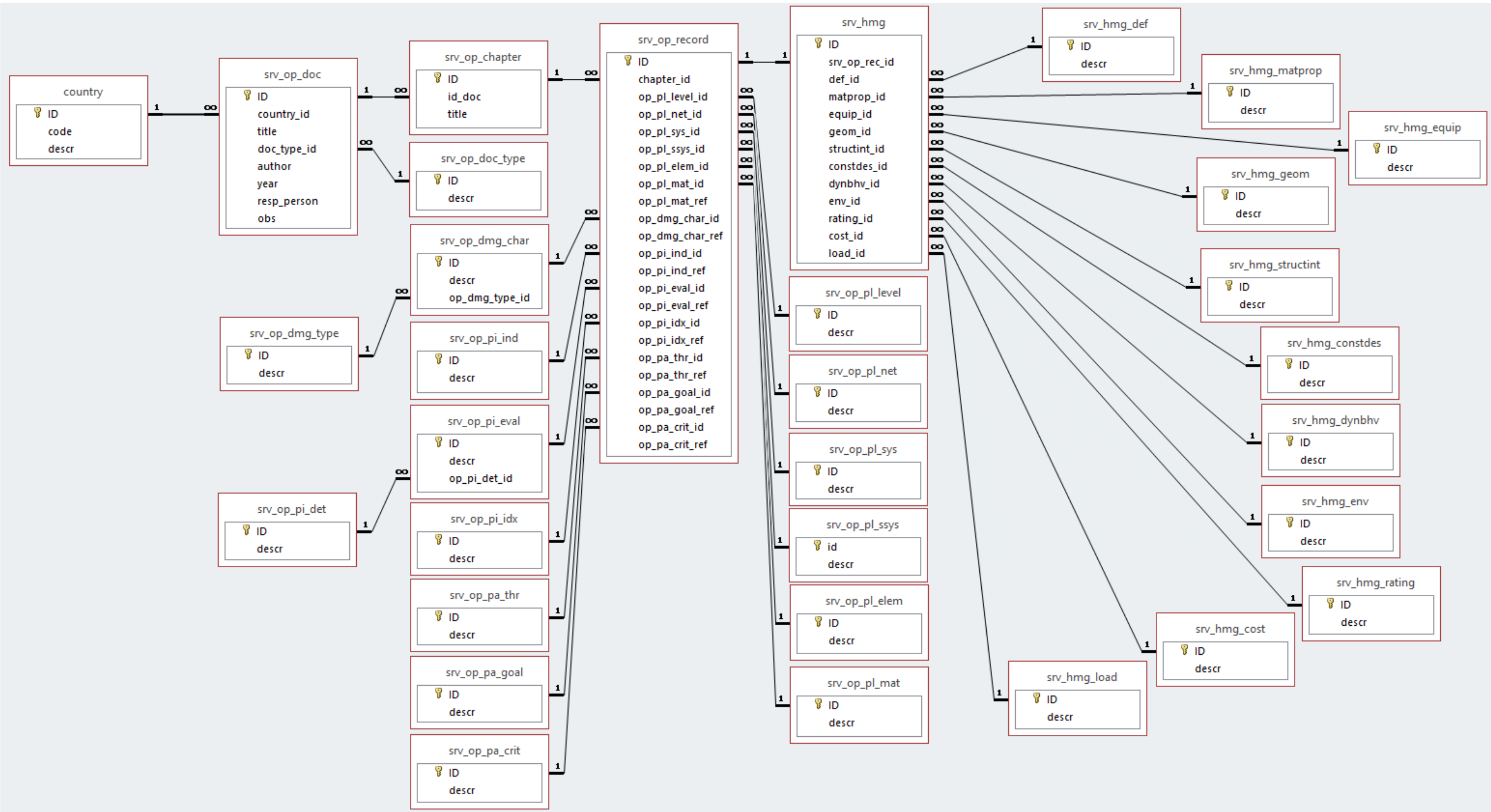
Research list of documents and database per country

Glossary

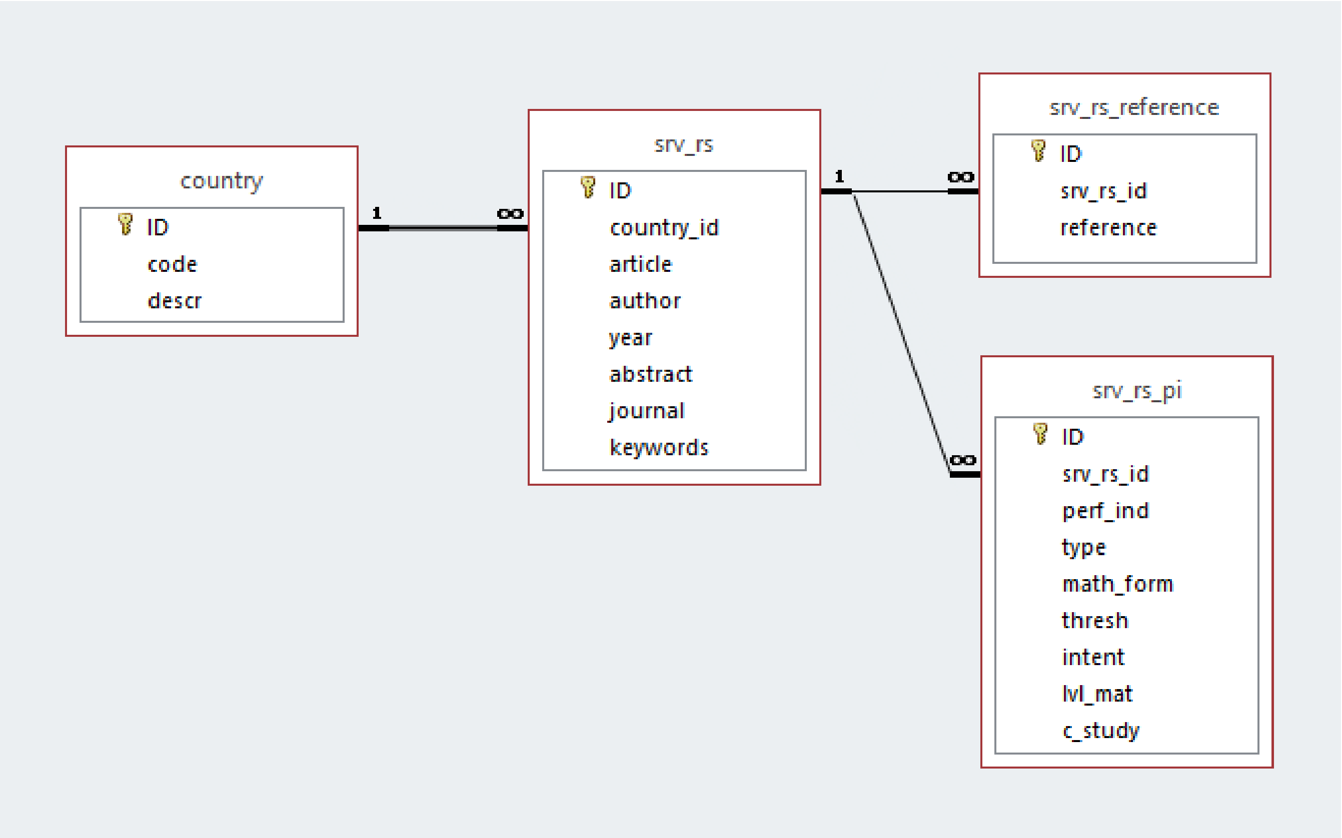
Glossary and specific term sheet per country

04
Glossary

OPERATOR DOCUMENTS *RELATIONAL DATABASE*



RESEARCH DOCUMENTS *RELATIONAL DATABASE*



FROM PERFORMANCE INDICATORS (PI) to KEY PI (KPI)

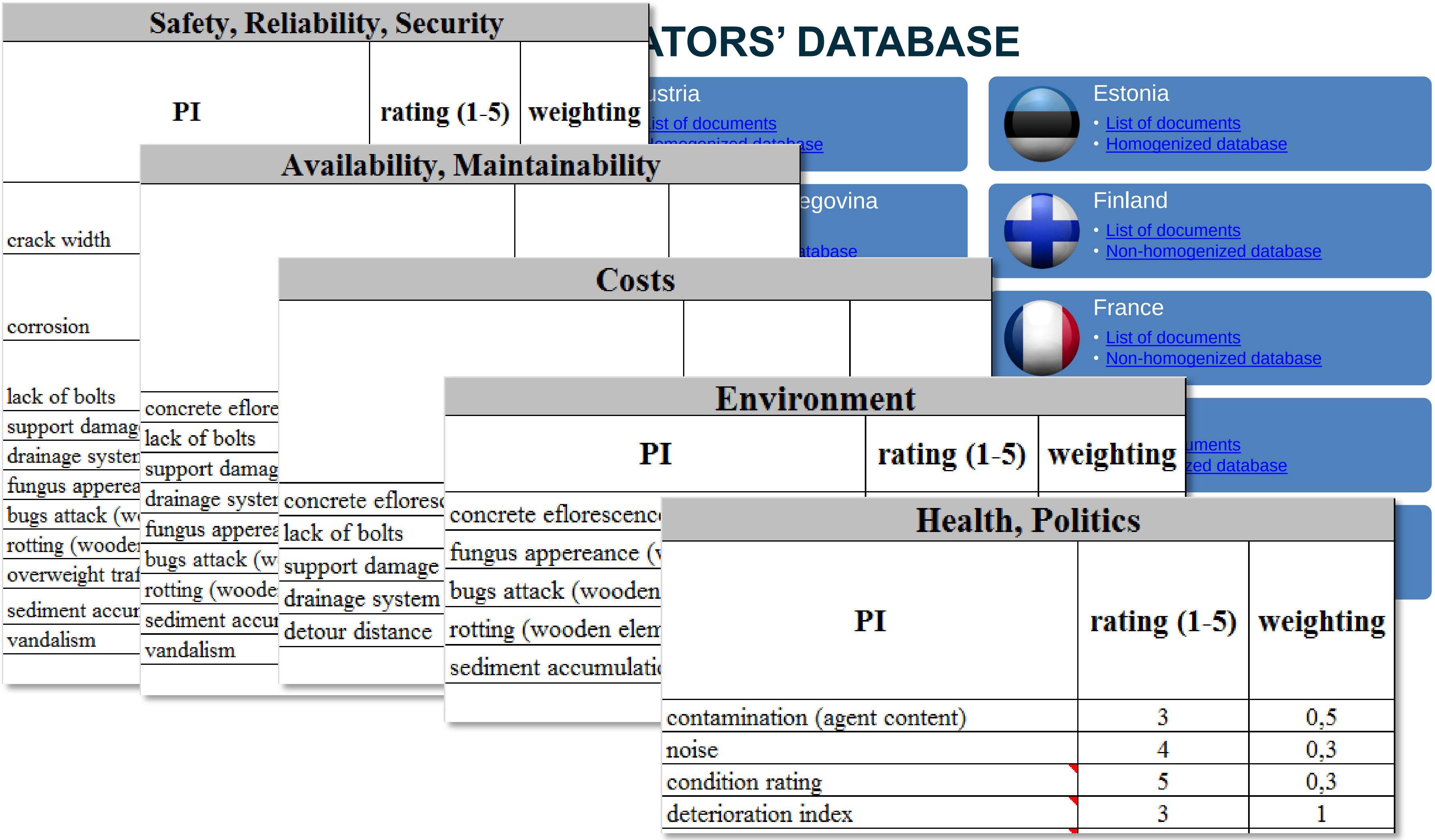
In order to move on with the reduction of the list of Performance Indicators, an Expert Group was asked to **specify a reduced list of 108 PIs according to the following points**:

- **Level** (Component Level, System Level or Network Level)
- Is the **PI measureable**? (Technical, Socio Economical or Sustainable)
- PI **belongs to the Key Performance Indicator(s)**? (Reliability, Availability, Maintainability, Safety, Security, Environment, Costs, Health, Politics, Rating/Inspection)
- **Assessment** (Threshold, Goal, Rating)

FROM PERFORMANCE INDICATORS (PI) to KEY PI (KPI)

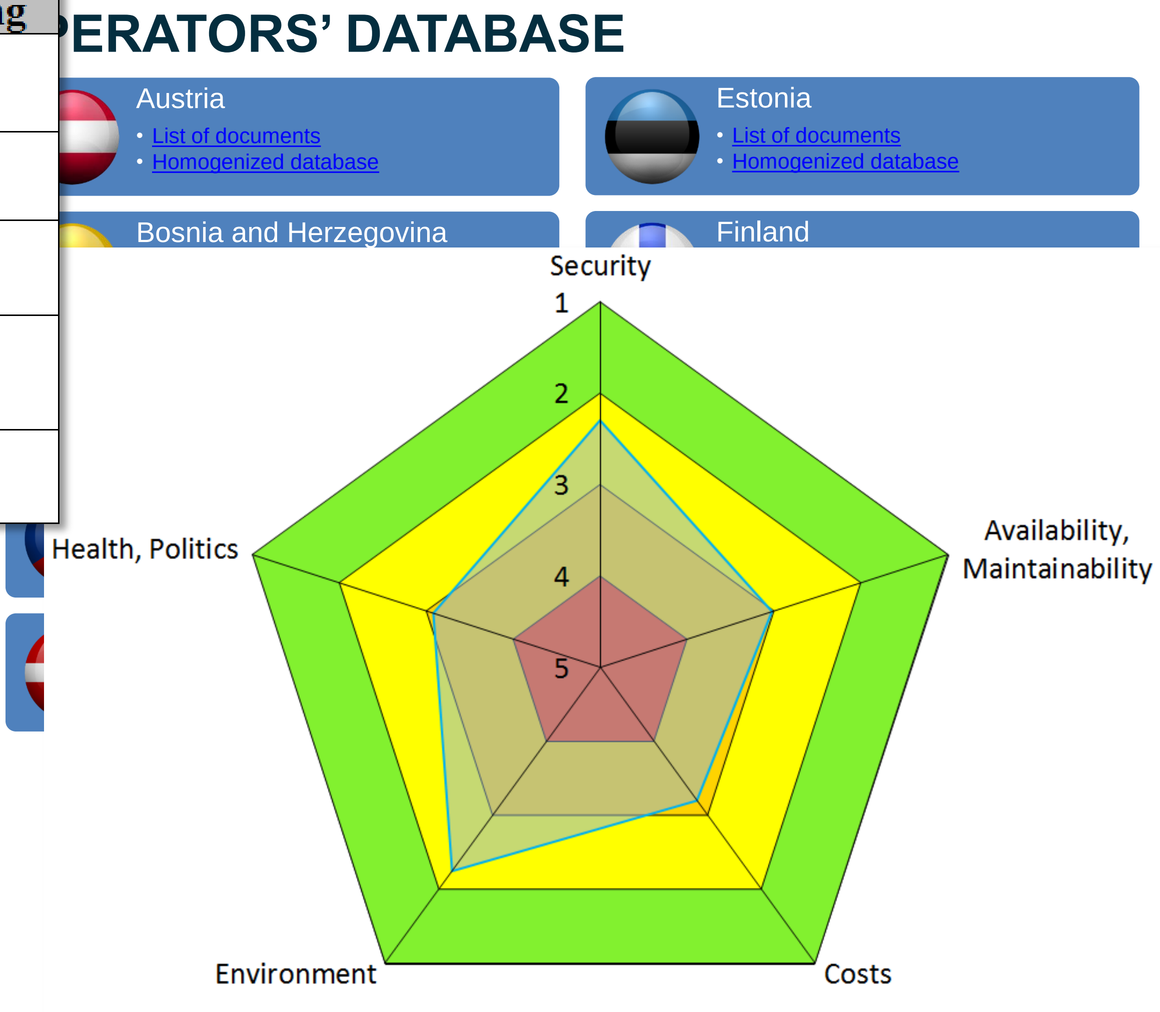
			Safety, Reliability, Security	
PI	**Level**	**Performance indicator PI if**	**PI belongs to the Key Peformance Indicator(s)**	**Assessm**
	Component Level (CL) System Level (SL)	Measurable? {Quantifiable?; Target value	Reliability (R), Availability (A), Maintainability (M), Safety (S),	Threshold (T Goal (G =
concrete cover (insufficient)	CL	Yes, Tech, Sust	R, A, (C, I)	T= thickness (mm), G=
cracks related to origin (e.g. due to loading, due to settlement, due to crumbling of concrete,…	CL, SL	Yes, Tech	R, A, S, (C, I)	T=width (mm), G=unde
fatigue cracking	CL, SL	Yes, Tech	R, A, S, (C, I)	T= number of cracks an
settlement	SL	Yes, Tech	R, A, S, (C, I)	T= dimension (mm) and
water penetrability	CL	Yes, Tech, Sust	R, (C, I)	T= area and affected co
wetting/leaking	CL	Yes, Tech, Sust	R, (C, I)	T= area and affected co
carbonation depth	CL	Yes, Tech, Sust	R, A, (C, I)	T= depth (mm) in relati
cathodic protection deficiency	CL, SL	Yes, Tech, Sust	R, A, (C, I)	T= existence of deficien
chloride depth profile	CL	Yes, Tech, Sust	R, A, (C, I)	T= depth (mm) in relati
contamination (agent content)	CL	Yes, Tech, Sust	R, A, (C, E, H, I)	T= % of agent content,
corrosion	CL	Yes, Tech, Sust	R, A, S, (C, I)	T= % section loss; G= F
fatigue (remaining service life)	SL	Yes, Tech, SoEc	R, A, S, (C, I)	T= (Remaining SL / Tin
absence (missing) of equipment component	CL, SL	Yes, Tech	R, A, S, (C, I)	T= evidence of the defe
approach slab settlement	SL, NL	Yes, Tech	R, A, S, (C, I)	T= height (mm); G=ass
asphalt pavement cracking	CL, SL, NL	Yes, Tech, Sust	R, (C, I)	T= width (mm), length (
asphalt pavement wearing and tearing (rutting, ravelling)	CL, SL, NL	Yes, Tech, Sust, SoEc	R, A, S, (C, I)	T= affected area (m2),
asphalt pavement wheel tracking and wrinkling and undulation	CL, NL	Yes, Tech, SoEc	R, A, S, (C, I)	T= affected area (m2),
blistering of protective coating	CL	Yes, Tech, Sust	R, (C, I)	T= affected area (m2);

FROM PERFORMANCE INDICATORS (PI) to KEY PI (KPI)

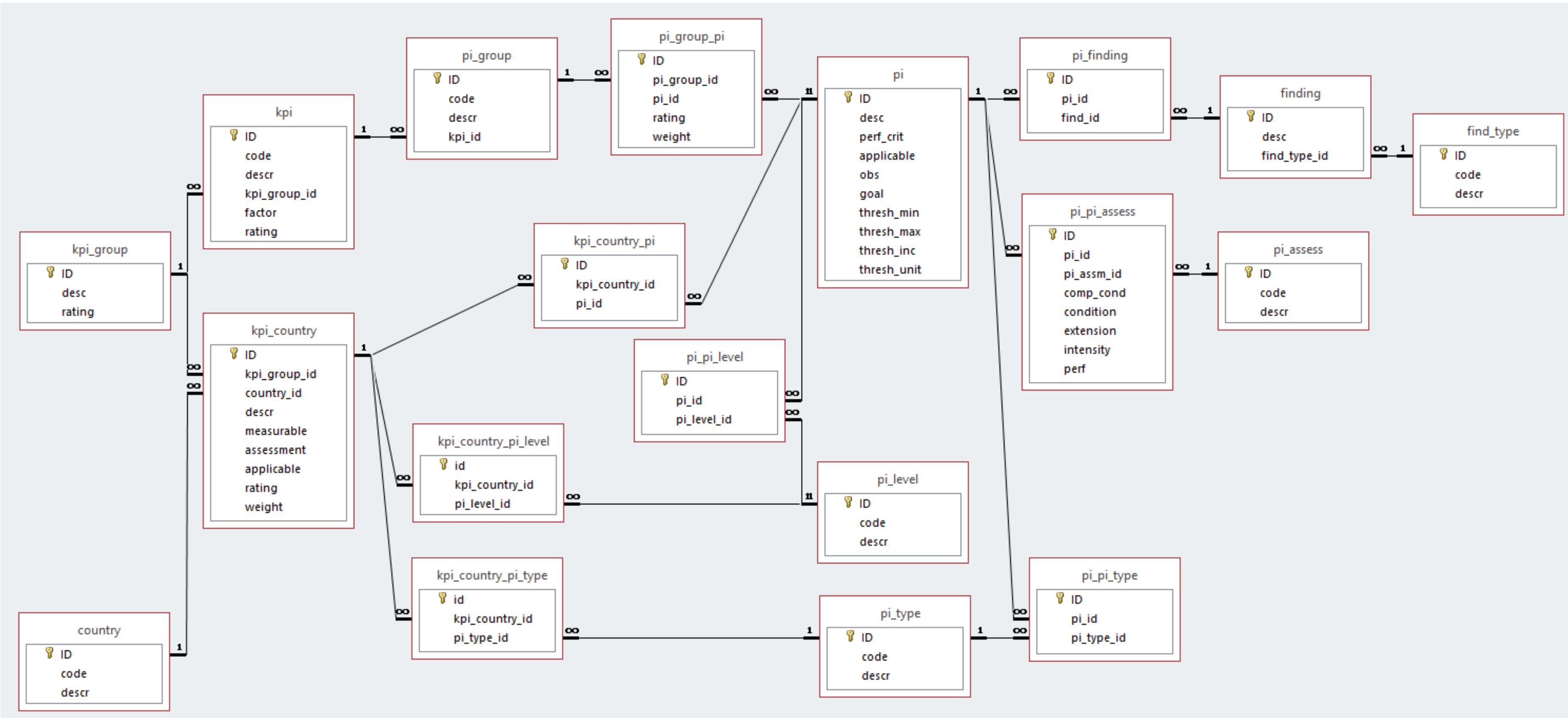


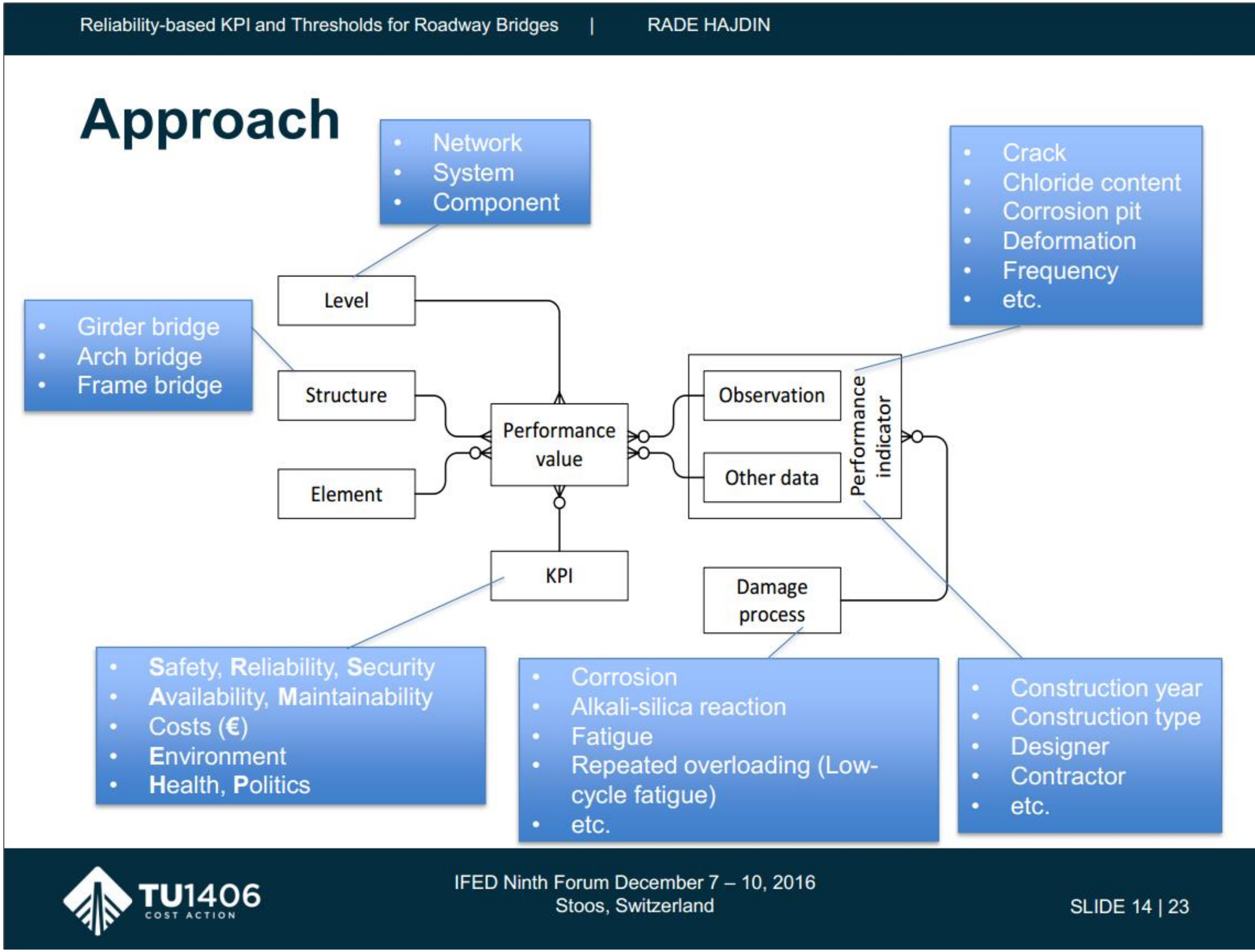
FROM PERFORMANCE INDICATORS (PI) to KEY PI (KPI)

KPI	total rating
Safety, Reliability, Security	2,30
Availability, Maintainability	3,02
Costs	3,20
Environment	2,25
Health, Politics	3,08



FROM PERFORMANCE INDICATORS (PI) to KEY PI (KPI)




Reliability-based KPI and Thresholds for Roadway Bridges | RADE HAJDIN
Approach
Network
System
Component
Crack
Chloride content
Corrosion pit
Deformation
Frequency
etc.
Level
Girder bridge
Arch bridge
Frame bridge
Structure
Element
Performance value
Observation
Other data
Performance indicator
KPI
Damage process
Safety, Reliability, Security
Availability, Maintainability
Costs (€)
Environment
Health, Politics
Corrosion
Alkali-silica reaction
Fatigue
Repeated overloading (Low-cycle fatigue)
etc.
Construction year
Construction type
Designer
Contractor
etc.
TU1406 COST ACTION
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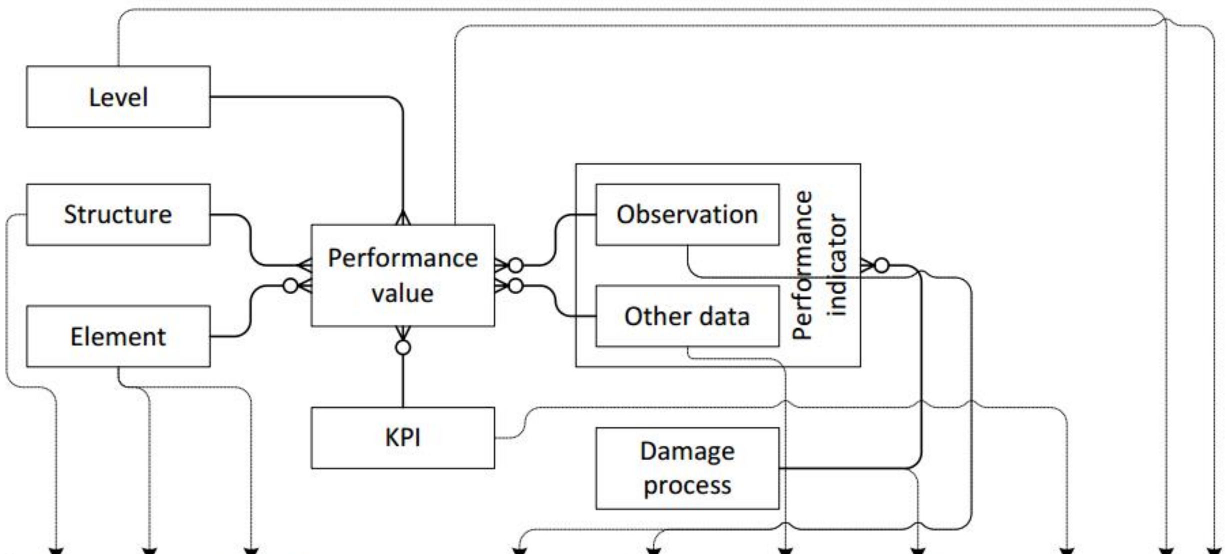
Stoos, Switzerland
SLIDE 14 | 23

Reliability-based KPI and Thresholds for Roadway Bridges | RADE HAJDIN

In more tangible terms ....



Single Girder Bridge	Beam	Reinforced concrete		Crack	0.4 mm	Construction year	Repeated overloading	Safety, Reliability, Security	C	2
Single Girder Bridge	Beam	Reinforced concrete		Spalling	1.0 m²	Construction year	Corrosion of Reinforcement	Safety, Reliability, Security	C	3

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ACKNOWLEDGMENT

This article is based upon the work from COST Action TU1406, Quality specification for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology). The support of the project “LeCIE – Life-cycle assessment for railway construction – strategies and methods” has been acknowledged.

THANK YOU FOR YOUR ATTENTION!

Alfred Strauss – University of Natural Resources and Life Sciences (BOKU), Austria
Ana Mandic Ivankovic – University of Zagreb, Croatia
Jose C. Matos – University Minho, Portugal
Joan R. Casas – UPC – BarcelonaTech, Spain
Sérgio Fernandes – ANSER Lda., Portugal

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Multi-criteria decision making: AHP method applied for network bridge prioritization

Zaharah Allah Bukhsh, Irina Stipanovic Oslakovic – University of Twente, Netherlands
Giel Klanker – Rijkswaterstaat, Ministry of Infrastructure, Netherlands
Niels Peter Hoj – HOJ consulting, Brunmen, Switzerand
Boulent Imam – University of surrey, United kingdom
Yiannis Xenidis - Aristotle University of Thessaloniki , Greece

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

INTRODUCTION

- Bridges are key structure of transport infrastructure networks
- Length of bridge is just 2% or whole network but it represent 30% of value (PIARC, 1999)
- Ratio of expenses per route km spent on bridge is 10 times more than the average expenses per route km (CEDR, 2010)
- Bridge management is exposed with budget constrains and need of improved service quality
- In last decade, number of bridge management system and life cycle tools have been developed to facilitate the reliable management of deteriorating assets

BRIDGE MAINTENANCE PLANNING :

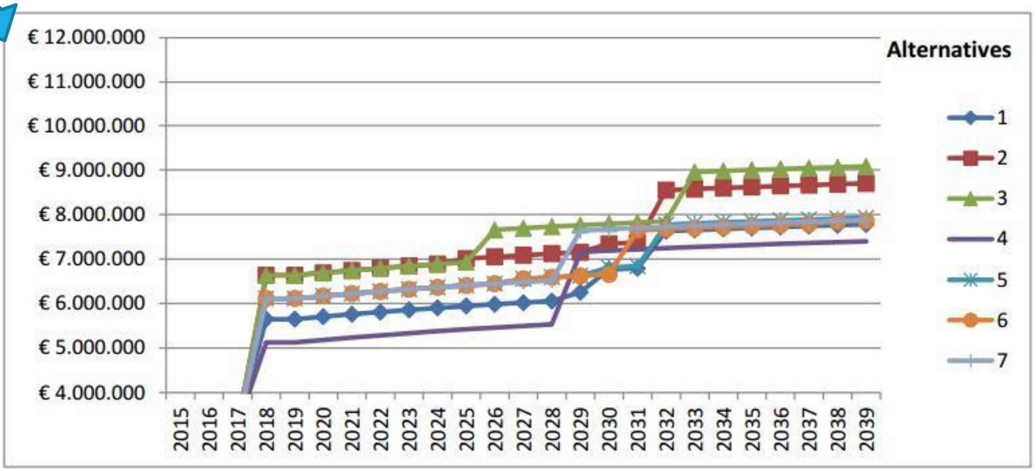
OBJECT LEVEL

- Consider only owner cost
- Ignore environment impact, availability, importance on the transport and overall societal impacts

- It is a process of deciding the scope, timing, costs, and benefits of future maintenance activities on a specific bridge.

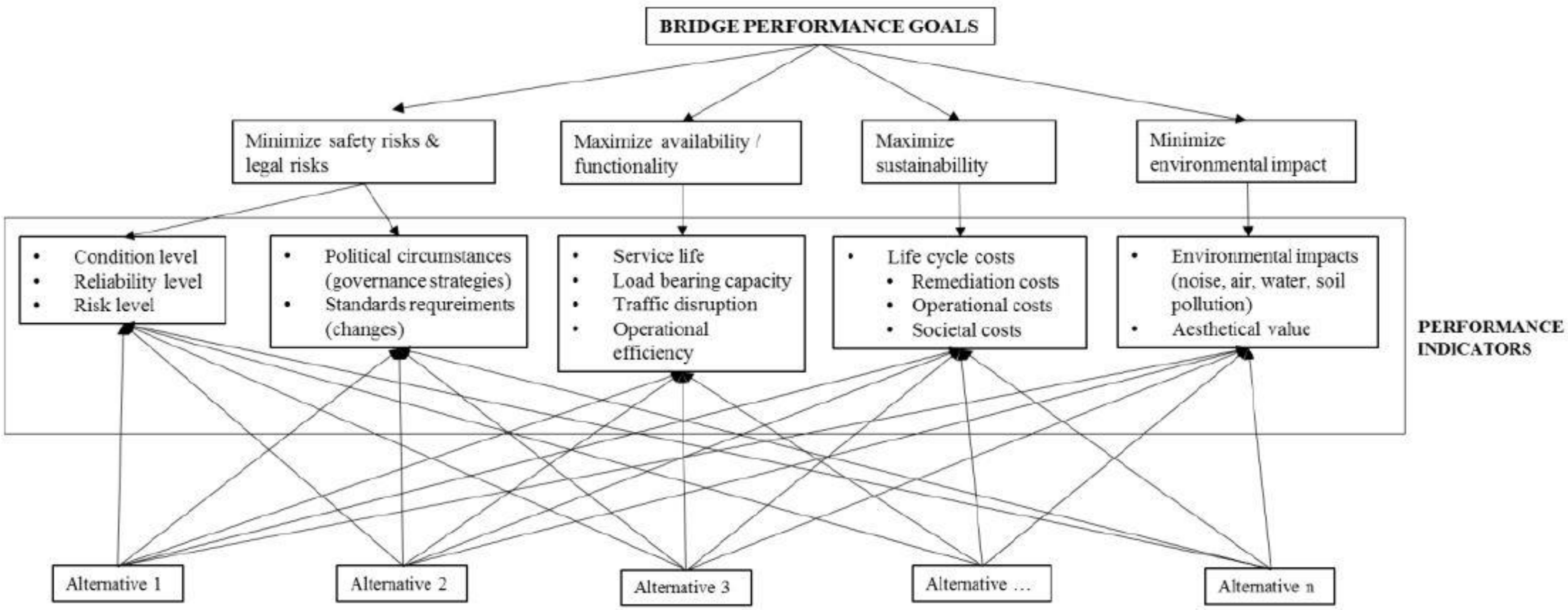
Object level

- Life cycle activity of each object
- Selection, budgeting and planning on network level



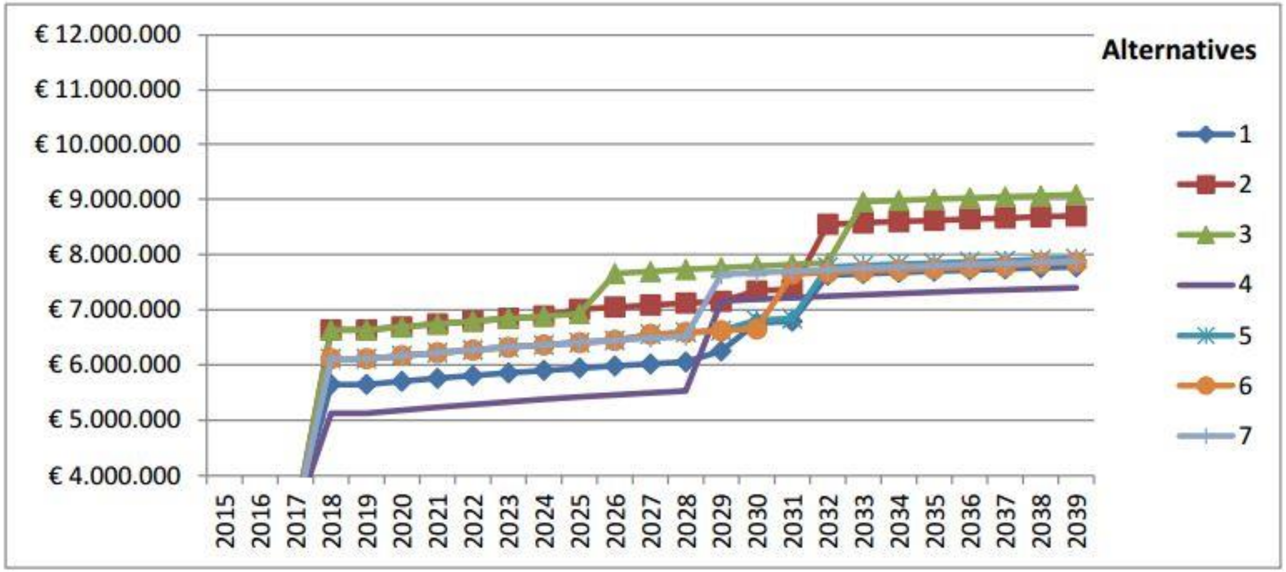
BRIDGE MAINTENANCE PLANNING

NETWORK LEVEL



BRIDGE MAINTENANCE PLANNING

NETWORK LEVEL



MULTI-CRITERIA DECISION MAKING

FOR MAINTENANCE PLANNING

- Systematic approach to evaluate multiple conflicting objectives in decision making
  - Limited budget vs. aging bridge
  - Demands of availability vs. need of maintenance
  - Risk of failure vs. criticality
- Enable the decision maker to provide preferences when exposed with conflicting objectives

Weighting → Scaling → Amalgamation

MULTI-CRITERIA DECISION MAKING

ANALYTICAL HIERARCHY PROCESS

Step	Description
Define objective and identify criteria	• Define the decision problems and the relative criteria
Comparison of decision criteria (Experts' judgment)	• Pairwise comparison of decision criteria based on fundamental scale (Saaty, 1999).
Matrix normalization into value function	• Normalization of each column into value function from 0 to 1 • Calculate the eigenvector by Geomean
Presentation	• Present the final result

ANALYTICAL HIERARCHY PROCESS

ILLUSTRATIVE EXAMPLE

Define objective and identify criteria

Objectives

- Minimize the maintenance cost
- Minimize the downtime

Performance indicator

Criteria

Scale

Alternatives

	Reliability	Economy	Availability	Society
Bridges	Reliability level	Maintenance cost	Downtime	Importance on the network
Name	Score card	Euros	Hours	Traffic Intensity (# cars / day)
A: B101	3	500k	30	9000
B: B109	4	1000k	70	10000
C: B207	4	200k	60	13000
D: B307	5	800k	180	15000
E: B150	3	500k	40	5000

ANALYTICAL HIERARCHY PROCESS

PREFERENCE STRUCTURE

Comparison of decision criteria (Experts' judgment)

Criteria		Important	Intensity
A	B		
Maintenance cost	Downtime	A	1: Equal importance
Maintenance cost	Reliability level	A	3: Moderate
Maintenance cost	Traffic Intensity	A	5: strong
Downtime	Reliability level	A	3: strong
Downtime	Traffic Intensity	A	3: Equal importance
Reliability level	Traffic Intensity	A	7: Very strong

Fundamental Scale for Pairwise Comparisons	
Intensity of Importance	Definition
1	Equal Importance
3	Moderate importance
5	Strong importance
7	Very Strong importance
9	Extreme importance
Intensities of 2, 4, 6, and 8 can be used to express intermediate values.	

ANALYTICAL HIERARCHY PROCESS

MATRIX NORMALIZATION

Comparison of decision criteria (Experts' judgment)

- Create the comparison matrix from the preference structure
- Reduce the matrix from 0 to 1 by

$$\overline{b}_{ij} = \frac{b_{ij}}{\sum_{k=1}^{n} b_{kj}}, (i, j = 1, 2, \cdots, n),$$

- Calculate the final Eigen vector from the preferences of each alte

$$\overline{\mathbf{w}}_i = \sum_{j=1}^{n} \overline{b}_{ij}, (i = 1, 2, \cdots, n).$$

$$\mathbf{w}_i = \frac{\overline{\mathbf{w}}_i}{\sum_{j=1}^{n} \overline{\mathbf{w}}_j}, (i = 1, 2, \cdots, n).$$

Comparison of decision criteria (Experts' judgment)

Comparison Matrix

Comparison among criteria	*Maintenance cost*	*Downtime*	*Reliability level*	*Traffic Intensity*
Maintenance cost	1.00	1.00	3.00	5
Downtime	1.00	1.00	3.00	3
Reliability level	0.33	0.33	1.00	7
Traffic Intensity	0.20	0.33	0.14	1

Matrix Normalization

$$\overline{b}_{ij} = \frac{b_{ij}}{\sum_{k=1}^{n} b_{kj}}, (i, j = 1, 2, \cdots, n),$$

Normalized matrix	*Maintenance cost*	*Downtime*	*Reliability level*	*Traffic Intensity*
Maintenance cost	0.39	0.38	0.42	0.31
Downtime	0.39	0.38	0.42	0.19
Reliability level	0.13	0.13	0.14	0.44
Traffic Intensity	0.08	0.13	0.02	0.06

Comparison of decision criteria (Experts' judgment)

$$\overline{\mathbf{w}}_i = \sum_{j=1}^{n} \overline{b}_{ij}, (i = 1, 2, \cdots, n).$$

$$\mathbf{w}_i = \frac{\overline{\mathbf{w}}_i}{\sum_{j=1}^{n} \overline{\mathbf{w}}_j}, (i = 1, 2, \cdots, n).$$

Calculate eigenvector

Normalized matrix	*Maintenance cost*	*Downtime*	*Reliability level*	*Traffic Intensity*
Maintenance cost	0.39	0.38	0.42	0.31
Downtime	0.39	0.38	0.42	0.19
Reliability level	0.13	0.13	0.14	0.44
Traffic Intensity	0.08	0.13	0.02	0.06

Scores
0.37
0.33
0.18
0.06

ANALYTICAL HIERARCHY PROCESS

MATRIX NORMALIZATION

Comparison of decision criteria (Experts' judgment)

	Maintenance cost	Downtime	Reliability level	Traffic Intensity
A: B101	0.20	0.37	0.07	0.11
B: B109	0.04	0.12	0.19	0.11
C: B207	0.47	0.15	0.19	0.24
D: B307	0.08	0.03	0.46	0.49
E: B150	0.20	0.28	0.07	0.03

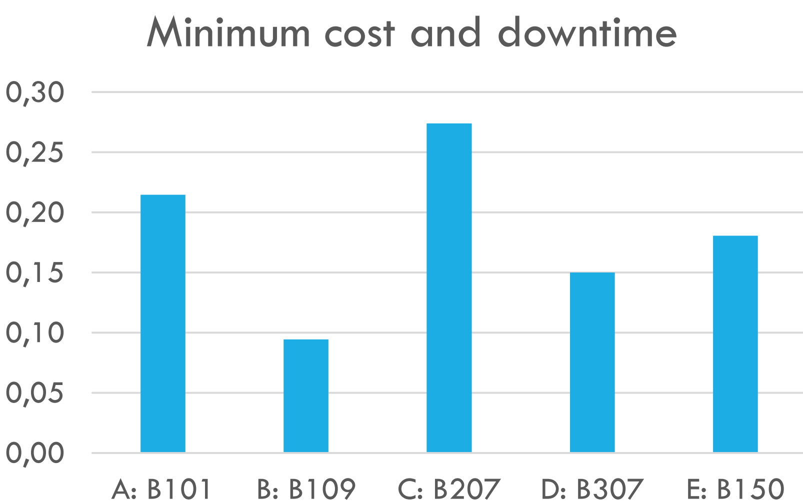
Scores
0.37
0.33
0.18
0.06

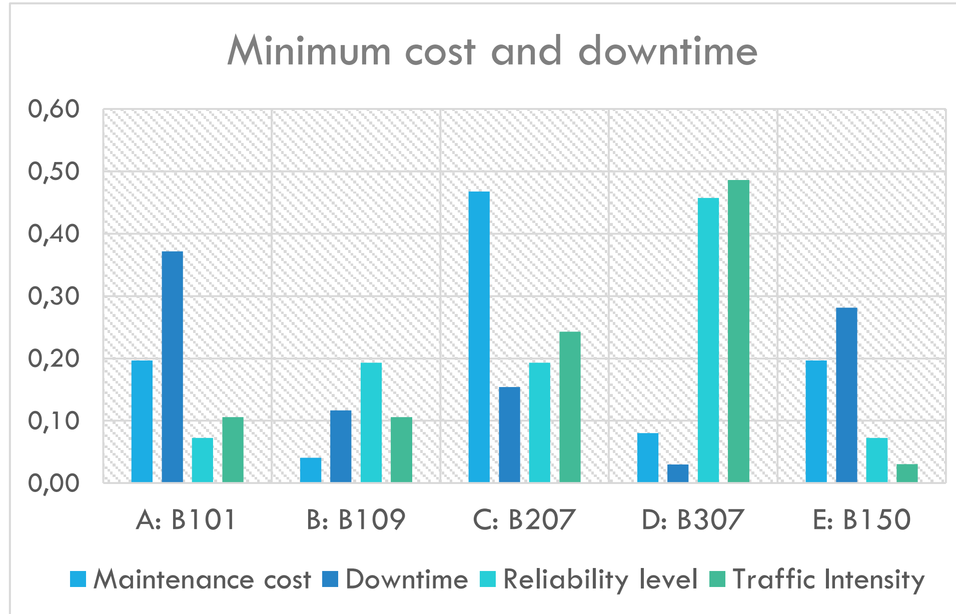
Overall score
0.21
0.09
0.27
0.15
0.18

Matrix Multiplication

NETWORK BRIDGES RANKING BASED ON OBJECTIVE

Results

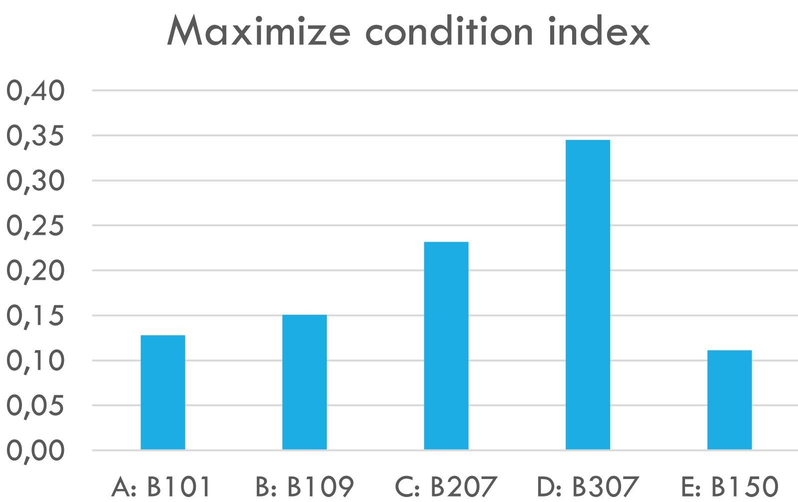


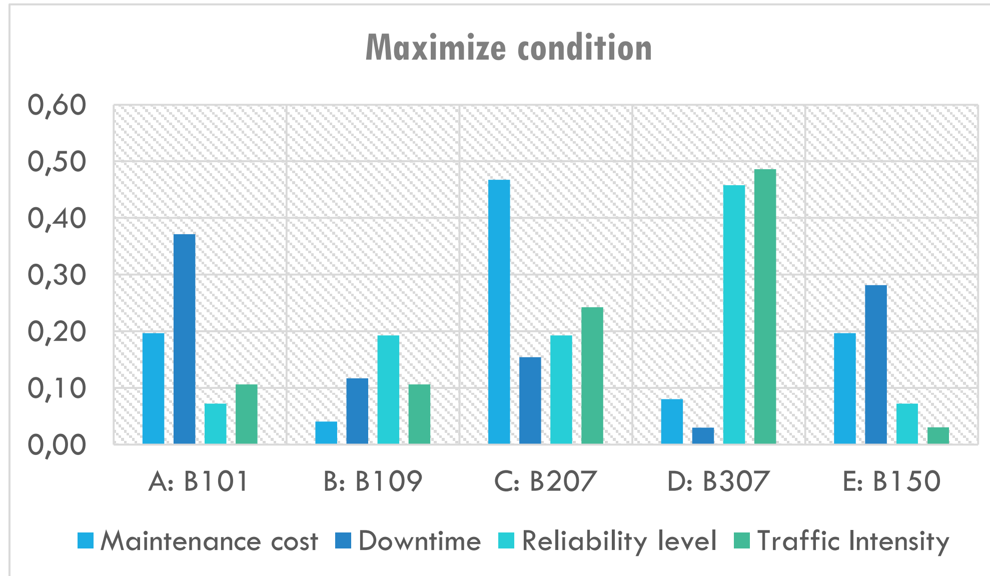


		Reliability	Economy	Availability	Society
	Bridges	Reliability level	Maintenance cost	Downtime	Importance on the network
	Name	Score card	Euros	Hours	Traffic Intensity (# cars / day)
2	**A: B101**	3	500k	30	9000
5	**B: B109**	4	1000k	70	10000
1	**C: B207**	4	200k	60	13000
4	**D: B307**	5	800k	180	15000
3	**E: B150**	3	500k	40	5000

NETWORK BRIDGES RANKING BASED ON OBJECTIVE

Results

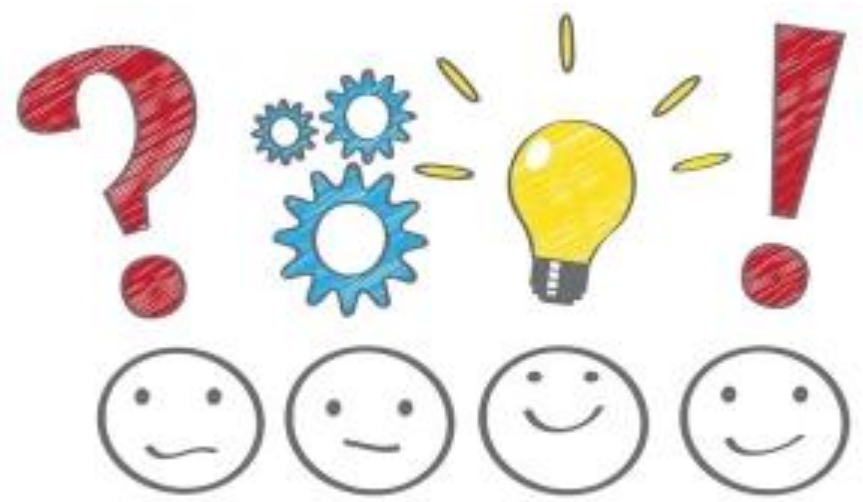




		Reliability	**Economy**	**Availability**	**Society**
	Bridges	Reliability level	Maintenance cost	Downtime	Importance on the network
	Name	Score card	Euros	Hours	Traffic Intensity (# cars / day)
4	**A: B101**	3	500k	30	9000
3	**B: B109**	4	1000k	70	10000
2	**C: B207**	4	200k	60	13000
1	**D: B307**	5	800k	180	15000
5	**E: B150**	3	500k	40	5000

CONCLUSION

- The framework of multi-criteria decision making (MCDM) provides a guidance on how to implement multiple performance goals
- The methods of MCDM incorporate decision makers preferences on multiple (conflicting) performance indicators
- The pairwise comparison of AHP grows exponentially when presented with large number of performance indicators
- For the maintenance optimization over the network, a link between the performance indicators at object level and the goal on network level needs to be established.
- The quantification of performance goals, other than technical goals, is a challenge.



ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

STRUCTURAL HEALTH MONITORING AND DESIGN CODE COMPLIANCE FOR PERFORMANCE ASSESSMENT OF BRIDGES UNDER SCOUR AND SEISMIC HAZARDS

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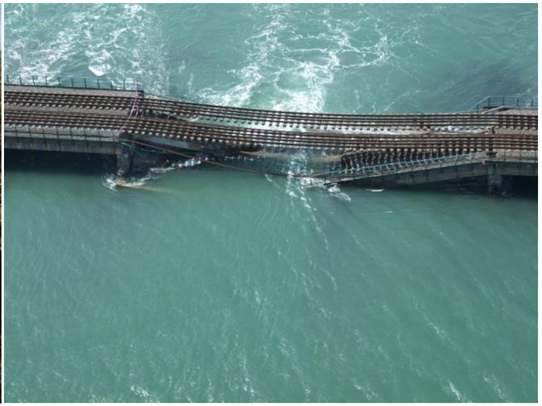
02nd – 03rd March 2017
Zagreb, Croatia

INTRODUCTION

- Bridge failures induced by natural hazards are increasing, due to lack of adequate monitoring and preventive maintenance actions, but also for the higher number of natural disasters.
- **Floods** and **Earthquakes** could be considered among the most critical events causing significant damage.
- Flooding phenomena usually affect substructures of bridges crossing rivers, causing local scouring at the base foundations in the river bed.
- In seismically prone regions, ground motions may induce damages on bridge components like piers, abutments and bearing systems.
- A brief overview of potential damage scenarios induced by flooding and seismic actions and by their combined action, is first illustrated.
- Assessment procedures currently in use against such natural hazards are briefly reviewed and some current research trends reported.

DAMAGE SCENARIOS DUE TO SCOUR

- Depending on bridge and foundation type, damage can be extremely detrimental to the operational capacity of the structure and may result in serviceability or ultimate failure.
- For bridges founded on shallow foundations, scour undermining the foundation can give rise to adverse settlements which can lead to cracking at the deck level and at other supports.
- In masonry arch structures, this damage can be even more severe and may compromise arch stability.



DAMAGE SCENARIOS DUE TO SCOUR

- Piled foundations, the loss of lateral pile support may give rise to the possibility of pile buckling.

- For severe scour around piles, the loss in shaft resistance may result in adverse settlement issues, which has ramifications for the bridge in terms of crack propagation in the superstructure.

- Differential settlement of different foundations may lead to severe cracking, deck buckling or total failure, whereas global settlement may induce serviceability failure.

- For pile groups, differential block settlement may arise inducing unacceptable tilting of the supported pier or abutment. This tilt may cause a deck to slide on its supports or buckle, depending on the nature of the structural connection.

DAMAGE SCENARIOS DUE TO SEISMIC ACTIONS

Past earthquakes have shown that for common girder bridges failure may occur due to:

- Collapse of the piers for bending or even for shear if capacity design prescriptions are applied;
- Collapse of the pier foundations if a capacity design is not applied;
- Collapse of the deck due to unseating induced by high seismic displacement.

SHM FOR SEISMIC ACTIONS

VISUAL-BASED/MANUAL →

- Time Consuming
- A priori damage location
- Resource-Intensive

SENSOR-BASED →

- Damage Detection
- Online/Remote
- Response based

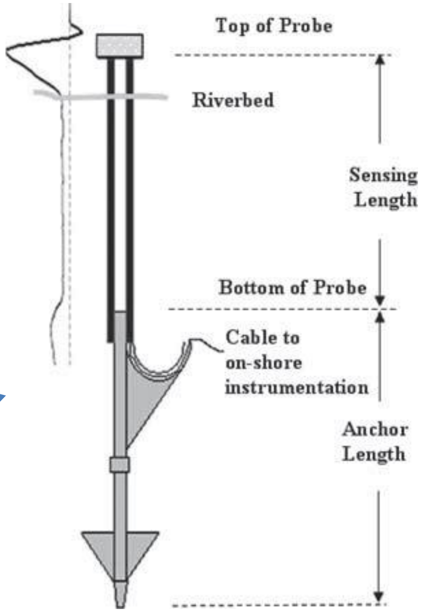
SHM FOR SEISMIC ACTIONS

Accelerometers →

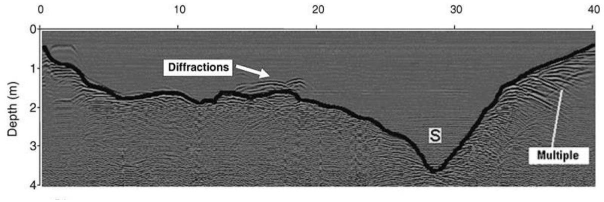
- MODAL ANALYSIS
- Soil-Structure Interaction
- Spatial Variability of Ground Motion
- Ground Failure Arrays

SHM FOR SCOUR PHENOMENA

VISUAL-BASED/MANUAL

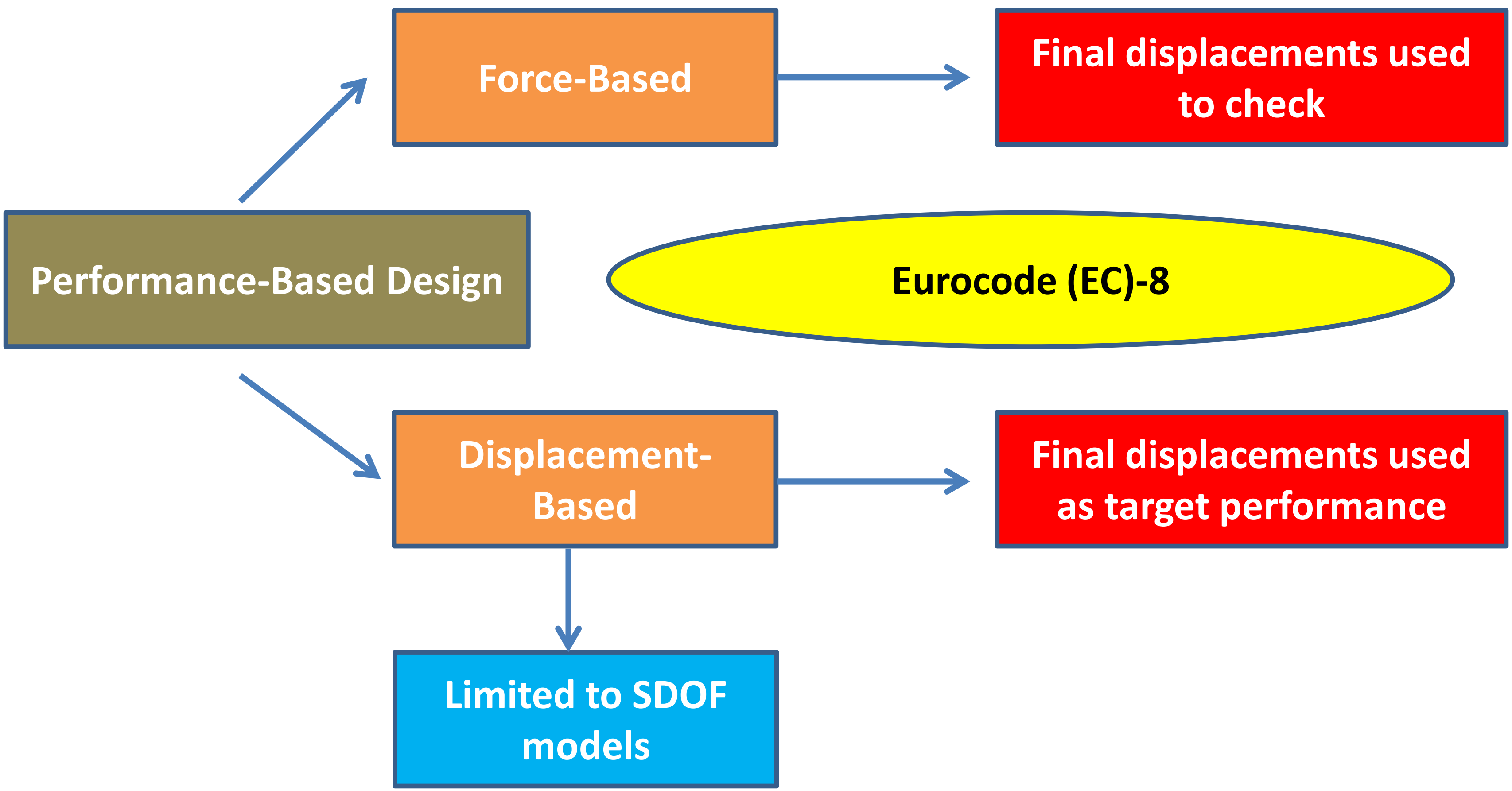


SENSOR-BASED



ACCELEROMETERS

PERFORMANCE ASSESSMENT ACCORDING TO DESIGN CODES - SEISMIC ACTIONS



PERFORMANCE ASSESSMENT ACCORDING TO DESIGN CODES - SCOUR ACTIONS

Design-Scour Depths

Colorado State University (CSU)

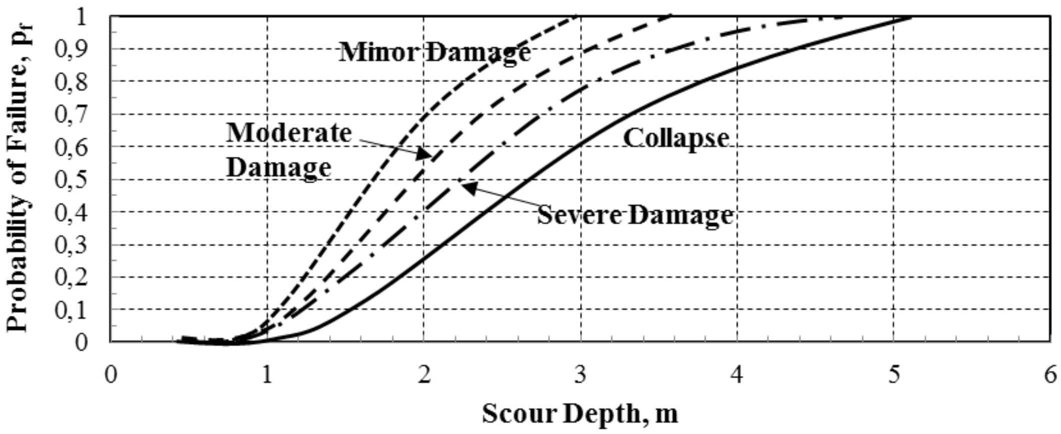
Hydraulic Engineering Circular (HEC-18)

National Practices??

Scour Design often undertaken as retrofit, based on national authority specs

PERFORMANCE ASSESSMENT ACCORDING TO DESIGN CODES - JOINT APPROACHES

Fragility-Curves



Can consider Multi-Hazards

CONCLUDING REMARKS

- Lack of preventative maintenance and monitoring has caused lots of collapses of bridges
- Multi-combined hazards can be more detrimental jointly than separately as in scour and seismic activities
- Scour reduces foundation stiffness, potentially exacerbating the effect of an earthquake
- These joint hazards are not explicitly designed for in design procedures but may become a critical load case

THANKS FOR YOUR ATTENTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

The impact of different maintenance policies on owners costs: Case Studies from Croatia and the Netherlands

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02nd – 03rd March 2017
Zagreb, Croatia

INTRODUCTION

Optimization of maintenance planning is an important part of bridge management

Efficiency of condition assessment process is directly influencing the choice of repair and maintenance measures

Paper examines two opposing maintenance strategies: corrective and preventive

Two case studies: Croatia and the Netherlands

MAINTENANCE STRATEGIES AND PLANNING

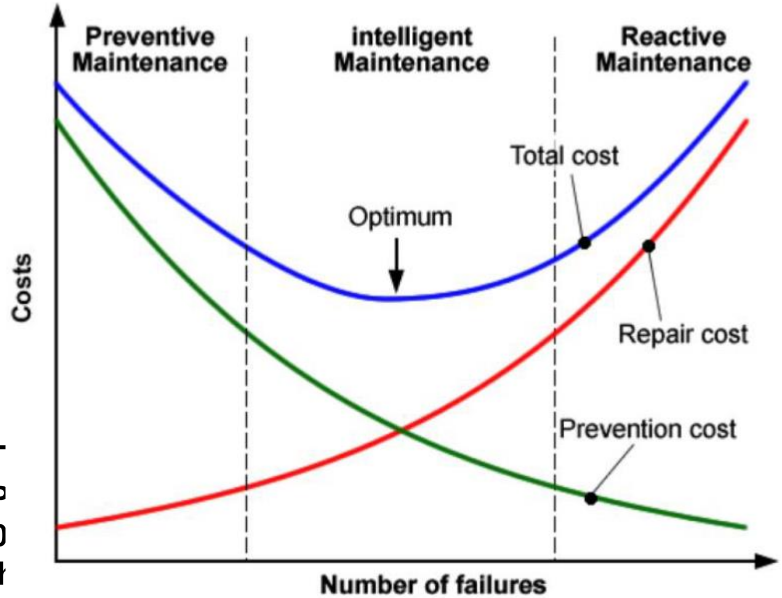
Basic approaches:

- Corrective: after a certain amount of damage has occurred
- Preventive: prevent unaccepted damages

Often a combination is used, aiming for optimal balance between costs and performance

Decision making system aiming for the best repair option, considering multiple performance aspects (safety, durability, functionality and economy etc.)

Good understanding of bridge condition and future degradation is necessary for this optimization



CASE STUDIES: Croatia vs. the Netherlands

Aim: to assess the impact of different maintenance policies (corrective vs. preventive) on the occurred maintenance costs

Approach:

- Compile data from previously performed inspections to correlate age, condition and costs for structural elements
- Compare condition rating and planned maintenance costs for structural elements of bridges at different ages

Croatia: mostly corrective maintenance strategy

The Netherlands: mostly preventive maintenance strategy

CROATIA

General approach:

- Visual inspections carried out on an 'as-needed' basis (inspection and testing when structural damage become self-evident)
- Condition rating 0 to 5
- Repair works are identified based on a condition assessment

Scope of case:

- 12 bridges, constructed in 1981 and 1988, inspected in 1998 and 2010
- Only routine maintenance was performed on viaducts from construction until the final inspection in year 2010

THE NETHERLANDS

General approach:

- Visual inspections carried out periodically (6 years)
- Condition rating 0 to 6
- Repair works are identified based on condition and risk assessment

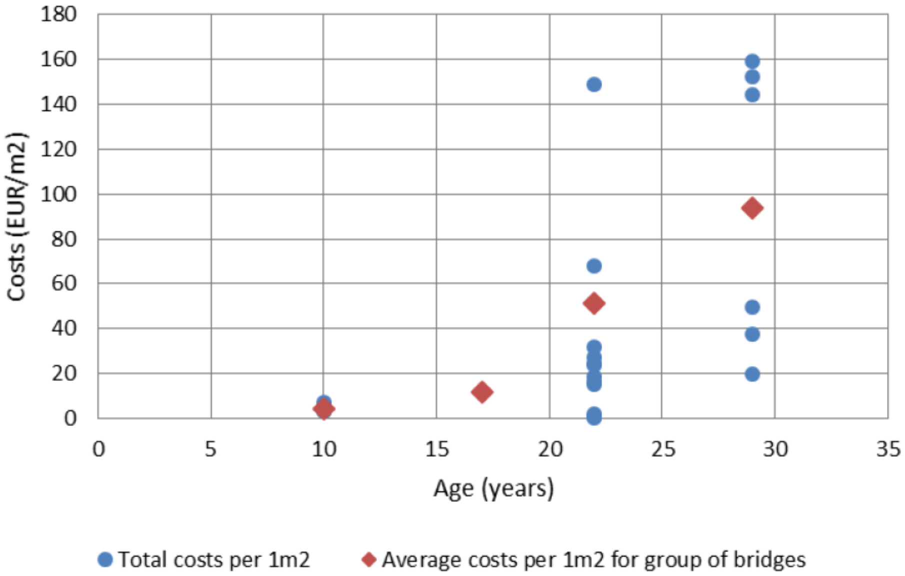
Scope of case:

- 24 bridges, constructed from 1960’s to 1990’s, inspected ca. 2010 and 2015
- Routine and planned maintenance was performed since construction

RESULTS

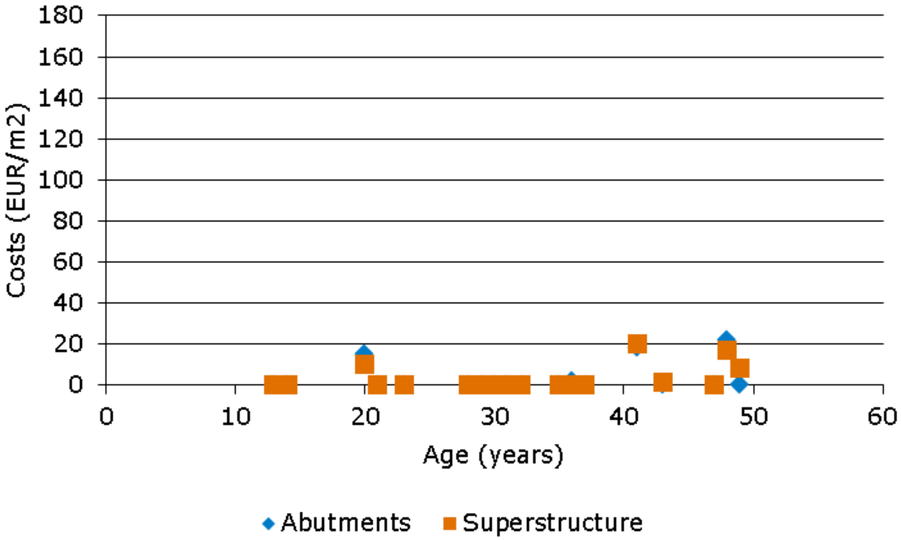
Croatia:

- Average value of direct repair cost increases rapidly after an age of 17 years



The Netherlands:

- Good condition for most bridges (1 or 2), no maintenance necessary for most bridges (15 out of 24)
- Most maintenance planned for bridges aged over 36 years



DISCUSSION AND CONCLUSION

Two opposing maintenance strategies examined: corrective and preventive

Although limited to a small number of bridges:

- Relation appears between the age of the structure and maintenance costs
- For a preventive strategy, the increase in costs appears to be later than for a corrective strategy

Notes for COST action TU1406:

- Keep assets at a desired performance level
- Specific performance indicators are established for components
- Performance goals will vary according to technical, environmental, economic and social factors, however, a preventive approach to maintenance planning supported by an effective inspection procedure will, in the longer term, likely be more efficient

THANK YOU FOR YOUR ATTENTION!

Giel Klanker – Rijkswaterstaat, the Netherlands
Irina Stipanovic Oslakovic – University of Twente, the Netherlands
Sandra Skaric Palic – Institut IGH, Croatia

Rijkswaterstaat
Ministry of Infrastructure and the Environment

UNIVERSITEIT TWENTE.

Institut IGH d.d.

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Performance indicators for bridges exposed to a flooding hazard

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02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
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University of Zagreb
Faculty of Civil Engineering

OUTLINE

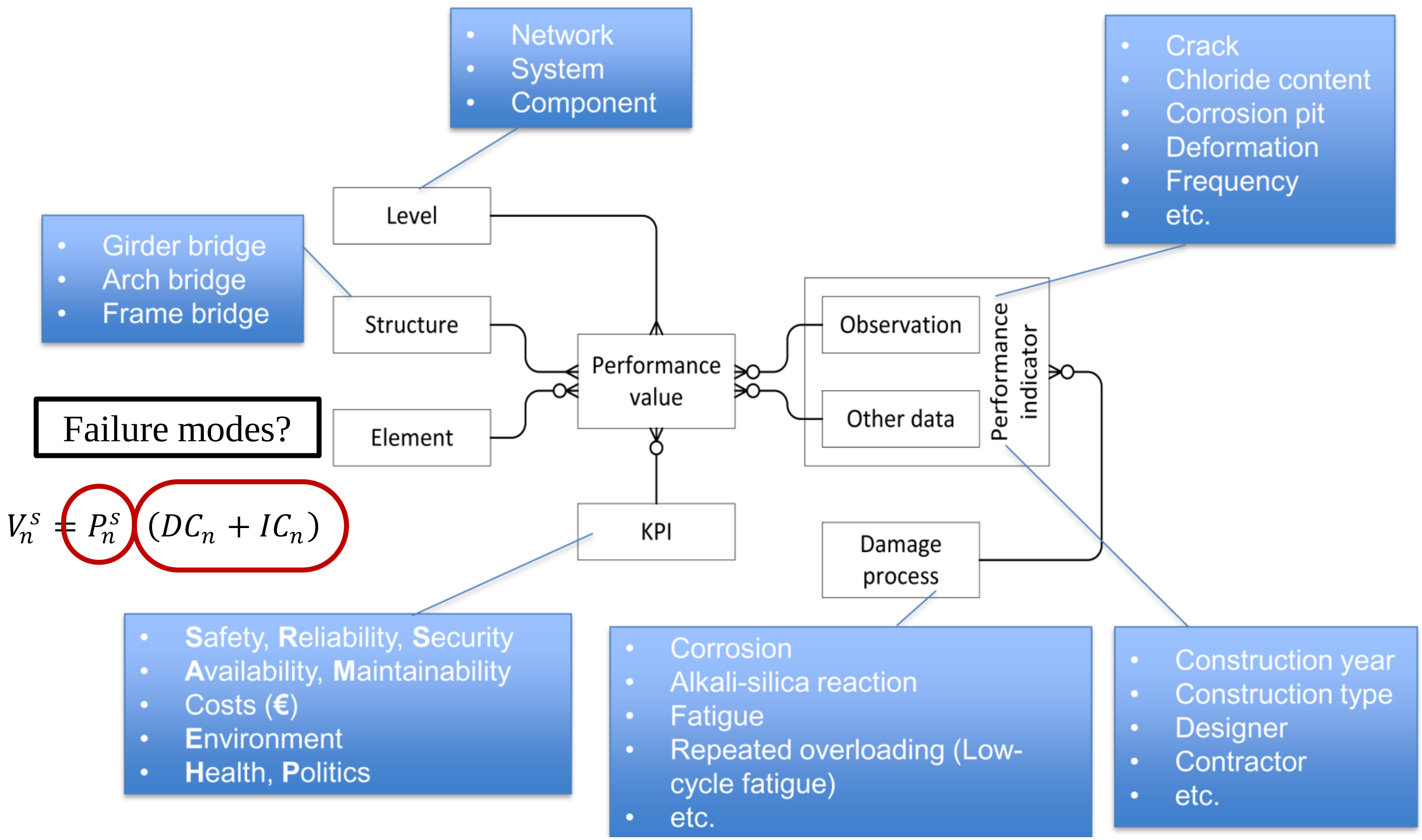
- The status of the Work Group 3 Task 4.
- Summary of the WG1 survey for PIs
  Scope: Flooding hazard & local scour
- Discussion on the PIs for flooding/scour hazard and related relevant data
- The impact of the PIs for flooding hazard on the KPIs and structuring of adequate QC plans
- Conclusions

Status of the Work Group 3, Task 4

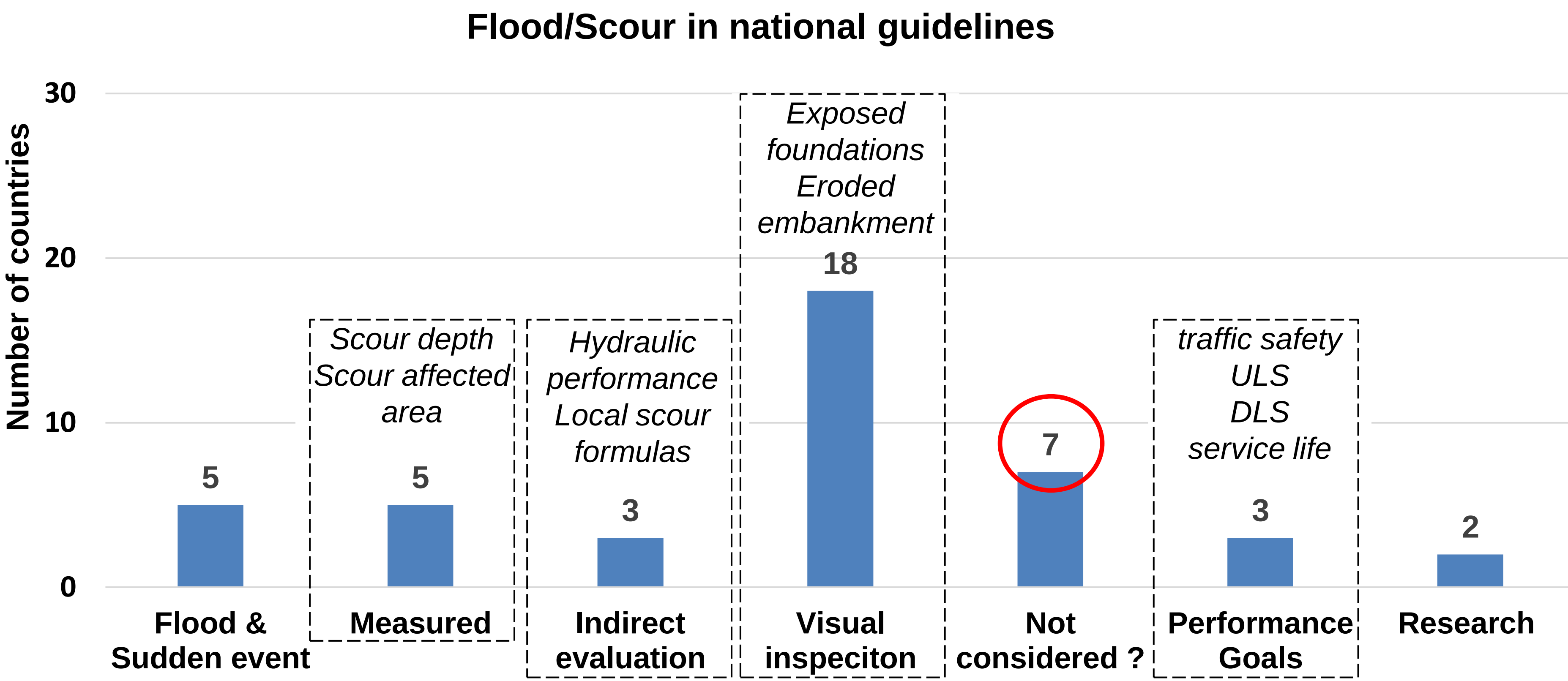
- **Delft meeting**
  - **9 KPIs**
    (Reliability, Safety, Security), (Maintainability, Availability), (Economy, Cost), (Health, Politics)
  - **WG 3, TASK 4. Non-interceptable processes**
    - ✓ Dynamics and uncertainty of sudden processes
    - ✓ Required quality levels (i.e. performance goals)
    - ✓ Triggering criteria (i.e. thresholds) for inspections and maintenance
    - ✓ QC Plans for girder, arch and frame bridges
  - **Quantitative methodology for vulnerability assessment is preferred**

Status of the Work Group 3, Task 4

- **Vulnerability assessment relationship to KPI**



Summary of the WG1 survey for PI-s



- **Terms <u>not mentioned</u>:** scour countermeasures, overtopping, washing away of access roads

Discussion on the PI-s for flooding/scour hazard and related relevant data

- **Reported terms on scour:**

  Visual Inspection (e.g. exposed foundation, eroded embankment)

  - Possible failure scenario revealed **- not reliable**

  - Prioritization for monitoring/measuring of scour **- not reliable**

  - Ineffective against flash flooding

  Measured e.g. scour depth and scour affected area

  - Scour cavity infill

  - Questions of cost and adequacy

  - Ineffective against flash flooding

  Indirect evaluation e.g. hydraulic adequacy, scour evaluation formulas

  - Appropriateness of the applied formulas

  - Overestimation of a scour depth

Discussion on the PI-s for flooding/scour hazard and related relevant data

- List of key terms for a bridge exposed to a flooding hazard and scour

		Performance indicators			
Structure	**Elements**	**Observation**	**Other relevant data**	**Damage process**	**KPI**
All bridge types and materials	*Foundations*	*Scour depth*	*Bridge geometry & dead load*	*Flood/Scour*	*Reliability*
	Embankment	*Scour affected area*	*Type of foundations*	*Erosion*	*Safety*
	Scour Countermeasures	*Exposed foundation*	*River bed properties*		*Availability*
	Substructure	*Eroded embankment*	*Foundation soil properties*		*Cost*
	Bearings/Joints/Hinges	*Hydraulic performance*	*Flood magnitude*		*Maintainability*
	Superstructure	*Specific damage location & severity*	*Debris/ice potential*		*Economy*
		Condition state	*Traffic data*		

- Other relevant data = Technical parameters which complement evaluation of indicators

- Indirect observations
  - ➢ (rotations & settlements) + related damage = failure mode already occurred!

Discussion on the PI-s for flooding/scour hazard and related relevant data

- **Key data for hazards: Exposure, Resistance, Consequences**

- **Exposure** (hazard scenario):
  – Flooding data (extreme flow & duration)
  – Water channel properties
  – Piers & abutments geometry and location in respect to a water flow

- **Resistance** (failure modes):
  – Soil at foundations (erodibility & geomechanic prop.)
  – Type/detailing of the substructure and superstructure
  – Location of damage on elements and its severity

- **Consequences** (related to failure modes)
  – Costs of repairs/replacement
  – Traffic data (vehicle operating costs, accident costs, travel time…)

Discussion on the PI-s for flooding/scour hazard and related relevant data

- **Questionnaire necessary !**
  - **Past failures i.e. references to papers and investigation reports**
    – *Crucial information on possible failure modes*
  - **Methodology for scour assessment**
    – *Equipment and procedures for measuring of scour at substructures*
    – *Indirect evaluation of scour and related data (esp. consequences)*
    – *Thresholds for initiating maintenance activities*
  - **Availability of data to conduct quantitative assessments i.e. risk/vulnerability assessments**
    – *Hazard maps*
    – *Resistance of bridges*
    – *Traffic data*
  - **BM practice regarding a climate change**
    – Research needs and current shortcomings

The impact of PI-s for flooding hazard on the KPI-s and structuring of adequate QC plans

- **FHWA Qualitative approach**

Data	PI: NBI Item 113 - Scour Critical Bridges	
Exposure	Visual inspection, overtopping history, foundation status	
Resistance	Indirect evaluations of local scour depth Condition state of countermeasures Foundation type/depth	**Superstructure not considered**
Consequences	Stability endangered = Bridge closure	**Not evaluated**

- **QC plans – not explicit**
  Evaluate scour (formulas); apply countermeasures; traffic restriction and bridge closure
- **KPI =** Reliability and Safety?

The impact of PI-s for flooding hazard on the KPI-s and structuring of adequate QC plans

- **NYSDOT Qualitative approach**

Data	PI: Hydraulic vulnerability score
Exposure	Channel hydraulic properties, foundation type/location historical scour
Resistance	Non-redundant / redundant bridge types Foundation material **uncomprehensive !**
Consequences	Failure type and traffic volume **uncomprehensive !**

- **QC plans -** general guidelines are given
  Flood watch, Post-flood inspection, Hydraulic analysis
- **KPI related:** Reliability, Safety, Cost?

The impact of PI-s for flooding hazard on the KPI-s and structuring of adequate QC plans

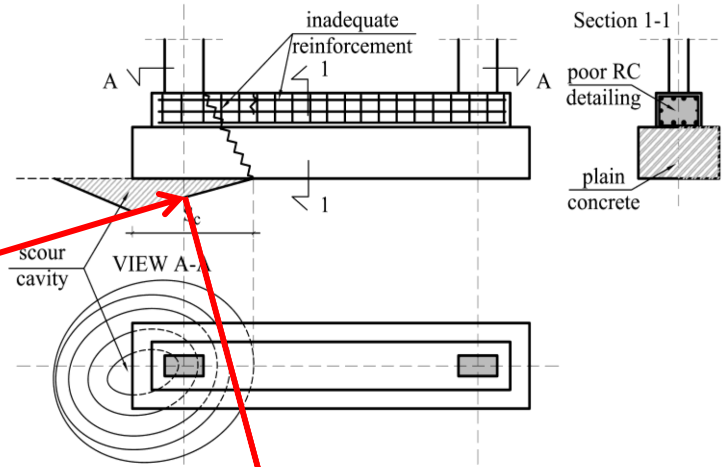
- **HYRISK Quantitative approach (bridges with unknown foundations)**

Data	PI: Risk of scour failure
Exposure	NBI Items
Resistance	Adjustment factors for types of a span and foundation Probability of failure – NBI items **uncomprehensive !**
Consequences	Traffic volume **Failure type not considered**

- **QC plans -** general guidelines are given incl. thresholds
  Foundation survey, countermeasures, automated monitoring
- **KPI related:** Reliability, Safety, Cost?

The impact of PI-s for flooding hazard on the KPI-s and structuring of adequate QC plans

- Deep / shallow / unknown foundations
- Structure resistance
  - Detailing of a foundation
  - Type/properties of the joints at a pier/abutment top
  - Type/properties of a superstructure and a number of spans



CONCLUSION

- Current procedures in BM related to non-interceptable processes are <u>unclear</u> and <u>uncomprehensive</u>
- There are different approaches among countries to account for a flooding hazard
- Resistance of the bridge structure must be accounted for
- Vulnerability – probability of failure and consequences – The direct relation to the KPI-s

THANK YOU
FOR YOUR ATTENTION

Nikola Tanasic – Faculty of civil Engineering, University of Belgrade, Serbia
Rade Hajdin – Faculty of civil Engineering, University of Belgrade, Serbia

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

QUALITY CONTROL PLANS FOR ARCH BRIDGES

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02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
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Faculty of Civil Engineering

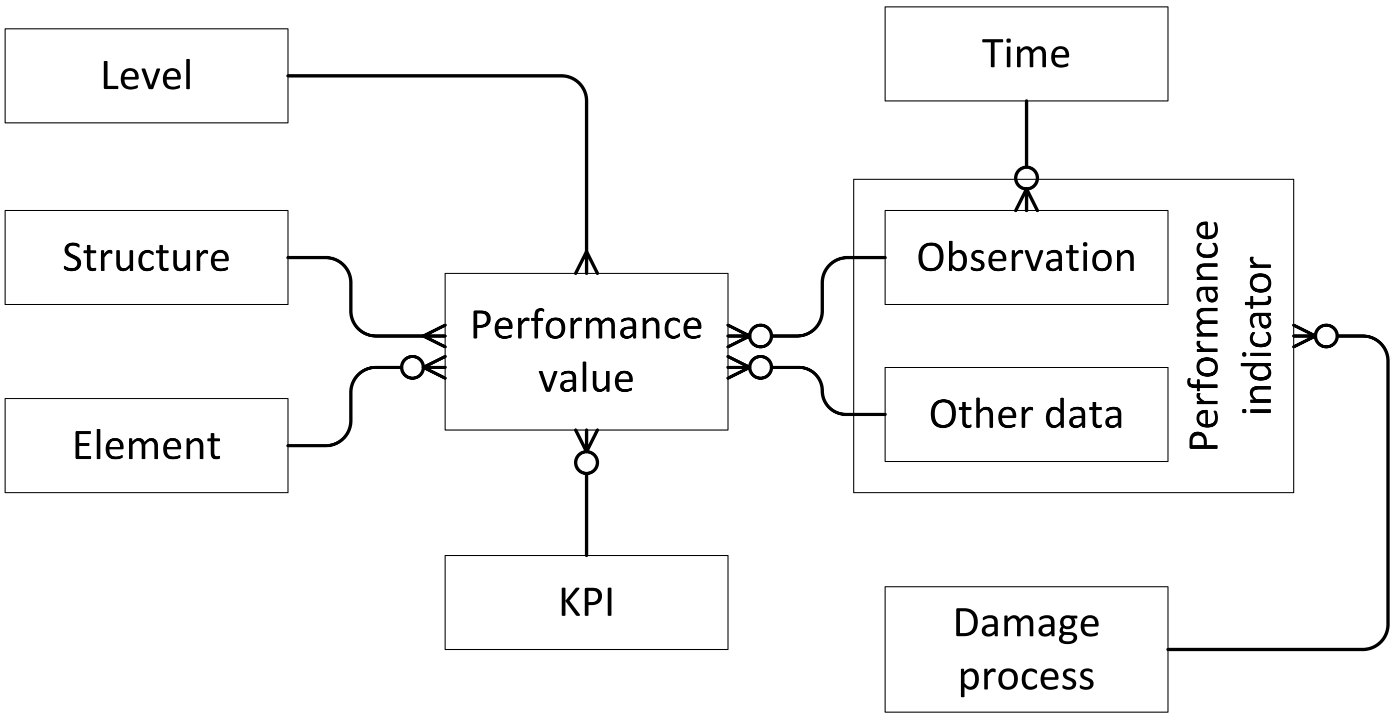
INTRODUCTION

→ WG3 Goal is to provide detailed step-by-step explanations for establishment of Quality Control Plans for different types of bridges;

→ The main challenge of a QC plan is the necessity to *connect general data about each bridge, observation/findings and other performance indicators with a set of key performance indicators that can be directly related to performance goals;*

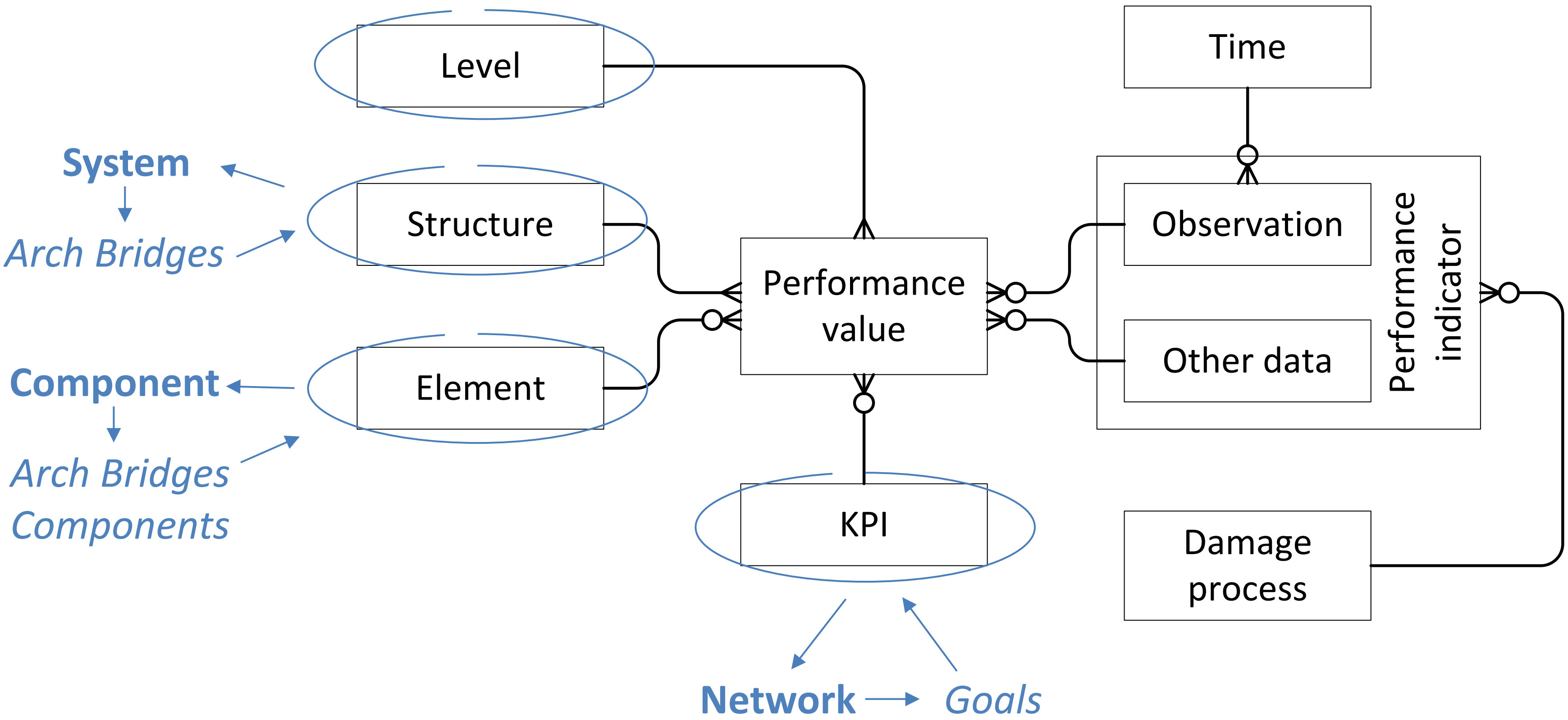
→ A general approach for QC plans was already developed in the scope of WG3 for all types of structures, and is now analyzed within the scope of arch bridges.

2. COMMON FRAMEWORK FOR THE DEVELOPMENT OF QCPs



Rade Hajdin, 2016

3. QUALITY CONTROL PLANS FOR ARCH BRIDGES



3.2 Structure – *Arch Bridges*

3.2 Structure – *Arch Bridges*

→ QCPs will focus on common bridges, not landmark bridges;

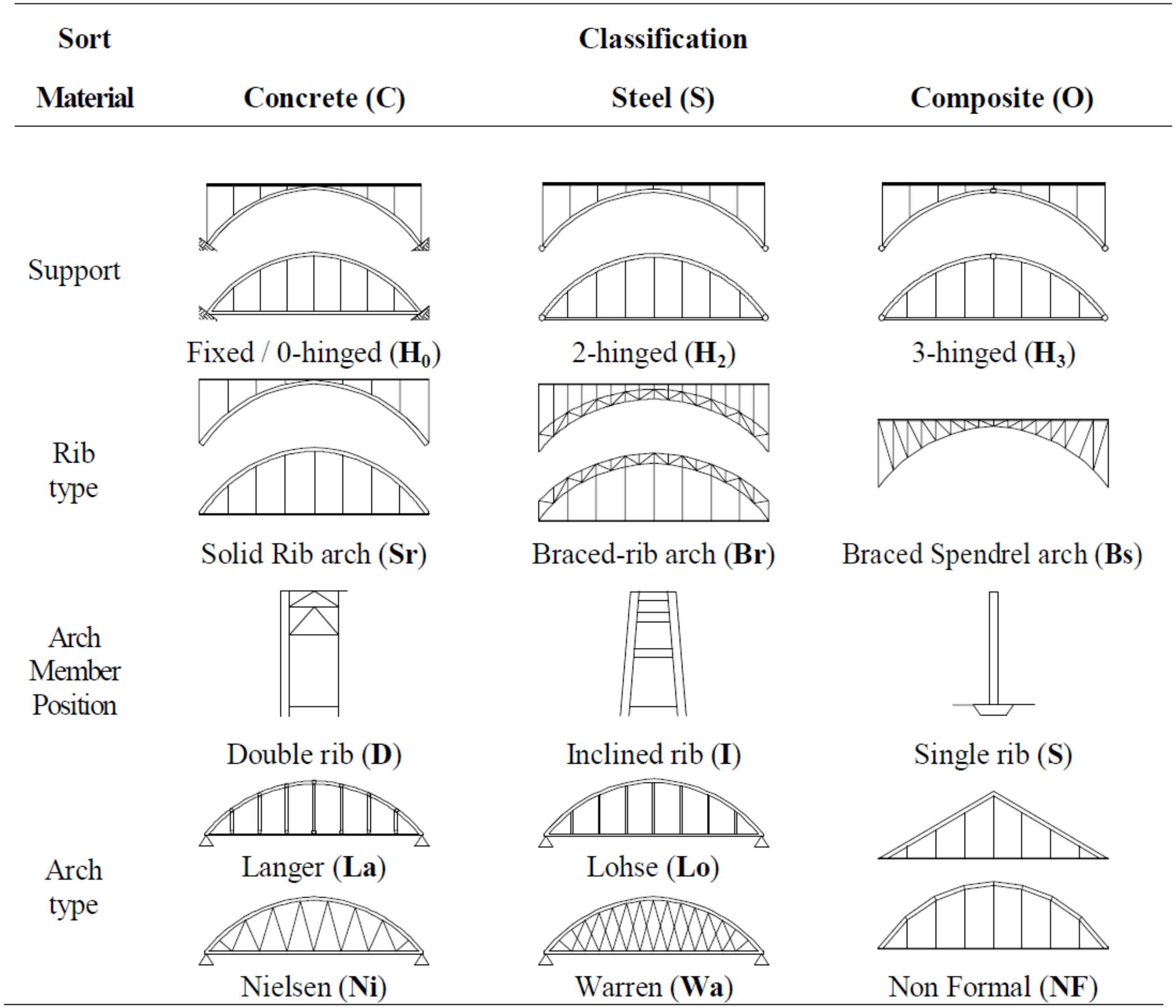
→ Arch bridges represent a significant percentage in most European countries national inventories, with special emphasis on masonry structures (UIC, 2011);

→ *Portuguese roadway network managed by IP include almost 34% of bridges classified as Arch Bridge;*

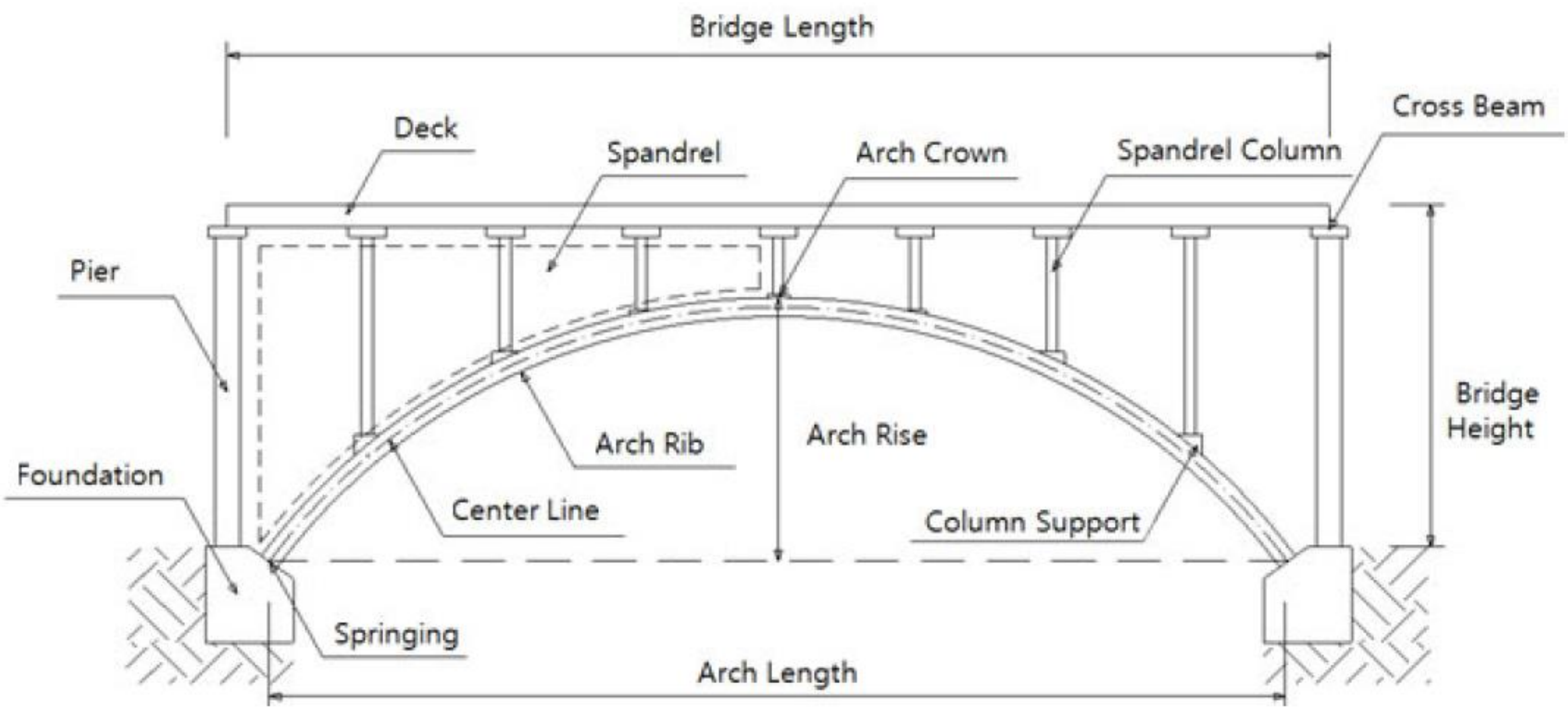
→ *More than 80% of these Arch Bridges are Masonry Arch Bridges.*

3.2 Structure – *Arch Bridges*

→ *Identification Code for the Type of Arch Bridge*, M. Han et. al., 2016

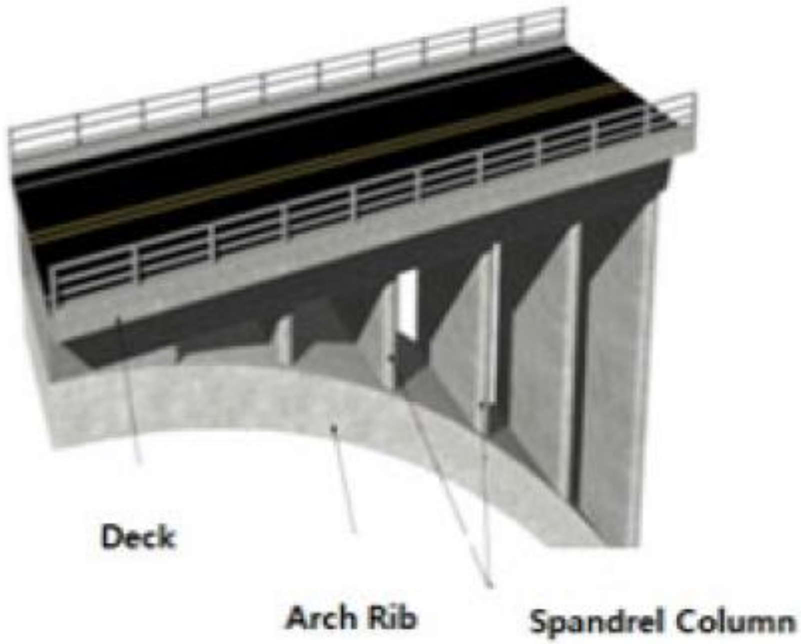

Sort
Classification
Material
Concrete (C)
Steel (S)
Composite (O)
Support
Fixed / 0-hinged ($\mathbf{H_0}$)
2-hinged ($\mathbf{H_2}$)
3-hinged ($\mathbf{H_3}$)
Rib type
Solid Rib arch (**Sr**)
Braced-rib arch (**Br**)
Braced Spendrel arch (**Bs**)
Arch Member Position
Double rib (**D**)
Inclined rib (**I**)
Single rib (**S**)
Arch type
Langer (**La**)
Lohse (**Lo**)
Nielsen (**Ni**)
Warren (**Wa**)
Non Formal (**NF**)

3.3 Element

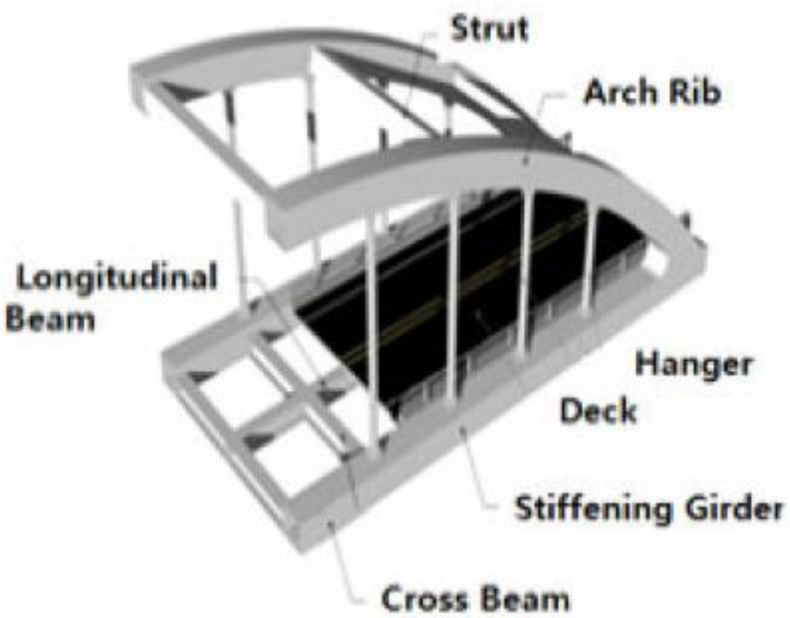


Outline of Arch Bridge, M. Han et. al., 2016

3.3 Element



Deck Arch Bridge

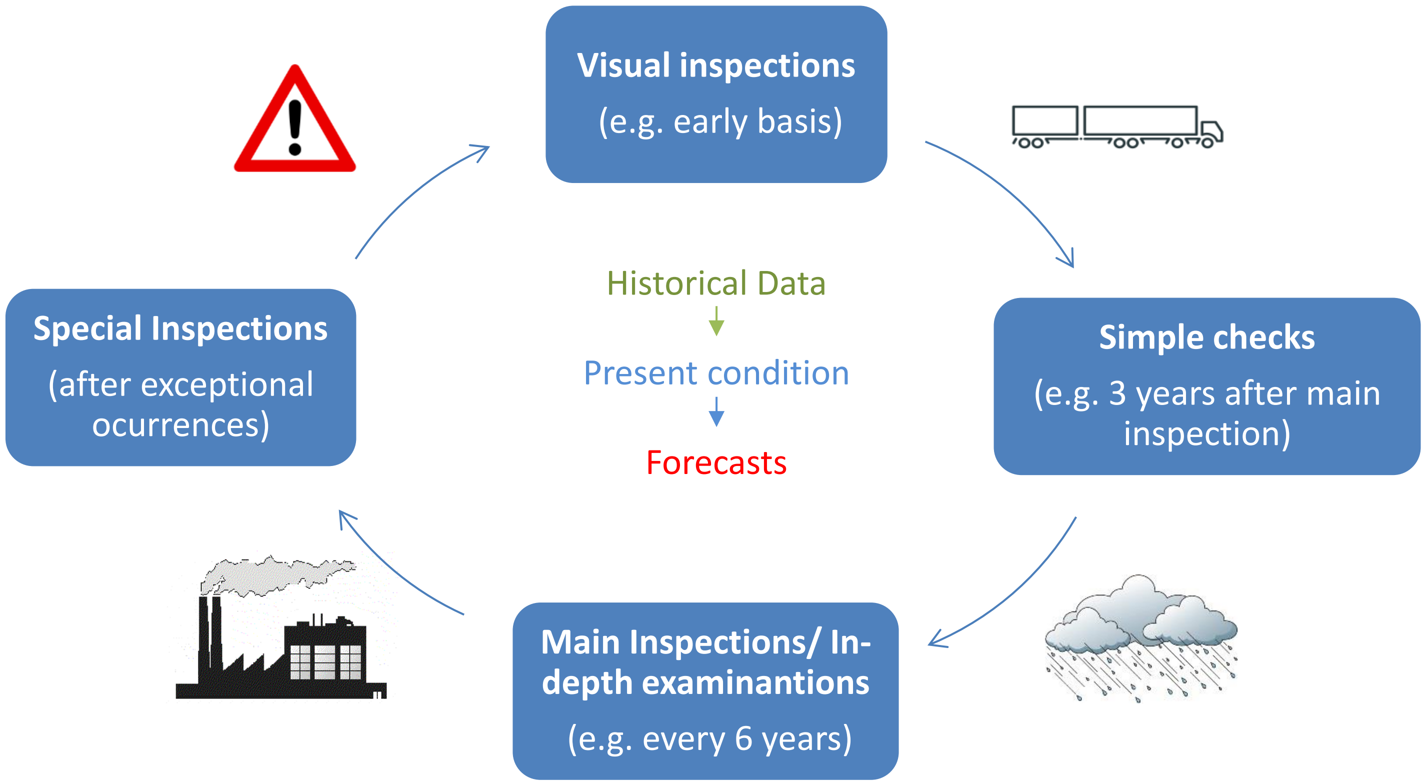


Through Deck Bridge

M. Han et. al., 2016

3.4 Time

→ From COST TU1406 WG1 Survey – common understanding on investigation workflow to access bridge condition:



3.5 Observations and Other Data

→ *Arch bridge defects versus health problems*, J. Bień and M. Gładysz-Bień, 2016

Defect type	Defect sub-type	Arch bridges	Medicine
Deformation	Changes of element (body component) axes geometry	●	●
	Changes of geometry along the element (body component)	●	●
Material destruction	Changes of chemical features	●	●
	Changes of physical features	●	●
Material losses	Losses of structural material (internal body components)	●	●
	Losses of protective material (external body components)	●	●
Material discontinuity	Scratch	●	●
	Crack	●	●
	Delamination	●	●
Contaminations	Inorganic	●	●
	Organic	●	●
Position changes	Abnormal position of elements (body components)	●	○
	Abnormal movement of elements (body components)	●	○
	Limitations in movement possibilities	●	●

3.4 Damage Processs

→ *Degradation mechanisms on Arch Bridges,* J. Bień and M. Gładysz-Bień, 2016

Degradation mechanisms		Material of arch bridge		
		concrete	steel	masonry
Physical	Accumulation of inorganic contamination	●	●	●
	Freeze/thaw actions	●	○	●
	Erosion	●	○	●
	Crystallization	●	X	○
	Extremal temperature influence	○	●	○
	Rheological processes	●	○	○
	Overloading	●	●	●
	Leaching	●	X	●
	Fatigue	○	●	○
	Changes of geotechnical conditions	●	●	●
Chemical	Carbonization	●	X	○
	Corrosion	●	●	X
	Aggressive environmental impact	●	●	●
	Reactions between material components	●	○	○
Biological	Accumulation of organic contamination	●	●	●
	Influence of microorganisms	●	●	●
	Influence of plants	●	○	●
	Influence of animals	○	●	○

● - basic mechanism; ○ - supplementary mechanism; X - not applicable

3.3.1 Performance Indicators

→ Clusters of PIs related terms, resulted from COST TU1406 WG1 Survey

Defects

Related to loads

Related to cost & Importance

Related to rating

Related to material properties

PI

Environmental based

Related to equipment & protection

Related to dynamic behaviour

Related to geometry changes

Related to bearing capacity, structural integrity & Joints

Related to Original construction & design

A. Strauss, A. Mandić, ed., 2016

384 PI related terms → 384 PIs in QCP ?

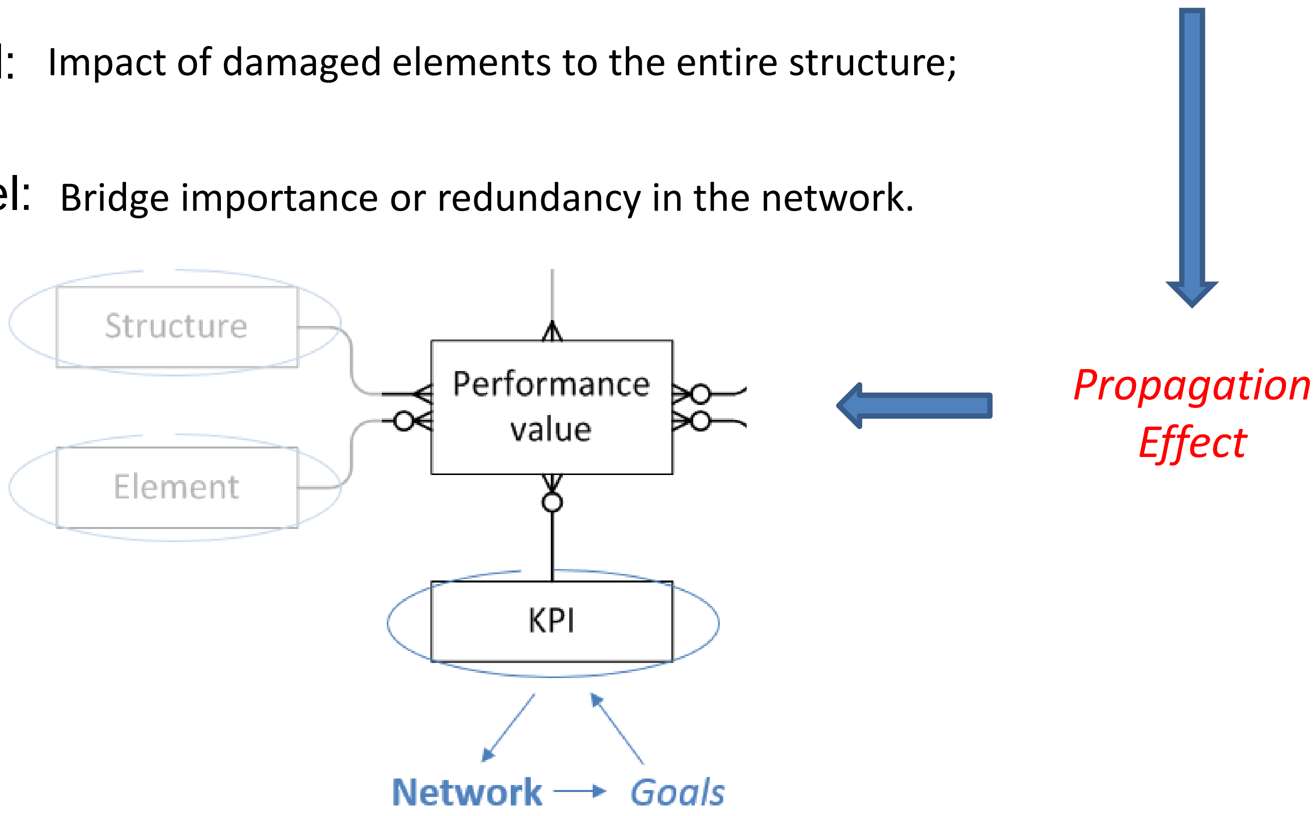
- → Performance Indicators
- → Damage Processes
- → Observations
- → Other Data

3.5 Level

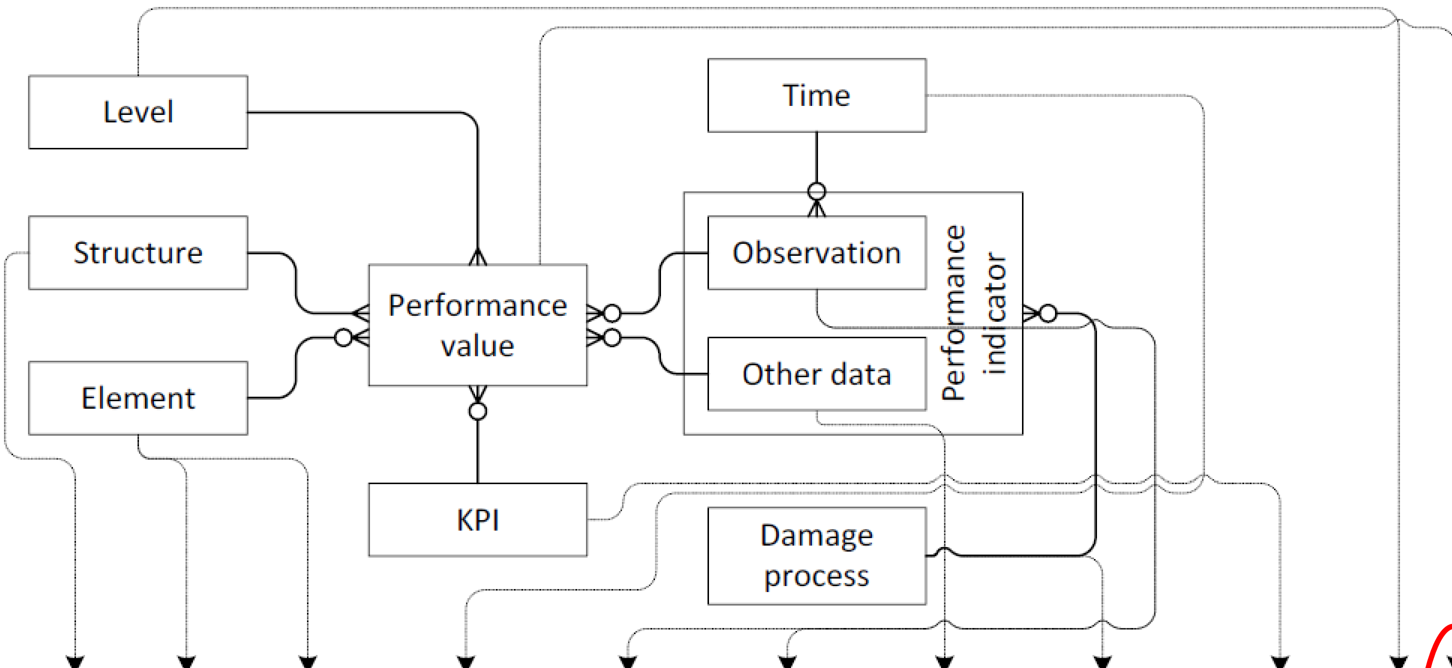
→ Component level: General inspections, observations/ assessment of bridge elements;

→ System level: Impact of damaged elements to the entire structure;

→ Network level: Bridge importance or redundancy in the network.



3.6 Performance Value



Single Girder Bridge	Beam	Reinforced concrete	*Time of the latest inspection*	Crack	0.4 mm	Construction year	Repeated overloading	Safety, Reliability, Security	C	2
Single Girder Bridge	Beam	Reinforced concrete	*Time of the latest inspection*	Spalling	1.0 m^2	Construction year	Corrosion of Reinforcement	Safety, Reliability, Security	C	3
Single Girder Bridge	Beam	Reinforced concrete	*Forecast in 10 years*	Spalling	2.0	Construction year	**Corrosion of Reinforcement**	Safety, Reliability, Security	C	4

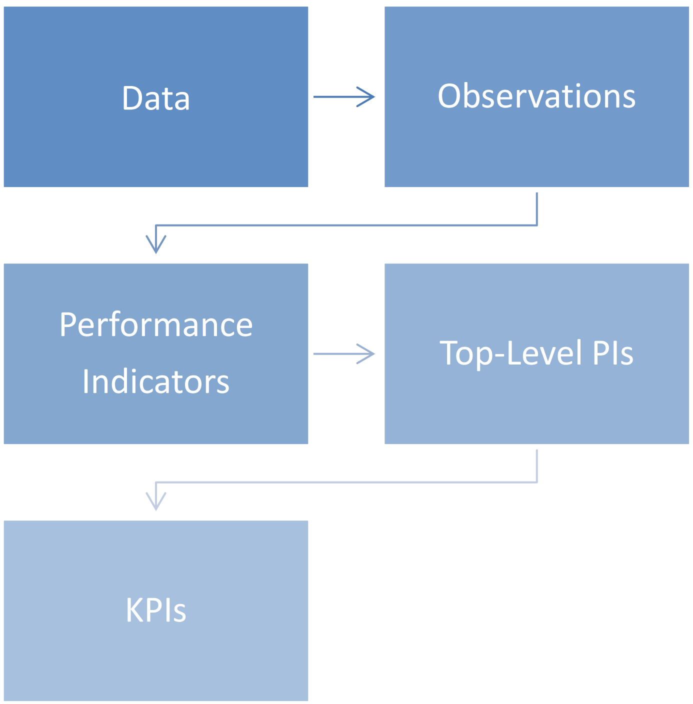
3.6 Performance Value

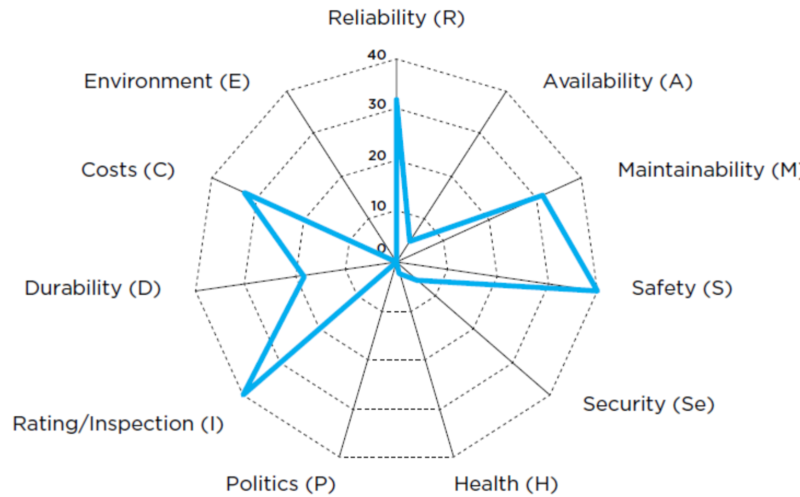
→ COST TU1406 WG1 Survey and report revealed a similar approach used in several countries and present a typical example of a five-level rating system:

Rating Index	Description
1	No or very slight damage, normal age-related wear and tear, aesthetic damage. No decrease in load carrying capacity, serviceability and predicted life time. No measures required.
2	Slight damage, production defects with no signs of further deterioration. No decrease in load carrying capacity and serviceability. If no suitable measures are taken, the predicted life time will decrease. Repair measures are required in the course of the next maintenance action.
3	Moderate to severe damage with no decrease in load carrying capacity and serviceability. Signs of deterioration regarding load carrying capacity and serviceability. Medium-term maintenance and repair actions are necessary in order to preserve the servicea-bility and expected life time of the structure.
4	Severe damage, with no decrease in load carrying capacity. Deterioration in terms of serviceability and expected life time can already be observed. Maintenance measures are to be instigated as soon as possible in order to safeguard the ser-viceability and the expected life time. Such measures may be substituted by additional special inspections within a defined time frame.
5	Extreme damage with impact on the load carrying capacity of the structure. Repair and maintenance measures must be performed immediately.

A. Strauss, A. Mandić, ed., 2016

3.7 Key Performance Indicators





→ Number of Performance indicators related to the pre-defined Key-Performance Indicators KPIs based on the categorized, homogenized and reduced Performance indicators of the findings from the screening and processing of the national applied documents. A. Strauss, A. Mandić, ed., 2016

CONCLUSION

Visual observations will continue to have an important role on the management of large stocks of bridges;

QCPs can help to systematize entities and their liaisations, looking forward to the fulfillment of quality requirements and the optimization of resources;

WG1 survey revealed a common approach among European countries, justifying the *Action* purpose of standardization;

As future developments it will be necessary to model the impact of different combinations of structure types and elements, observations and other relevant data with KPIs.

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Alfred Strauss, A. Mandić Ivanković, editors. Performance Indicators for Roadway Bridges of Cost Action TU1406. WG1 Technical Report, Cost Action TU1406, 2016.

Bień and Gładysz-Bień. Gerontology of Arch Bridges. In *Proceedings of the 8th International Conference on Arch Bridges*, pages 1269-1278, Wrocław, Wrocław, Poland, 2016.

Thank you for your attention!

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Rade Hajdin – University of Belgrade, Serbia

Infraestruturas de Portugal

Грађевински Факултет
Универзитет у Београду

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

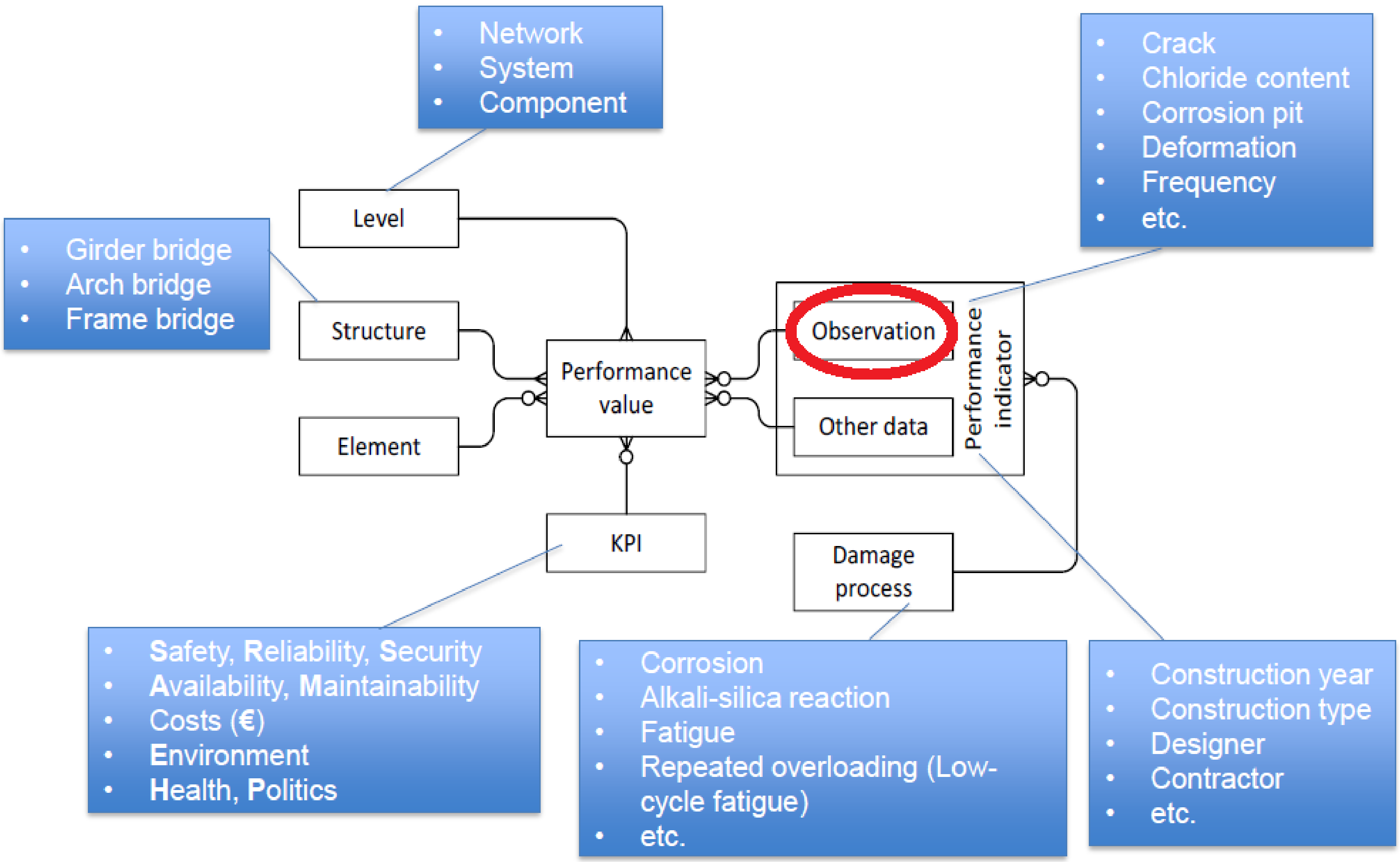
BRIDGE INSPECTION QUALITY IMPROVEMENT USING STANDARD INSPECTION METHODS

Matej Kušar – Faculty of Civil and Geodetic Engineering, University of Ljubljana, Slovenia

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

BRIDGE INSPECTION IN OC PLAN FRAMEWORK *



*** Hajdin, R. 2016. From performance indicators to performance goals (COST TU1406 3rd workshop, Delft, Netherlands)**

DAMAGE DETECTION AND ASSESSMENT

Main approaches:

1. Visual inspection
2. Non-destructive testing
3. Probing
4. Structural health monitoring

Suitable for large-scale periodical damage detection

VISUAL INSPECTION

Most common method of data acquisition:

Simple and cost effective (+)

Somewhat less reliable as other approaches (-)

Data quality can be improved

1. Improvement of inspection protocol
2. Complementary use of NDT

ISSUES REGARDING VISUAL INSPECTION

- Inspection frequency
- Inspectors qualification and experience
- Inspectors rotations
- Data input
- Office review
- Field review
- Refresh training
- Performance indicators

NON-DESTRUCTIVE TESTING

- Not regularly integrated in periodical bridge inspection
- Should be applied when degradation processes intensify

WHY?

Application may bring valuable additional information regarding:

- Damage depth
- Damage intensity
- Concealed damage

NON-DESTRUCTIVE TESTING

- Reliable, simple and if possible inexpensive NDT should be used
- Advanced methods are not suitable for large scale implementation

Indicative overview of 30 NDT already available

Aim is to identify methods appropriate

- Time consumption
- Cost efficiency
- Result reliability
- **Cover shortcomings of visual inspection**

CONCLUSION

- The damage state of majority of bridges is determined by conducting periodical visual inspections only

- Selection of the most appropriate protocol for conducting regular inspections (visual inspection + NDT) could be extremely demanding

WHY?

- Best possible effort for quality data acquisition will stay in the domain of detailed bridge inspection

(number of bridges) X (limited financial resources) = **reasonable effort**

THANKS FOR YOUR ATTENTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Observation-based Decision-making for Infrastructure

Eleni Chatzi – ETH Zürich
Konstantinos Papakonstantinou – Penn State University
Rade Hajdin – University of Belgrade
Daniel Straub – Technical University of Munich, Germany

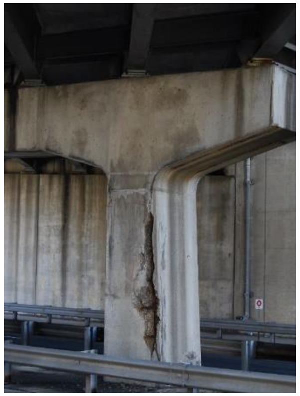
02nd – 03rd March 2017
Zagreb, Croatia

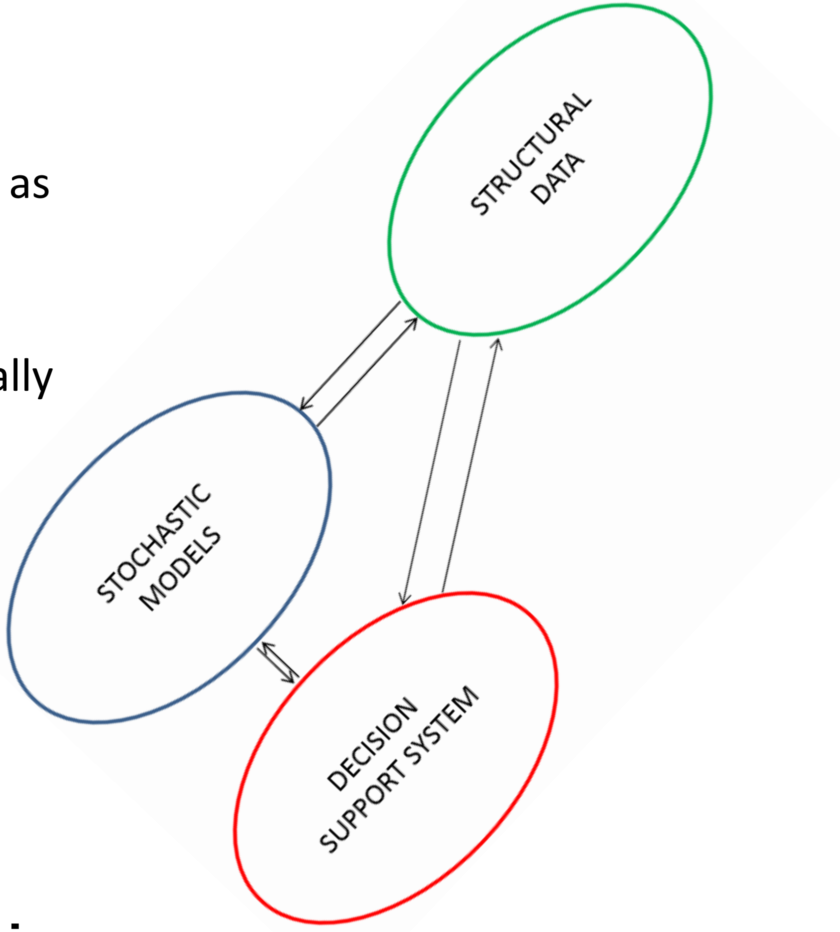
Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

Structures and infrastructure systems face challenges due to **aging, deterioration and adverse operational conditions**

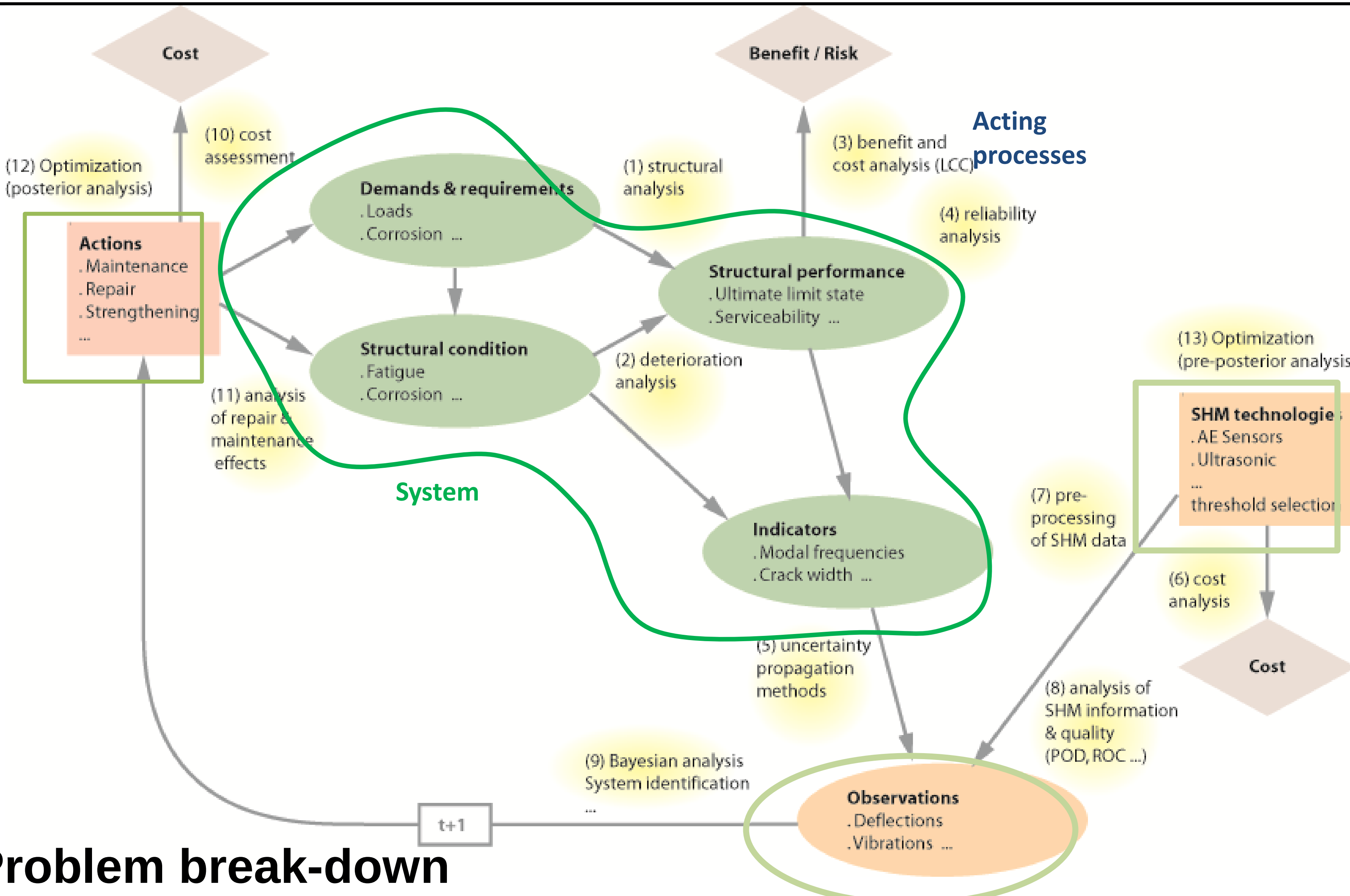
Notions like **life-cycle performance**, infrastructure **management and sustainability** have been deemed as necessities

To address these problems effectively and scientifically new **Methods & Tools** are needed





Optimal Inspection & Maintenance Planning

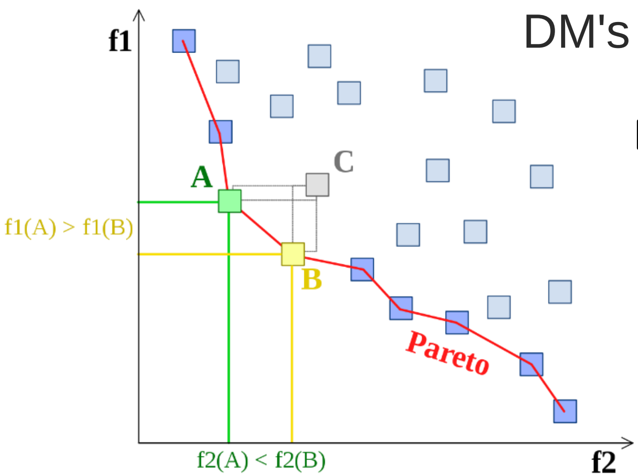


Problem break-down

Approaches to Decision-Making

Multi Criteria Decision Making: consider the DM's preference structure in advance so as to transform multiple objective functions into a single objective function (single criterion-synthesis MAUT, MAVT, or outranking methods ELECTRE) *Teixeira de Almeida et al. (2015)*

Multi-Objective optimization:
identify the non-dominated solutions, the Pareto frontier, without taking the DM's preferences into account *Frangopol & Liu (2005)*



Bayesian Statistical Decision theory

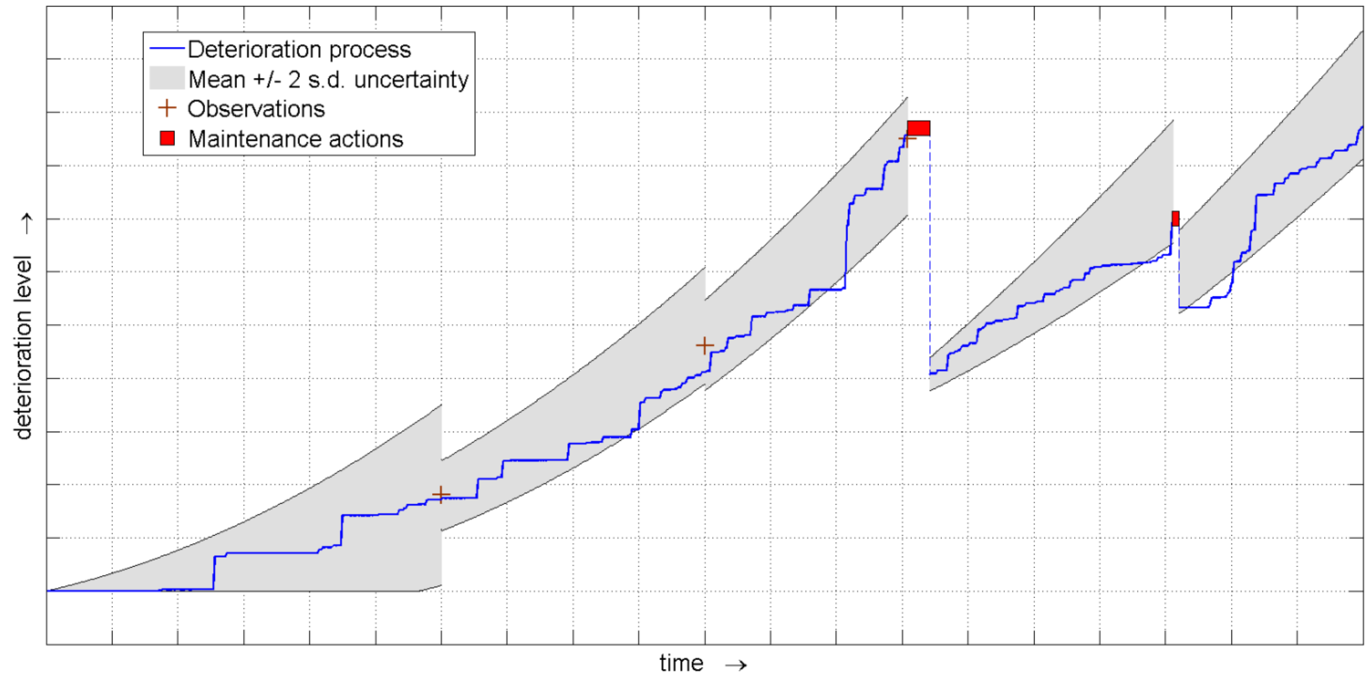
Bayesian Networks *Sørensen (2009)*

Stochastic Optimization *Taflanidis & Beck (2008)*

and more..

Optimal Inspection & Maintenance Planning

Identified Challenges



Papakonstantinou & Shinozuka (2014)

Incorporate stochastic models and uncertain data, based on firm mathematical foundations

Provide accurate assessment of system state at all times

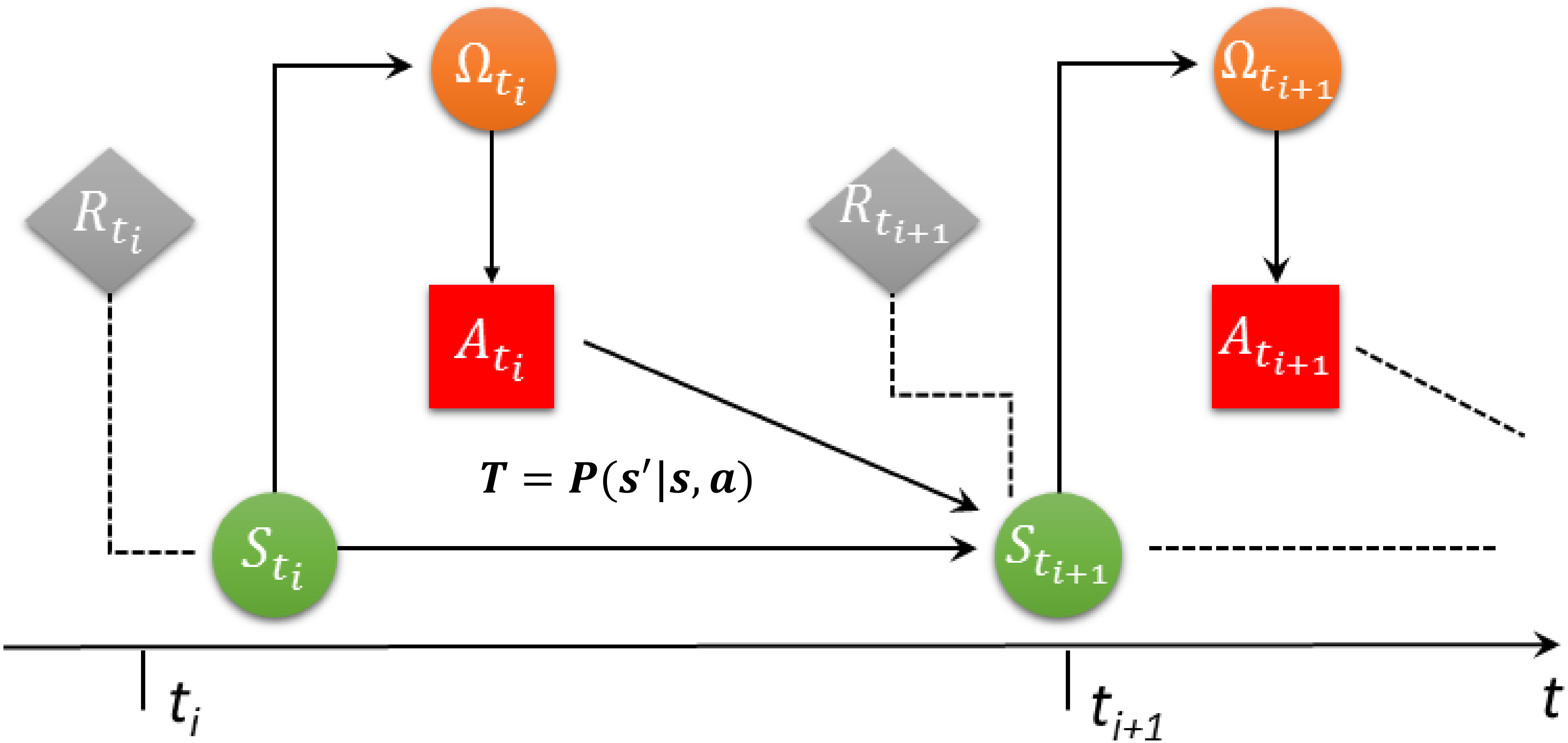
Optimizes long-term objectives

Uses near-real-time observations

Allow for an updating process

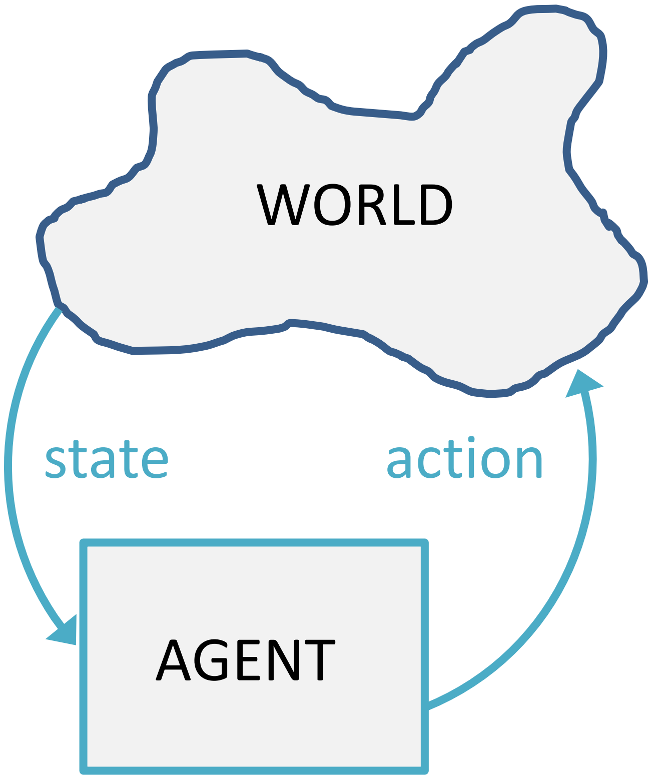
Allow for near-real-time optimal decision support

Sequential decision process with alternating actions and inspections



Optimal Inspection & Maintenance Planning

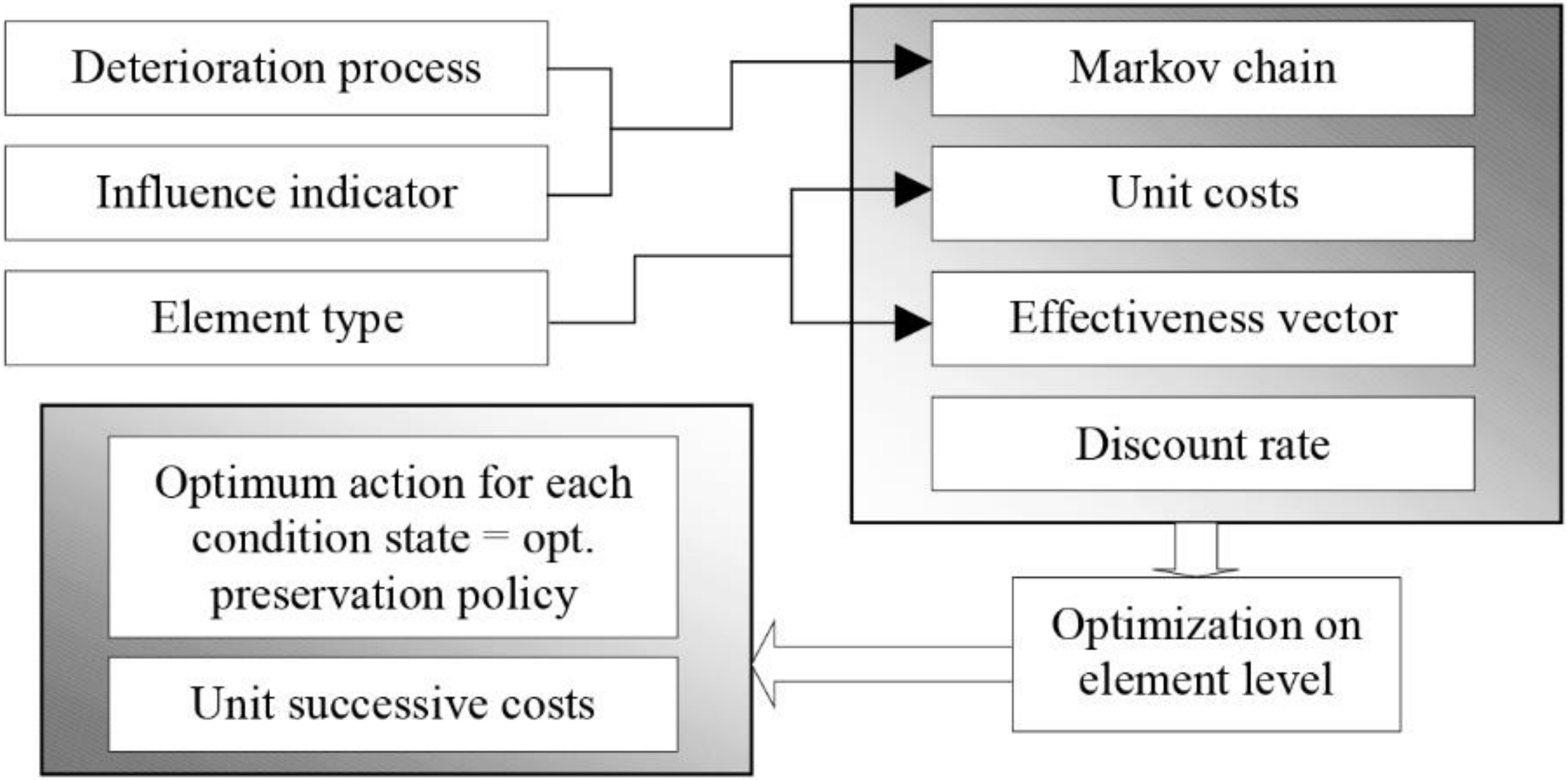
Fully Observable MDP



- Decision depends on current state, no history
- Initial state is known
- Action's consequences are known
- World is known
- The state is fully observable

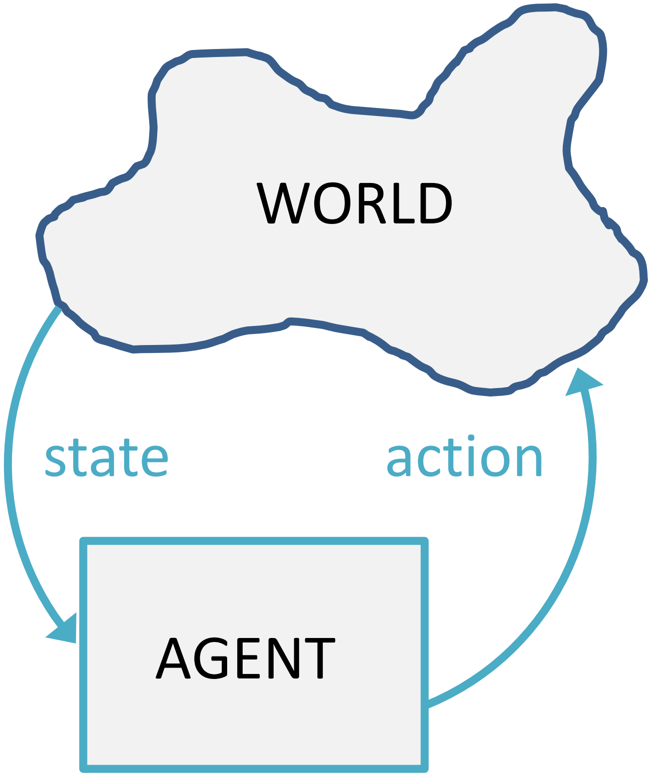
Markov Decision Process

Fully Observable MDP

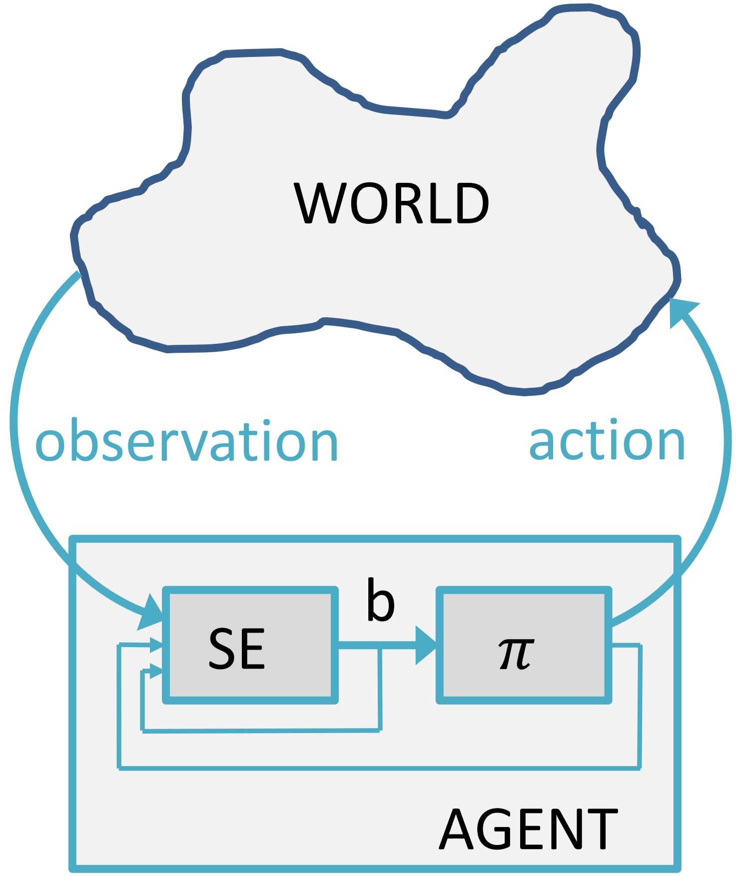


KUBA-MS, Hajdin et al.

Fully Observable MDP



Partially Observable MDP

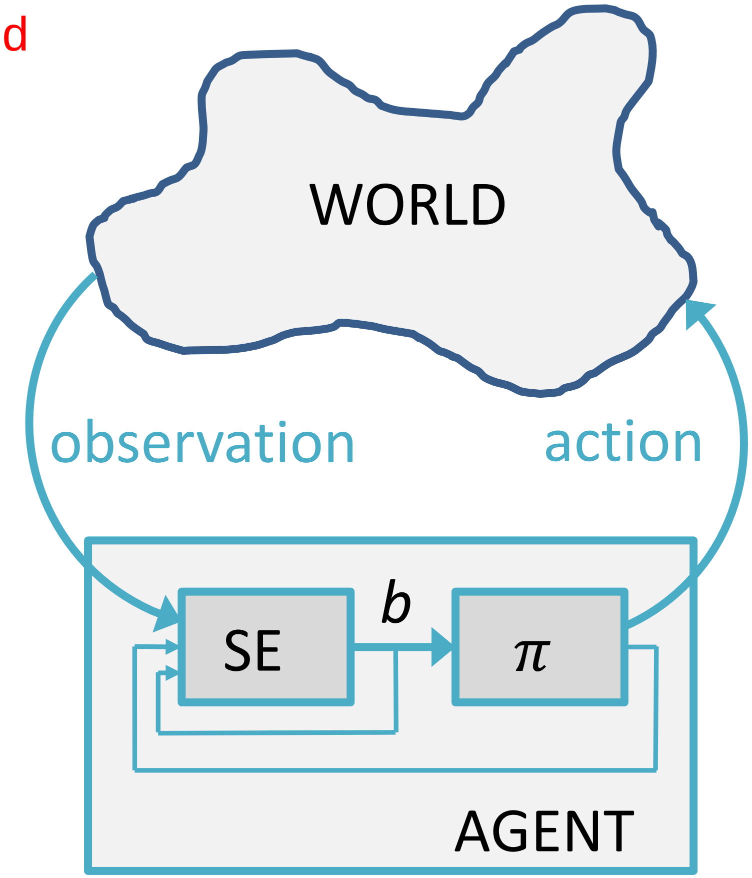


(Kaelbling, 1999)

Markov Decision Process

Partially Observable MDP ⟶ state not fully observable

- Decisions depend on current state and history
- Initial state is uncertain
- Actions are uncertain
- World is known
- Observations are uncertain
- Sequential process: action →
- observation → action . . .

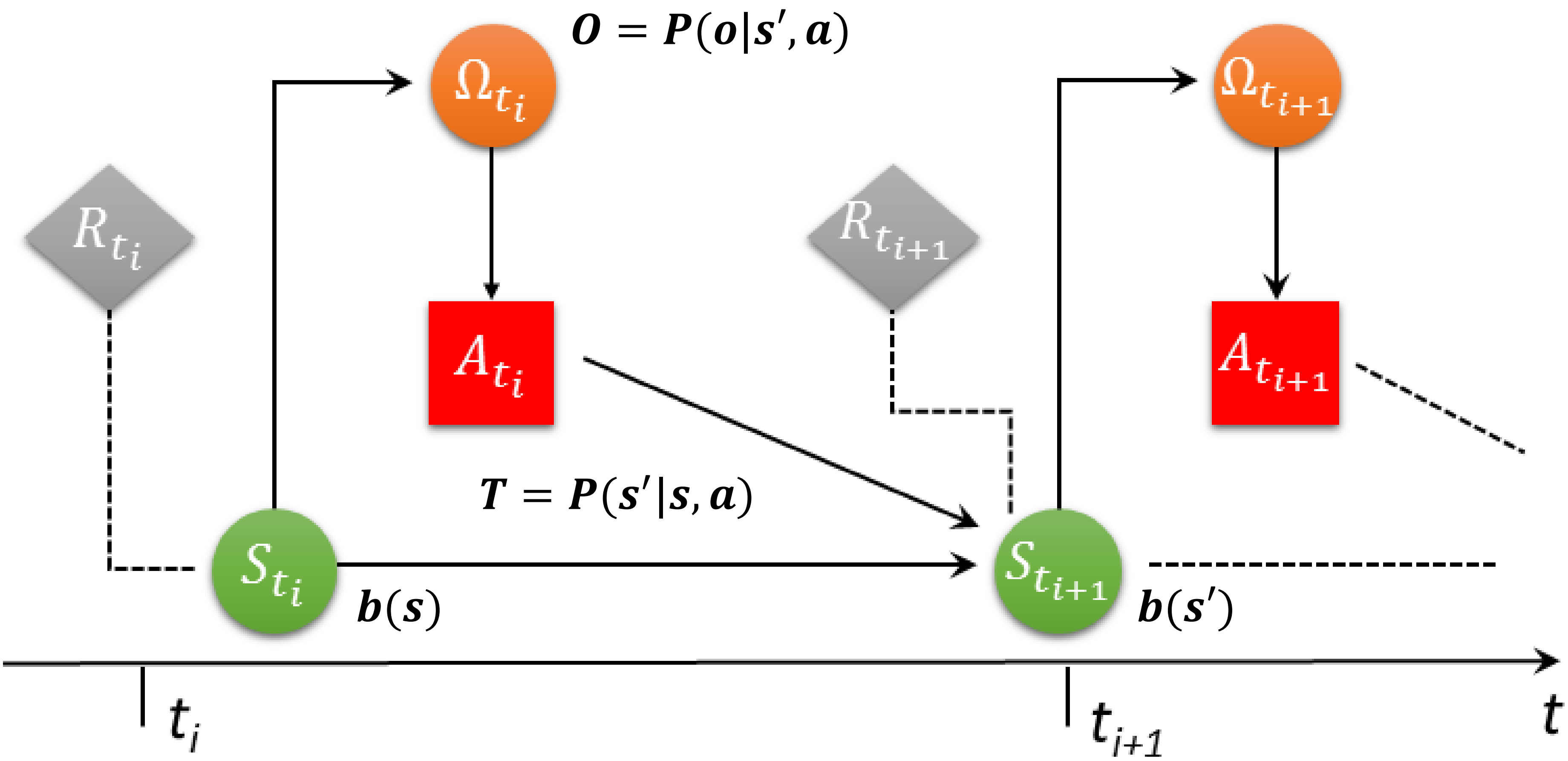


SE = State Estimator
b = belief state
π = policy

(Smallwood and Sondik, 1973; Sondik, 1978)

Markov Decision Process

Sequential decision process with alternating actions and inspections:



The POMDP Framework

A **POMDP** framework consists of the **tuple** $\{S, A, T, \Omega, O, R\}$

S	is the set of system states
A	is the set of actions
$T: S \times A \rightarrow \Pi(S)$	is the transition model describing $p(s'\|s,a)$
Ω	is the set of discrete observations
$O: S \times A \rightarrow \Pi(\Omega)$	is the observation model describing $p(o\|s,a)$
R	is the reward function $r_a(s) \in \mathbb{R}$

The updating of a given belief state may be obtained, using Bayes' rule is (continuous states):

$$b^{a,o}(s') = \frac{p(o|s',a)}{p(o|b,a)} \int_S p\,(s'|s,a) b(s \Big)$$

Back Propagation

The **total reward** over the entire lifetime of the agent ($t = 1, \dots, T$) is:

$$Q_{tot} = Q_{terminal} + \sum_{t=1}^{T} dQ_t \quad (2)$$

$$dQ_t = \int_s r(s,a)\, b(s) \quad (3)$$

Planning aims to maximize the expected future rewards:

$$V_n(b) = \max_a Q_n(b,a) \quad (4)$$

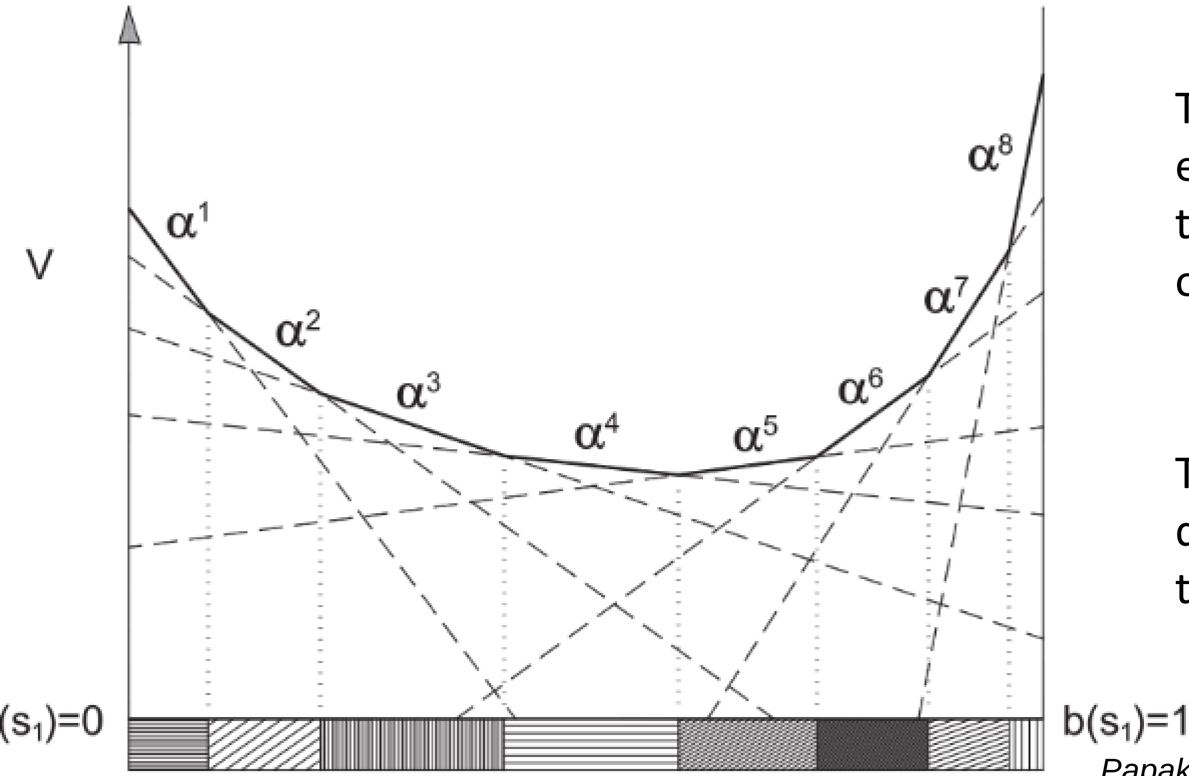
$$Q_n(b,a) = \int_S r_a(s)\, b(s) + \gamma \sum_o p(o|b,a)\; V_{n-1}\, b^{a,o} \quad (5)$$

where n describes the number of decision time steps left till the end of the agent's lifetime, a.k.a. **horizon**. It is then $n + t = T$.
($V_0 = Q_{\text{terminal}}$)

Discrete Equivalent

The optimal future reward is represented by a set of $\alpha-$ vectors:

$$V^*(\mathbf{b}) = \max_{\{\boldsymbol{\alpha}^i\}_i^*} \sum_{s \in S} b(s)\, \alpha^i(s),$$



Papakonstantinou & Shinozuka, 2014

The belief space is a simplex, and each vector defines a region over the simplex which represents a set of belief states.

The value function, is generally defined as the upper surface of these vectors.

POMDP Solvers

The solution of this recursive problem aims at establishing the optimal policy, i.e., planning of sequence of inspections and actions to be performed (policy).

Discrete POMDPs

- Approximations based on MDP and Q-functions
- Grid-based approximations
- Point-based value iteration methods

(Pineau, Gordon, & Thrun, 2003; Vlassis & Spaan 2004).

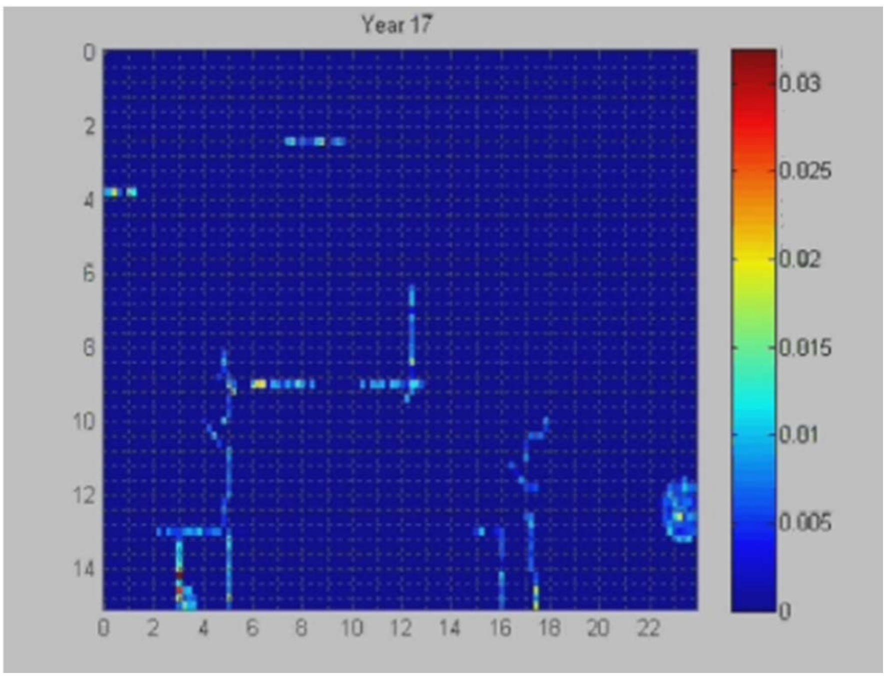
Continuous POMDPs

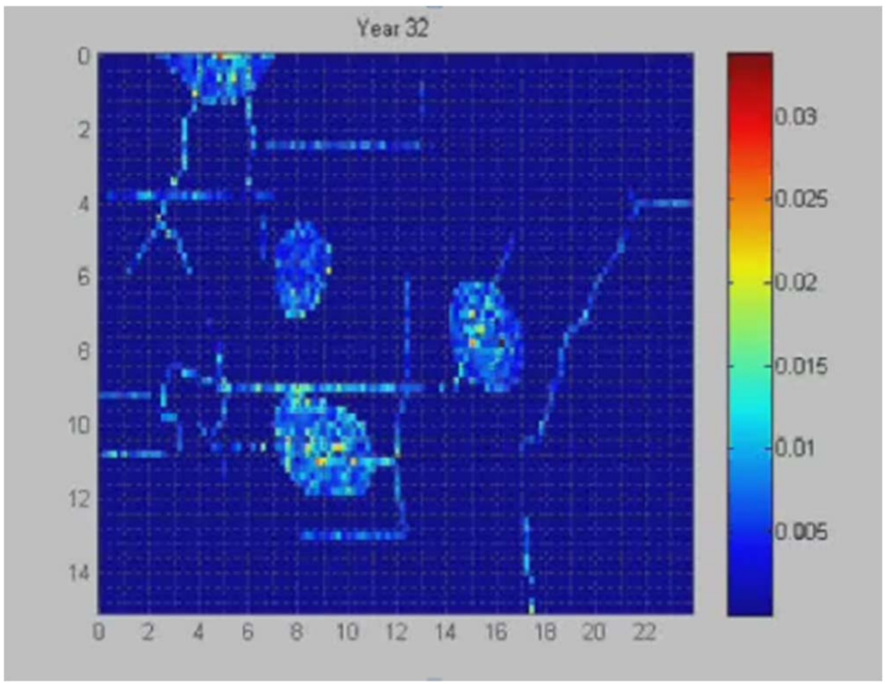
- Policy search methods (Aberdeen & Baxter, 2002; Baxter & Bartlett, 2001; Ng & Jordan, 2000; Williams & Singh, 1999).
- Approximate, i.e., grid- (Zhou & Hansen 2001) and point-based (Porta et al., 2005), value iteration algorithms may also be extended to fit the continuous space.

Beyond consideration of linear transition models, Schöbi & Chatzi (2016) extend the solution of continuous state POMDPs to nonlinear action models via use of Gaussian Mixtures and the Unscented Transform.

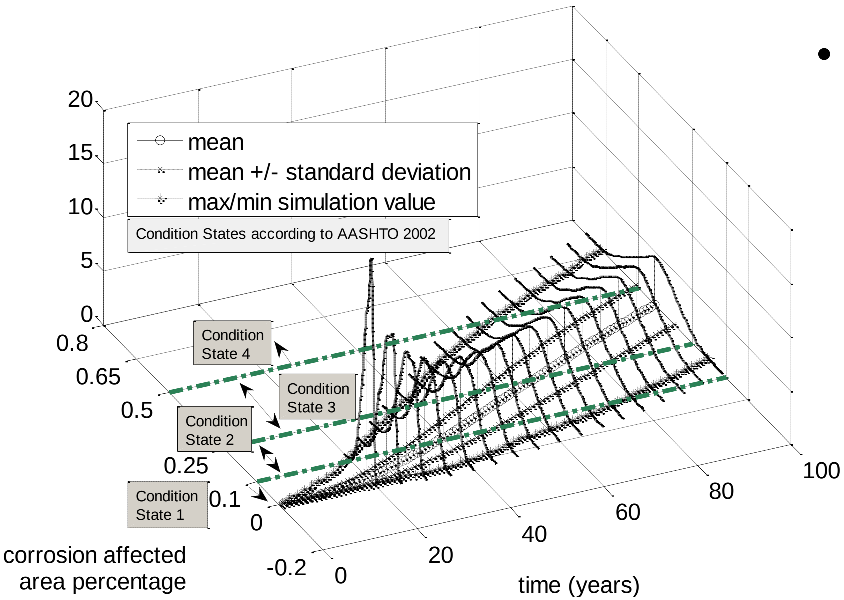
How to use POMDPs in Infrastructure Management

The examples involves an infinite horizon & non-stationary case, simulating the problem of corrosion





How to use POMDPs in Infrastructure Management



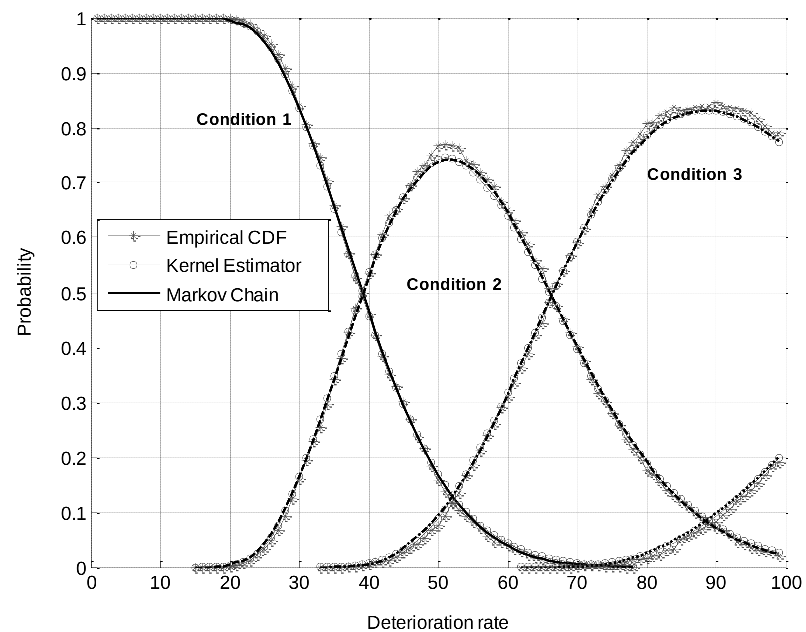
- 4 conditions:

≤10%, >10% ≤25%, >25% ≤50%, >50%

$$p(s'|s) = \frac{\text{number of transitions from } s \text{ to } s'}{\text{number of data in state } s}$$

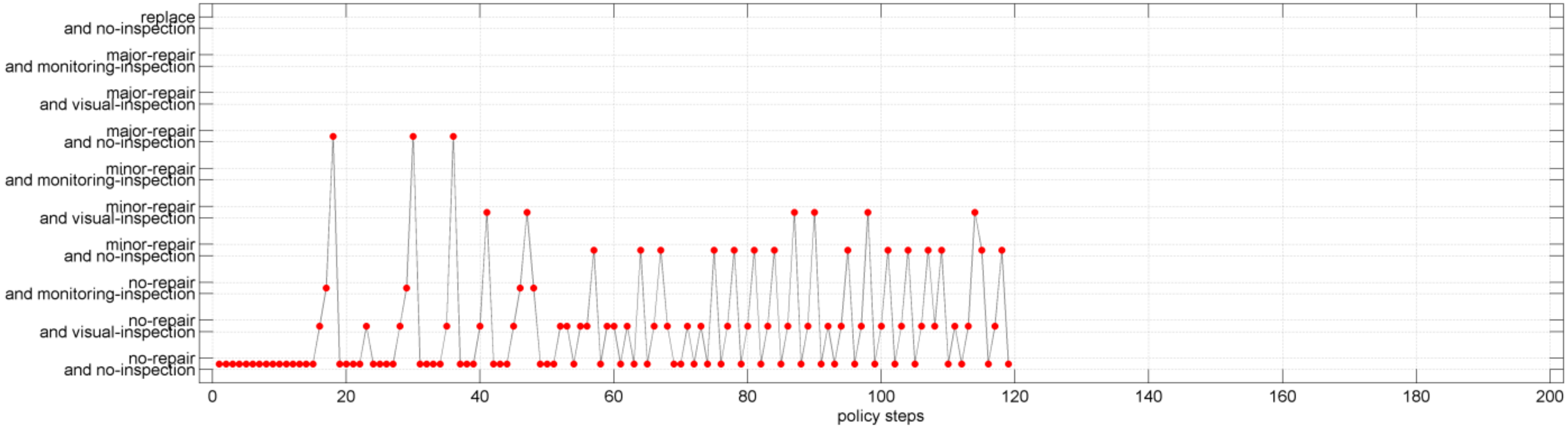
$$\mathbf{P} = \begin{bmatrix} \ddots & \vdots & \iddots \\ \cdots & \mathbf{P}(a \to b) & \cdots \\ \iddots & \vdots & \ddots \end{bmatrix}$$

$$\mathbf{P}(a \to b) = \begin{bmatrix} \mathbf{0}^{aa}_{(4x4)} & \begin{matrix} p_{11}^{ab} & p_{12}^{ab} & 0 & 0 \\ 0 & p_{22}^{ab} & p_{23}^{ab} & 0 \\ 0 & 0 & p_{33}^{ab} & p_{34}^{ab} \\ 0 & 0 & 0 & p_{44}^{ab} = 1 \end{matrix} \\ \mathbf{0}^{ba}_{(4x4)} & \mathbf{0}^{bb}_{(4x4)} \end{bmatrix}$$

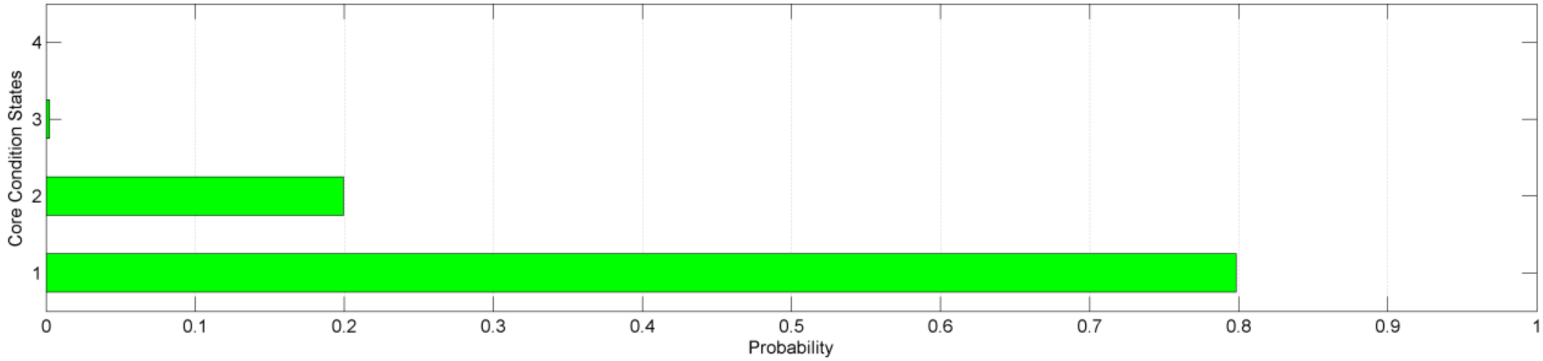


Papakonstantinou & Shinozuka, IALCCE 2016

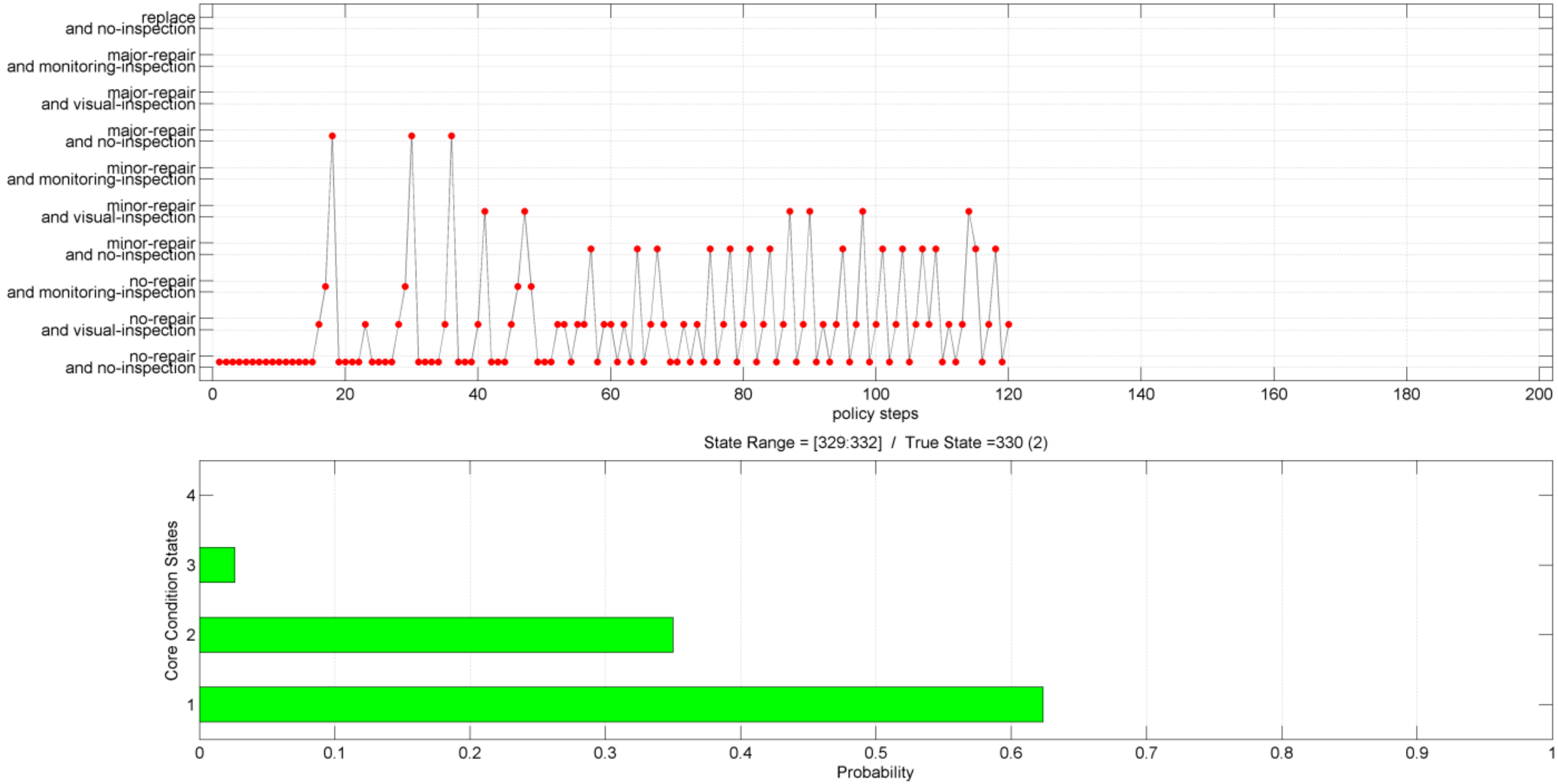
How to use POMDPs in Infrastructure Management



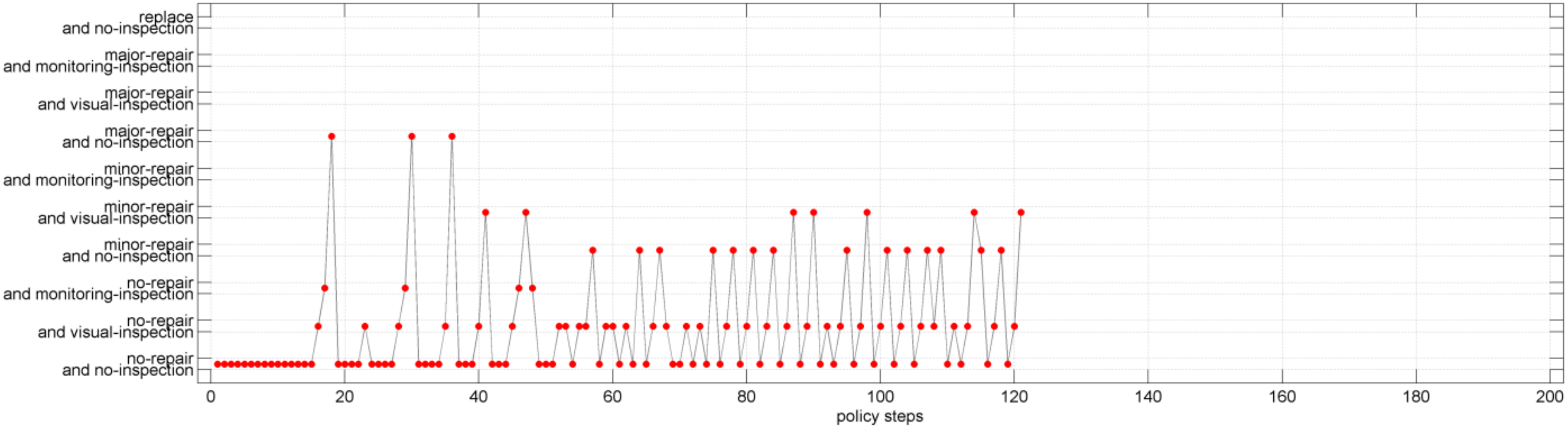
State Range = [329:332] / True State =330 (2)

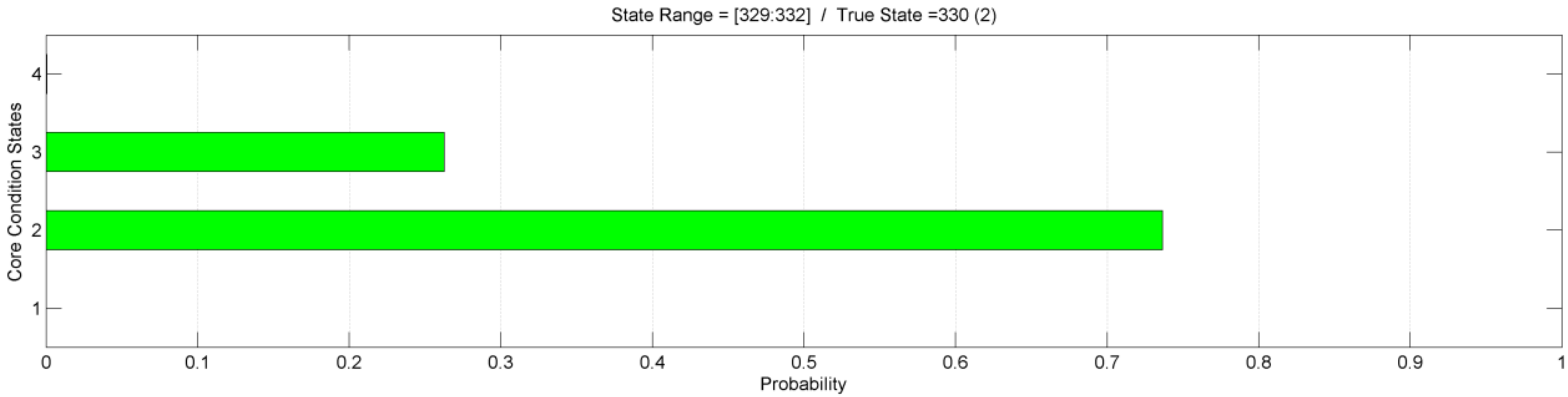


How to use POMDPs in Infrastructure Management

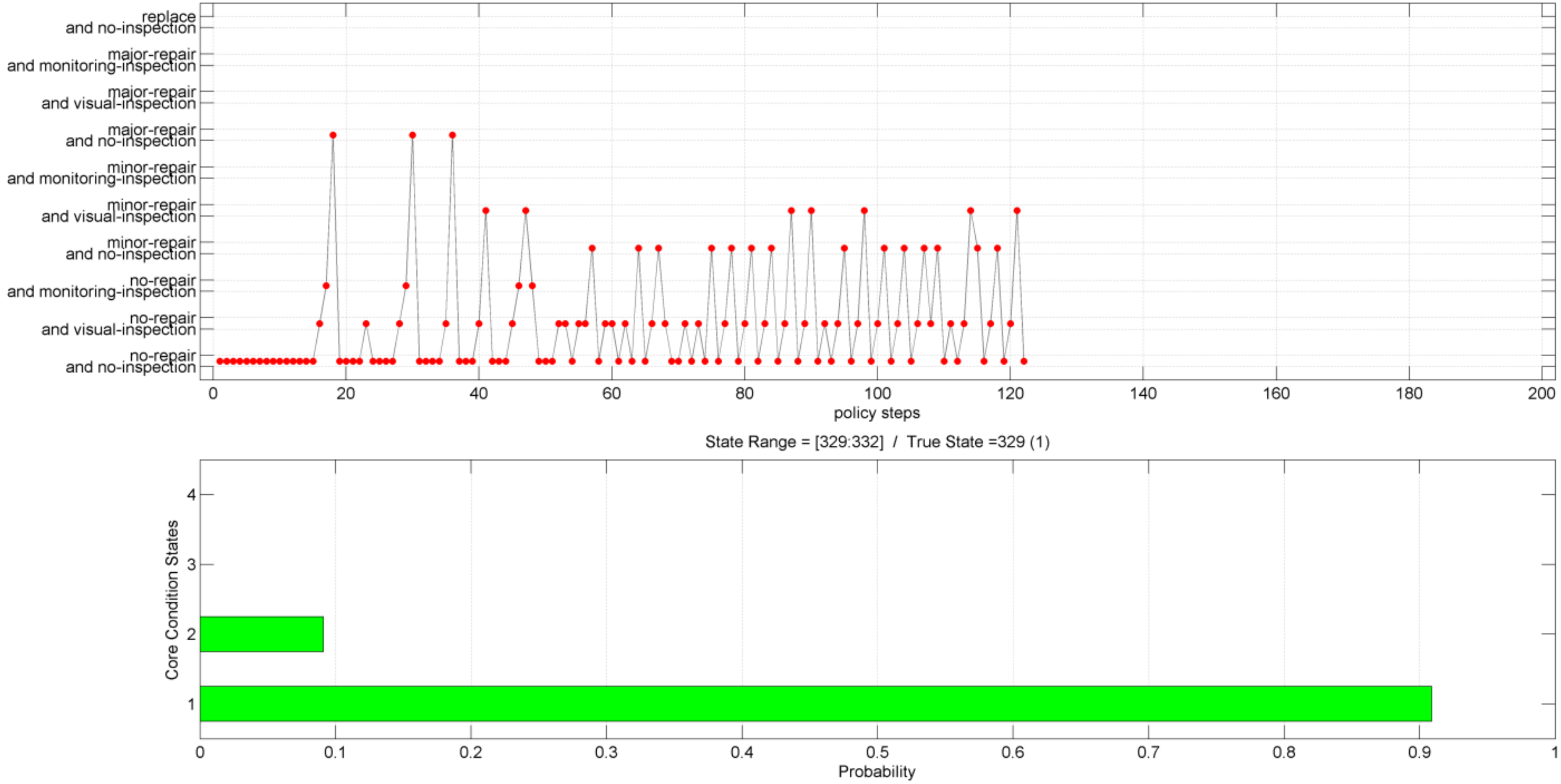


How to use POMDPs in Infrastructure Management

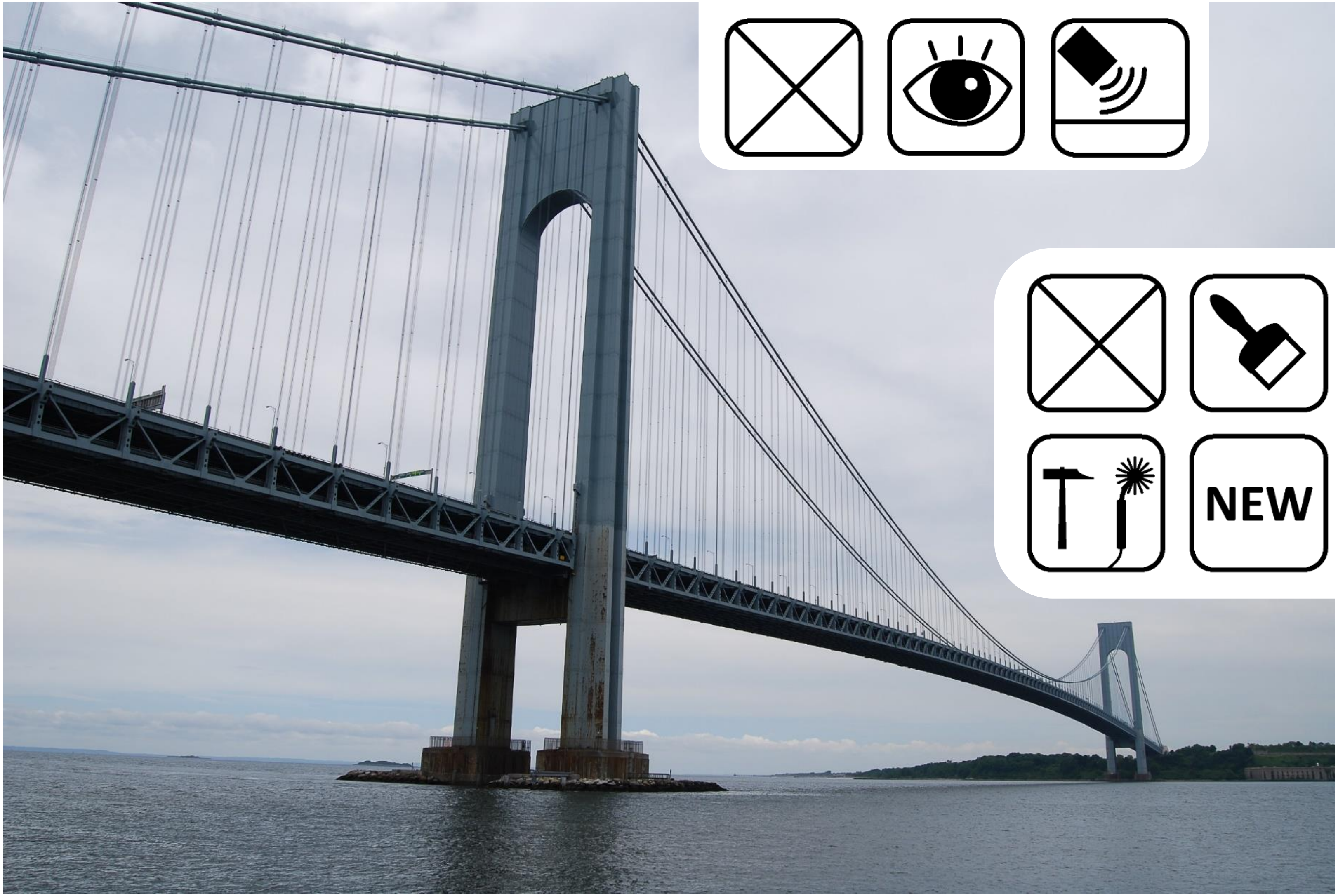




How to use POMDPs in Infrastructure Management



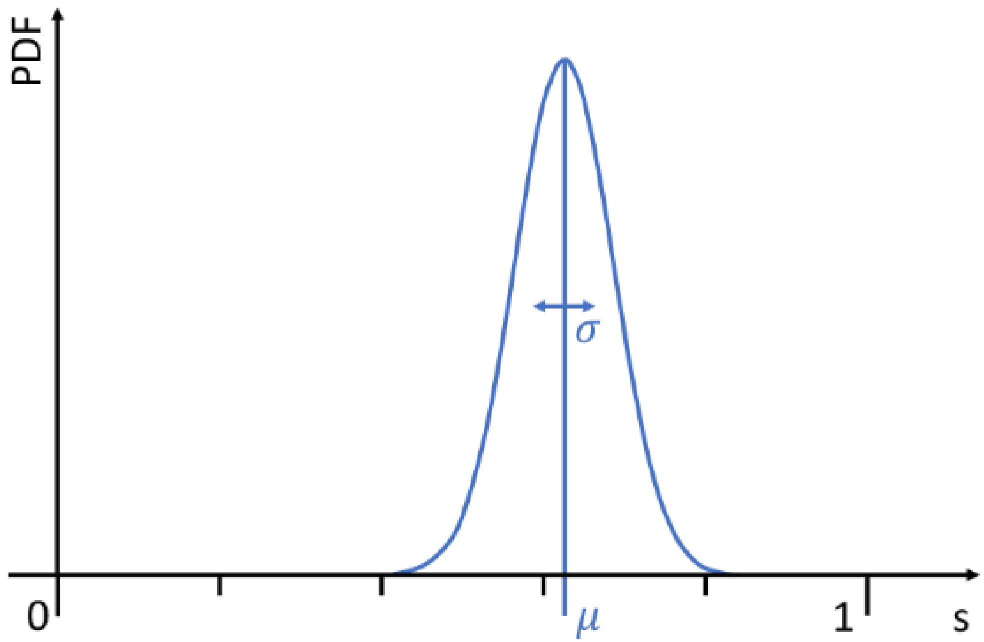
How to use POMDPs in Infrastructure Management



Example 2: Policy Maps

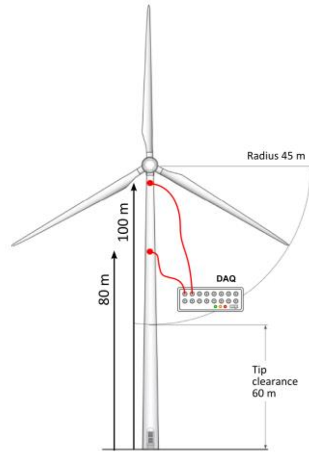
The **system state** S is 1-dimensional with a range $0 \leq s \leq 1$. (0 for failure, 1 for optimal condition of the bridge), e.g. damage index through vibrational data (natural frequencies)

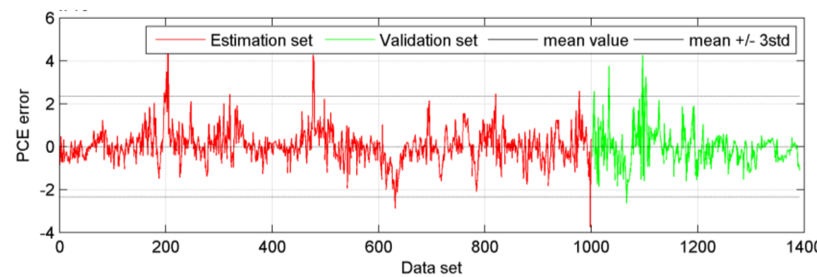
Cost for failure of the structure $C_{\text{failure}} = 1000$.



How to extract such a Condition Index?

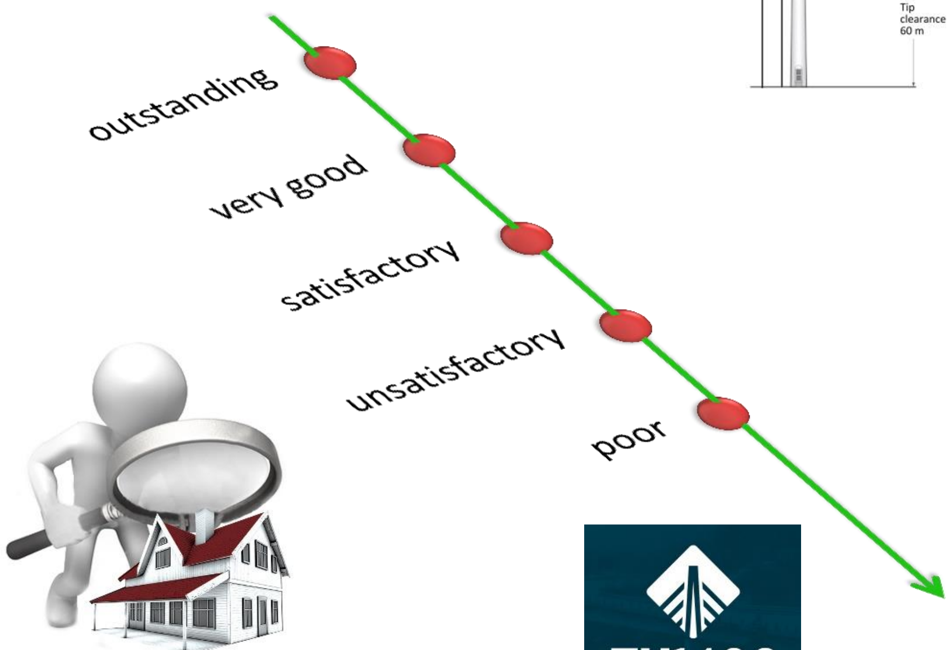
Permanent Monitoring



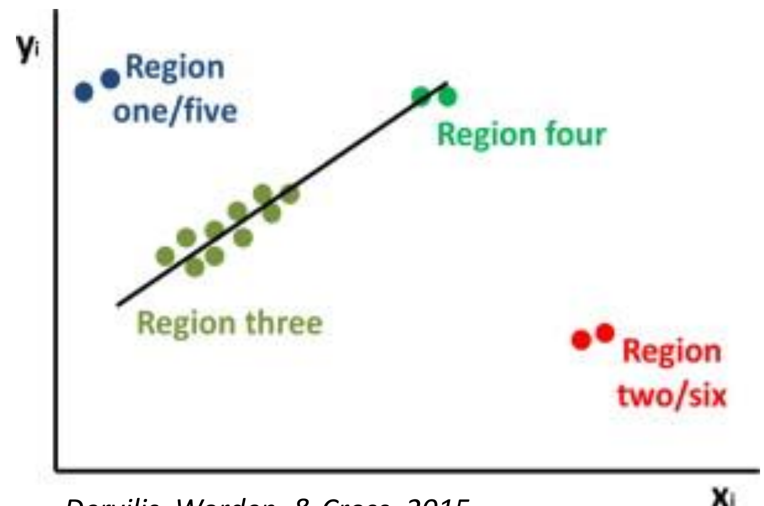


Spiridonakos & Chatzi, 2015

Inspection or NDE



TU1406 COST ACTION



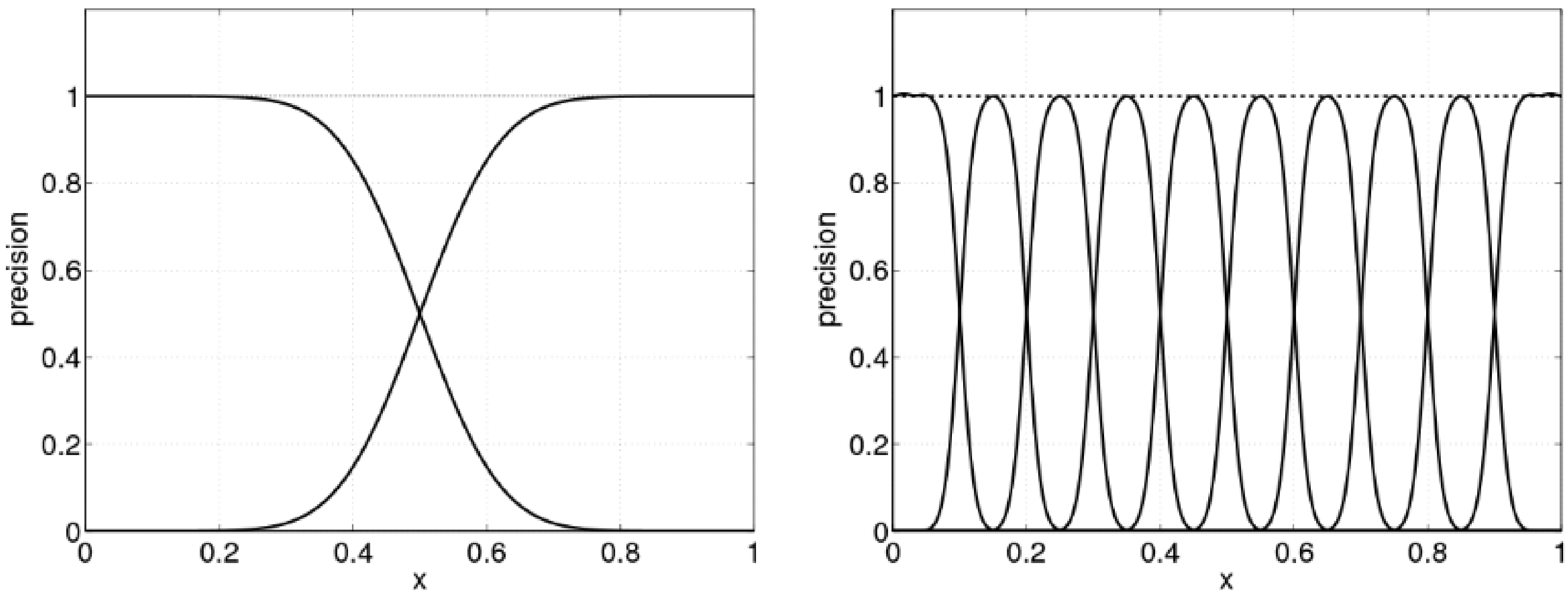
Dervilis, Worden, & Cross, 2015

TU1402 Factsheet

Example 2: Policy Maps

Observations

Three possible inspection methods: "doing nothing" ($C_{\text{doing nothing}} = 0$), "visual inspection" ($C_{\text{visual}} = 1$), and "ND testing" ($C_{\text{ND}} = 5$)



(a) Visual inspection (b) ND testing

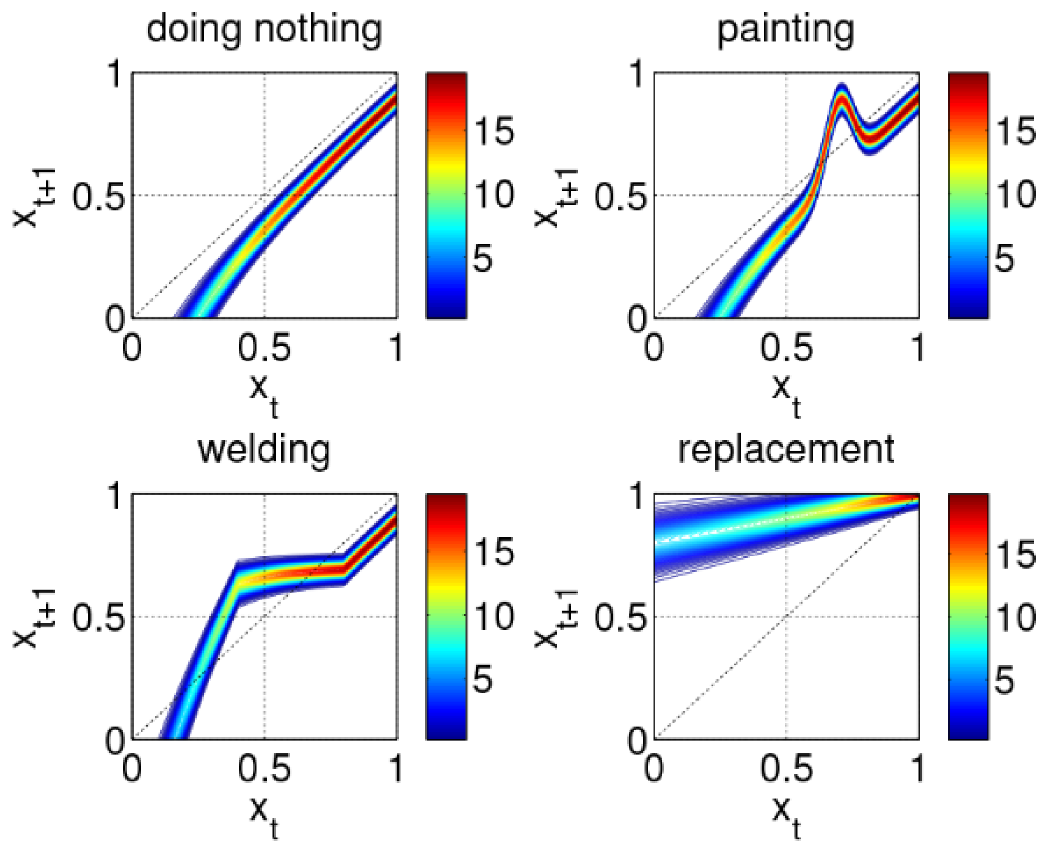
Example 2: Policy Maps

Actions

The action models are defined as the sum of a deterministic component and a stochastic component:

$$s' = f(s) = \mu_f(s) + \epsilon_f(s)$$

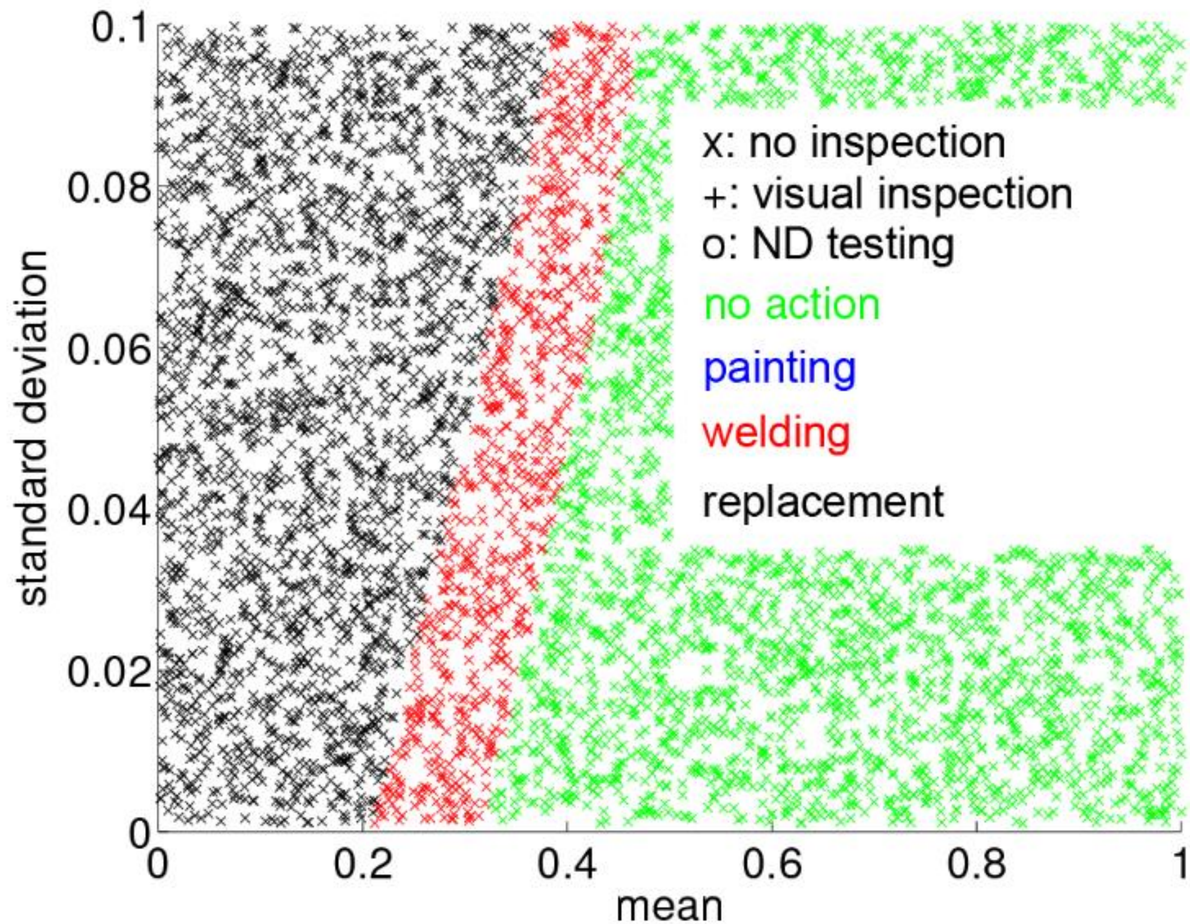
$$C_{\text{doing nothing}} = 0$$
$$C_{\text{painting}} = 10$$
$$C_{\text{welding}} = 50$$
$$C_{\text{replacement}} = 100$$



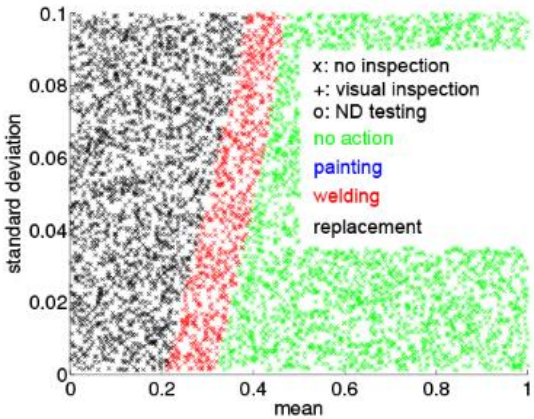
Deliver Standardized Maps to be used by decision/makers

This is a Continuous state Finite Horizon Problem:

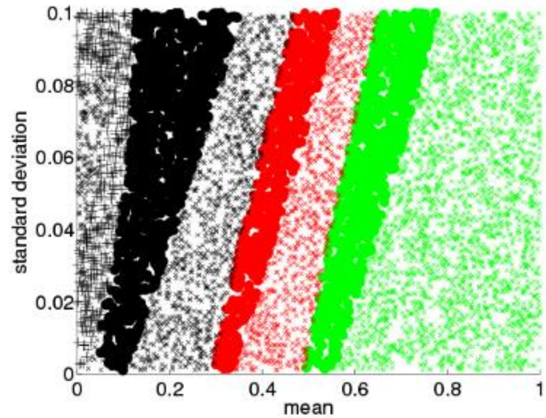
1 decision step left:



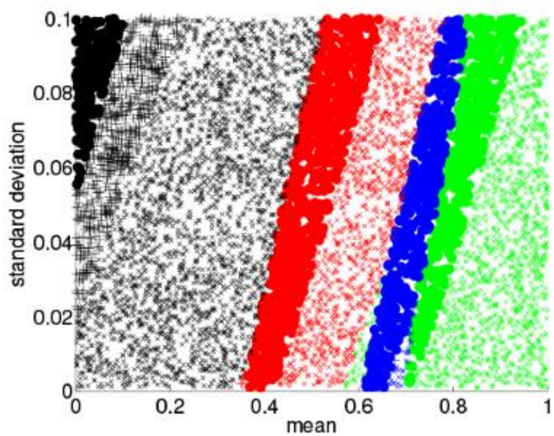
Example Application



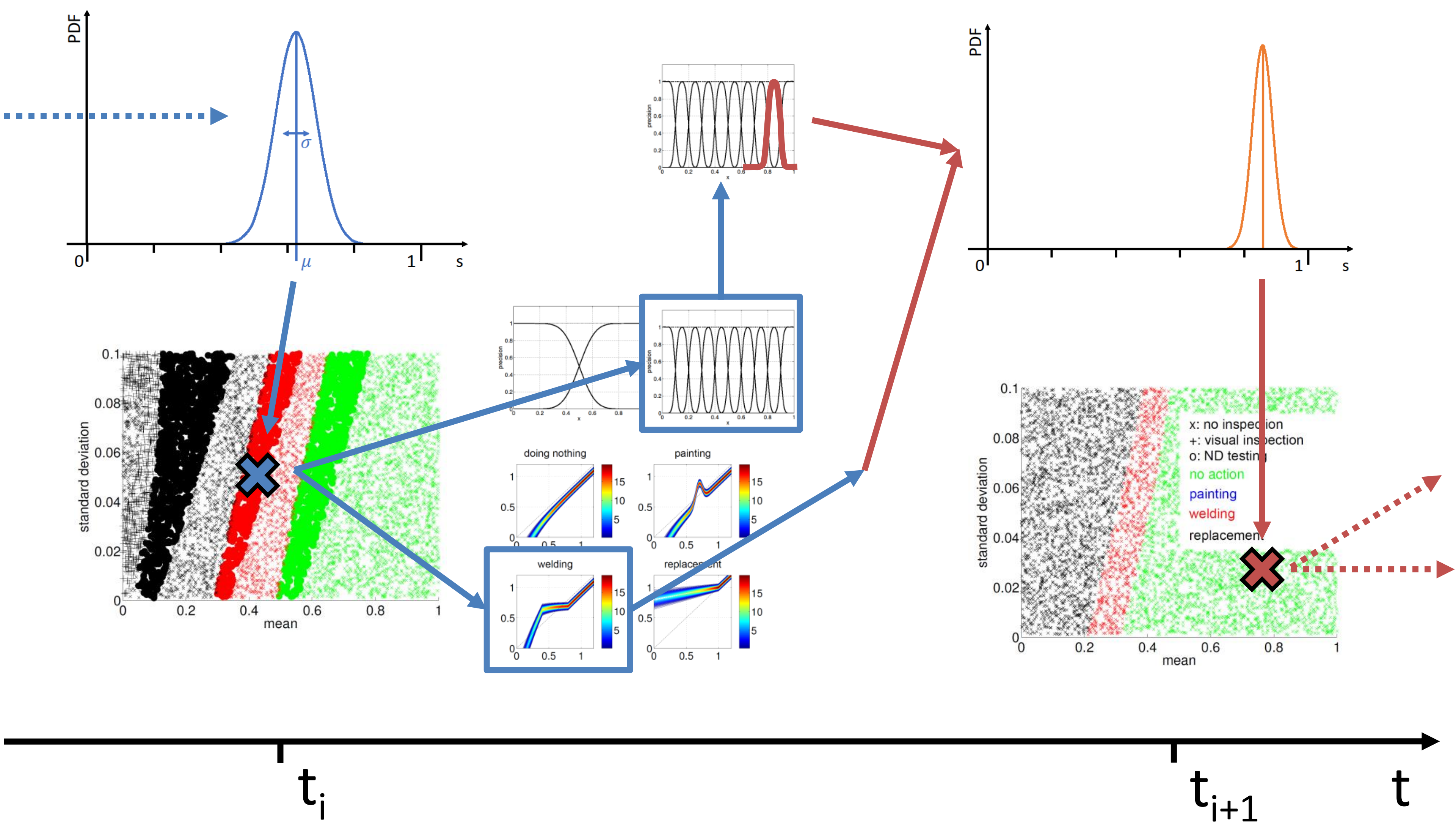
$n = 1$



$n = 2$



$n = 3$


PDF
σ
0
μ
1
s
standard deviation
mean
doing nothing
painting
welding
replacement
PDF
0
1
s
x: no inspection
+: visual inspection
o: ND testing
no action
painting
welding
replacement
standard deviation
mean
t_i
t_{i+1}
t

VoI in Infrastructure Management

As a tool for pre-posterior analysis

The *net VoI* is defined as the difference in maximum expected utility of the decision to install the monitoring system and the maximum expected utility without the monitoring system: $net\ VoI = \mathrm{E}[U|d_{opt,SHM}, SHM] - \mathrm{E}[U|d_{opt,no\ SHM}, No\ SHM]$

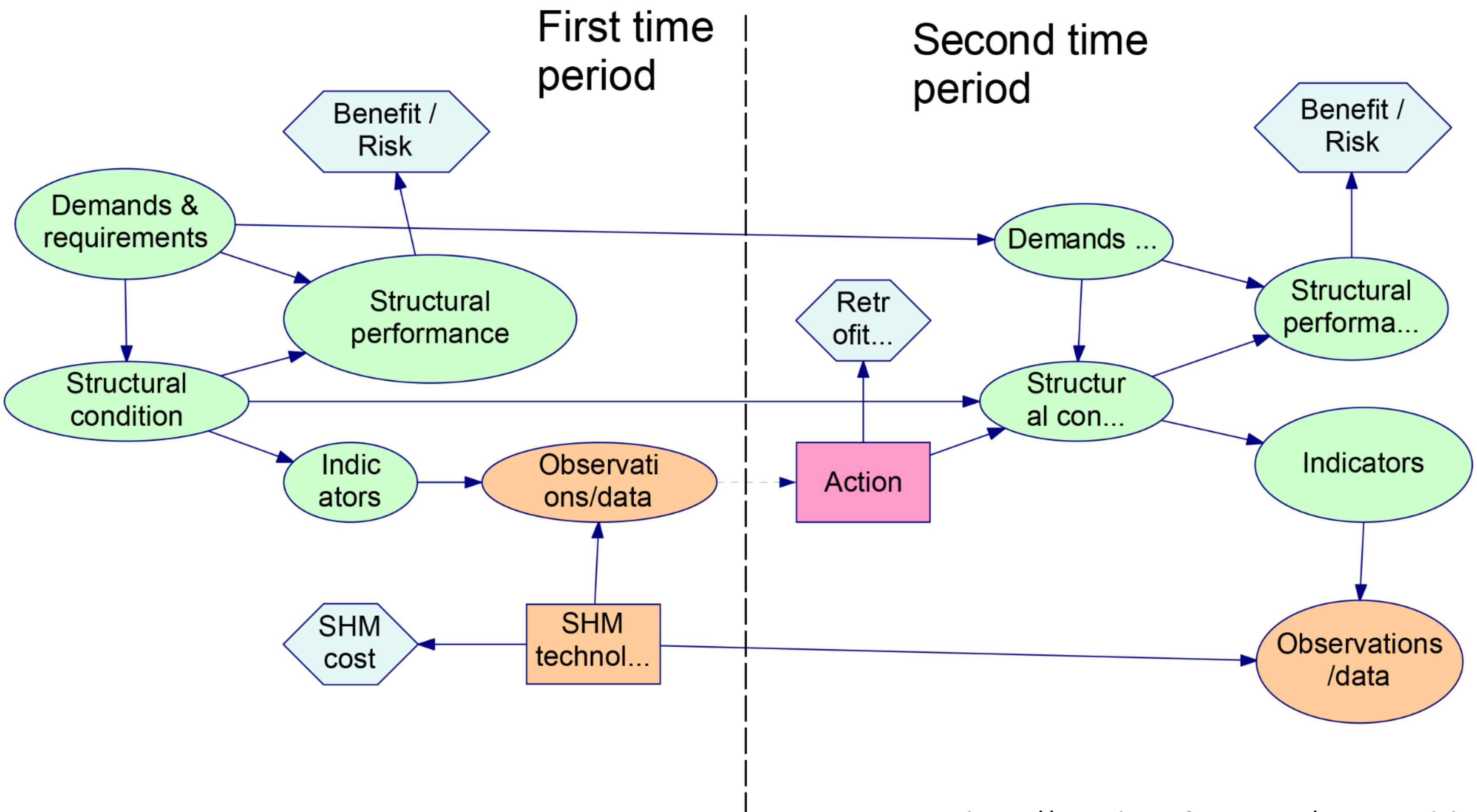
(Straub et al. ICOSSAR 2017)

Use the **POMDP optimal policy trajectories** in order to generate the **conditional probabilities** of the effects of actions, and deterioration processes on the system's condition, with and without inclusion of **inspection/monitoring information**.

Couple this Information with Bayesian Analysis Tools for Quantifying the Value of Information of Monitoring Systems

coming up!

VoI in Infrastructure Management



https://www.bayesfusion.com/genie-modeler

We welcome questions/comments/collaboration:
chatzi@ibk.baug.ethz.ch

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Monitoring during life cycle of bridges to establish performance indicators

Peter Haardt – Federal Highway Research Institute (BASt), Germany
Ralph Holst – Federal Highway Research Institute (BASt), Germany

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

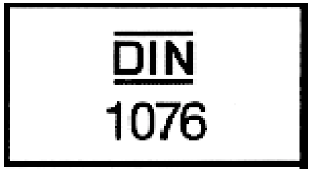
AGENDA

- Background,
- Bridge Maintenance Regulations for German Highways,
- Key Performance Indicators,
- Monitoring for quality assurance and function checks,
- Monitoring of action (impact),
- Monitoring of the condition,
- Monitoring safety-related system elements,
- Monitoring as compensation procedure according to NRR,
- Conclusion

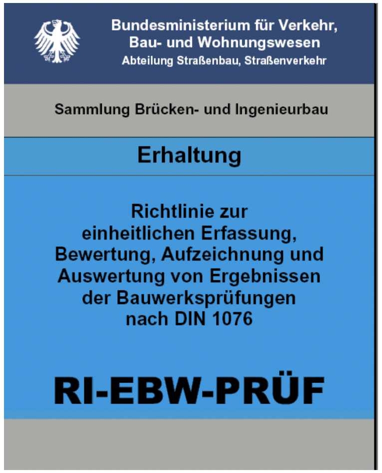
BACKGROUND

- Ageing infrastructure,
- Increase of traffic, especially heavy traffic
- Boundaries of Visual inspection
  - Limited to visible damages,
  - No/limited information from inside of structure,
  - Information to a certain point in time not continuously,
- Complex behaviour of structures (bridges, Tunnel, ...),
  - KPI necessary

BRIDGE MAINTENANCE REGULATION FOR GERMAN HIGHWAYS

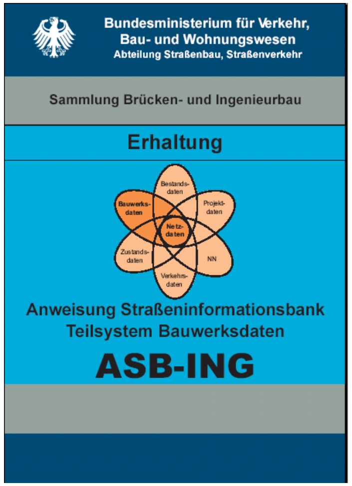


Engineering structures in connection with roads; inspection and test, 1999



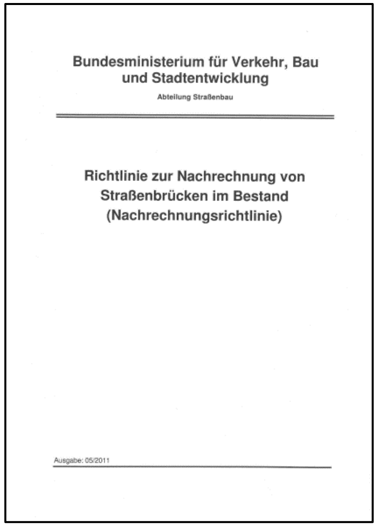
Guideline RI-EBW-PRÜF, 2013

Recording and assessment of damages, condition assessment



ASB Structures Inventory instructions, 2013

Extent and structure of data



Guideline for Recalculation (NRR), 2011

Pictures: BASt

BRIDGE MAINTENANCE REGULATION FOR GERMAN HIGHWAYS

Guideline RI-EBW-PRÜF

- Distribution in bridge components,
  - Substructure, superstructure, bearings, expansion joint, railings, pavement, ...,
- Damage assessment regarding
  - stability (S), traffic safety (V), durability (D),
- Condition index
  - total bridge
  - and components

based an S, V, D.

D = 1

S \ V	0	1	2	3	4
4	4,0	4,0	4,0	4,0	4,0
3	3,1	3,3	3,5	3,7	4,0
2	2,2	2,3	2,4	2,8	4,0
1	1,5	1,7	2,2	2,7	4,0
0	1,1	1,3	2,1	2,6	4,0

Picture: BASt

BRIDGE MAINTENANCE REGULATION FOR GERMAN HIGHWAYS

Guideline for Recalculation (NRR) (1)

- for existing bridges
  - Level 1: Calculation (based on “DIN –Fachbericht” or Eurocodes),
  - Level 2: Consideration of supplementary regulations,
  - Level 3: Consideration of measurement results,
  - Level 4: including (new) scientific methods,

Compensation measures: e.g. Monitoring


Bundesministerium für Verkehr, Bau und Stadtentwicklung
Abteilung Straßenbau

Richtlinie zur Nachrechnung von Straßenbrücken im Bestand (Nachrechnungsrichtlinie)

Ausgabe: 05/2011

Picture: BASt

KEY PERFORMANCE INDICATORS (KPI) (1)

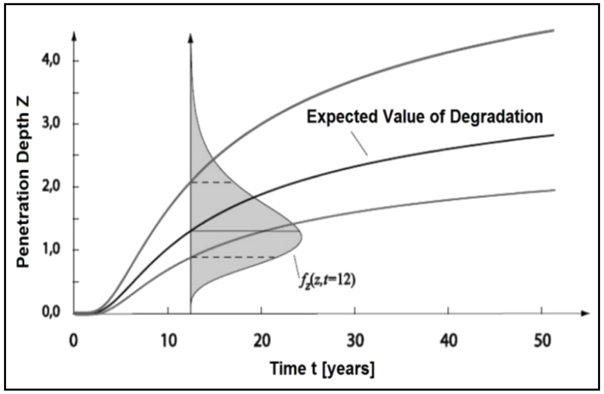
- Condition Index
  - For total bridge,
  - For bridge components (e.g. bearings, pavement, ...),
  - Load carrying capacity
- Evaluation criteria
  - Structural safety,
  - Traffic safety,
  - Durability,

KEY PERFORMANCE INDICATORS (KPI) (2)

- (measurable) Indicators to describe the KPI
  - Cracks (width, length),
  - Corrosion (reinforcement),
  - Deformation,
  - Strain,
  - Flaking,
  - Chloride depth,
  - ...

MONITORING FOR QUALITY ASSURANCE AND FUNCTION CHECKS

- Calculation of Service life; adaption of service life,
- Permanent monitoring of critical areas,
- For inaccessible areas,
- Survey of bridge reaction,
- Assessment to evaluate the structural measures.



Picture: BASt

KPI: Condition index (bridge/components); load carrying capacity

MONITORING OF ACTION (IMPACT)

Picture: scanntronik

- Knowledge of traffic load reaction,
- Collected information
  - Load spectra, vehicle/axle loads, vibration coefficients, climate
  - Intensity and frequency; WIM,
  - Bridge and components (e.g. Expansion joints)
- Permanent measurement of strain => influence line

KPI: Deformation, strain, cracks (open/close)

MONITORING OF CONDITION

Picture: sensortec

- Deformation of the deck,
- Moisture penetration,
- Crack and crack width,
- Flaking with exposed reinforcement and reduced cross-section,
- Depth of chloride ingress,
- moisture penetration.

Picture: selfsan consult

KPI: Deformation, cracks (width, length), flaking, chloride depth, change in dimensions (e.g. cross section)

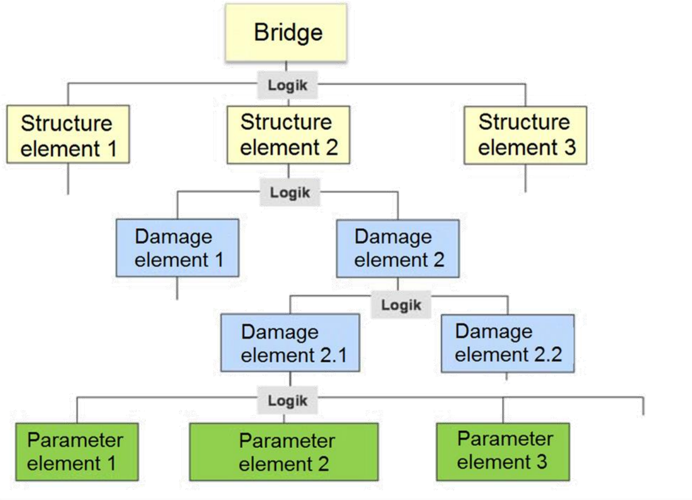
MONITORING OF SAFETY-RELATED SYSTEME ELEMENTS (1)

- Early warning function,
- Limit states (thresholds),
- Non destructive online analysis,
- Two strategies
  - Predictive strategy (adaptive model)
  - Threshold monitoring (continuous monitoring)

KPI: Deformation, cracks (width, length), flaking, chloride depth, change in dimensions (e.g. cross section)

MONITORING OF SAFETY-RELATED SYSTEME ELEMENTS (2)

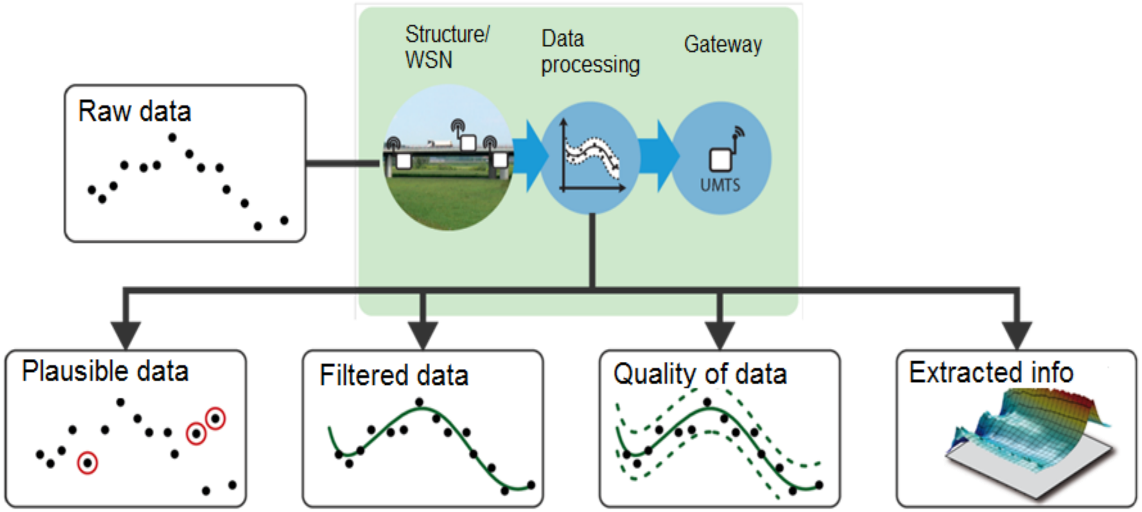
- Threshold values
  - Connection between zero state/target value and measurement,
  - Realistic simulation of damages/bond behaviour of reinforced concrete,
  - Plausibility checks, control/redundant measurement procedures,
  - Physical model,
    - Parameter based; FEM-Updating,



Picture: BASt

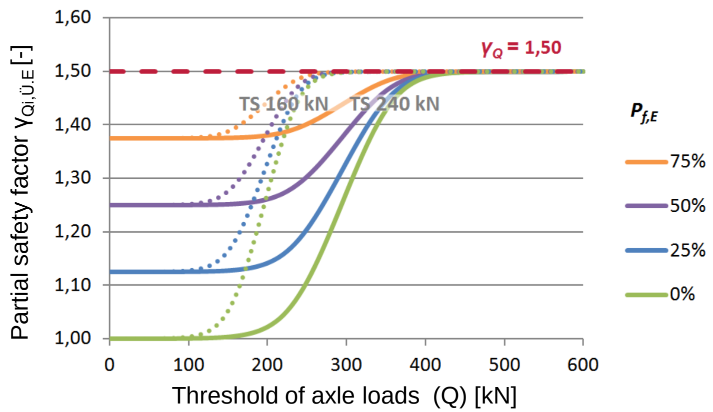
MONITORING OF SAFETY-RELATED SYSTEME ELEMENTS (3)

- Non physical model
    - Recognize patterns,
    - "Training" using learning techniques,
    - Data from sufficient long period of time.



Picture: BASt

MONITORING AS COMPENSATION PROCEDURE ACCORDING TO NRR



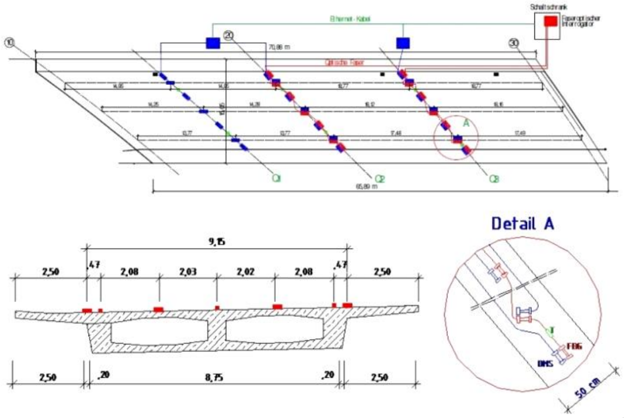
Picture: BASt

- Tool for assessing load carrying capacity,
- Compensation procedures,
  - Traffic compensation procedures,
  - Compensation monitoring procedures,
    - Target reliability; added safety through monitoring,
    - Must generate a sufficient safety gain (e.g. reduce partial safety factors,
    - Full probabilistic analysis.
    - Impact monitoring

KPI: limit state, decompression, crack width, shear strength, load carrying capacity, fatigue, stress crack corrosion

CONCLUSION

- More and better information about structures are needed,
- KPI can describe important impacts or reactions,
- Monitoring procedures can provide necessary additional information,
- Procedures have different advantages and boundary conditions,
- Monitoring must be cost-effective,
- Bring Monitoring Procedures into practice (not only a technical problem!)



Pictures: BASt (durabast-Testfield)

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Application of DIC to monitor reinforced concrete structures

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CSIC
CONSEJO SUPERIOR DE INVESTIGACIONES CIENTÍFICAS

GOBIERNO DE ESPAÑA
MINISTERIO DE ECONOMÍA Y COMPETITIVIDAD

02nd – 03rd March 2017
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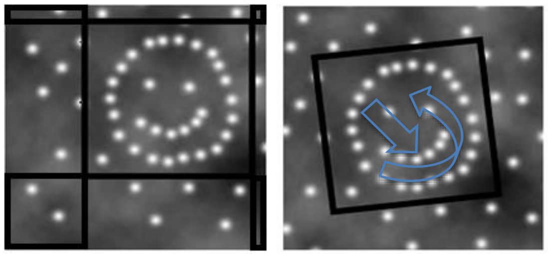
Faculty of Civil Engineering

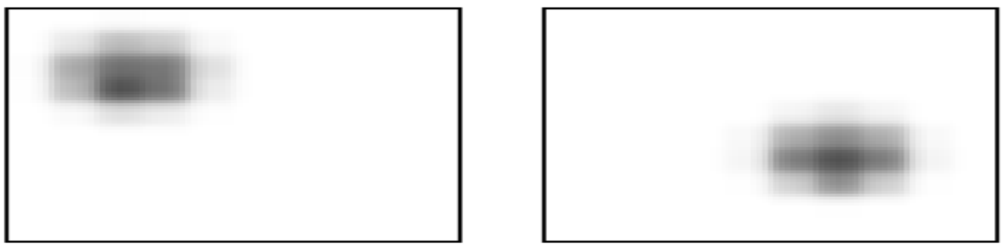
INTRODUCTION

DIC theory



Reference Image Displaced Image

Image Source: O. Ronneberger, M. Raffel, and J. Kompenhans, "Advanced Evaluation Algorithms for Standard and Dual Plane Particle Image Velocimetry."



Values of the pixels

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255	225	174	191	247	255	255	255	255	255
253	153	93	101	217	255	255	255	255	255
254	172	64	96	230	255	255	255	255	255
255	249	217	232	254	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255

Displacement

255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	255	255	255	255	255
255	255	255	255	255	242	224	242	255	255
255	255	255	255	247	159	123	158	247	255
255	255	255	255	242	107	64	106	241	255
255	255	255	255	254	196	120	195	254	255
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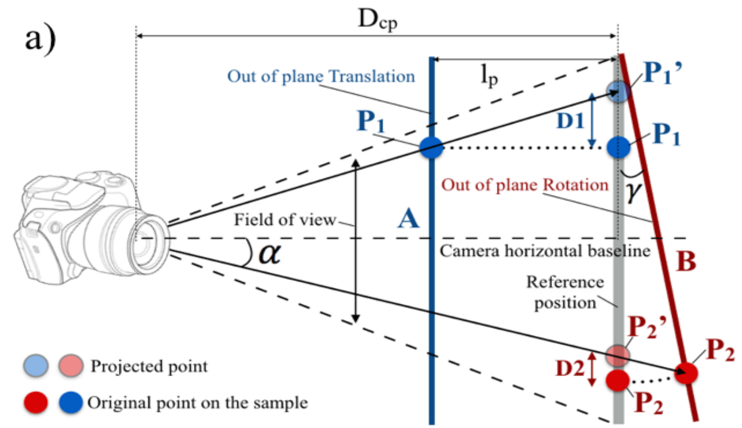
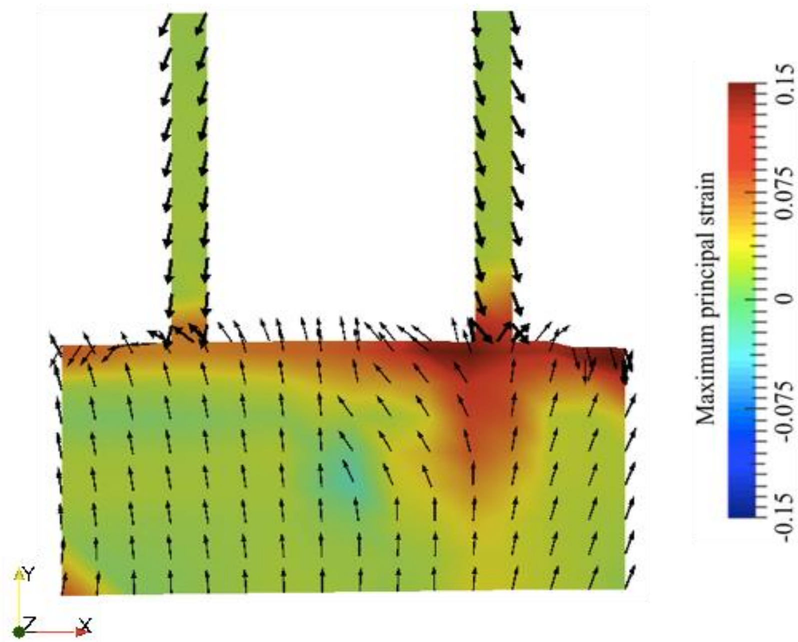
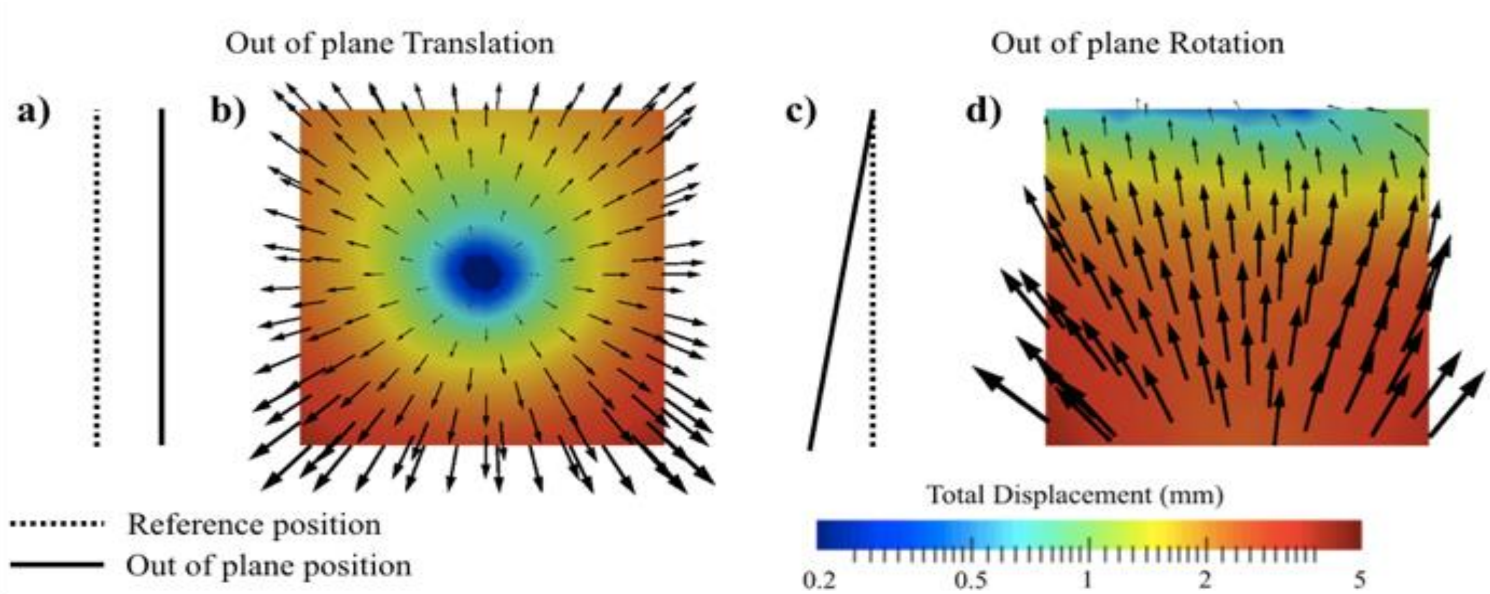
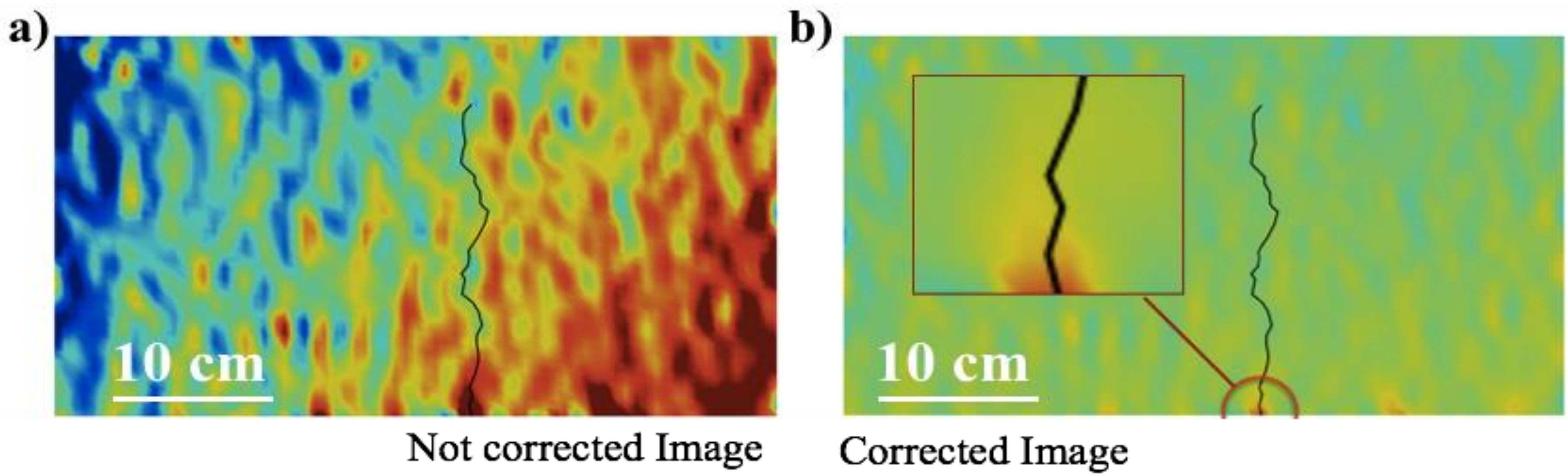
255	255	255	255	255	255	255	255	255	255



Prestressed

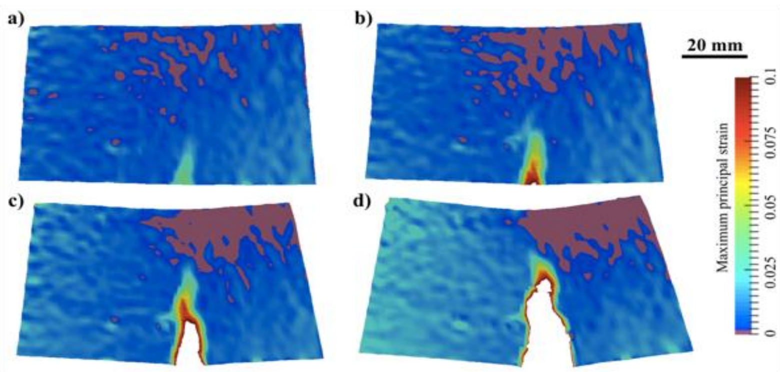


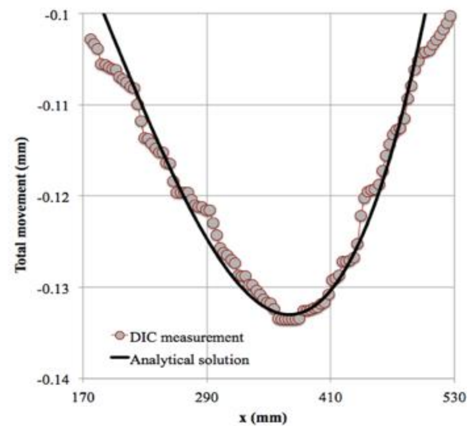

Out of plane Translation
Out of plane Rotation
a)
b)
c)
d)
Reference position
Out of plane position
Total Displacement (mm)
0.2
0.5
1
2
5


a)
b)
10 cm
10 cm
Not corrected Image
Corrected Image

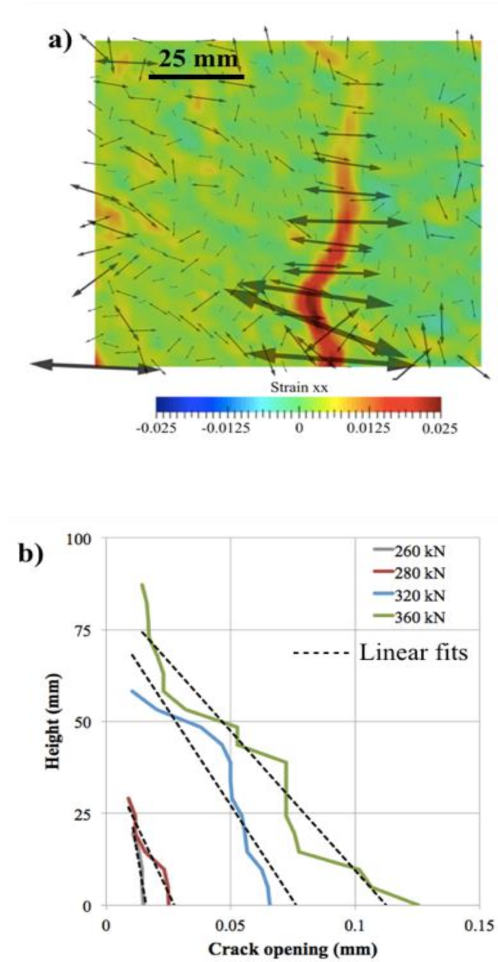
Applications

Crack localization

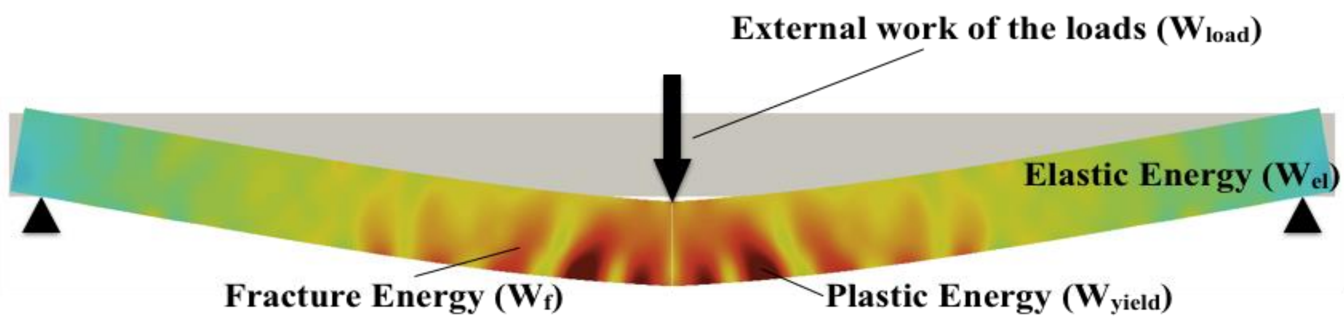




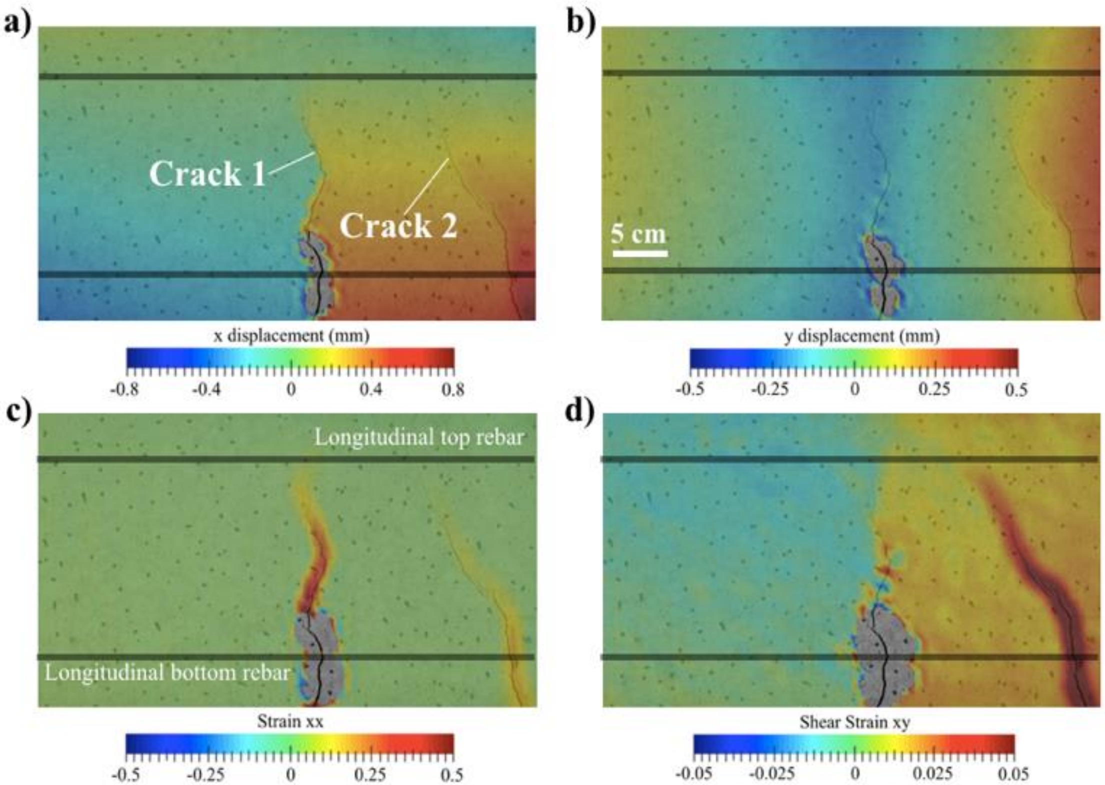
Measurement of the crack opening

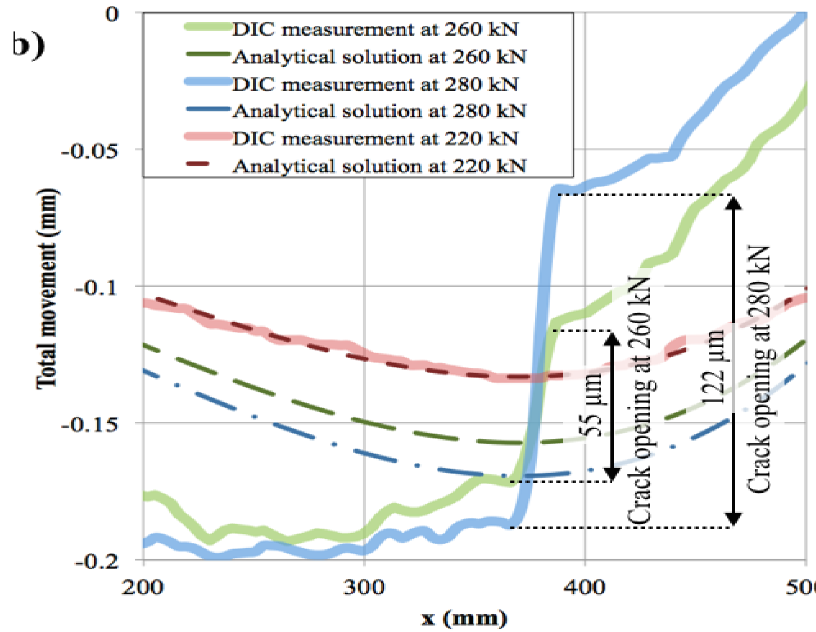


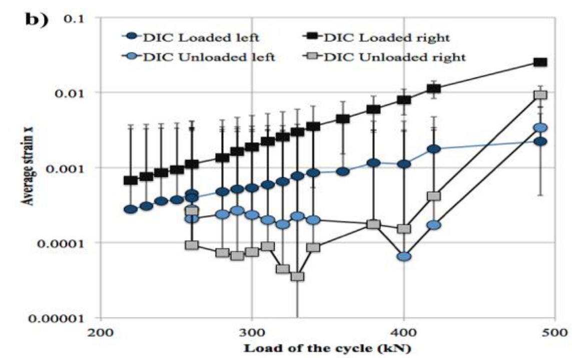
Energy dissipated by the structure



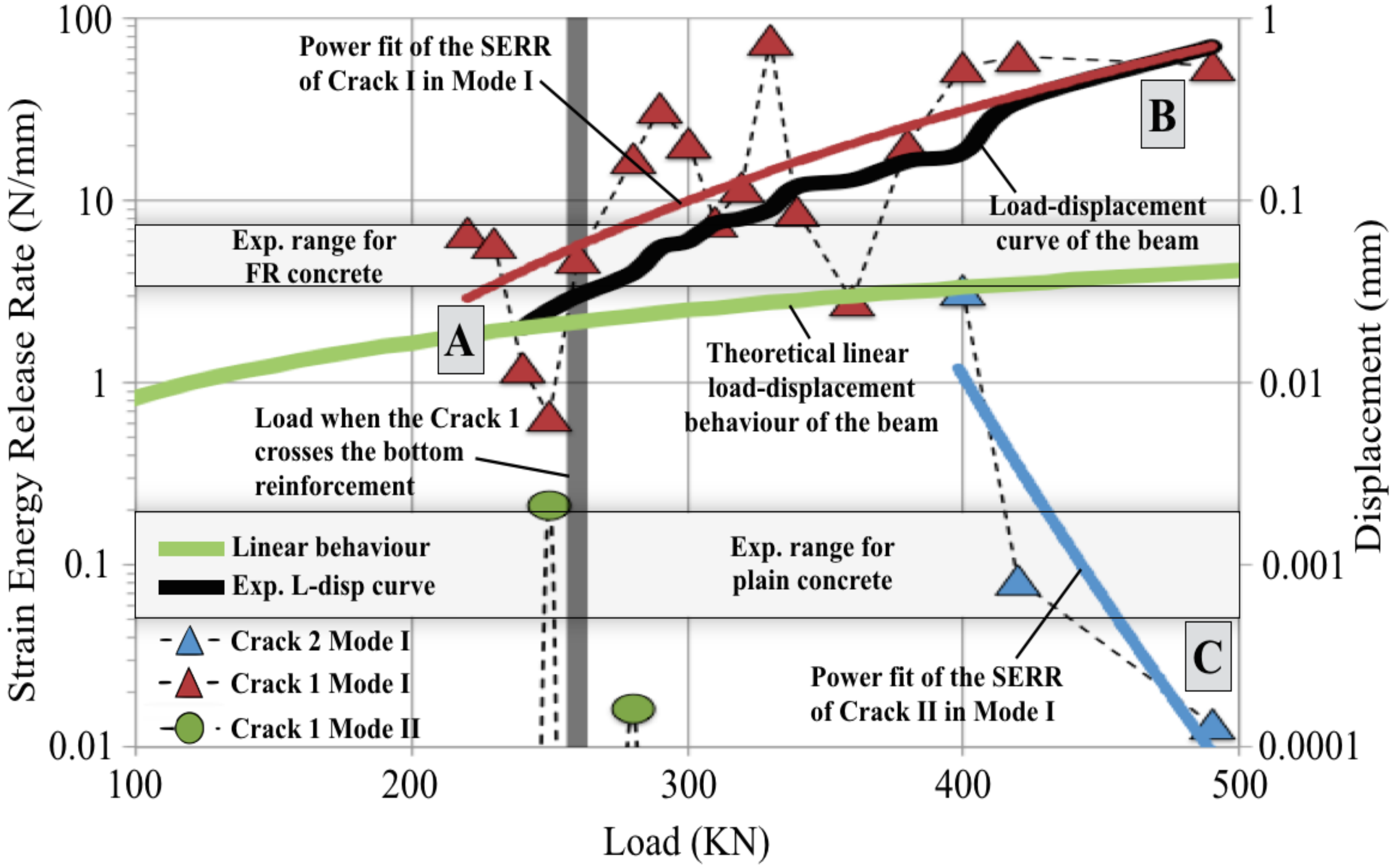
Crack evolution in a short prestressed beam (750x250x250mm)

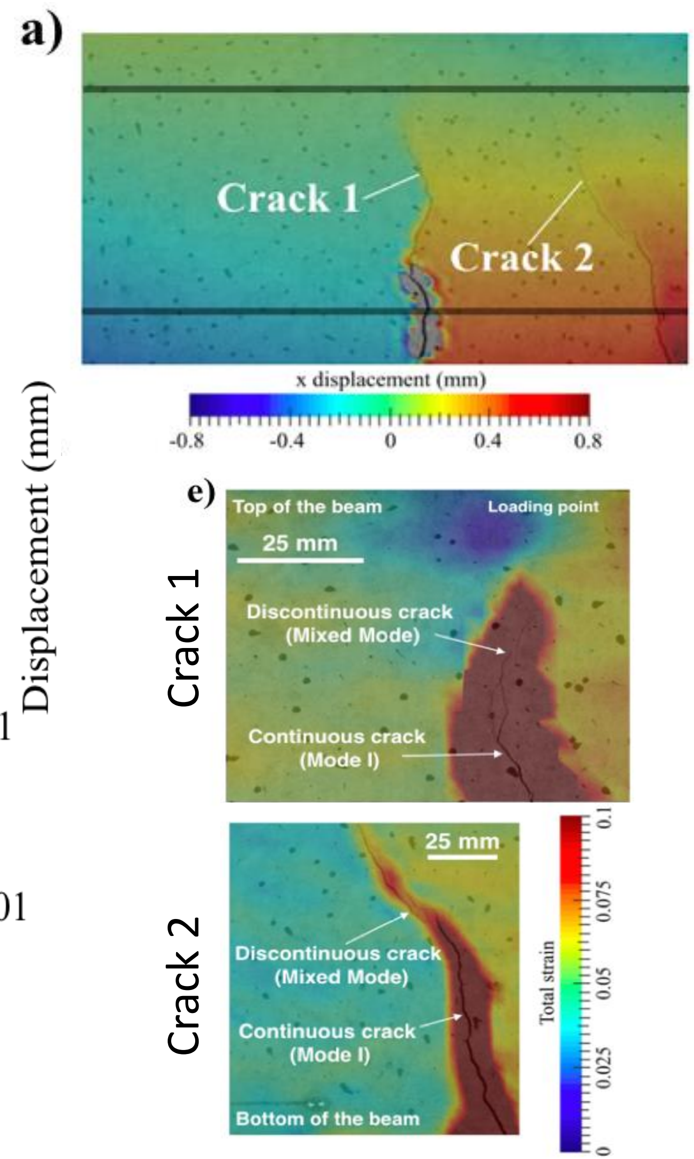




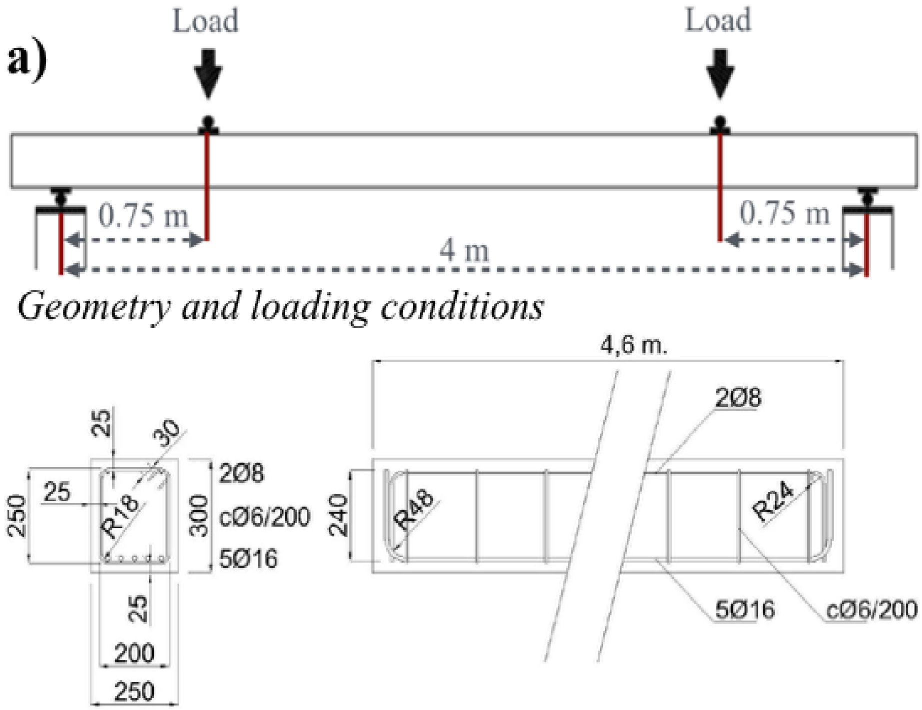


Energy dissipation



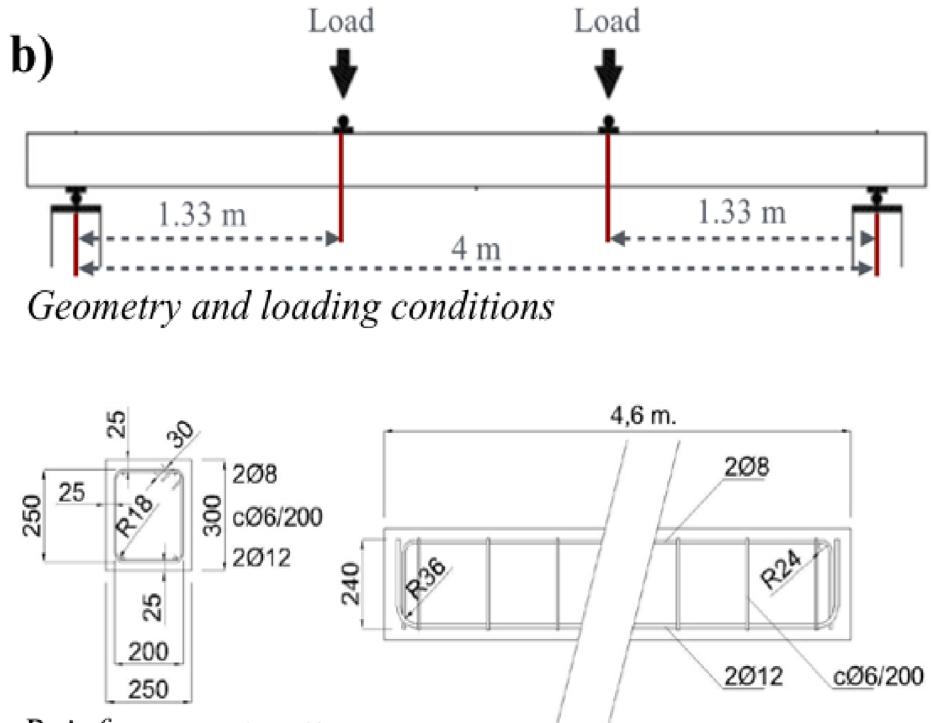


SEDUREC beams



Geometry and loading conditions

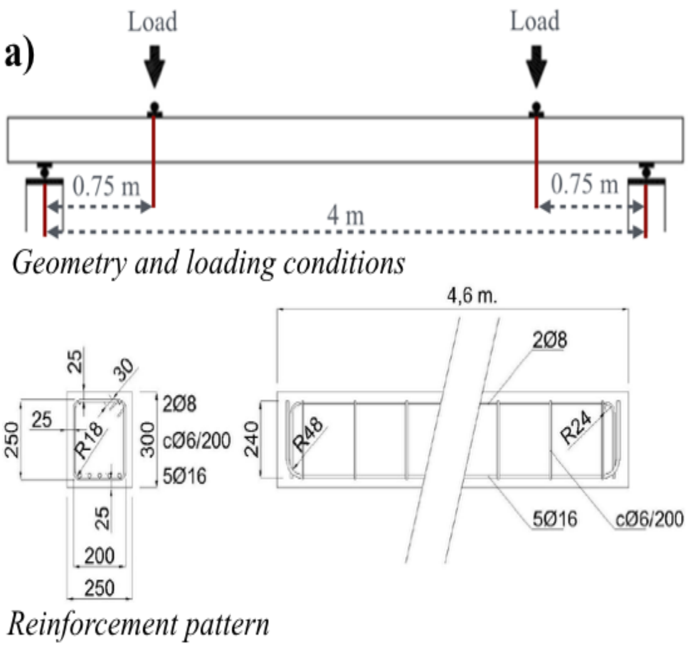
Reinforcement pattern

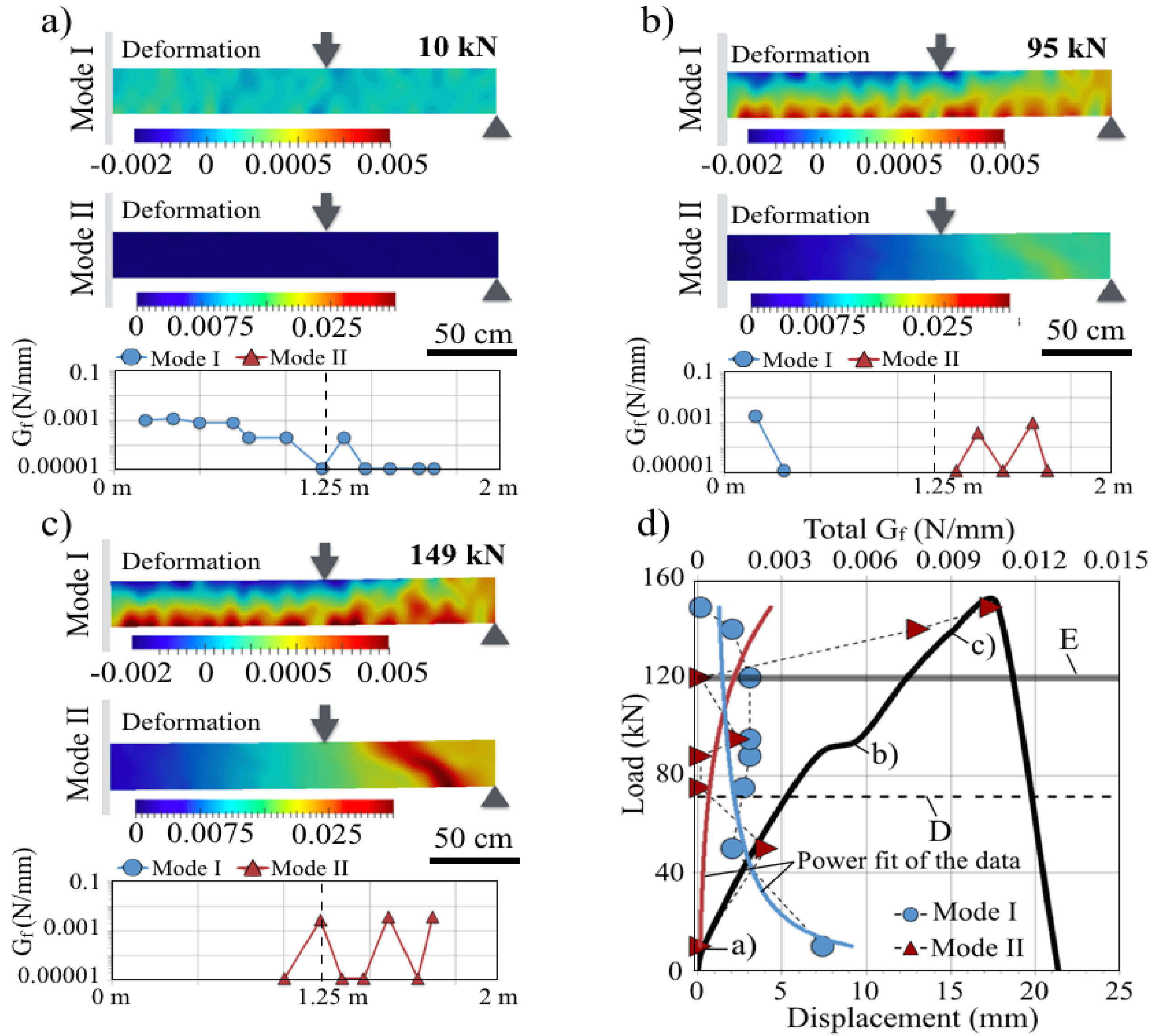


Geometry and loading conditions

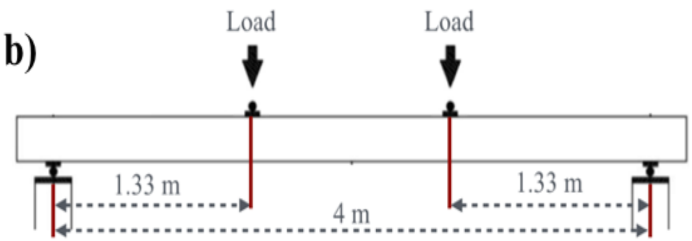
Reinforcement pattern

SEDUREC beams

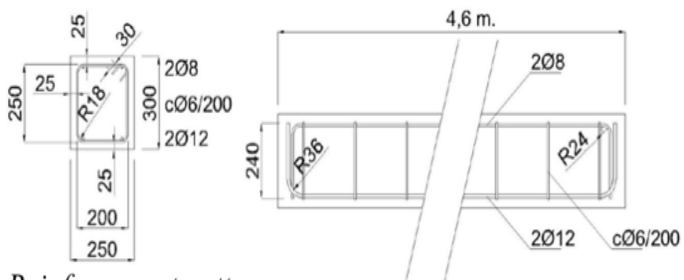




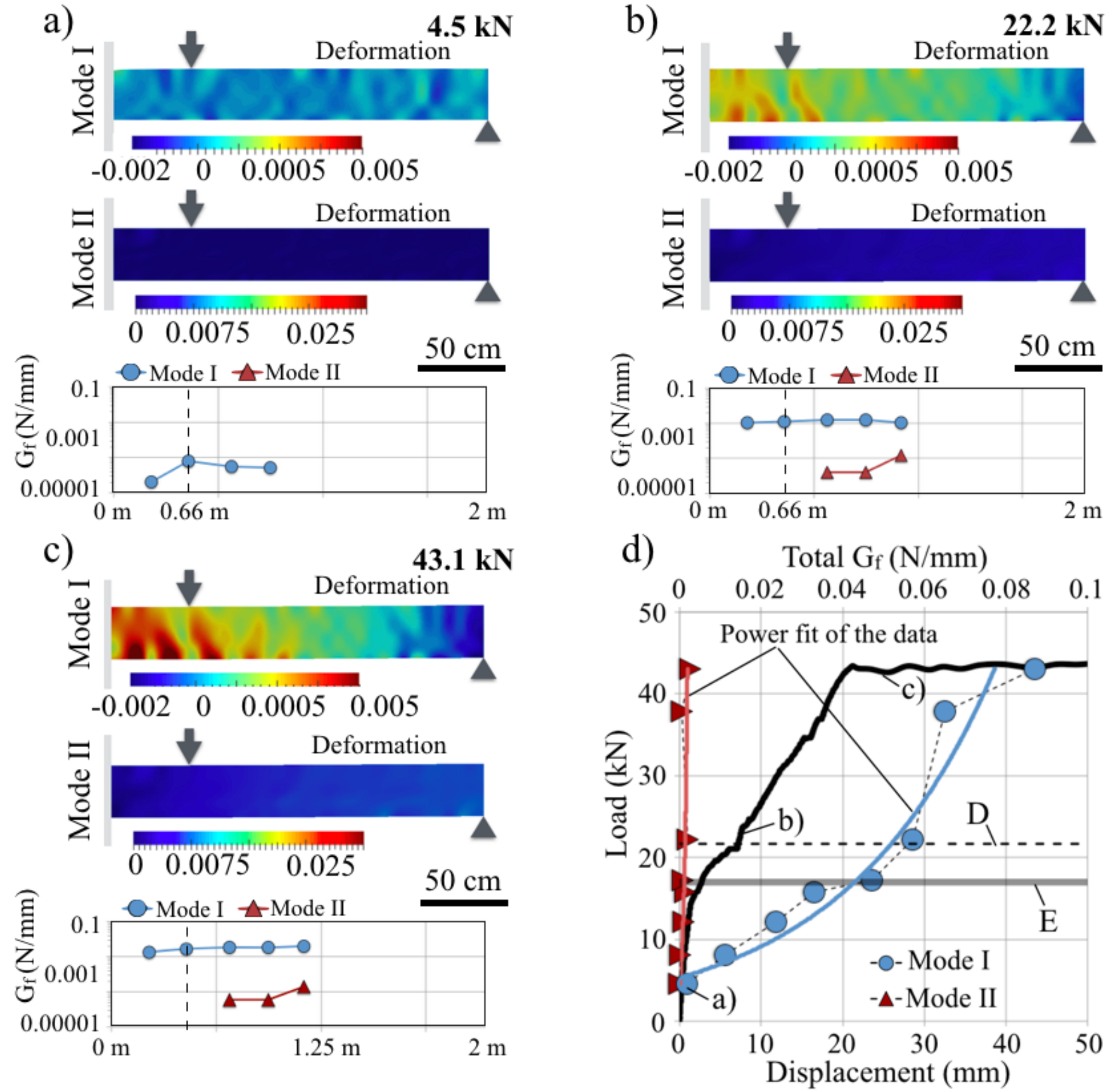
SEDUREC beams



Geometry and loading conditions



Reinforcement pattern



Conclusions

DIC is a non-contact and non-destructive technique that can give valuable inflrmation for structural health monitoring.

The early identification of the cracks and their energy released can be used to identify the regions to be repaired and the more suitable repair.

The technique is moving toward its application in large structures.

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

System Reliability of Bridge Structure Subjected to Chloride Ingress

Bernt J. Leira – NTNU, Norway
Sebastian Thöns – DTU, Denmark
Michael Faber – DTU, Denmark

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

Objective

Statistical analysis of test data obtained from the Gimsøystraumen Bridge in Norway

Assessment of lifetime with respect to Chloride Ingress by First Order Reliability Method (FORM)

Quantification of System Reliability Effects

Feasibility of an Enhanced Monte Carlo Simulation method to be assessed

Gimsøystraumen bridge: probabilistic models

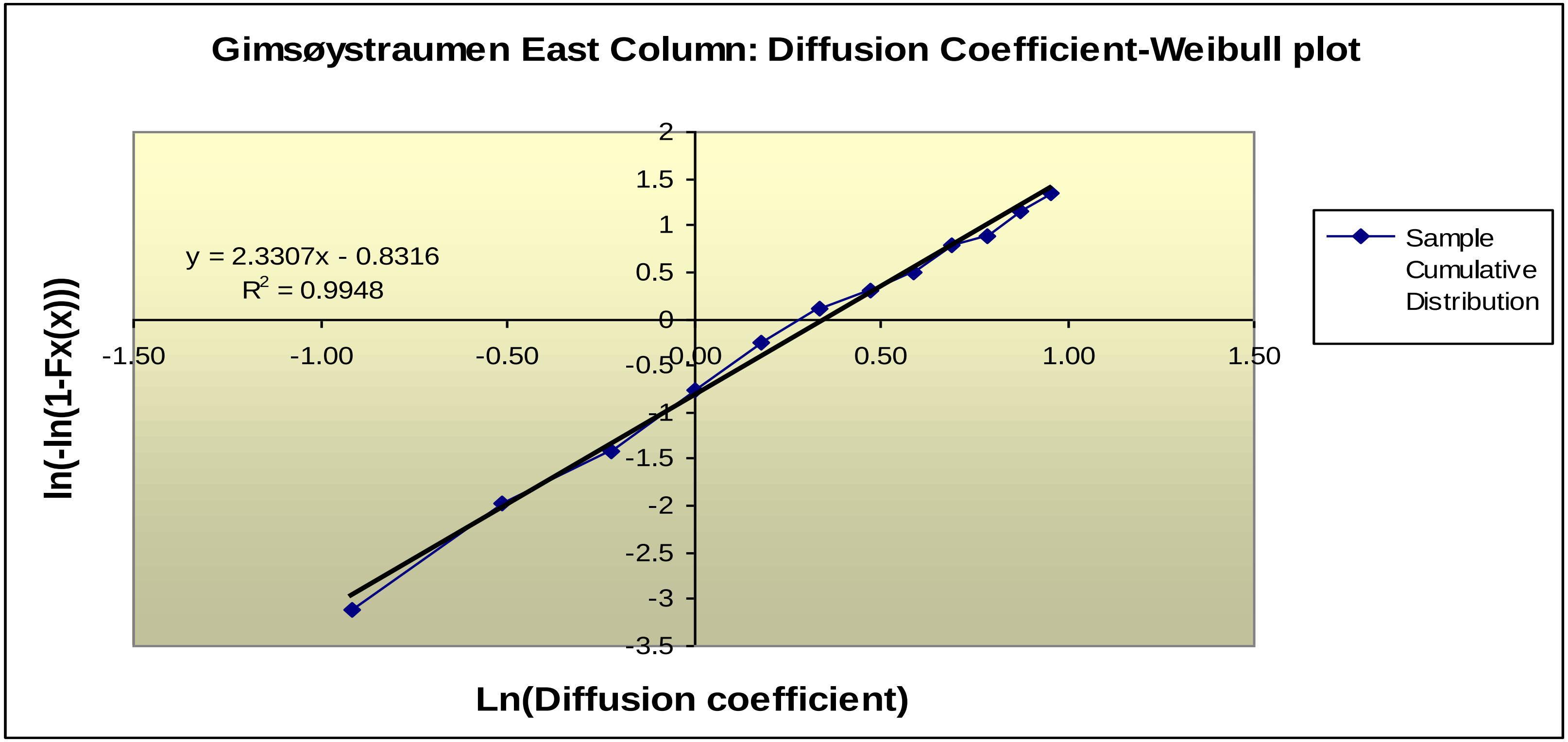
Tests performed:

- Diffusion coefficient
- Surface concentration
- Concrete cover

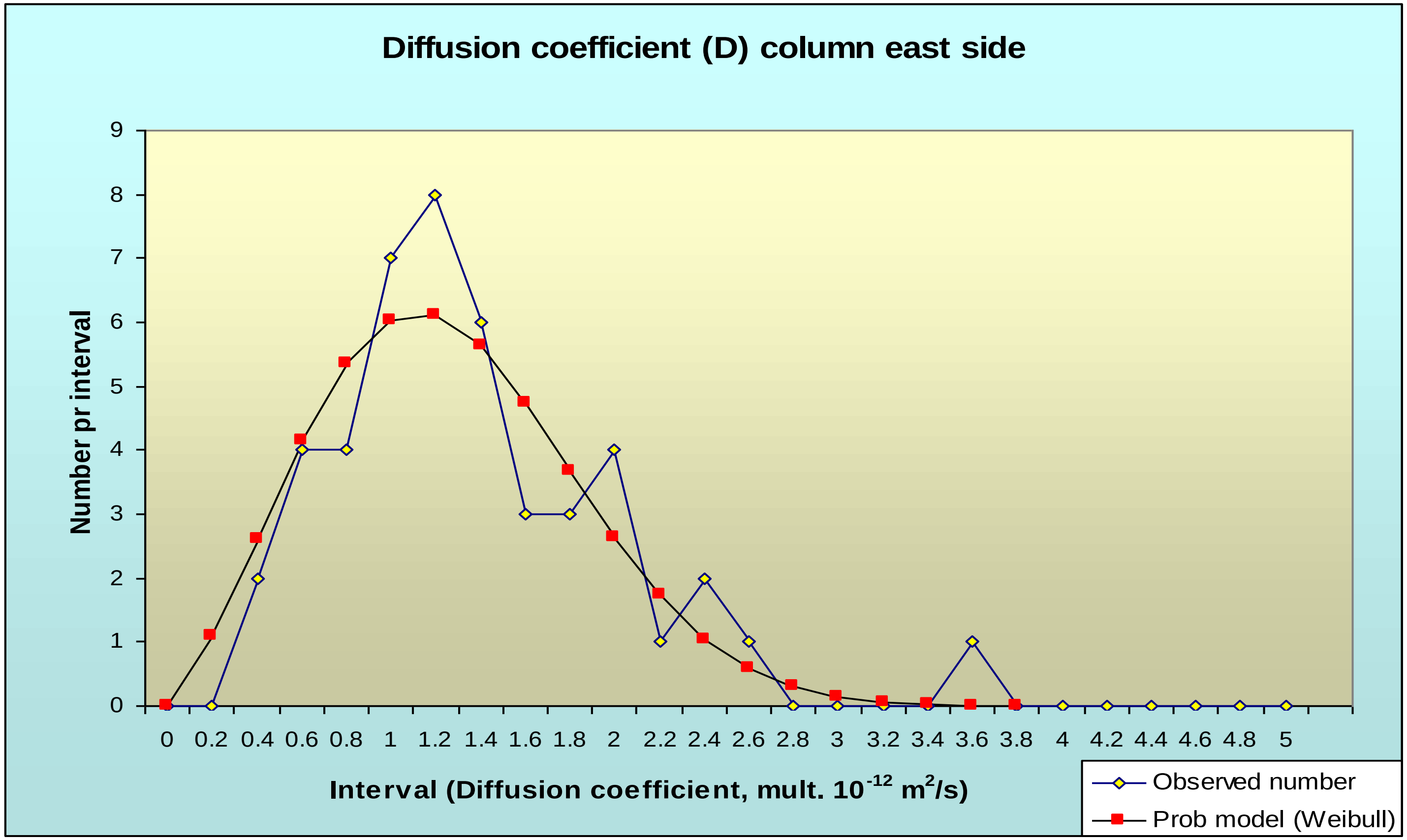
Observed/fitted cumulative distribution, Diffusion coefficient (Multiplication by 10^{-12} gives the values in m^2/s)

Prob. model	Regression line	Mean value	St.dev.	R^2
Normal	y = 1.61x – 2.04			0.98
Gamma	y = 0.74x – 0.65			0.79
Gumbel	y = 2.15x – 2.11	1.27	0.64	0.99
Weibull	y = 2.33x – 0.83			0.99
Lognorm.	y = 1.98x – 0.16			0.98

Observed/fitted cumulative distribution, East column



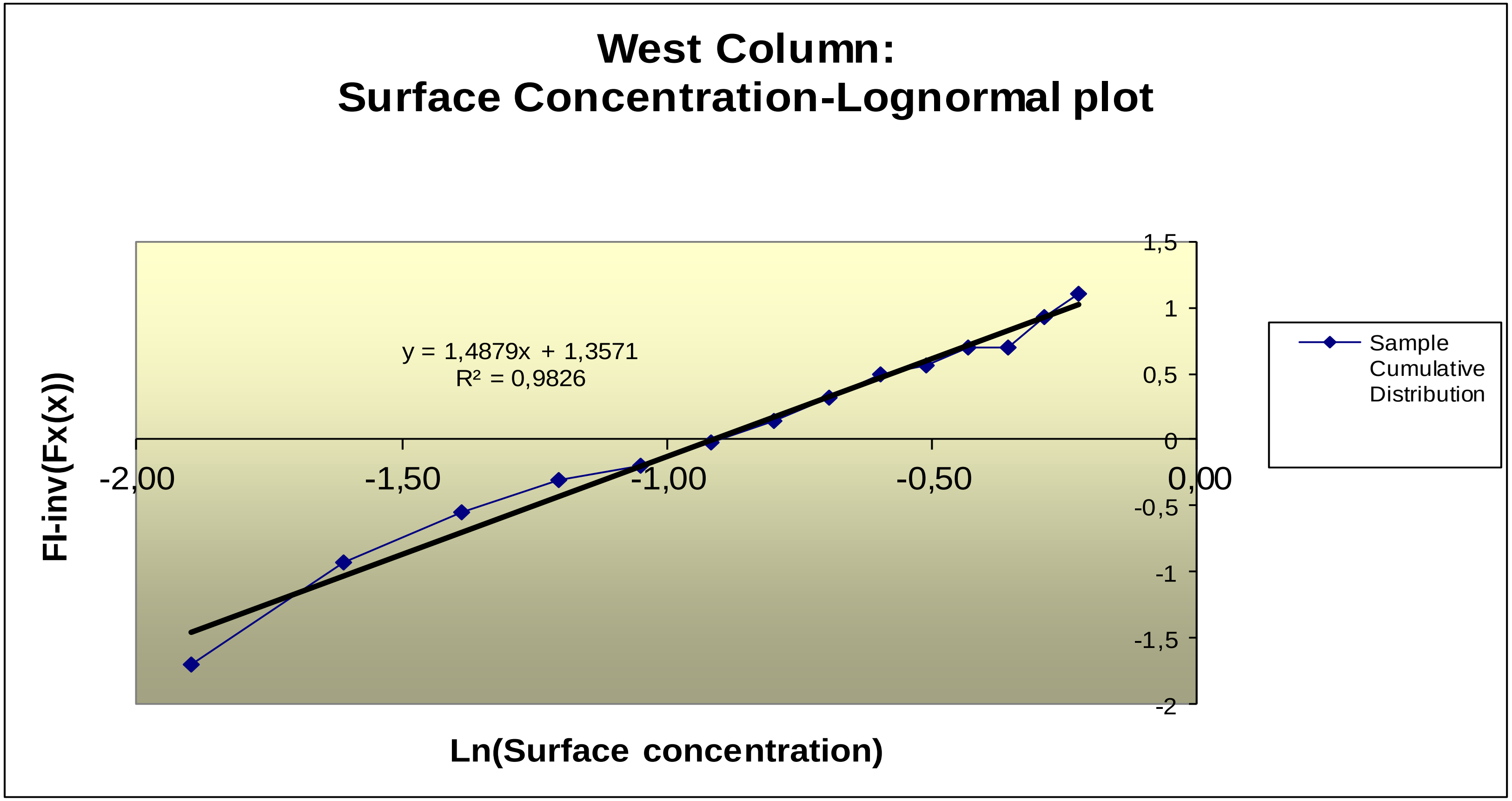
Observed/fitted relative frequency



Observed/fitted cumulative distribution:Surface concentration West column

Prob. model	Regression line	Mean value	St. Dev.	R^2
Normal	$y = 3.55x - 1.60$			0.92
Gamma	$y = 1.49x - 0.34$			0.94
Gumbel	$y = 4.08x - 1.36$	0.50	0.34	0.97
Weibull	$y = 1.90x + 1.24$			0.93
Logn.	$y = 1.49x + 1.36$			0.98

Observed/fitted cumulative distribution, West Column



Statistical distributions for the superstructure

Statistical variable	Distribution	Mean value	Standard deviation
Surface concentration, global variation	Lognormal	0.25 (% of concrete weight)	0.18 (% of concrete weight)
Diffusion coeffisient, global variation	Lognormal	0.88 (m^2/sec, mult 10^{-12})	0.68 (m^2/sec, mult 10^{-12})
α-faktor (time variation)	Normal	0.0	0.0
Initial concentration	Normal	0.015 (% of concrete weight)	0.0015 (% of concrete weight)
Concrete cover	Lognormal	23 mm	6 mm
Chritical chloride level	Lognormal	0.18 (% of concrete weight)	0.06 (% of concrete weight)
Model uncertainty	Normal	1.0	0.01/0.10

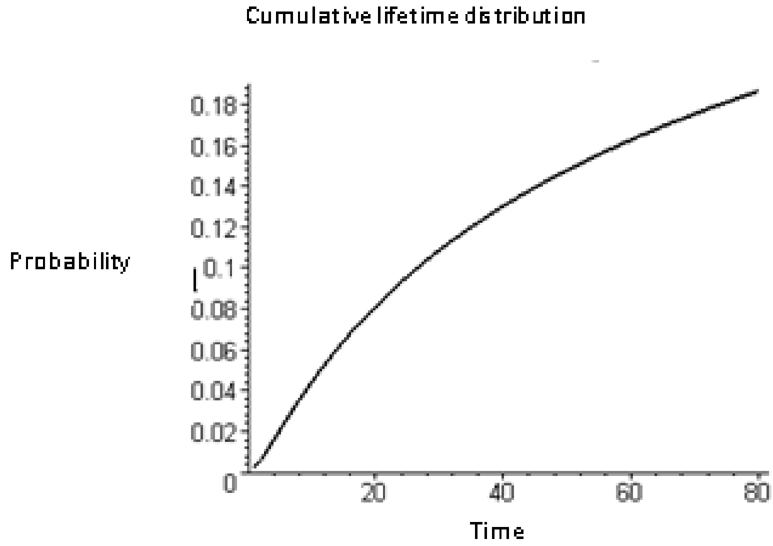
Reliability model based on Fick's diffusion model:

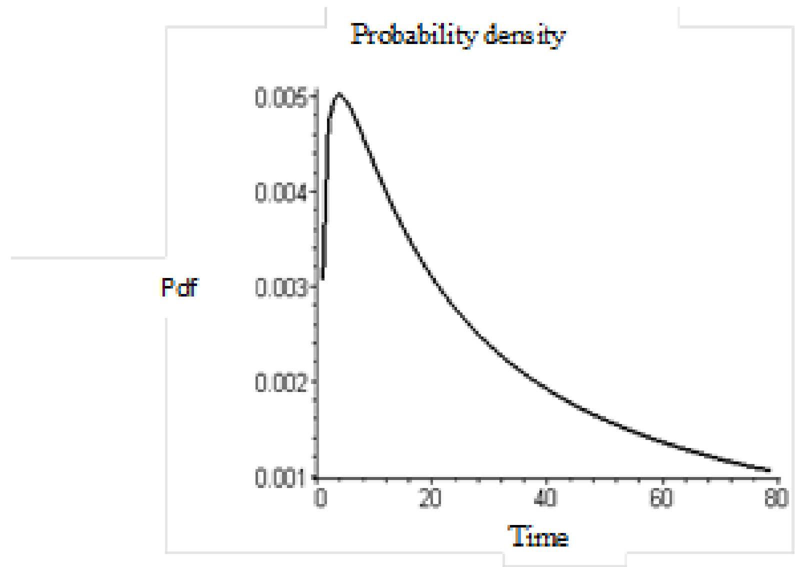
$$c(x,t) = c_i + (c_s - c_i) \cdot \mathrm{erfc}\left(\frac{x}{2\sqrt{D \cdot t}} \right)$$

Statistical distributions for the columns

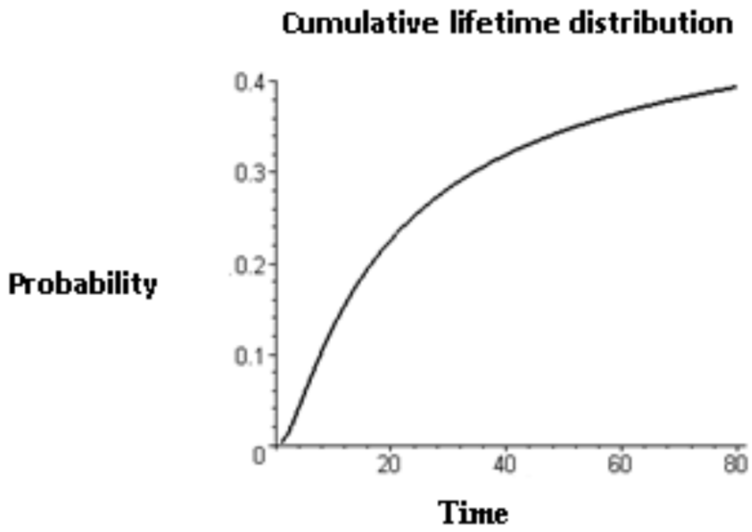
Statistical variable	Distribution	Mean value	Standard deviation
Surface concentration, global variation	Lognormal	0.25(0.50) (% of concrete weight)	0.18(0.34) (% of concrete weight)
Diffusion coefficient, global variation	Lognormal	1.27 (m^2/sek, mult 10^{-12})	0.64 (m^2/sek, mult 10^{-12})
α-factor (time variation)	Normal	0.	0.
Initial concentration	Uniform	0.015 (% of concrete weight)	0.0015 (% of concrete weight)
Concrete cover	Lognormal	45 mm	6 mm
Chritical chloride level	Lognormal	0.18 (% of concrete weight)	0.06 (% of concrete weight)
Model-uncertainty	Normal	1.0	0.01/0.10

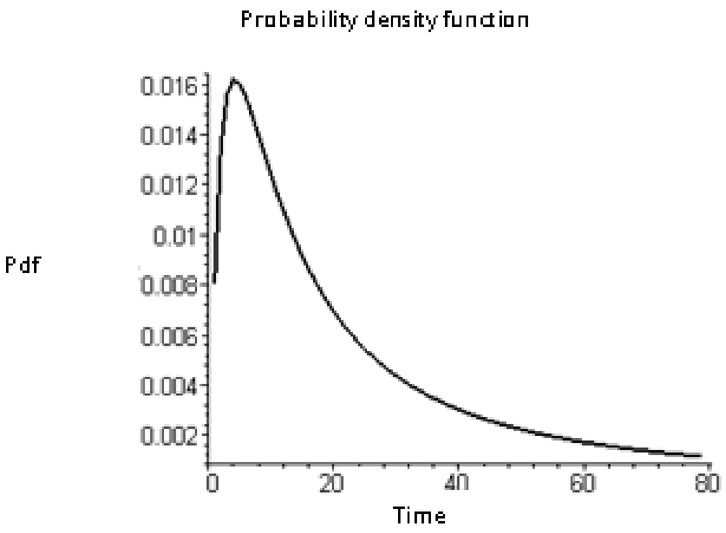
Probability distribution of lifetime (superstructure) corresponding to input statistical models, time-varying diffusion coefficient





Probability distribution of lifetime (superstructure) corresponding to input statistical models



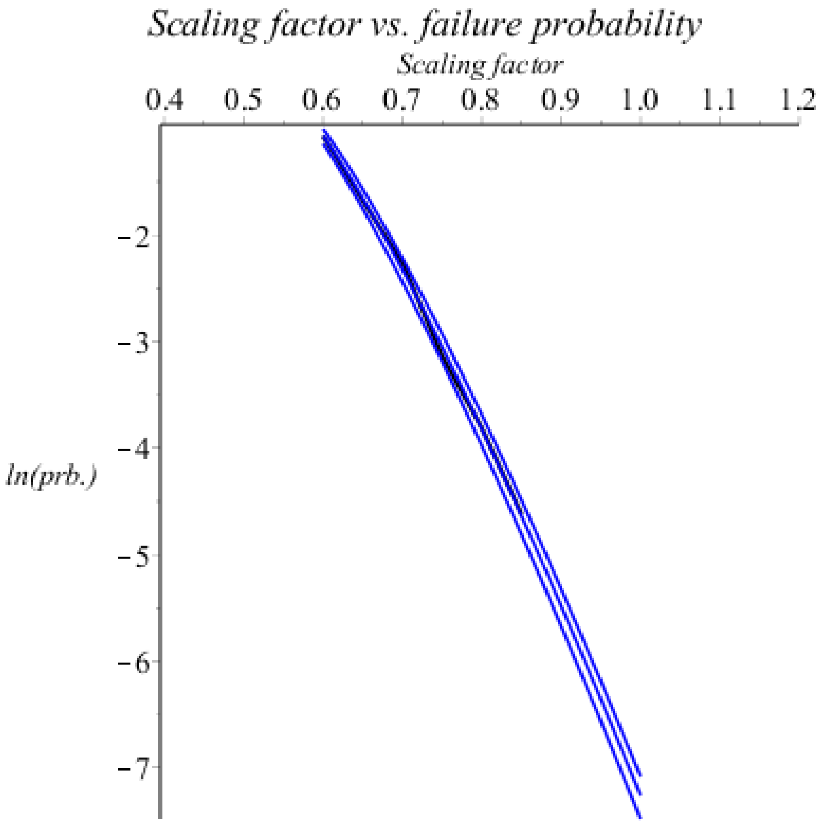


Simplified system reliability analysis:

90 independent components

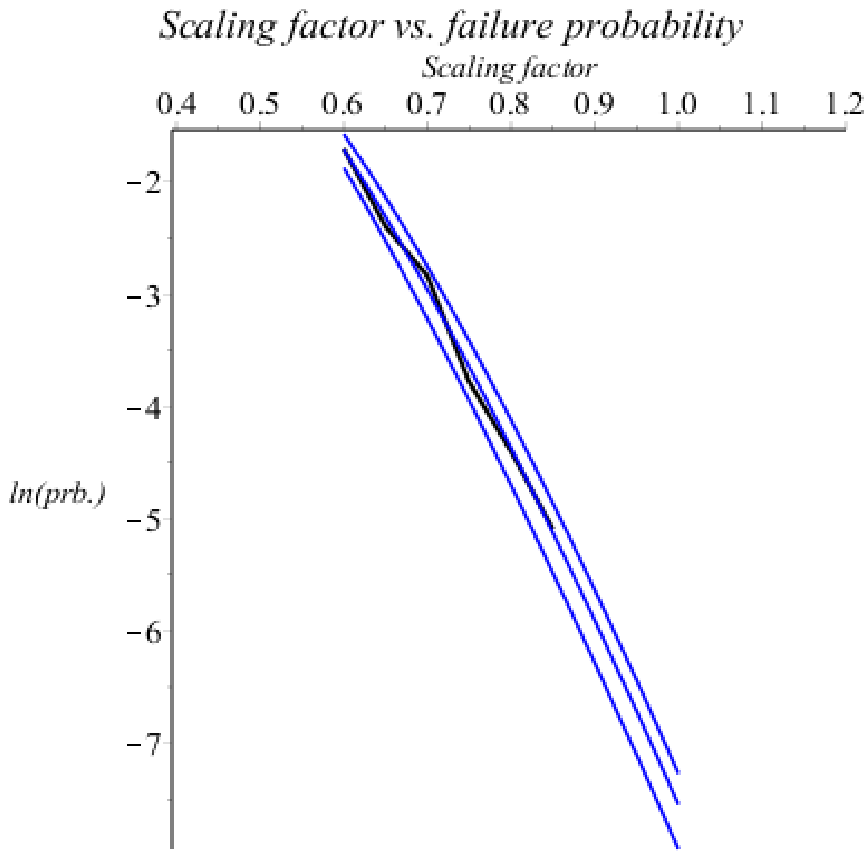
Surface chloride concentration is represented by a mean value of 0.14% and a standard deviation of 0.028 %

T = 30 years without any additional information (from monitoring or inspection) being available.



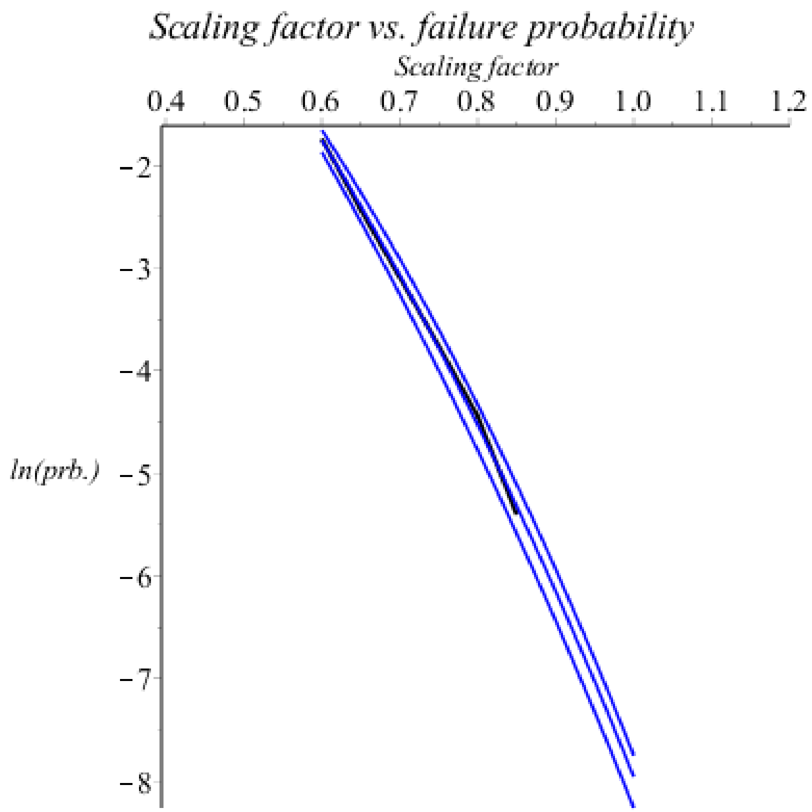
Simplified system reliability analysis:

T = 30 years for the case that the surface concentrations for half the components are found to be smaller than 0.196%.



Simplified system reliability analysis:

T = 30 years for the case that the surface concentrations for half the components are found to be smaller than 0.18%.



Simplified system reliability analysis:

Failure probability as a function of time for cases with different correlation coefficients with and without monitoring of half the “components”

Case	t=30	t=50	t=70	t=90
No inspct., ρ= 0.	7.0e-4	3.7e-2	1.0e-1	2.9e-1
No inspct., ρ= 0.5	4.4e-4	2.1e-2	8.7e-2	1.6e-1
No inspct., ρ= 1.	7.8e-6	4.1e-4	1.1e-3	5.4e-3
Monitoring ρ= 0.	3.5e-4	1.9e-2	5.0e-2	1.6e-1
Monitoring ρ= 0.5	2.2e-4	9.8e-3	4.5e-2	1.1e-1
Monitoring ρ= 1.	8.6e-9	1.9e-6	4.1e-5	1.6e-4

Simplified system reliability analysis:

Failure probability at time t = 50 years for cases with different correlation coefficients and without/with monitoring for varying number of “components”.
Total component number is N_{Tot} = 90.

Case	N_{Mon}=1	N_{Mon}=5	N_{Mon}=10	N_{Mon}=45
Monit., ρ= 0.	3.7e-2	3.5e-2	3.3e-2	1.9e-2
Monit., ρ= 0.5	1.9e-2	1.8e-2	1.8e-2	9.3e-3
Monit., ρ= 1.0	1.9e-6	1.9e-6	1.9e-6	1.9e-6
No monit., ρ= 0.	3.7e-2	3.7e-2	3.7e-2	3.7e-2
No monit., ρ= 0.5	2.1e-2	2.1e-2	2.1e-2	2.1e-2
No monit, ρ= 1.0	4.1e-4	4.1e-4	4.1e-4	4.1e-4

CONCLUSION

- Probabilistic models based on full-scale measurements from the Gimsøystraumen bridge are addressed.
- Models apply to diffusion coefficient, chloride surface concentration and concrete cover.
- Based on these models and supplementary models for other relevant parameters, probabilistic lifetime calculations are performed.
- A system reliability analysis method was introduced and subsequent reliability updating was performed by means of enhanced Monte Carlo simulation.
- The computational effort (as measured by CPU-time) was typically reduced by a factor of six or more.
- Future research topics :
  - ✓ Effect of correlation between the system components
  - ✓ Effect of non-identical system components,
  - ✓ Combination of parallel and series system models of bridge systems
  - ✓ Ultimate Limit State criteria in addition to Serviceability criteria.

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

QUANTIFYING THE VALUE OF SHM FOR EMERGENCY MANAGEMENT OF BRIDGES AT-RISK FROM SEISMIC DAMAGE

Piotr Omenzetter – University of Aberdeen, United Kingdom
Ufuk Yazgan – Istanbul Technical University, Turkey
Serdar Soyoz – Bogazici University, Turkey
Maria Pina Limongelli – Milan Polytechnic, Italy

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
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Oxford Dictionaries 2016 Word of the Year:

POST-TRUTH

Use of (emphatic) assertion in lieu of evidence, reasoning…

SHM as belonging in post-truth ‘reality’?

Quantifying the Value of SHM as a DISRUPTIVE IDEA?

FACT ABOUT EARTHQUAKES

Earthquakes often come in swarms:

- Preshocks
- Mainshock
- Aftershocks

WHAT USE/VALUE OF SHM IN BRIDGE EMEREGENCY MANAGEMENT?

- No use for mainshock; this risk already eventuated in the past
- Hazard(s) (aftershocks, traffic loads, etc.) must remain probable in future and will from now on act upon a possibly weakened structure
- There is probability of future detrimental consequences due to future aftershocks and information from SHM can help us manage that risk

RYTTER'S DAMAGE DETECTION CLASSIFICATION

Early levels: damage detection, localisation, severity assessment

- Current condition due to past events
- 'So what?', 'Should I be bothered?'
- Cannot on their own provide basis for 'fully rational' decisions about management of (future) risk
- May be achieved with data driven SHM methods

Top level: remaining useful life, capacity, reliability, risk of (future) failure

- Projections into the future
- Tells you if you 'should be bothered'
- Requires physics based structural models (or calibrated empirical relationships linking physical quantities like fatigue S/N curves)
- This level is required to quantify/realise value of SHM information

FACTS ABOUT PRACTICAL DAMAGE ASSESSMENT AND BRIDGE EMERGENCY MANAGMENT

Structural condition/damage is often expressed as a discrete **D**amage **S**tate

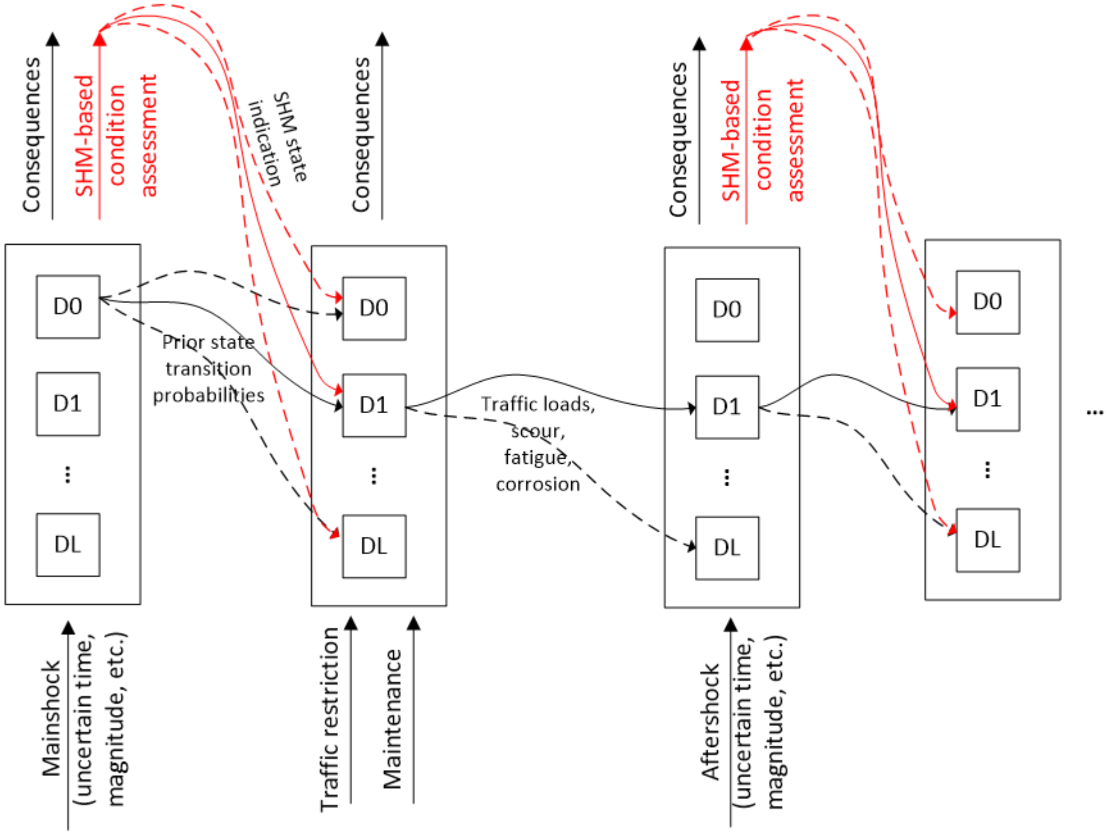
- DS0: full health
- DS1: light damage
- …
- DSL: collapse

Emergency management often entails choosing from a finite set of **T**raffic **R**estriction decisions

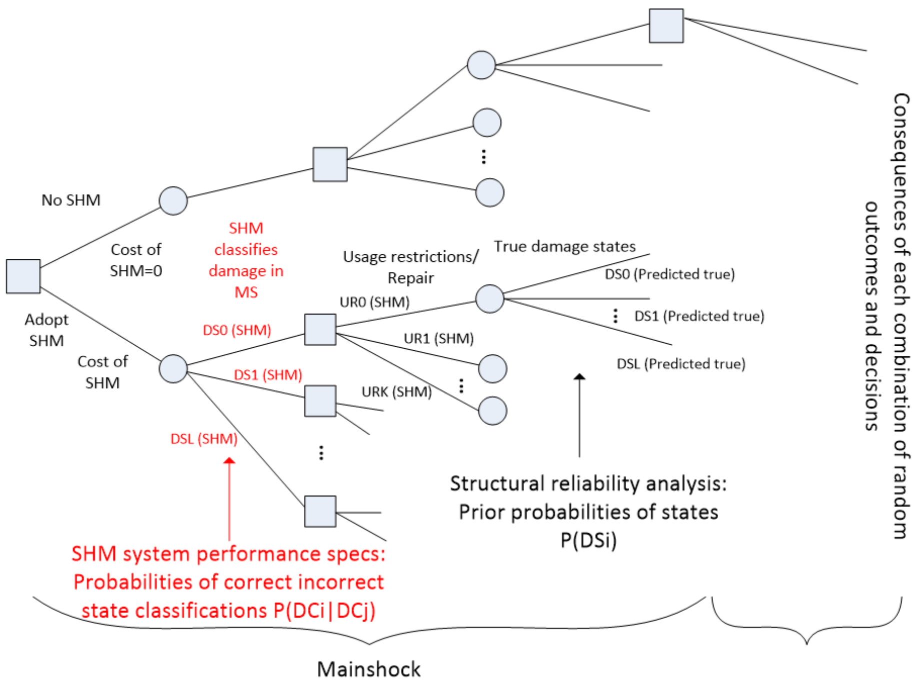
- TR0: fully operational
- TR1: restricted to light vehicles (posting)
- ….
- TRK: full closure to traffic

EVOLUTION OF BRIDGE CONDITION AS A STOCHASTIC DISCERET-STATE DYNAMICAL SYSTEM

With an SHM system the bridge becomes observable (but data are noisy) and with maintenance/repairs controllable



DECISION TREE FOR PRE-POSTERIOR BAYESIAN ANALYSIS TO DETERMINE THE VLAUE OF SHM INFORMATION AND OPTIMAL DECISION MAKING



Basis of decision = minimize overall cost/risk (probability x consequences)

Integration of SHM information via Bayes theorem

$$P\left(DS_i\left(True\right)/DS_j\left(SHM\right)\right)=\frac{P\left(DS_j\left(SHM\right)/DS_i\left(True\right)\right)P\left(DS_i\left(True\right)\right)}{\sum_{i=1}^{L}P\left(DS_j\left(SHM\right)/DS_i\left(True\right)\right)}$$

Practicalities (1)

HOW TO OBTAIN PROBABILITIES REQUIRED IN THE FRAMEWORK

State transition probabilities, e.g. P(DS1(AS)|DS0(MS)) etc.

Structural reliability analysis: seismic hazard analysis & Monte-Carlo nonlinear time history FE analysis

Specifications for SHM system performance, e.g.

P(DS1(SHM)|DS1(True)) – correct classification

P(DS0(SHM)|DS1(True)) – misclassification, etc.

Virtual experimental programme:

- Monte-Carlo nonlinear time history FE analysis ->
- Extraction of features from these 'virtual measurements' contaminated with noise and state classification based on them (DS(SHM)) ->
- Comparison with damage states predicted by FE analysis (DS(True))

Practicalities (2)

What to measure?

1. From measurements we need to extract features/indicators that map well into structural states DS0… DSL

2. We need to be able to assess remaining structural capacity (strength! not stiffness)

Displacements (rotations, strains) correlate better with practical damage assessment measures (ductility)

How do you measure them?

No fixed base for LVDTs or similar

Obstructed line of sight for optical methods, need to compensate for their own motion

Strain gauges will not survive

GPS has no required accuracy (yet?)

Double integration of accelerations introduces drifts

Banerjee & Shinozuka, 2008

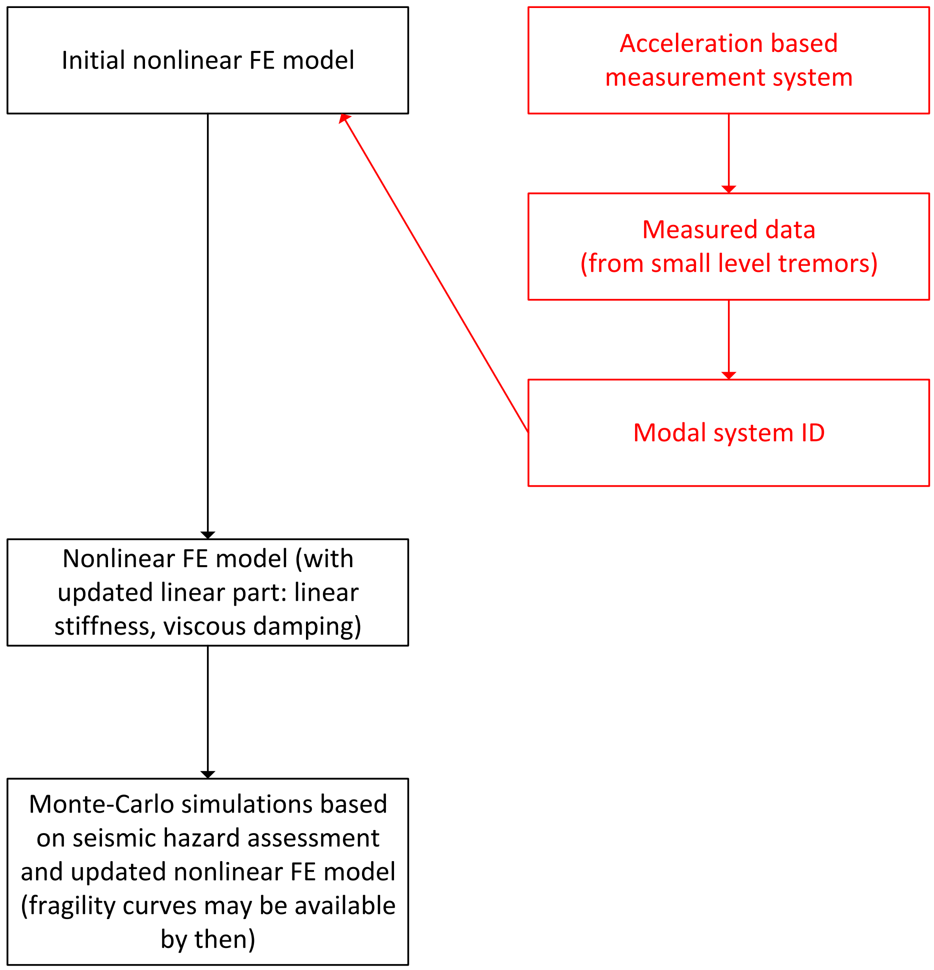
Damage state	Rotational ductility demand
None	<1
Negligible	1-1.5
Minor	1.5-3.10
Moderate	3.1-5.7
Major	5.7-8.3
Collapse	>8.3

Practicalities (3)

<u>Accelerometers</u> can easily be deployed, and are robust to damage

Acceleration-extracted features do not easily map into damage states

Envisaged SHM method (based on work by Soyoz et al.)



CONCLUSIONS

- Adoption and application of SHM & NDE needs to consider/anticipate quantitatively economic benefits of doing so
- Minimizing overall risk/cost is a guiding principle
- To realize the value of SHM and obtain a (hopefully positive) return on investment in SHM the highest level of Rytter's damage assessment level (future prognosis) must be attained
- A risk-based, pre-posterior Bayesian decision making framework has been proposed for assisting in such decisions taking into account the various uncertainties of the process
- An acceleration-based SHM method has been envisaged
- We need to apply it to a realistic example

TU1406
COST ACTION

IABSE

TU1402
COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

APPLICATION OF BRIDGE WEIGH-IN-MOTION MEASUREMENTS IN ASSESSMENT OF EXISTING ROAD BRIDGES

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Aleš Žnidarič, Maja Kreslin – *Slovenian National Building and Civil Engineering Institute*

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ZAG
ZAVOD ZA GRADBENIŠTVO SLOVENIJE
SLOVENIAN NATIONAL BUILDING AND CIVIL ENGINEERING INSTITUTE

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

INTRODUCTION AND MOTIVATION

- Age of existing road bridges in Croatia and region
- > 50 % - 40 or more years old – designed according to old codes
- Current standards (Eurocode) - conservative assumptions
- Site specific assessment
- Bridge Weigh-in-Motion – Structural Health Monitoring
- Link – performance indicator and corresponding performance goal

STRUCTURAL HEALTH MONITORING TOOL

Bridge Weigh-in-Motion Measurements

↔

STRUCTURAL PERFORMANCE

Existing Bridge Reliability

INTRODUCTION AND MOTIVATION

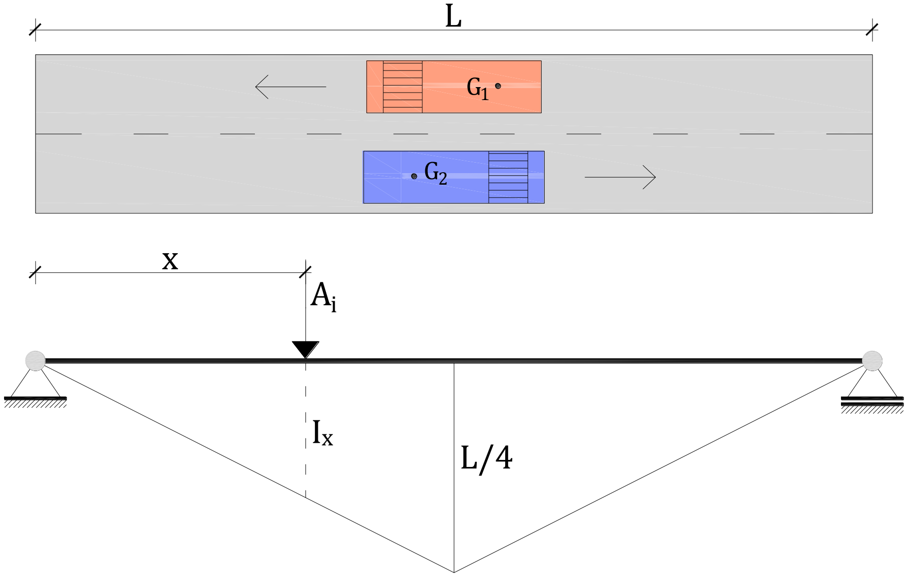
- Weigh-in-Motion (WIM) – measuring vehicles at full speed as they drive over measurement sites.
- Data provided:
    - Vehicle Gross Weight
    - Axle Weights
    - Axle Number and Spacing
    - Vehicle Speed
    - Time Stamps
- Bridge Weigh-in-Motion (WIM):
    - Using existing bridges as weighing scales – minimum traffic interruption
- Additional structural data:
    - Realistic Influence Lines
    - Dynamic Factor
    - Transverse load distribution

B-WIM – DATA POST PROCESSING

- Main Challenge:
  - Extrapolation of acquired traffic data
  - Estimation of maximum load effects on a bridge in certain time period

Time stamp	Lane	Speed [m/s]	Class	Number of axles	GSW [kN]	AW1 [kN]	AW2 [kN]	Axle spacing [m]
2007-03-22-00-39-28-955	1	17,5	41	2	123,8	37,07	86,69	6,07

- Solution:
  - Convolution method – influence lines
  - Maximum load effect – two vehicles
  - Calculation of each vehicle passing



$$Q_s = \sum_{i=1}^{n} A_i \cdot I_i$$

Q_s is static load effect of vehicle,

A_i is weight of the axle i,

n is number of axles of each vehicle,

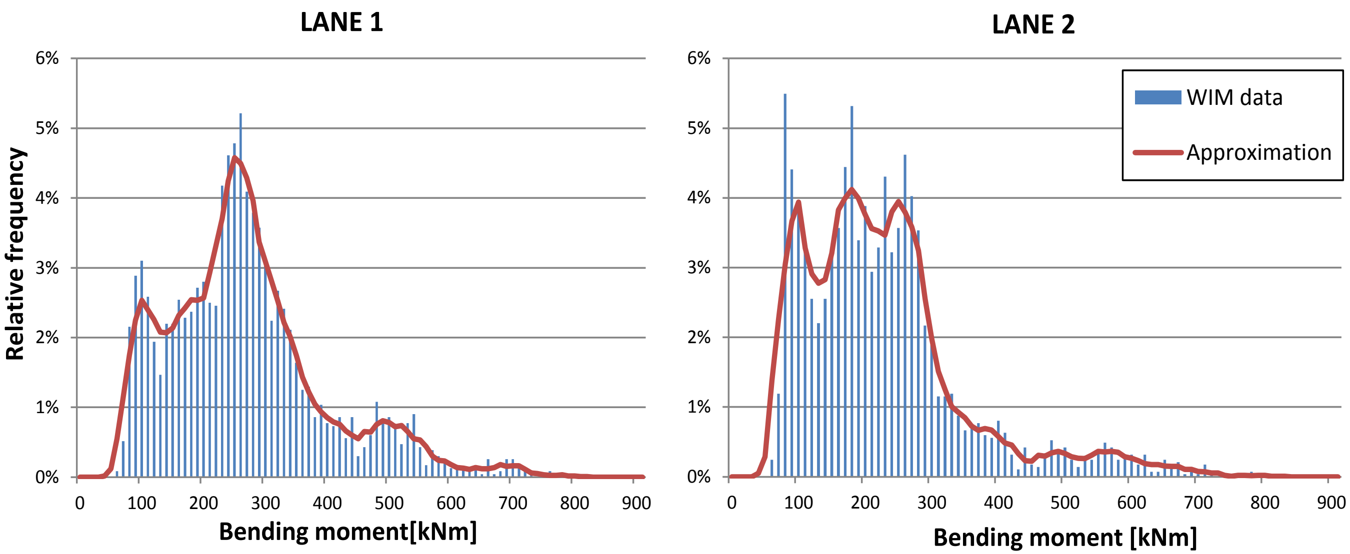
I_i is value of the influence

B-WIM – DATA POST PROCESSING

- Output example:

n	W1 [kN]	W2 [kN]	W3 [kN]	A_{1-2} [m]	A_{2-3} [m]	M_{Max} [kNm]	$x_{1,M}$ [m]	$x_{2,M}$ [m]	$x_{3,M}$ [m]	V_{Max} [kN]	$x_{1,V}$ [m]	$x_{2,V}$ [m]	$x_{3,V}$ [m]
3	46,18	52,37	42,85	6,04	1,26	143,78	11,55	5,51	4,25	72,21	6,15	0,11	0,00
2	37,07	86,69	/	6,07	/	237,62	11,55	5,48	/	102,39	6,15	0,08	/

- Large number of vehicles
- Generation of load effects histograms for each lane
- Modified „moving average” approximation – probability mass function (PMF)



B-WIM – DATA POST PROCESSING

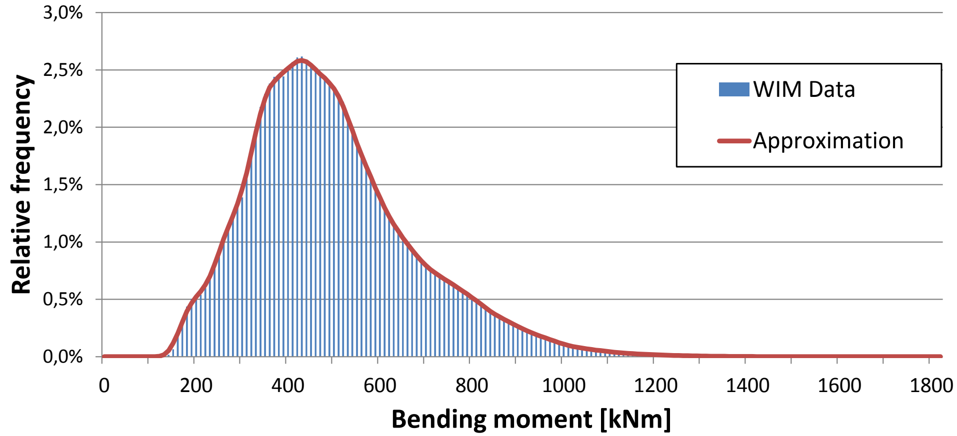
- Convolution of histograms
  - Simulation of maximum load effect
  - Vehicles in both lanes

$$f_z = \sum_{k=1}^{m} f_x(k) \cdot f_y(z-k)$$

f_x is PMF of vehicles in lane 1,

f_y is PMF of vehicles in lane 2,

f_z is PMF of vehicles in both lanes



- Expected Maximum Load Effects
  - Estimation for different time periods
  - Extreme Value Theory

$$F_{\max}(z) = F_Z^N(z)$$

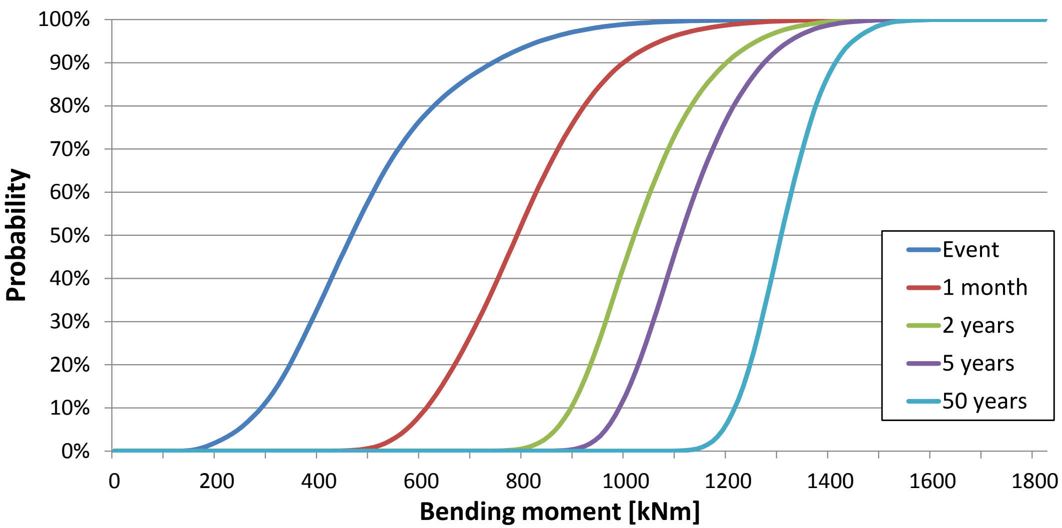
N is number of expected maximum load effects

F_Z is cumulative distribution function (CDF) derived from f_Z

$F_{\max}$ is maximum expected load effect in given time period

B-WIM – DATA POST PROCESSING

- Convolution curves (CDF for different time periods)

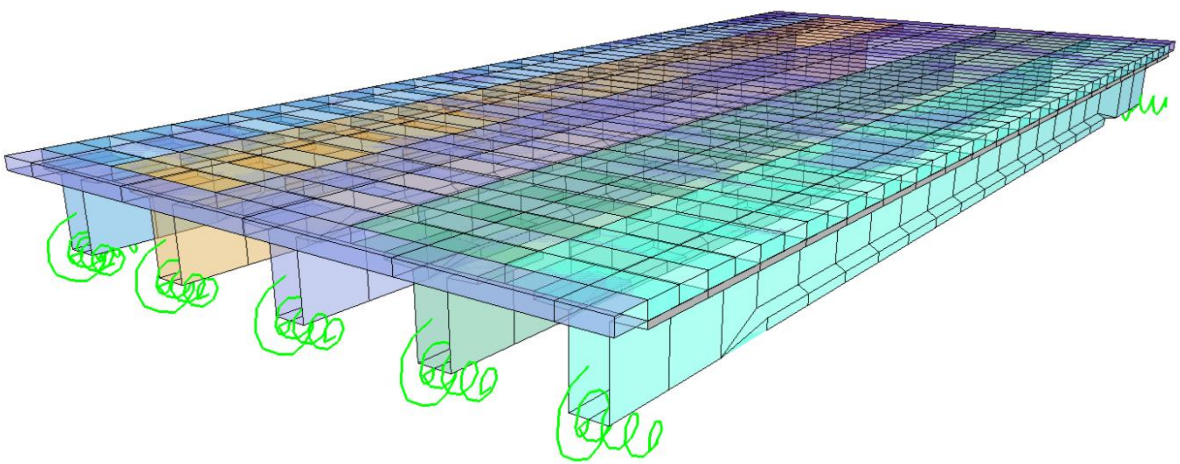


- Statistical parameters for bending moment (assuming normal distribution):

Time period	Mean value [kNm]	Standard deviation [kNm]
Single event	463,60	169,65
One month	922,14	125,72
Two years	1016,96	116,97
Five years	1107,48	115,54
Fifty years	1304,17	76,06

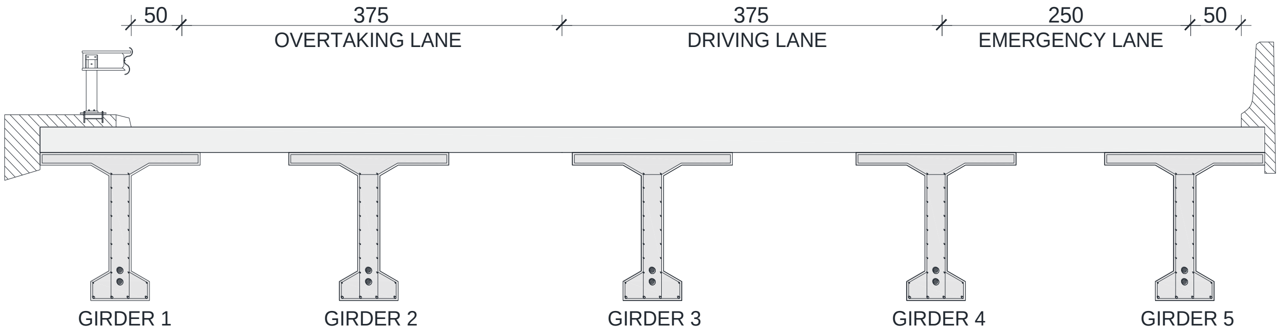
APPLICATION OF B-WIM DATA IN ASSESSMENT

- Assessment of existing bridges using numerical models
  - Development of models using Finite Element Method (FEM)
  - Models based on documentation and original design plans
  - Data provided
    - Load distribution
    - Modal Shapes
    - Deflections etc.
  - Calibration of bridge models using on-site measurements
    - Realistic behavior of the bridge
    - Site – specific load models
    - Bridge dynamics etc.



APPLICATION OF B-WIM DATA IN ASSESSMENT

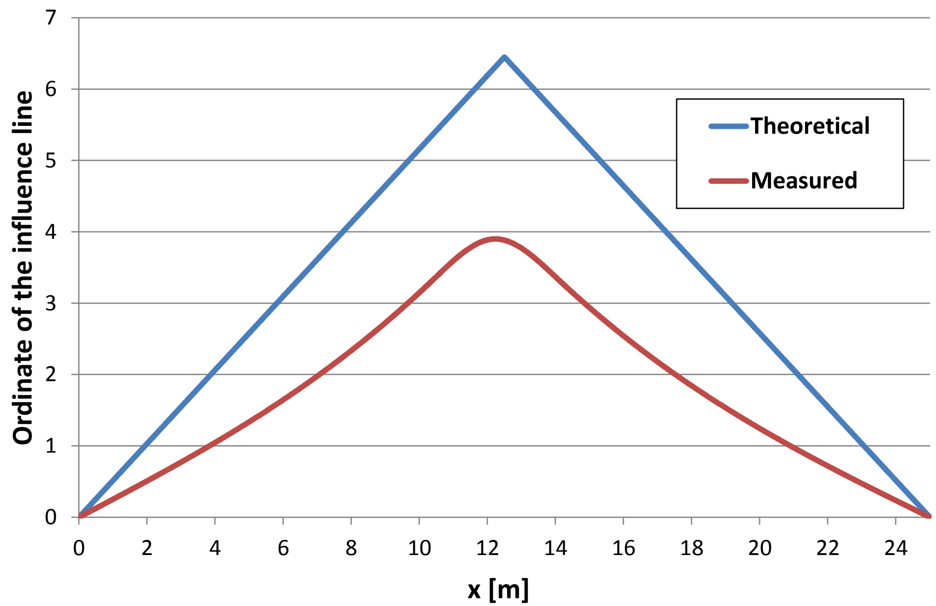
- Case Study Bridge
  - Simply supported highway bridge
  - Single span – 24,8 meters
  - Prefabricated I-type girders with monolithic concrete deck



- Initial assessment procedure – Step 1
  - Linear analysis based with partial factors based on EN 1992-2
  - Self weight and additional permanent load
  - Traffic load according to Load Model 1 of EN 1991-2
  - Cross section resistance calculated based on built in reinforcement

APPLICATION OF B-WIM DATA IN ASSESSMENT

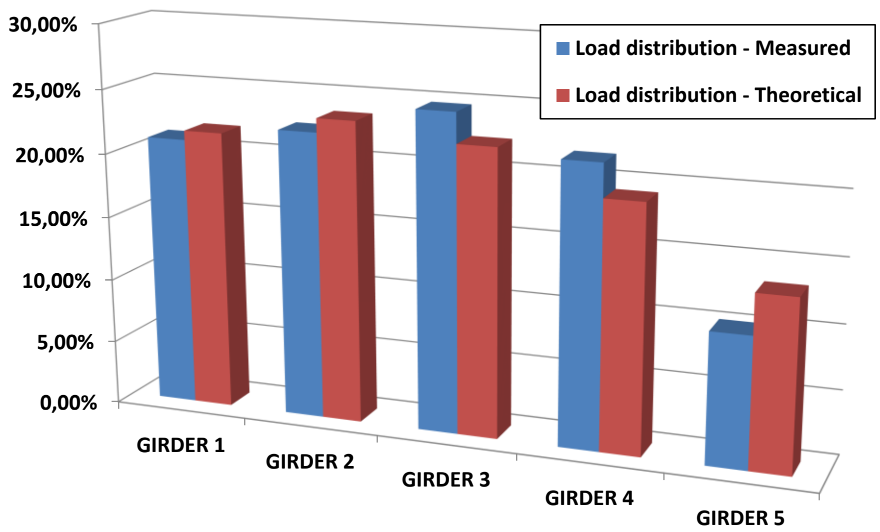
- Additional B-WIM structural dana – Realistic Influence Lines – Step 2
  - Calibration of B-WIM system with vehicles of known size and weight
  - Measured Influence lines – calibration output



  - Realistic bridge behavior – not simply supported
  - Degradation of the bearings and/or expansion joints
  - Result - reduced bending moment in the middle of the span

APPLICATION OF B-WIM DATA IN ASSESSMENT

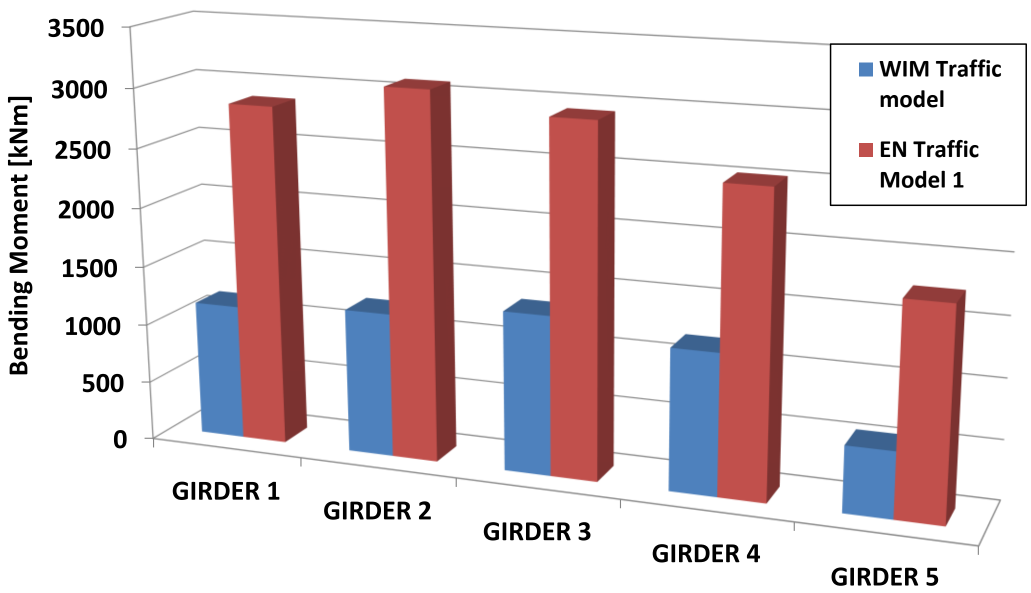
- Additional B-WIM structural data – Transverse Load Distribution – Step 3
  - Distribution of total bridge loading on each girder
  - Numerical model – based on the stiffness and geometry of cross sections
  - B-WIM sensors placed on every girder – realistic deflection and amount of load



  - No significant difference – expected results
  - This method can reveal degradations – even non visible (cracks etc.)

APPLICATION OF B-WIM DATA IN ASSESSMENT

- Additional B-WIM structural data – Site specific Load Model – Step 4
  - Maximum expected bending moment determined from B-WIM
  - Time period of 75 years chosen – remaining service life
  - Comparison with EN 1991-1 – bending moment at the middle of the span



APPLICATION OF B-WIM DATA IN ASSESSMENT

- Additional B-WIM structural data – Analysis of the Results
    - Ratio of bending moment load/resistance on each girder – deterministic

	Girder 1	Girder 2	Girder 3	Girder 4	Girder 5
M_{Rd}/M_{Ed} – EN 1991-1	1,046	0,992	1,022	1,124	1,390
M_{Rd}/M_{Ed} – B-WIM	2,036	1,965	1,894	2,042	2,797

- Probabilistic approach – modelling of parameters as stochastic variables
- B-WIM load models – described with statistical parameters
    - Mean Value
    - Standard Deviation
    - Distribution Type
- Definition of Limit State Equation

$$Z = \theta_R \cdot R - \left(\theta_{E,PERM} \cdot E_{PERM} + \theta_{E,B-WIM} \cdot E_{B-WIM}\right)$$

R - Resistance of bridge cross section

E_{PERM} - Permanent load effects

E_{B-WIM} - Traffic load effects

θ - Additional model uncertainties

CONCLUSIONS AND FUTURE REMARKS

- **Existing bridges:**
  - Results clearly show the quantification of B-WIM measurements in load carrying capacity assessment.

- **Economic aspect:**

COST - BENEFIT ANALYSIS

Initial Investment in B-WIM Measurements

Reduction of Bridge Maintenance Costs

- **Bridge Management:**
  - WIM data can be used for early discovery of non-visible degradation of bridge elements, with a comparison of theoretical and numerical structural data.

THANK YOU FOR YOUR ATTENTION.

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Aleš Žnidarič, Maja Kreslin – *Slovenian National Building and Civil Engineering Institute*

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ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

OPTIMAL DECISIONS BASED ON MONITORING – CASE STUDY OF A STEEL ROOF

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02nd – 03rd March 2017
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OUTLINE

- Scope
- Standards
- Case study stadium roof: Input
- Case study stadium roof: Results and discussions
- Conclusions

INTRODUCTION (*SCOPE OF THE CONTRIBUTION*)

- **Implementation in standards**
- **Use of monitoring results**
- **Reliability and risk assessment**
- **Decision making (risk acceptance)**
- **Development of guidelines (WG5 in TU 1402)**
- **Application in practice**

STANDARDS

A) Types of monitoring

- **Permanent monitoring**
- **Periodic monitoring**
- **Spot monitoring**

B) Issues

- **Field test planning**
- **Sensor classification**
- **Sensor availability/reliability assessment**
- **Treatment of data**
- **Inspection/maintenance planning**

C) Improvements

- **Acceptance/decision criteria**

STANDARDS: RISK BASED CLASSIFICATION EN 1990

Consequence Class	**Description**	**Examples of buildings and civil engineering works**
CC1	Low consequences for loss of human life, social and environmental consequences small or negligible	Agricultural buildings, silos, greenhouses
CC2	Medium consequences for loss of human life, economic, social or environmental consequences considerable	Residential and office building, public buildings
CC3	High Consequences for loss of human life, or economic, social or environmental consequences very great	Stadia, congress centers

STANDARDS: RECOMMENDATION FOR INSPECTION INTERVALS (VDI 6200)

	Visual Check	Inspection (engineer)	Verification (expert)
CC1	5 years		
CC2	3 years	5 years	15 years
CC3	2 years	3 years	10 years

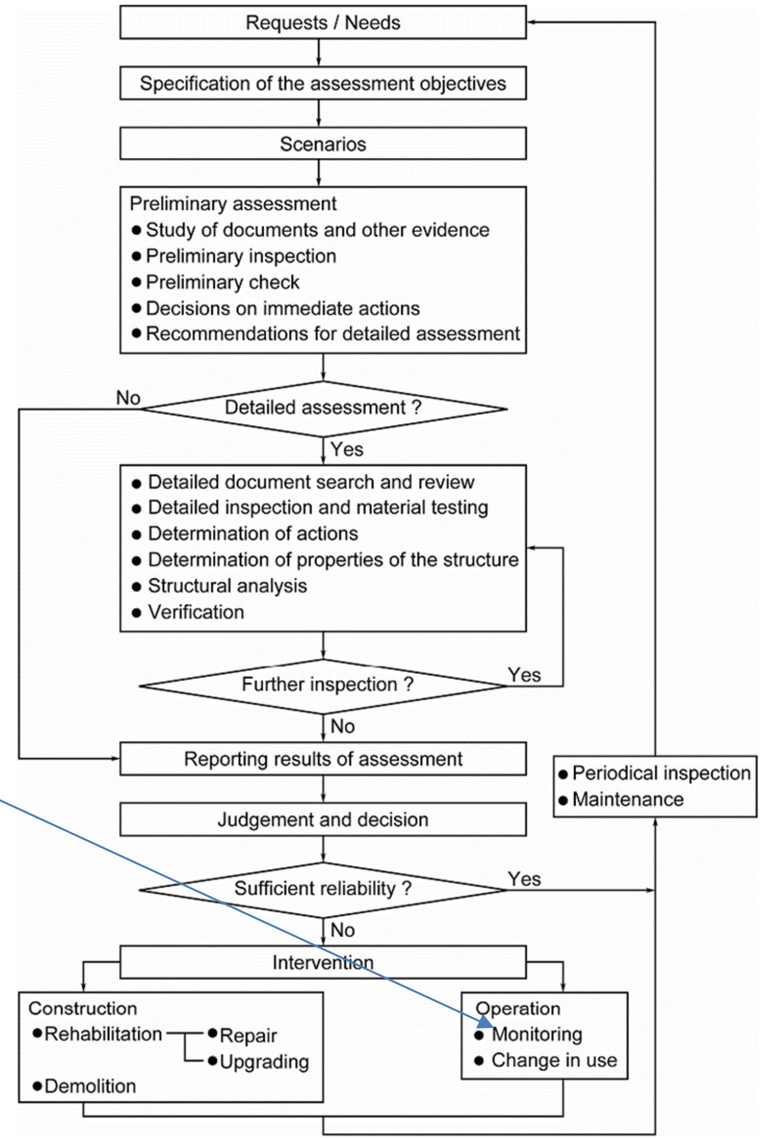
STANDARDS

ISO 13822

Flow chart

Assessment of existing structures

Decisions based on monitoring

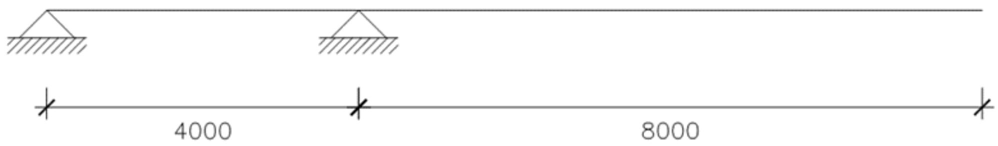


CASE STUDY: ROOF UNDER SNOW LOADS

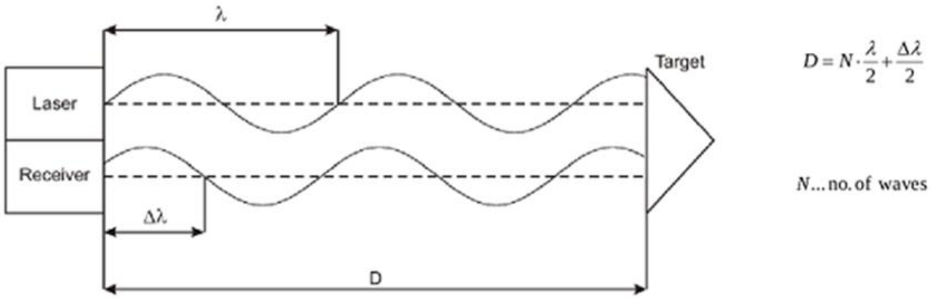
Roof failures under snow load
Bavaria, 2006

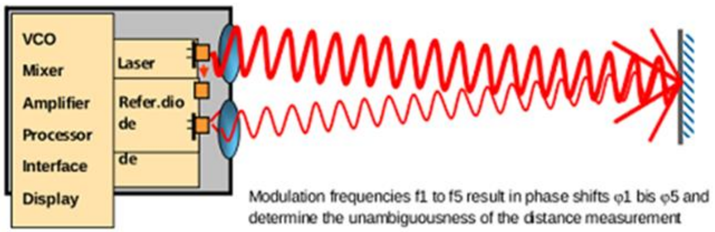
STUDY CASE Stadium roof: input

static system



- Consequence Class 3 Structure
- snow load according to new code 33% higher compared to code at design stage
- online monitoring of snow depth
- Example laser snow depth sensor (Haij, 2011)





CASE STUDY Stadium roof: input

Failure through limit state of bending

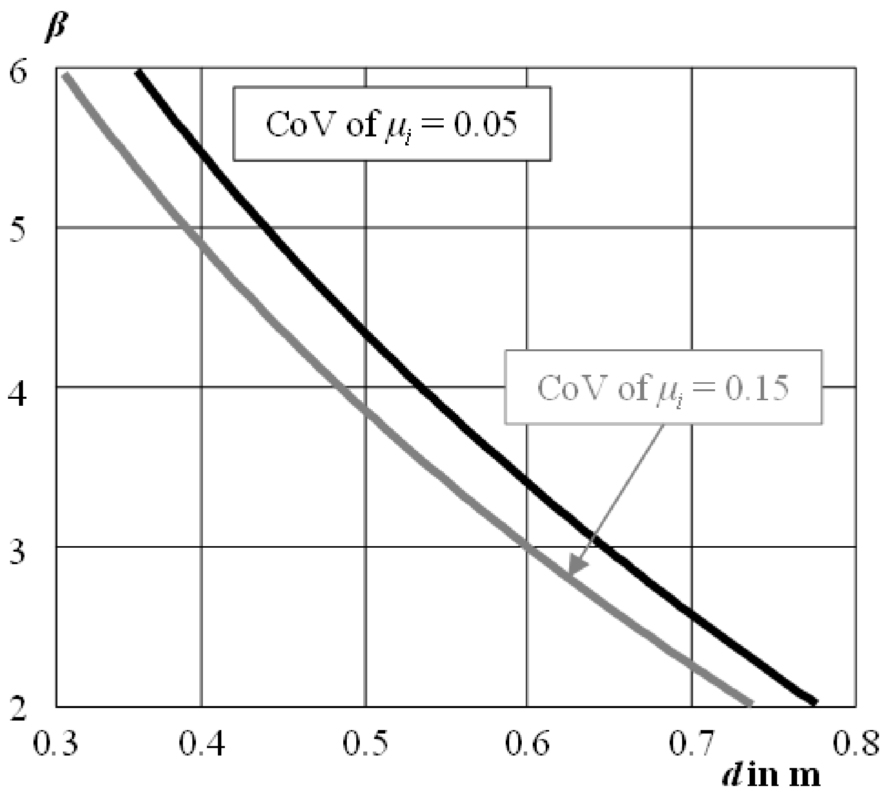
$$Z(\boldsymbol{X}) = \vartheta_R \, W_{\mathrm{pl}} \, f_{\mathrm{y}} - \vartheta_E \, L^2/2 \, [\gamma_{\mathrm{steel}} \cdot A_{\mathrm{s}} + g_{\mathrm{roof}} \, b + \mu_i \times \gamma_{\mathrm{snow}}(d) \times b \times d]$$

main random variables

- *f_{y} yielding stress*
- *ϑ_R and ϑ_E model uncertainties for R and E*
- *μ_i shape factor*
- *$\gamma_{snow}(d)$ snow density (through different prediction models)*
- *d snow depth*

CASE STUDY Stadium roof: results

Reliability index as a function of snow depth



Acceptance criteria through specified target reliability index β_T

Example: Eurocode 1990, JCSS PMC, ISO 13822, ISO 2394, fiB bulletin 80

DECISION CRITERIA

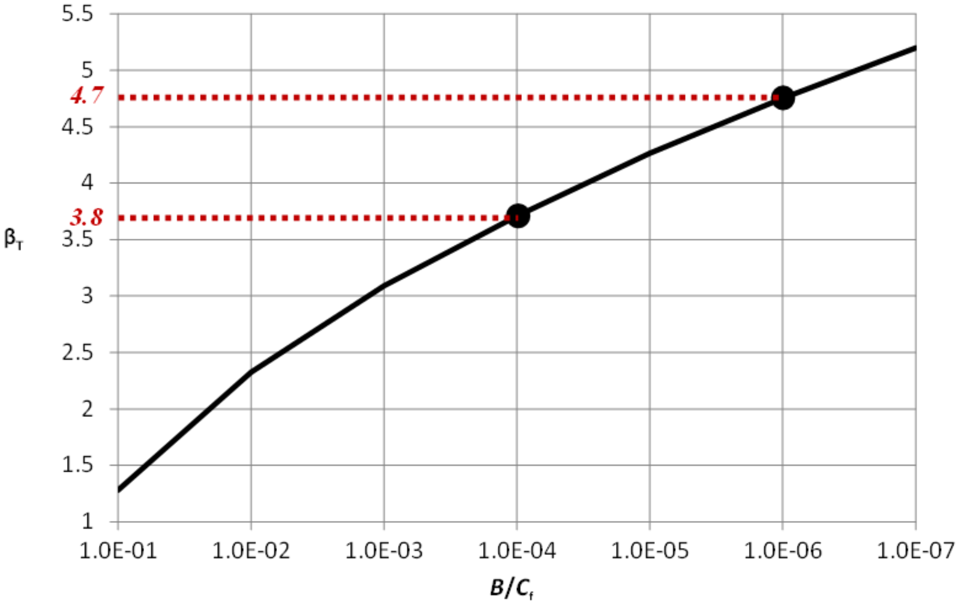
Condition: the benefit B exceeds the probability of failure dependent on snow depth $p_F(d)$, multiplied by the consequences of failure C_f:

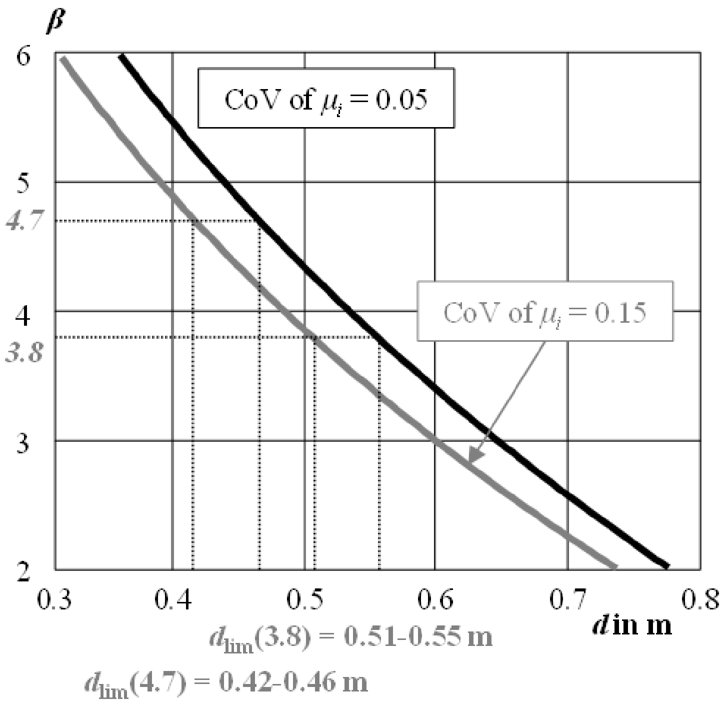
$$B \cdot [1 - p_F(d)] \approx B \geq C_f \cdot p_F(d)$$

$$\beta_T = -\Phi^{-1}\left(p_{F,eco}\right) \approx -\Phi^{-1}(B/C_f)$$

CASE STUDY stadium roof: results

Variation of the “optimal” target reliability index β_T with the ratio B/C_f



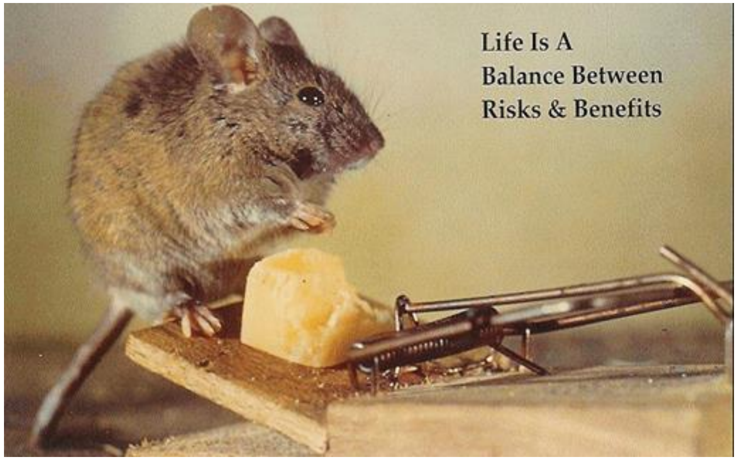


Discussion:

- Combination of human, economical and environmental consequences
- Acceptance by authorities for temporal conditions

CONCLUSIONS

- Improvement of standards on SHM, existing structures
- Real time evaluation of reliability level
- Question: placement of sensors (spatial correlation)
- Target reliability based on cost-benefit approach
- Quantification (combination) of consequences
- Application to all types of structures
- Implementation in guidelines
- First step: JCSS Probabilistic Model Code



TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

INDICATORS FOR THE PERFORMANCE ASSESSMENT OF ROAD BRIDGES THROUGH MONITORING

Peter Tanner – Eduardo Torroja Institute for Construction Science IETcc-CSIC, Madrid, Spain
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CSIC
CONSEJO SUPERIOR DE INVESTIGACIONES CIENTÍFICAS

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
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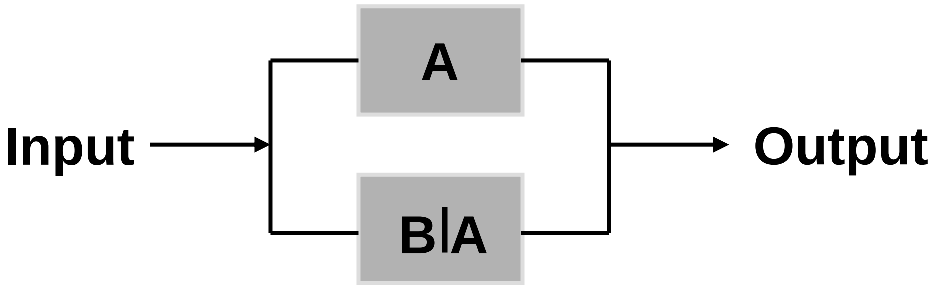

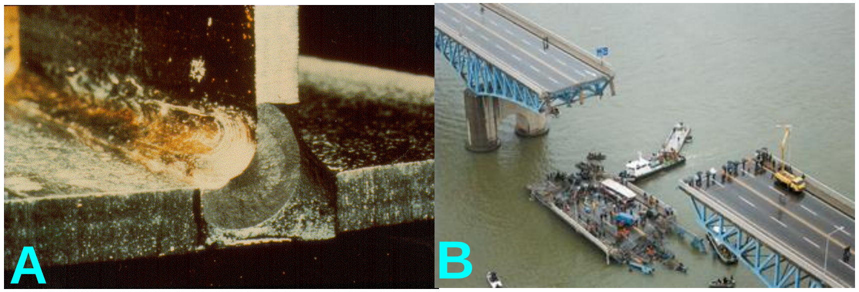

CONTENTS OF THE PRESENTATION

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- Monitoring plan
- Example
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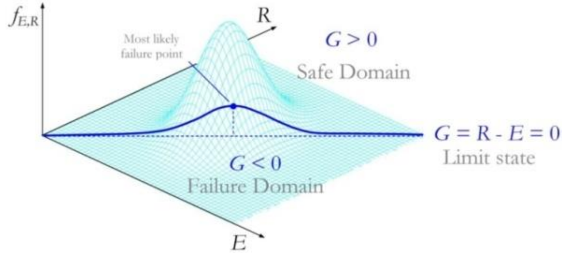
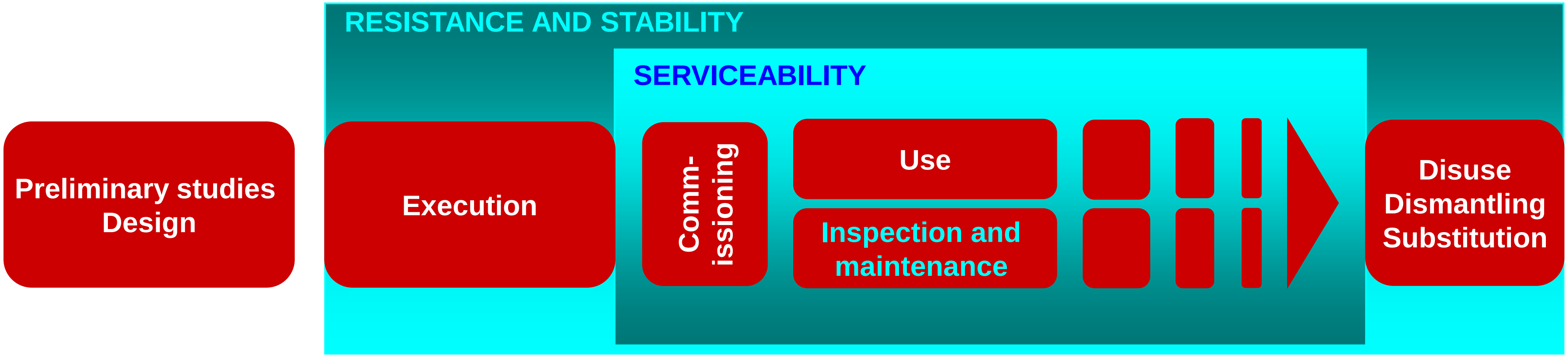


REQUIREMENTS

- Structures are to be designed, built, used and maintained in such a way that they
  - Remain fit for the use for which they are planned
  - Sustain all actions and influences likely to occur during execution and use
- Requirements can be achieved by adopting measures
  - Technical or organizational measures
  - Measures referring to all stages of the whole process
  - → E. g. risk control by means of inspections, warning system, …

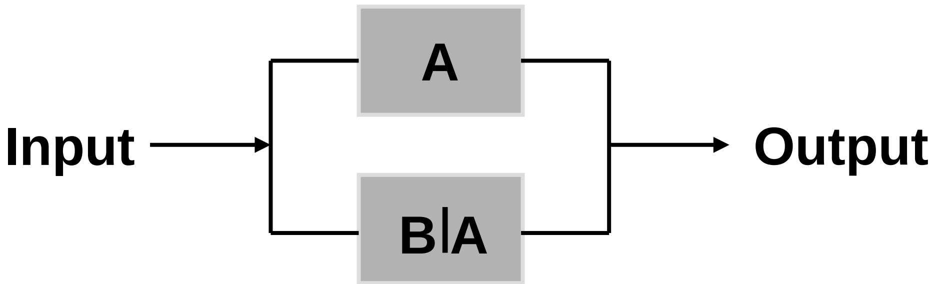


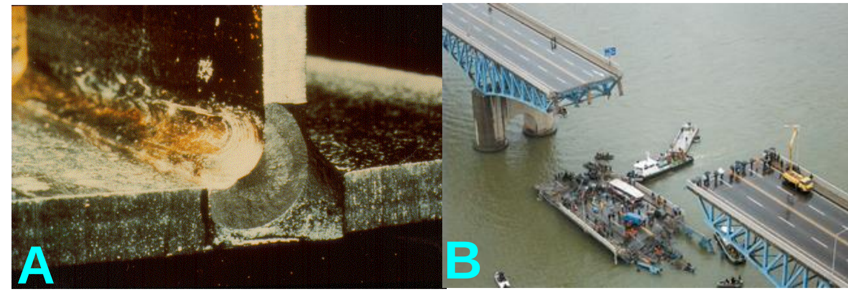
NON-COMPLIANCE OF REQUIREMENTS

– Different causes may lead to the non-compliance of any particular requirement
  - Deviations from expected actions
  - Geotechnical actions
  - Environmental influences
  - Dynamic actions
  - Deviations from expected resistance
  - Loss of load bearing capacity due to accidental actions
  - Loss of resistance due to deterioration mechanisms such as corrosion or fatigue
  - Others

→ Quantification of parameters related to such influences may provide evidence about the degree of compliance

→ Indicators, in analogy with economy or medicine





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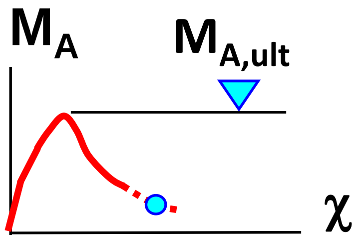
INDICATORS

– Quantification may refer to different system parameters, related with
  - Geometry
  - Materials
  - Actions and influences
  - Structural behaviour

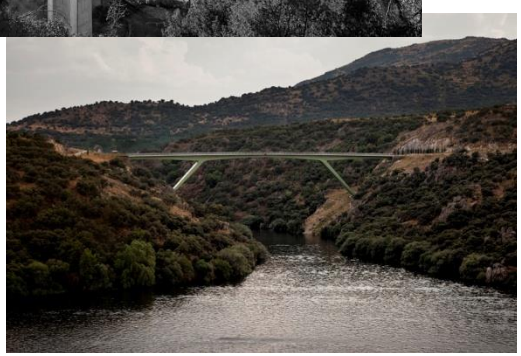


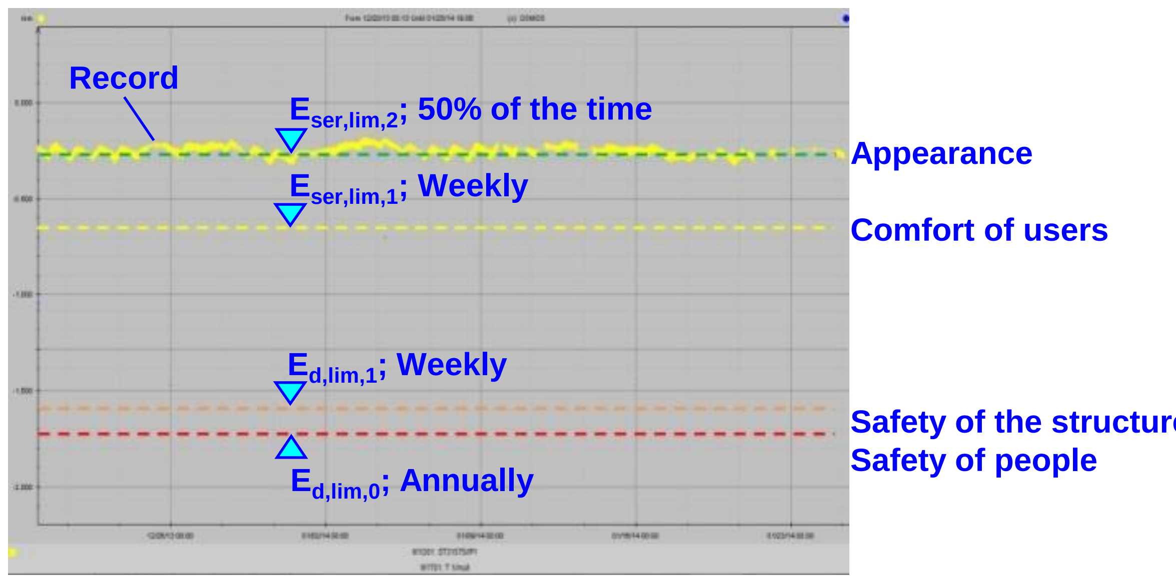
– Choice of parameters depending on the sensitivity of structural reliability to their variation

→ Most sensitive parameters depend on
  - Structural system and behaviour
  - Intended use of the structure
  - Exposure conditions
  - Materials
  - Available data acquisition system

PRACTICAL TOOLS FOR ROAD BRIDGES

- Definition of indicators related with different requirements
- Establishment of threshold values by applying normal structural analysis methods
- Determination of admissible average frequencies for outcrossing

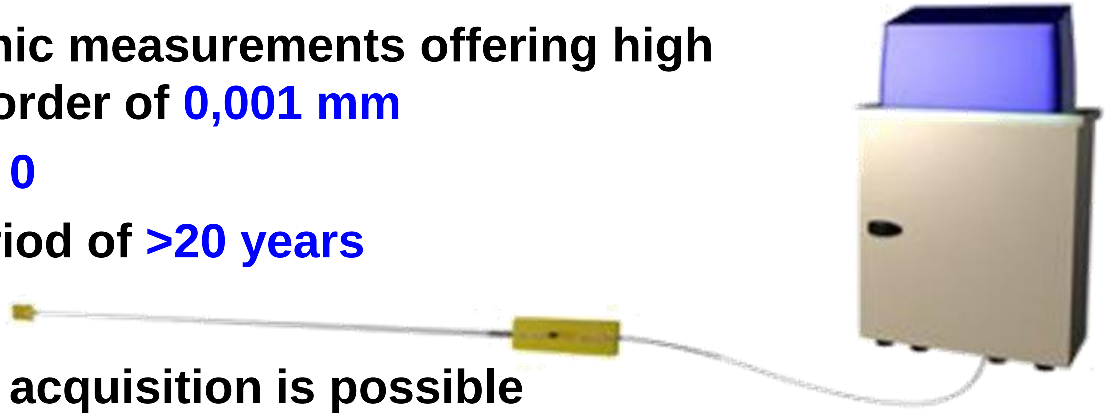




FIBRE OPTIC SENSORS

– Developments originally intended for, but not limited to, monitoring by using fibre optic sensors

– Properties of sensors measuring the intensity of light
  - Excellent signal-to-noise ratio and no electromagnetic interference
  - Static and dynamic measurements offering high precision of the order of 0,001 mm
  - No loss of origin 0
  - Long service period of >20 years



→ Advantages
  - Continuous data acquisition is possible
  - Continuous comparison with threshold values
  - Alarm in case of outcrossing
  - Adoption of measures depending on the type of non-compliance

→ Automation is possible

REQUIREMENTS, INDICATORS AND THRESHOLD VALUES

– Developed criteria depending on the failure consequences

– Material independent requirements

	Demand	Consequences	Requirement	Indicator	Threshold		
					Value $E_{ser,lim}$; $C_{ser,lim}$		Mean frequency ω_{ser}
SLS	Appearance	Reversible	Deformations	Deflection	L/700 1)		50 % of time
	Appearance	Reversible	Deformations	Strain	$E_{ser,lim,2}$		50 % of time
	Comfort	Reversible	Deformations	Deflection	L/1000 2)		Weekly
	Comfort	Reversible	Deformations	Strain	$E_{ser,lim,1}$		Weekly
	Comfort	Reversible	Vibrations	Acceleration	a_v 3)	a_h 3)	
	– Maximum				0,5	0,1	-
	– Medium				1,0	0,3	-
	– Minimum				2,5	0,8	-

	Demand	Consequences	Requirement	Indicator	Threshold	
					Value $E_{d,lim}$	Mean fr. ω_d
ULS	Structural reliability	Reversible	Safety of structure and facilities	Traffic loads 1)	$E_{d,lim,1}$	Weekly
	Structural reliability	Reversible	Safety of structure and facilities	Strain	$E_{d,lim,1}$	Weekly
	Structural reliability	Irreversible	Safety of people	Traffic loads 1)	$E_{d,lim,0}$	Yearly
	Structural reliability	Irreversible	Safety of people	Strain	$E_{d,lim,0}$	Yearly

◁ Example

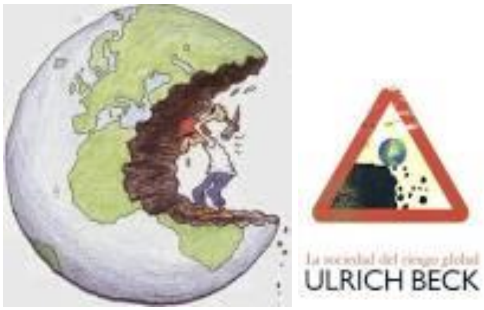
Increasing consequences in case of non-compliance

LIFE SAFETY RISK AS AN EXAMPLE

– Normally, bridges are monitored during limited period of time

→ Exposure to extreme events is less likely, compared to the remaining service period

– Starting point for the establishment of threshold values
  - Marginal Life Safety Cost principle essentially does not require consideration of life safety risk
  - However, higher failure rates than those associated with *BCP* would not be acceptable, even if they were rational
  - Implementation of *MLSC* principle by using *LQI* requires specification of acceptable life safety risk [ISO 2394:2015]



LIFE SAFETY RISK AS AN EXAMPLE

→ **Acceptable life safety risk per time unit as for permanent structures**

– **Approach**

- **Inference of life safety risk associated with *BCP* for permanent structures**
- **Adapt derived risk-based acceptance criteria to circumstances of limited period of time**
- **Basis for comparison: statistical fatality rate γ_{LR} [Faber et al. 2015]**

$$\gamma_{LR} = \frac{n_F}{N_L}$$

n_F **Expected number of lost lives per time unit due to activity**

N_L **Total number of life years per time unit exposed to activity**

→ **Condition in terms of admissible statistical fatality rate**

$$\gamma_{LR,adm,P} = \gamma_{LR,adm,T} \rightarrow \beta_{t,LR,T} \rightarrow E_{d,\lim}$$

THRESHOLD VALUES RELATED WITH STRUCTURAL SAFETY

– **Quantified parameters indicate acceptable reliability if**

$$E_{mon} \leq E_{d,\lim}$$

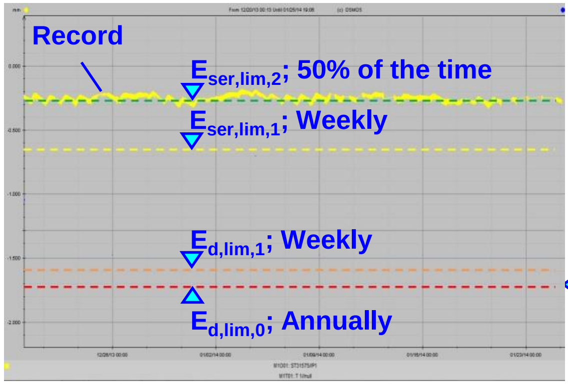
or

$$E_{mon} > E_{d,\lim} \text{ with } \omega_{mon} < \omega_d$$

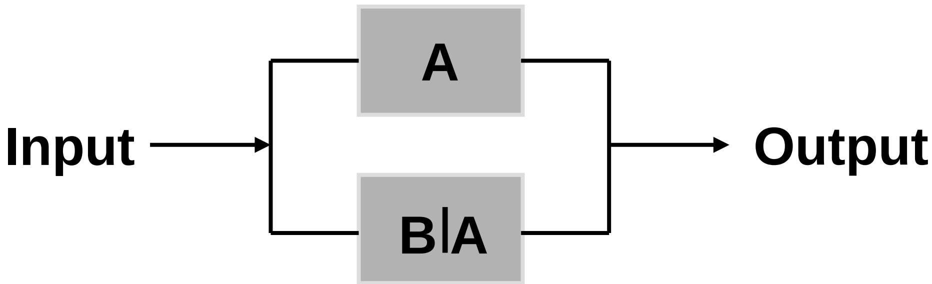
– **Example**

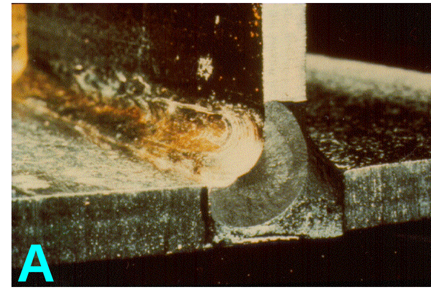
$$E_{d,\lim,0} = E\left(\begin{array}{l} \sum_{j\geq 1} \gamma_{G,j} \cdot G_{k,j} "+" \gamma_P \cdot P "+" \gamma_{Q,1} \cdot \psi_{0,1} \cdot Q_{k,1} "+" \\ "+" \sum_{i>1} \gamma_{Q,i} \cdot \psi_{1,i} \cdot Q_{k,i} \end{array}\right)$$

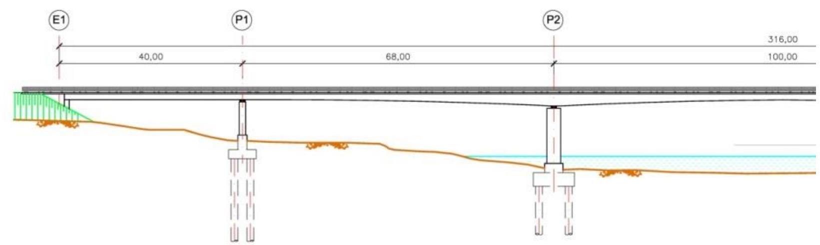
$$\omega_d : \text{annually}$$

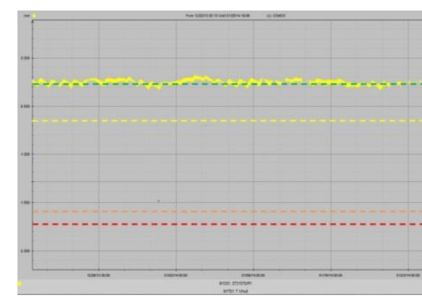


→ **Threshold values depend on stage when monitoring starts**









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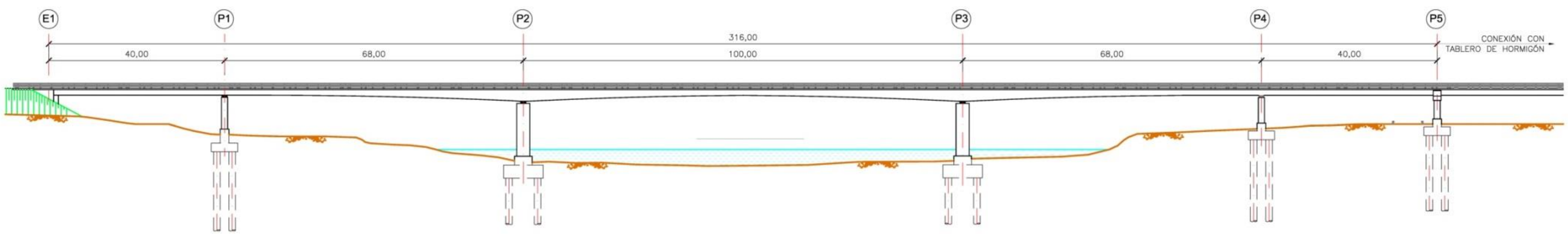
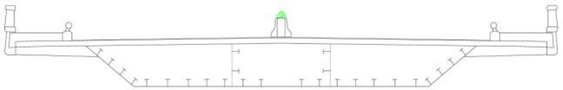
IMPLEMENTATION OF A MONITORING AND ALARM SYSTEM

- **Doubts about structural safety of an existing bridge**
- **Deck constituted by continuous five-span composite girder**
  - **Total length 316 m: 40 – 68 – 100 – 68 – 40 m**
  - **Deck width: 30,1 m**
  - **Tricellular steel box girder of varying height: 2250 to 4550 mm**
  - **0,22 m deep reinforced concrete slab with prestressing over piers**
  - **Cantilevers rest on composite ribs**
  - **Deck supported by 4 piers and 2 abutments**



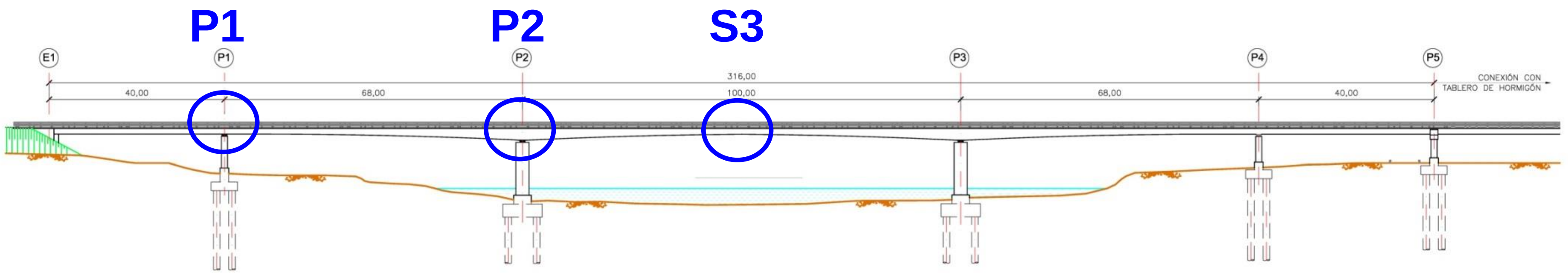


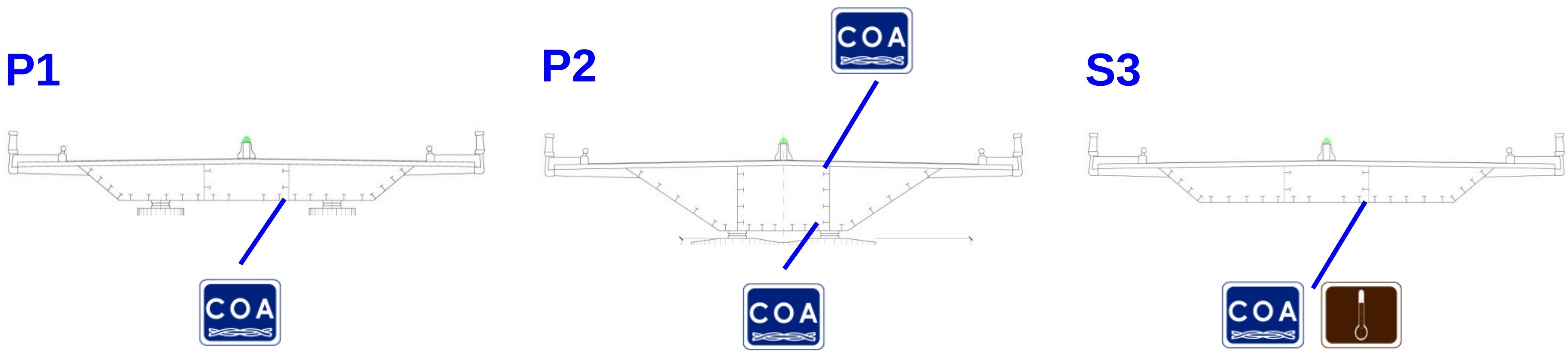
INSTRUMENTATION

– Installation of fibre optic sensors in 3 cross-sections:

→ Indicator: strains

– In addition, temperature measurement in the box girder:

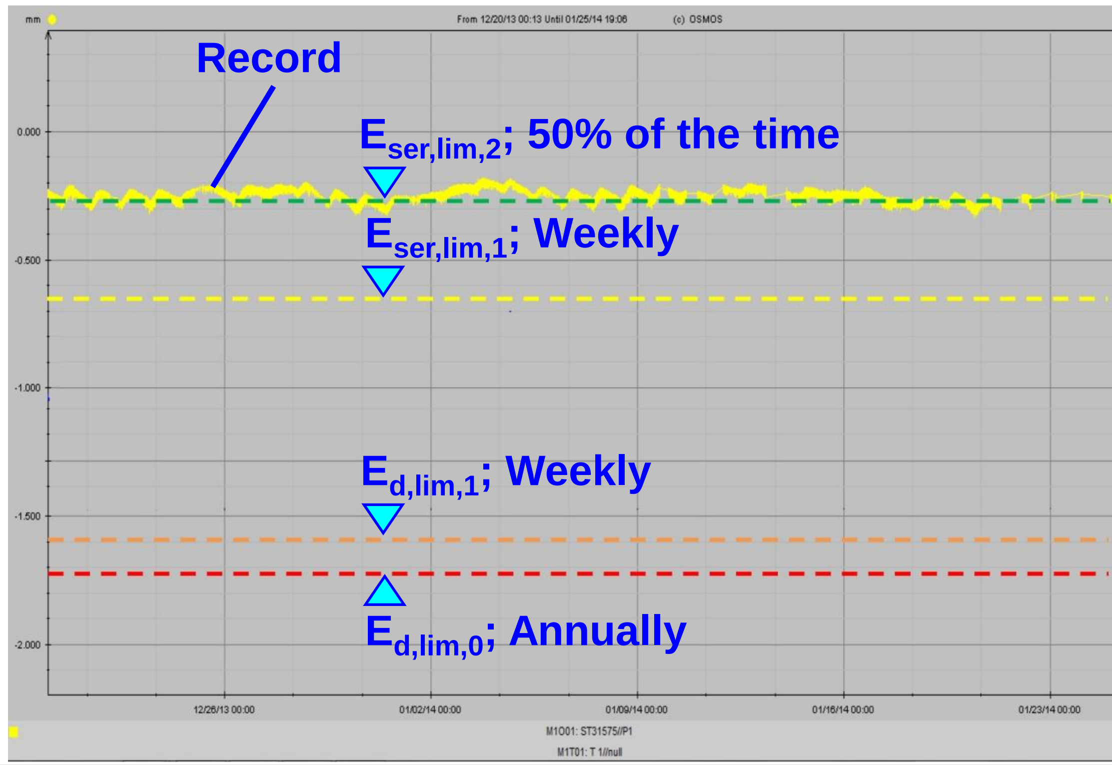




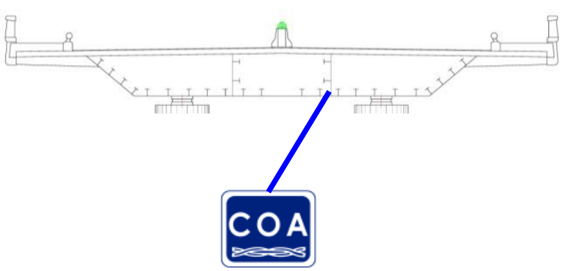
RESULTS

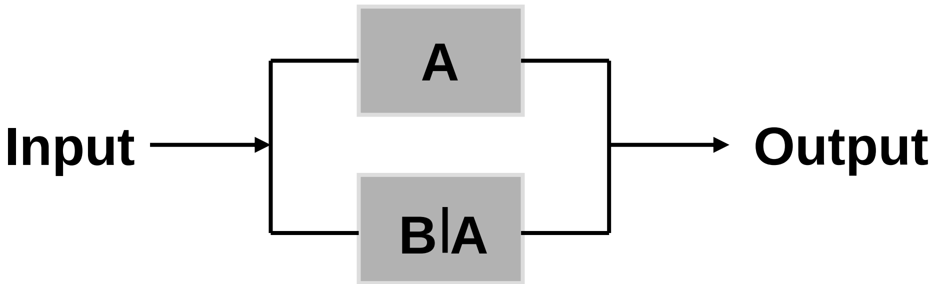
- For illustration purposes, consider results for the bottom flange over pier P1 → negative sign for compression
- Continuous record only for 2 months due to budget cuts…

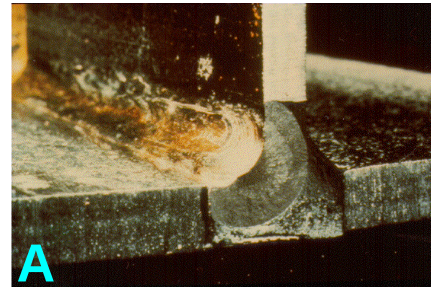
→ Signal is green



P1







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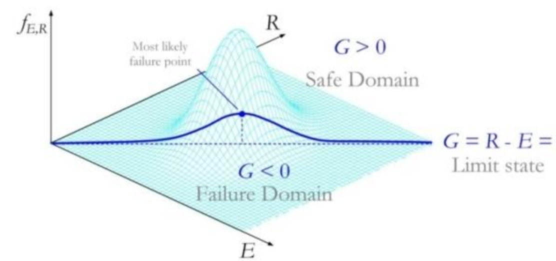

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FINAL REMARKS

– Monitoring is a rational tool for risk control

– Contributes to the optimization of operation costs for infrastructures, new or existing

→ Further calibration is needed for different indicators taking into account

- Consequences of non-compliance
- Time dependency
- Resolution of the data acquisition system

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

BRIDGE SCOUR RELIABILITY UNDER CHANGING ENVIRONMENTAL CONDITIONS

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02nd – 03rd March 2017
Zagreb, Croatia

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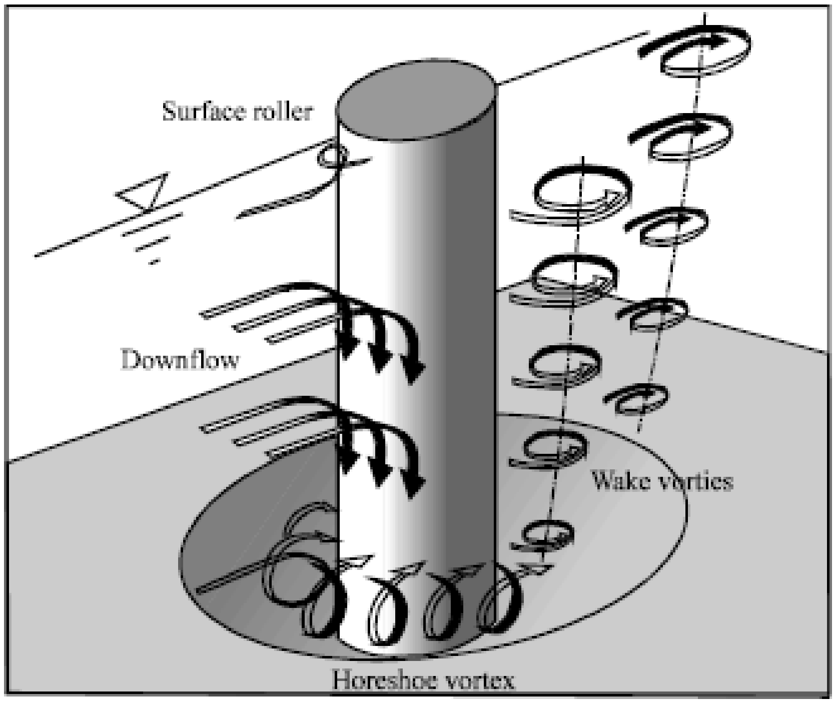
INTRODUCTION

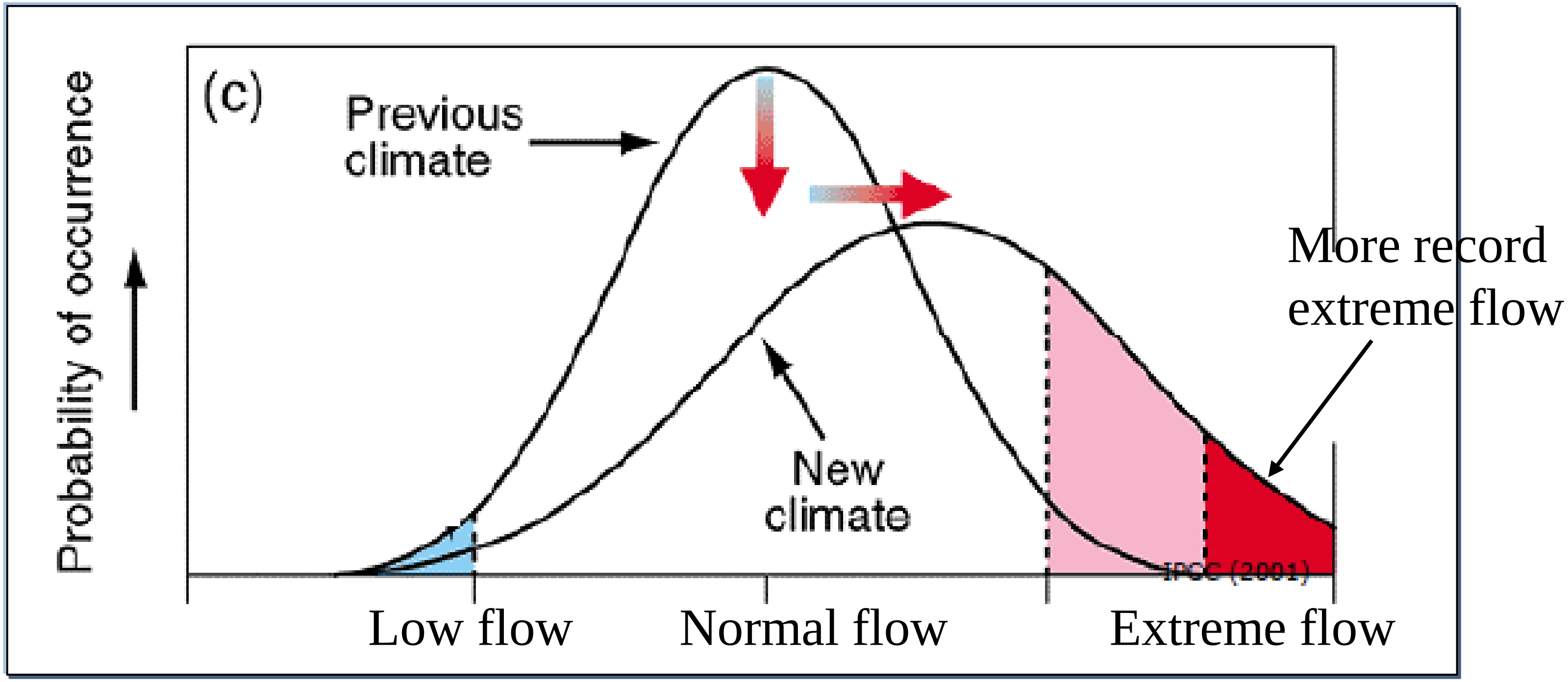
- Bridge scour is the most common cause for bridge failure!
- Most prediction methods are empirical
- Sources of uncertainty
    - ➢ River/flow characteristics
    - ➢ Effect of changing environmental conditions (climate change)
    - ➢ Unknown foundation depths
    - ➢ Empirical models
- Framework for scour reliability assessment under changing conditions

BRIDGE SCOUR FAILURES

BRIDGE SCOUR



CLIMATE CHANGE



Statistical variability of climate change

PROBABILISTIC FRAMEWORK

Maximum scour depth (HEC-18) $$y_{\max} = 2 y_0 K_1 K_2 K_3 K_4 \left(\frac{D}{y_0} \right)^{0.65} F_0^{0.43}$$

y_0: depth of the flow upstream of the bridge pier (BD, 2012)
K_1, K_2, K_3, K_4: coefficients for pier shape, angle of attack, streambed conditions and river bed material size
D: pier diameter
F_0: Froude number

$$F_0 = \frac{V}{(g\, y_0)^{0.5}} \qquad y_0 = \left(\frac{nQ}{B s^{1/2}} \right)^{3/5}$$

Q: river flow
B: river width
n: Manning's coefficient
s: longitudinal slope of the channel.

PROBABILISTIC FRAMEWORK

- Limit state for local scour of a bridge pier

$$G(t) = D_F - y_{\max}(t) = D_F - \left[2y_0(t)\lambda_{sc}K_1K_2K_3K_4\left(\frac{D}{y_0(t)}\right)^{0.65} F_0(t)^{0.43} \right]$$

- Annual probability of failure

$$p_f(t) = P[G(t) \leq 0]$$

- Cumulative probability of failure

$$p_f(0, t_L) = 1 - \prod_{i=1}^{k} \left\{ 1 - P\left[G_{t_i}(t) \leq 0\right] \right\}$$

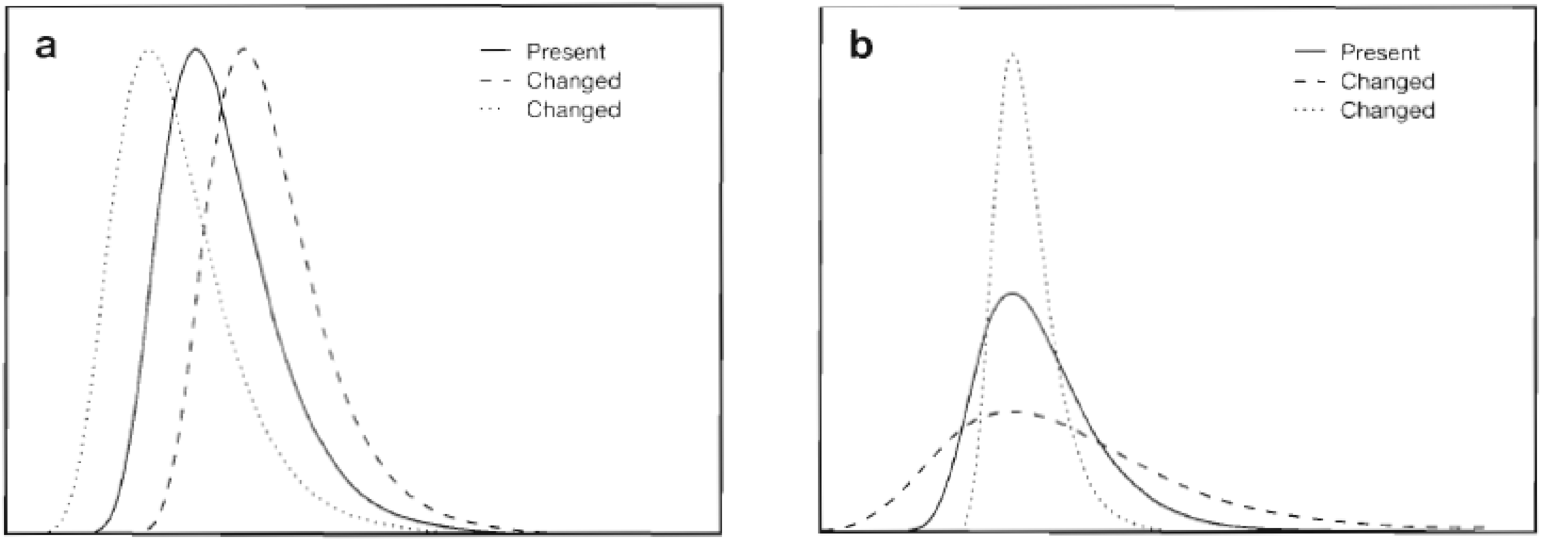
- Probability of failure estimated through Monte Carlo simulation with 2×10^6 samples

BRIDGE CASE STUDY

Random variables

Variables		Mean	COV	Distribution	Reference
River	Width B (m)	65	0.05	Normal	Assumed
	Streambed conditions (K_3)	1.1	0.05	Uniform	NCHRP (2003)
	Bed material size (K_4)	1.0	-	Deterministic	Assumed
	Slope s	0.0032	0.05	Lognormal	Assumed
	Manning's coefficient n	0.035	0.28	Lognormal	NCHRP (2003)
Bridge piers	Foundation depth D_F (m)	4.5	-	Deterministic	Assumed
	Pier nose shape (K_1)	1.0	-	Deterministic	Assumed
	Angle of attack (K_2)	1.0	-	Deterministic	Assumed
	Pier width, D (m)	2	0.05	Normal	Assumed
Scour eqn.	Modelling uncertainty, λ_{sc}	0.55	0.52	Normal/ lognormal	NCHRP (2003)

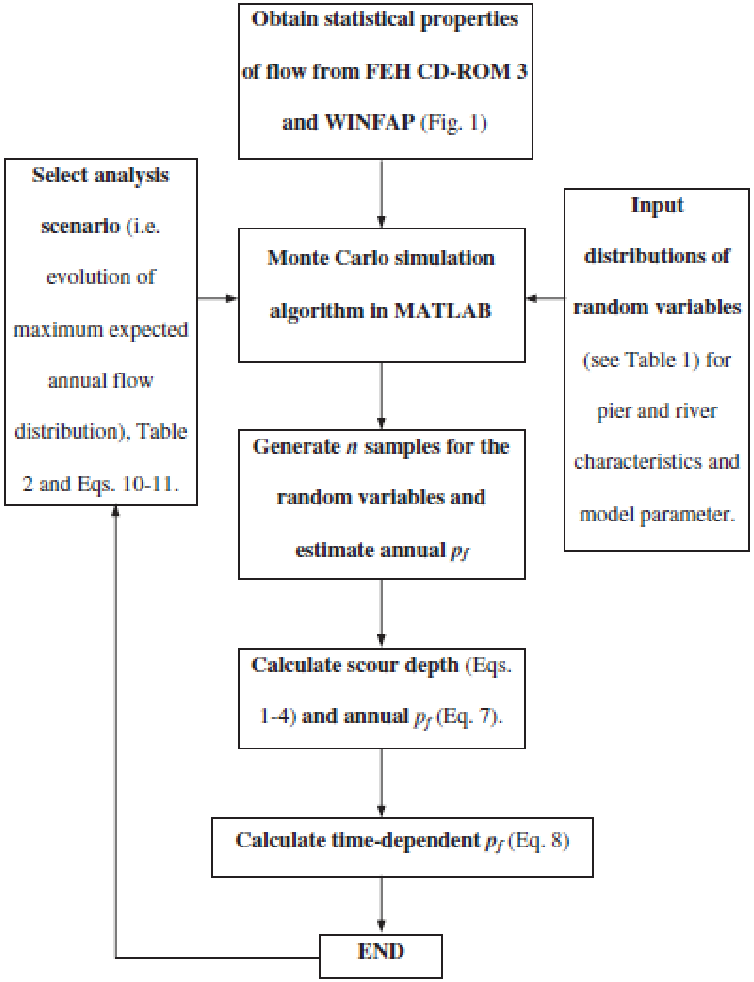
BRIDGE CASE STUDY



- Scour assessment based on max annual flow, Q, with return period T=2 years
- Q obtained from annual maxima series of pooling group flood data (FEH)
- Expected annual flow best described by Generalised Extreme Value (GEV) distribution
- Flow events in different years assumed to be independent
- Parametric study to quantify changing conditions through changes in distribution parameters

BRIDGE CASE STUDY

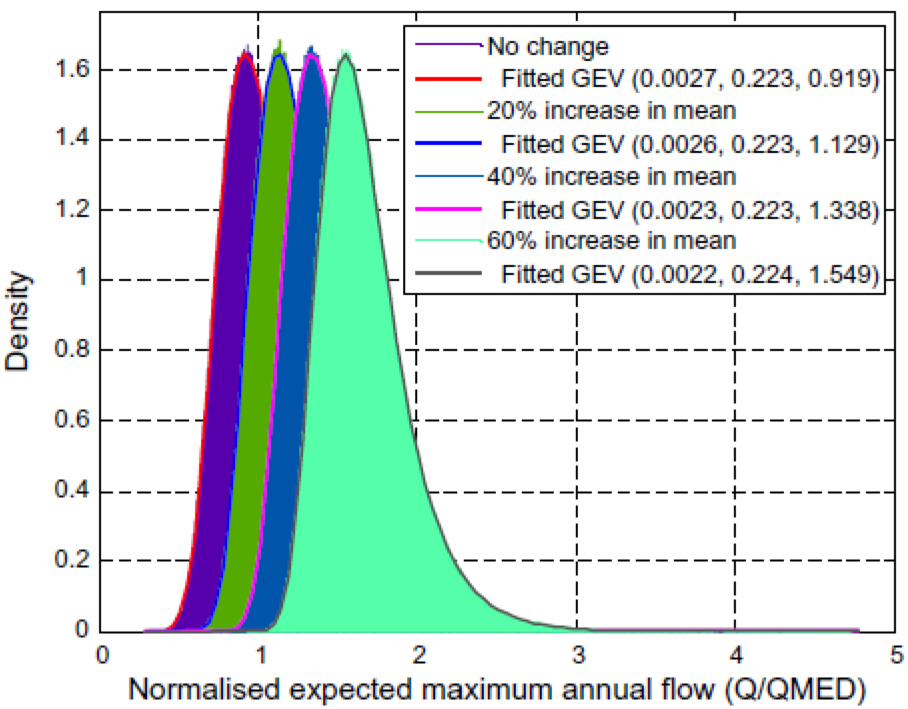
Probabilistic framework



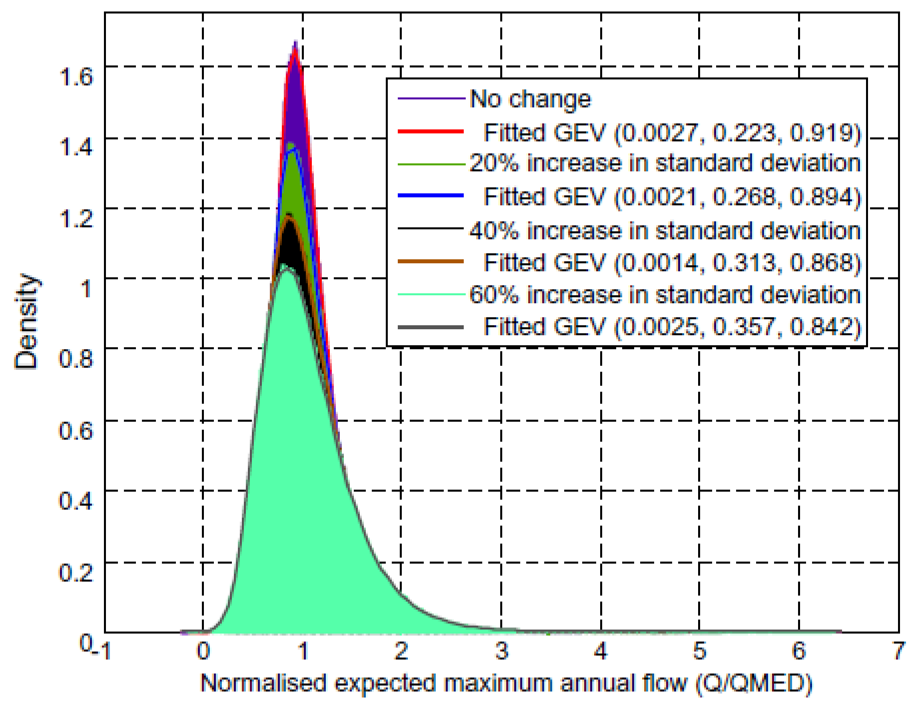
RESULTS

Maximum annual flow distribution

Increasing mean

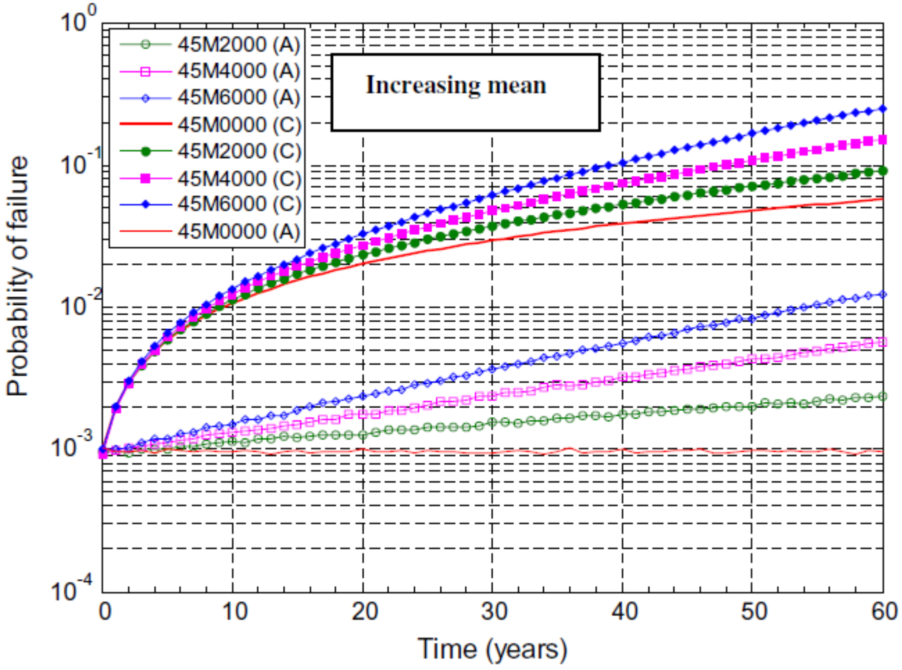


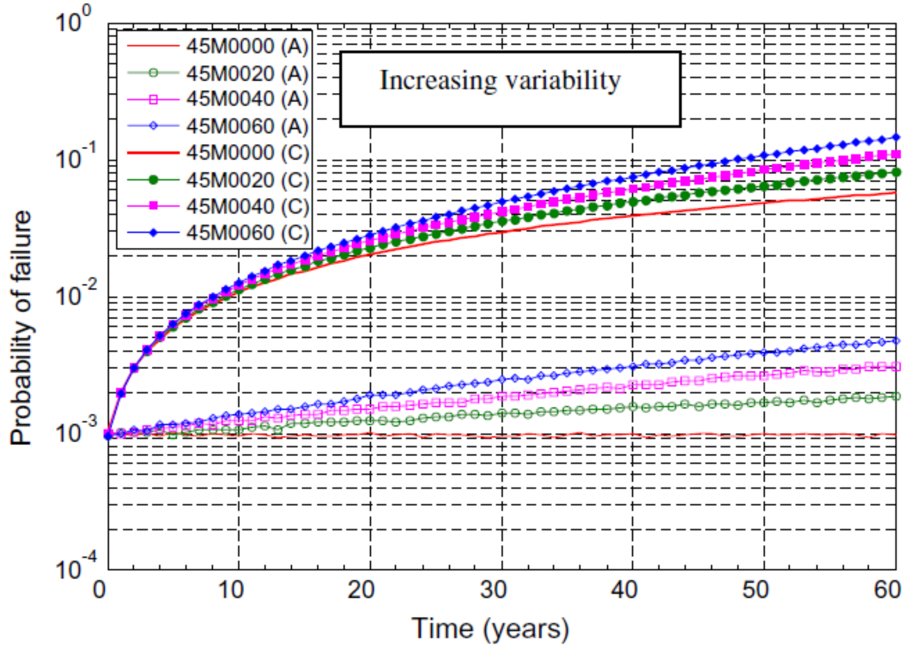
Increasing standard deviation



RESULTS

Effect of river flow characteristics



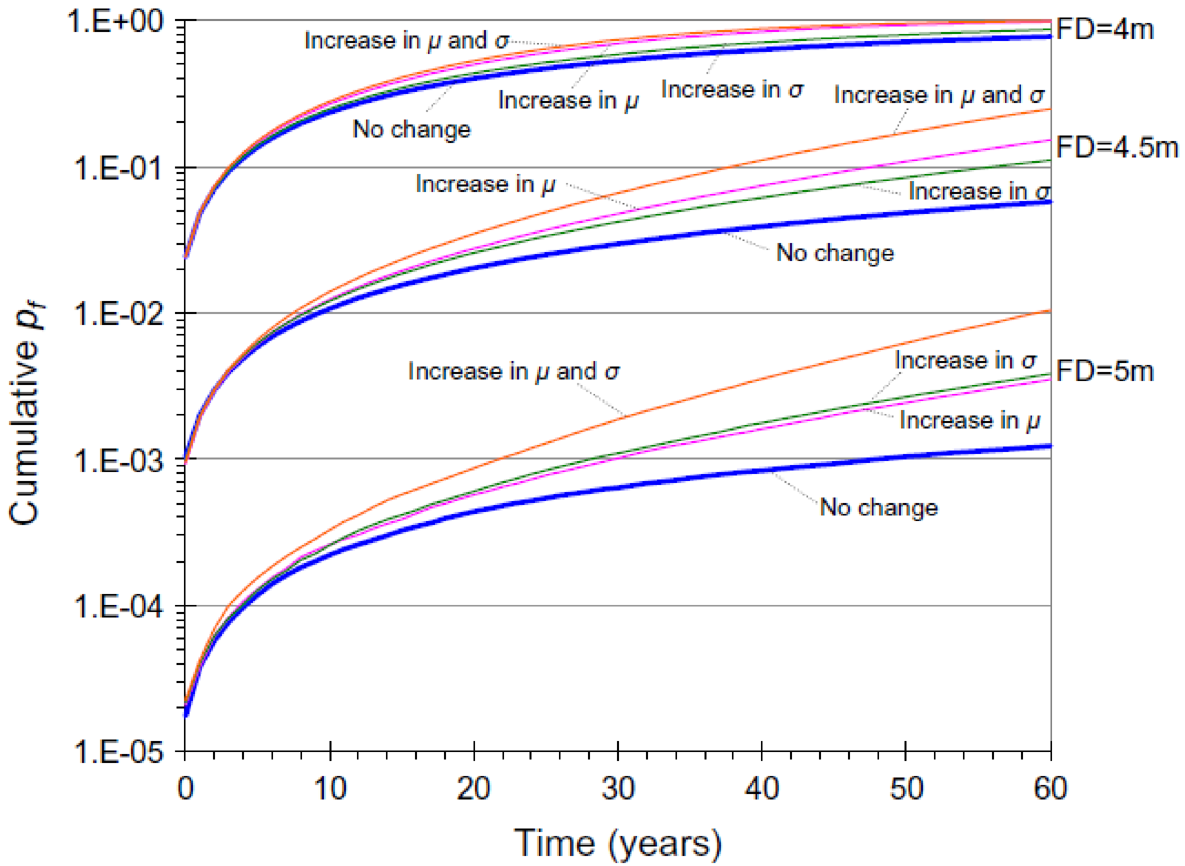


Parametric analyses: 20%, 40% and 60% increases over a 60-year period

A: Annual; **C**: Cumulative probabilities of failure

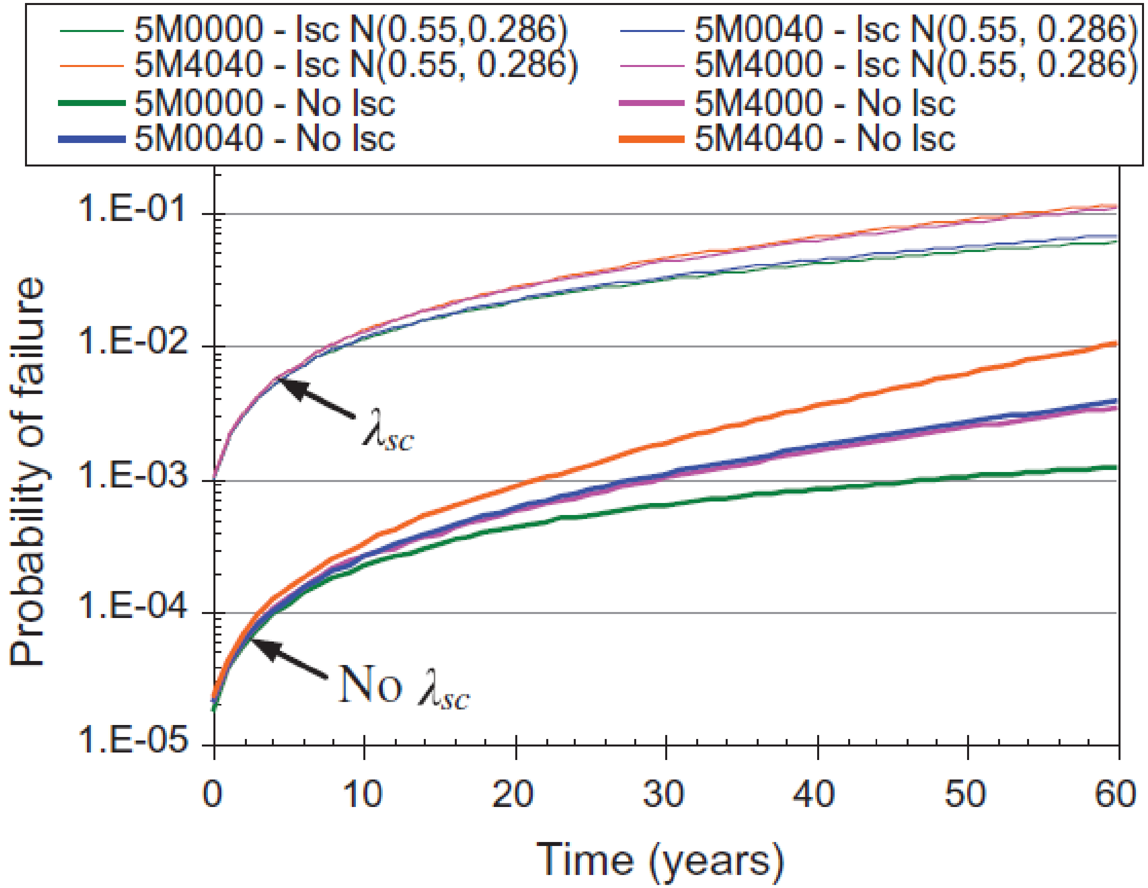
RESULTS

Effect of foundation depth



RESULTS

Effect of scour modelling uncertainty (FD = 5m)



CONCLUSIONS

- Framework for scour reliability assessment under changing environmental conditions.
- Effects of gradually increasing mean or variability of the expected maximum annual flow on the predicted probabilities of scour failure were found to be relatively small for the initial 10-15 years and becoming progressively higher thereafter.
- Foundation depth and scour modelling uncertainty was found to have a significant effect on scour failure probability.
- Case study results indicate that the effects of changing flow characteristics on the scour failure probabilities reduce with reducing foundation depths.

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

BRIDGE SMS: Intelligent Bridge Maintenance and Management System

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[3] School of Engineering, University College Cork, Ireland

[4] University College Dublin and MaREI, Ireland

02nd – 03rd March 2017
Zagreb, Croatia

1. BACKGROUND TO THE PROJECT IDEA

2013-2016: Design of Bridge Scour Management Measures and Repair works

- Irish Rail, Croatian Railways

2011-2013: Bridge Scour Management Programme - Irish Rail

- Pilot Scour Inspections for 20 bridges around Dublin
- General Scour Inspections for 105 railway bridges
- Detailed Scour Inspections for 25 railway bridges

2009-2014: Inspections of bridges for Croatian Railways

- Collapse of bridge on the Sava River in 2009
- Detailed inspections and recommendations for 20 bridges

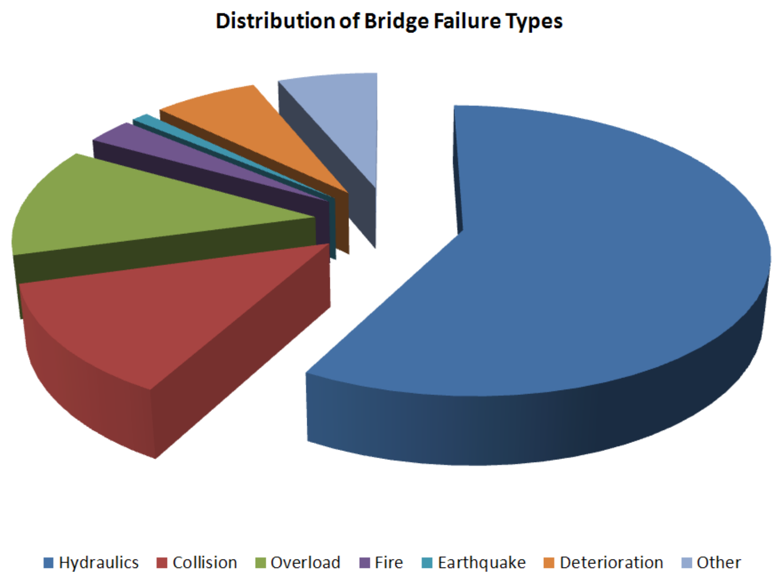
2009-2012: Malahide Viaduct Reinstatement

- Hydrology, Hydraulic, Morphology, Environment
- Physical model weir + Mathematical model estuary

Experiences from USA

Biezma 2007 study

- The scour is the most common cause of bridge collapses in the USA.
- A 83% of all bridge collapses are due to the natural causes, and the bridge scour (flooding) was the cause at 58% of bridge failures out of all natural causes.



Failure Type	Number of Failures	Percentage
Hydraulics	1027	58%
Collision	224	13%
Overload	220	12%
Fire	52	3%
Earthquake	19	1%
Deterioration	116	7%
Other	111	6%
Total	**1769**	**100%**

Activities in Bridge Management Programme

Inspections and Decisions

- Routine monitoring/maintenance.
- Inspections (structural, scour, special investigations).
- Training for inspections (LA engineers, inspectors).

Repairs of damages

- Design of repairs by experts (studies, modelling).
- Repair works (structural, scour protection).

Software System for bridges

- implement, fill, use, maintain, etc.

Reporting on past activities and planning of future activities (finance, implementation, …).

Experiences from work with Irish Rail/Croatian Railways

Limited budgets available to undertake all necessary activities.

- inspections, monitoring, repairs
- software system
- trainings, reporting, planning

Limited time available to collect all required data for inspections.

Difficult decisions must be taken:

- How and where limited resources should be invested?
- Where investment is not justified?

Currently no of-the-shelf solution for all aspects for efficient management of Bridge Scour Risk for the end-user needs.

2. GENERAL ON BRIDGE SMS PROJECT

“Intelligent Bridge Assessment Maintenance and Management System” (BRIDGE SMS) is an EU Marie Curie FP7 project, under the Industry Academia Partnerships and Pathways (IAPP).

- Duration: 2015-2018
- Budget: € 1.418.821

Project partners:

University College Cork, Ireland
Coláiste na hOllscoile Corcaigh

Cork County Council
Comhairle Contae Chorcaí

arctis

Infraestruturas de Portugal
Ligamos destinos

Supporting institutions:

Universidade do Minho
Instituto de Educação

Comhairle Contae Átha Cliath Theas
South Dublin County Council

Dublin City
Baile Átha Cliath

Connecting industry and academia

END-USER - Bridge owner and operator

- Expertise in day to day management of bridge structures over water
- End User perspective
- Test-bed for new system development

Intelligent Bridge Management System

BRIDGE-SMS

ACADEMIC EXPERTS

- River hydrology & hydraulics
- River & bridge modelling
- Scour protection design & installation
- Risk modelling and quantification
- Foundation and Structural Engineering

SOFTWARE DEVELOPERS

- Software platform experts
- New software system integration
- Open Source experts

BRIDGE SMS – Work Packages

WP 1 Project Management	Website, Coordination, Quality Assurance, Reporting
WP 2 Technical Research	T.2.1 Identification of input bridge and scour data sets. T.2.2 Development of methodologies and tools for Bridge Scour Management System. T.2.3 Development of requirements for Intelligent Decision Support System.
WP 3 Development of Bridge Scour Management System	T.3.1 Development of Inventory Module. T.3.2 Development of Scour Inspections Module. T.3.3 Development of Maintenance and Repair Module. T.3.4 Development of Monitoring and Predictions Module. T.3.5 Development of Decision Support Module.
WP 4 Knowledge Transfer and Training	T.4.1 Appointment of Transfer of Knowledge Director. T.4.2 Training Seminar organised by seconded staff in host organisations. T.4.3 On-going training activities and debriefing. T.4.4 Organization of internal workshops at key dates. T.4.5 Develop sample pilot as a means to educate and train staff-simulator training.

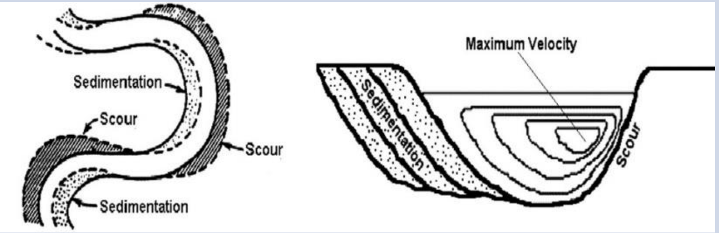
3. TECHNICAL RESEARCH

Three types of scour at bridges

a) General scour

Lateral instability

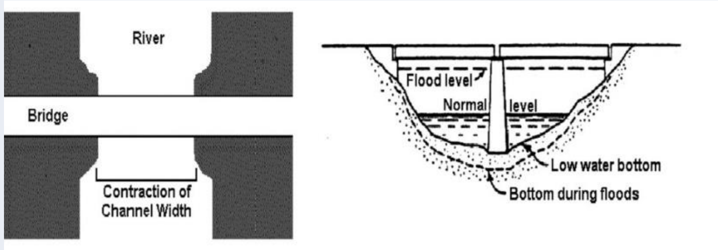
Vertical instability



b) Constriction scour

constriction of flow

due to bstructure

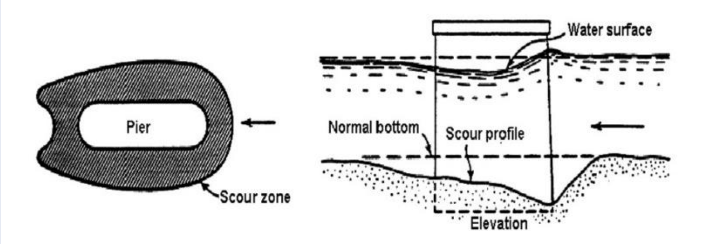


c) Local scour

bridge abutments

bridge pier/s

bridge walls



Modules of Eirspan Inspection – in operation by DOT/CCC

Routine Inspection

- Annually by Local Authority engineer

Principal Inspection

- Visual inspection by bridge engineer
- Condition rating 0-5
- Inspection interval 1-6 years

Special Inspection

- Underwater → NRA was using UK standard (BA 74/06, BD 97/12)
- Structural testing
- Investigations

Position of Scour Risk Assessment in Standards

Included (partially) in principal inspection

- US DOT, US FHWA
- Initial screening only (limited data collection)

Completely part of special inspection

- NRA Eirspan
- UK Highways Agency

Standards for Scour Risk Assessment

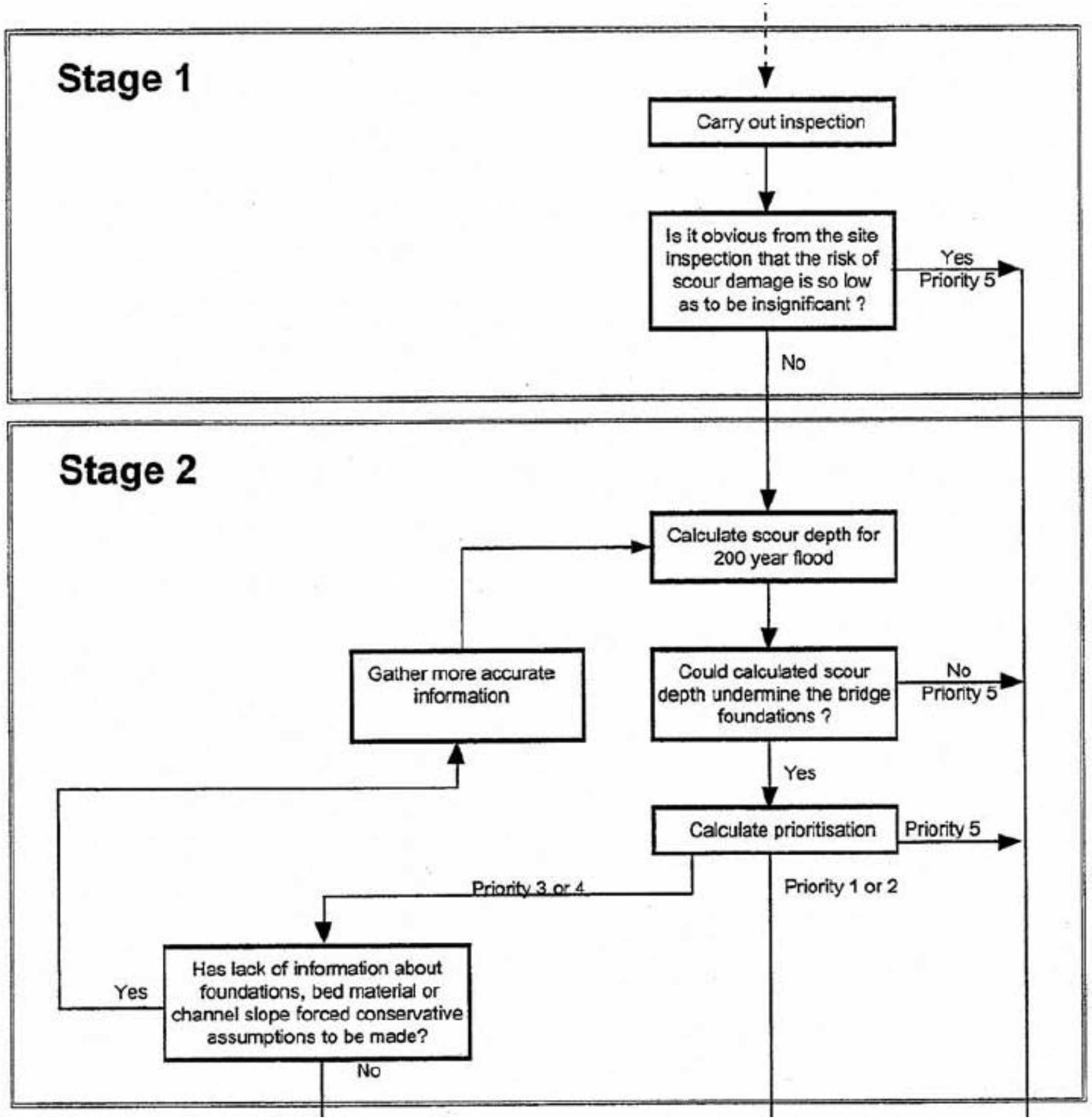
- Design Manual for Roads and Bridges (BA 74/06, BD 97/12)
- US Forest Service (Colorado method)
- USACE HEC 18, 20, 23

UK Highways Agency

BA 74/06

BD 97/12

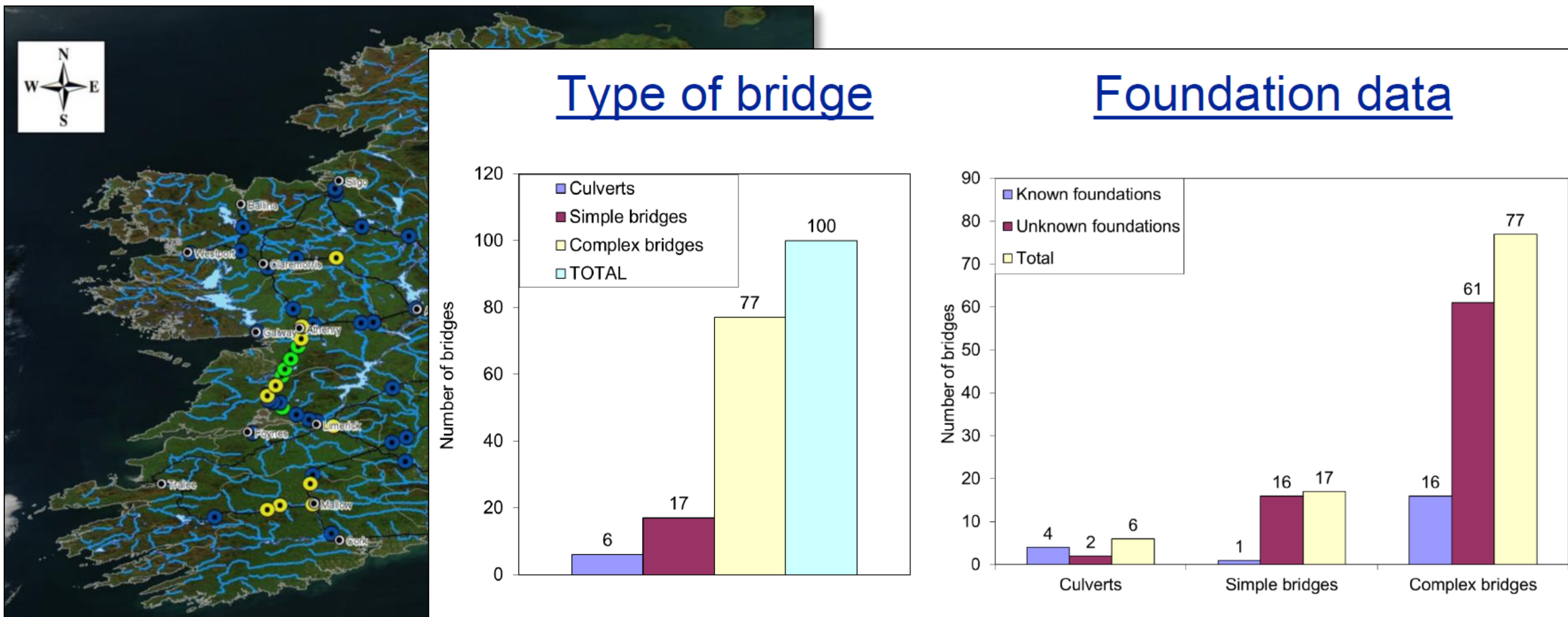
Overall scour assessment methodology



Application of Standards for Scour Inspections

Bekić D., et al. (2012):

- „Experiences from Bridge Scour Inspections by Using Two Assessment Methods on 100 Railway Bridges", CETRA 2012, Dubrovnik.



General on Scour Risk in Standards

Scour Risk Assessment

- Some standards → Not enough input data
- Requirement for too many input data (increase in time and costs)

Key elements for scour safe bridges

- Input data = Methodology of Bridge Scour Inspection
- Collection and storage of structured data

Out of 1,244 bridges in the Cork County (Ireland)

Liam Dromey et al., 2016

Geometry

- Single span = 770 bridges (62%)
- Span length <3.0m = 350 bridges (28%)

Design and materials

- Arch, one or more spans = 60%
    - *stone masonry 95%*
- Simple span, constant cross-section = 30%
    - *in-situ RC 55%, stone masonry 24%*

Guidelines for Inspection of Bridges (Draft v0.1)

Inspections:

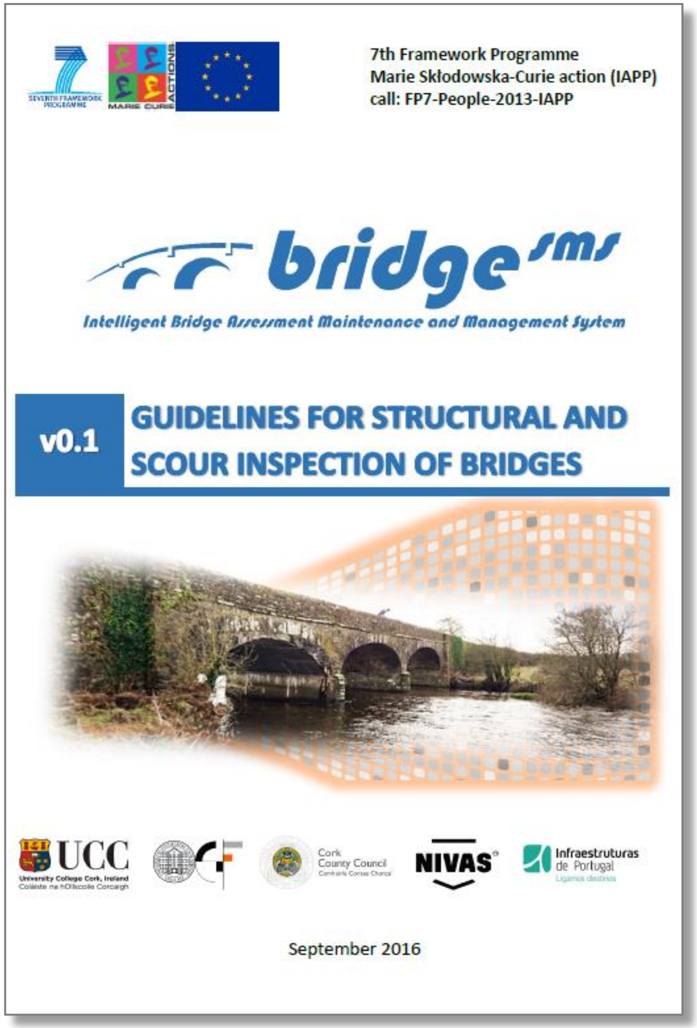
- Structural
- Scour

Two different Condition Rating:

- Structural, StCR:0 to StCR:5
- Scour, ScCR:0 to ScCR:5

Two levels of inspections:

- Level 1 General Inspection
- Level 2 Detailed Inspection
- Investigation, Monitoring and Design (Level 3)



7th Framework Programme
Marie Skłodowska-Curie action (IAPP)
call: FP7-People-2013-IAPP

bridge sms

Intelligent Bridge Assessment Maintenance and Management System

v0.1

GUIDELINES FOR STRUCTURAL AND SCOUR INSPECTION OF BRIDGES

September 2016

Guidelines for Inspection

Implementation of the Guidelines

- Training course for Cork County Council engineers
- Testing of the Gudelines for pilot and other bridges in Cork County

Further testing

- bridges in Croatia
- bridges in Portugal

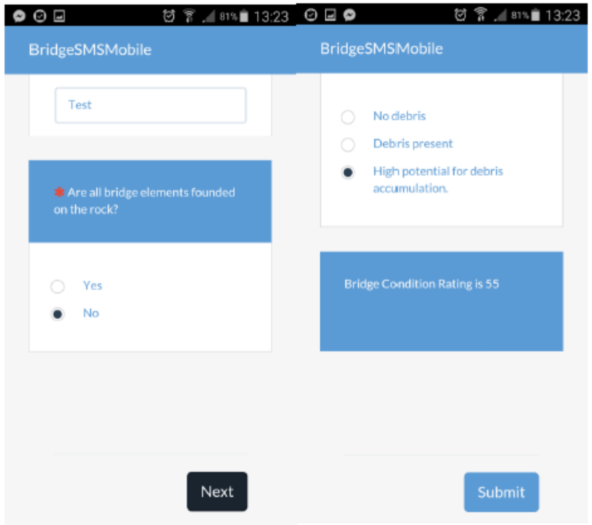
4. BRIDGE SMS – KEY FEATURES

Inspection Management

- Standardised forms for bridge inspections.
- Geolocation, data and pictures can be added during inspections.
- Information from performed/future inspections.
- Planning and budgeting.

Mobile Technology

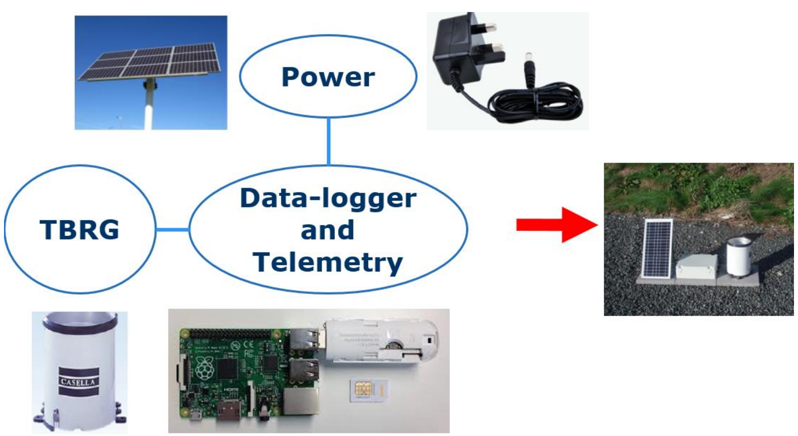
- Supports Android and iOS
- Users can access BRIDGE SMS data and forms when network or Wi-Fi connection is non-existent.



Key Features: Monitoring

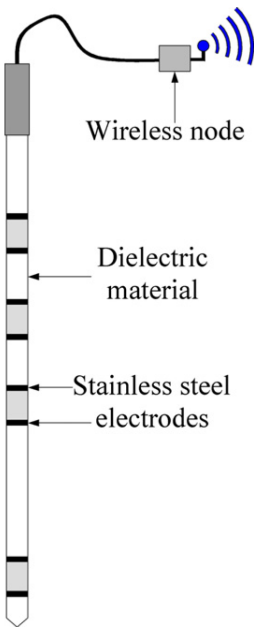
Environmental Monitoring

- Real time monitoring (rainfall, water levels, soil moisture, temperature
- Information on primary screen about the current status, changes and trends.

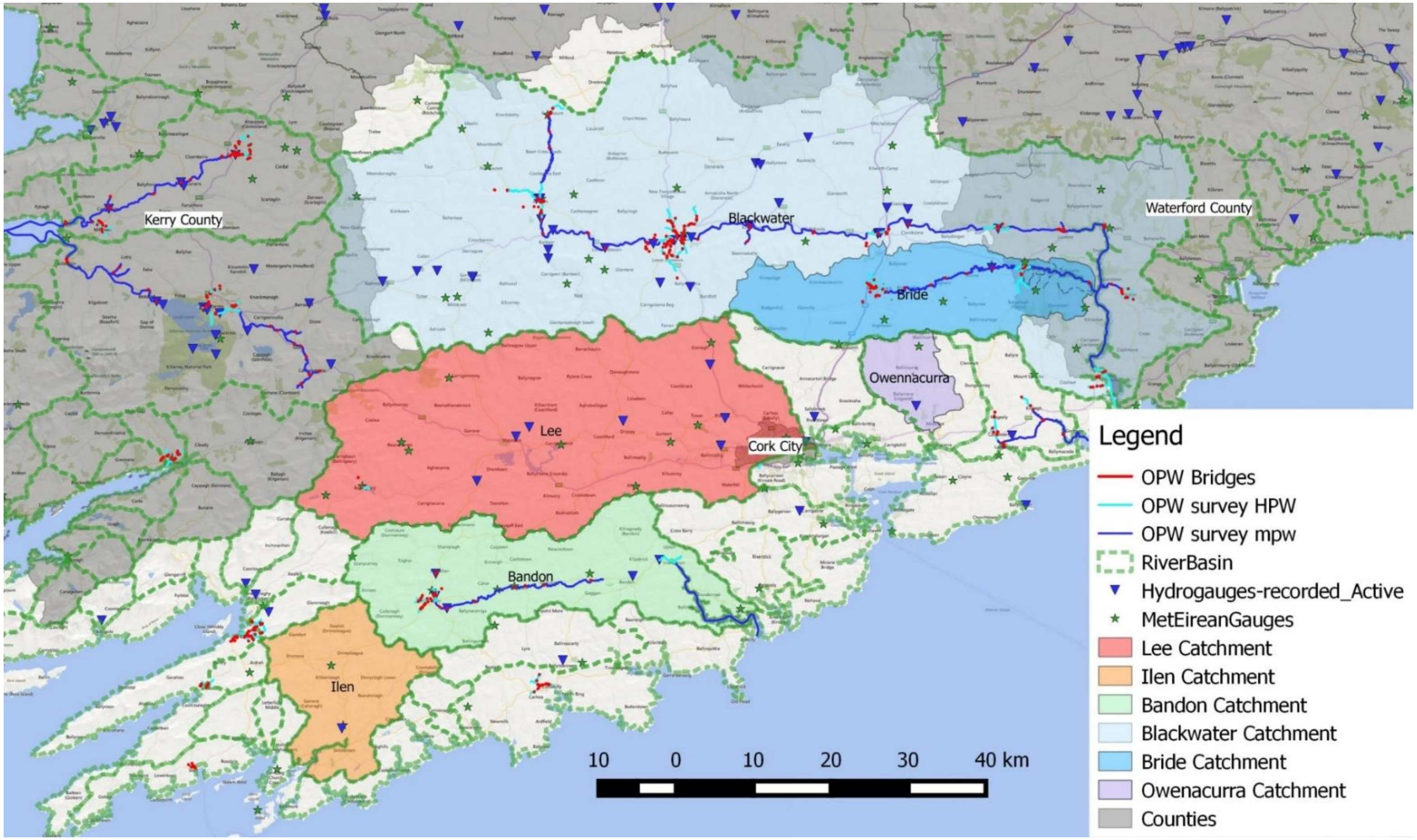


Bridge Monitoring

- Scour depth monitoring
- Structural monitoring
  - Vibration Based Health Monitoring
  - Forced Dynamic Assessment

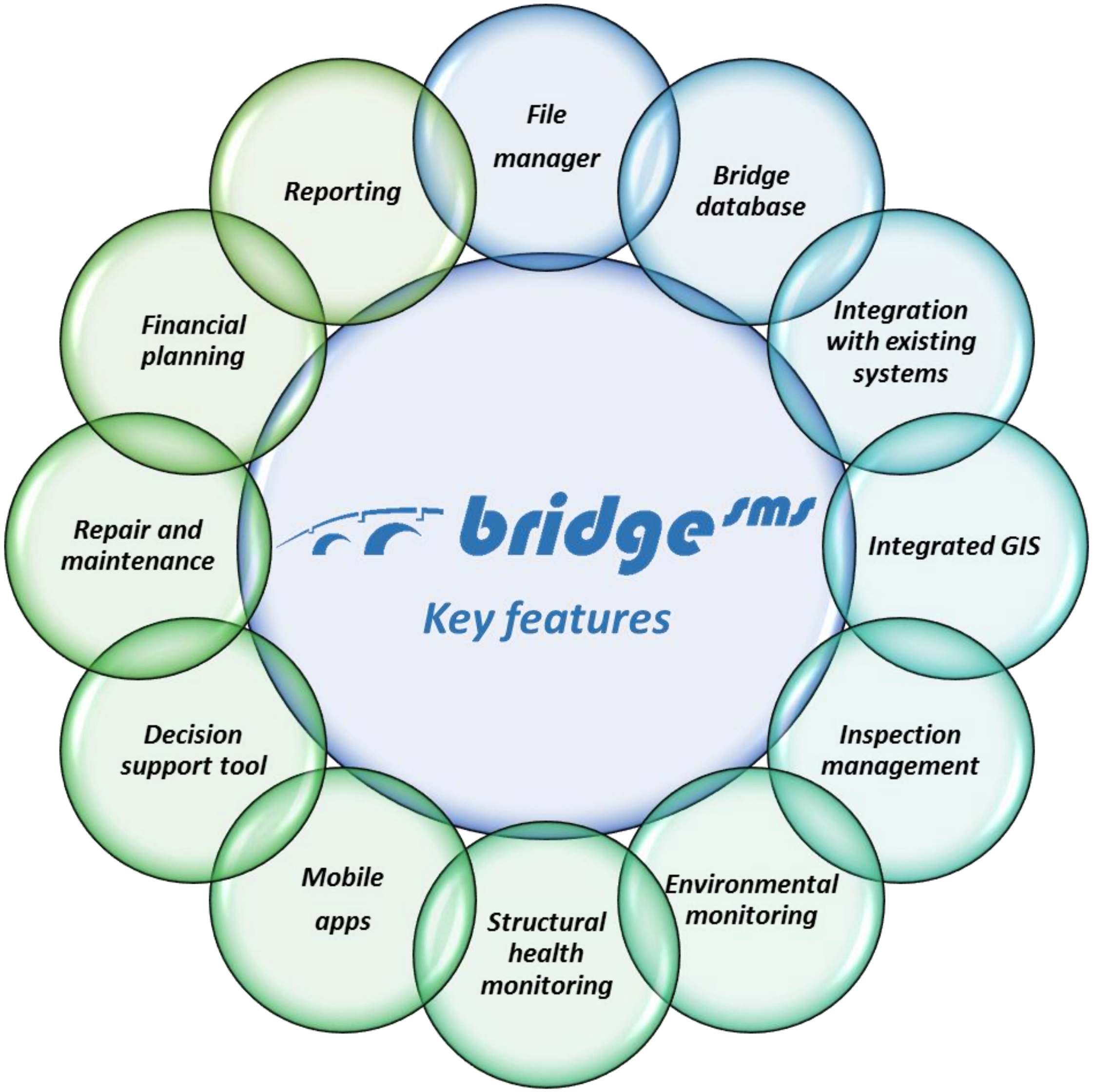


Key Features: Flood Forecasting - BRIDGE SMS is part of EFAS



Key features

Modular system



5. CONCLUSIONS

This project is developing a novel cloud based Bridge Management System.

The system includes:

- Repository of data in different levels, including archive data.
- Collects and integrates real-time data from environmental (rainfall, water levels) and bridge monitoring (structure).
- Rapidly notifies relevant personnel of possible maintenance and failure issues (tablets).
- Advice for decision making.
- Support for efficient and easy inspections.
- Prioritize and optimize the operational and maintenance budget spend.
- Specialist training for engineers and inspectors.

Thank you for your attention!!!

Damir Bekic[1], Igor Kerin[2], Panagiotis Michalis[1], Eamon McKeogh[3], Paul Cahill[2], Vikram Pakrashi[4]

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[2] MaREI, University College Cork, Ireland

[3] School of Engineering, University College Cork, Ireland

[4] University College Dublin and MaREI, Ireland

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

NON GAUSSIAN MULTI-PARAMETER BAYESIAN ESTIMATION THROUGH A MECHANICAL EQUIVALENT OF LOGICAL INFERENCE

Denise Bolognani– University of Trento (Italy), Princeton University (USA)
Andrea Verzobio – University of Strathclyde (UK)
Carlo Cappello – University of Trento (Italy)
Daniel Tonelli – University of Trento (Italy)
Branko Glisic – Princeton University (USA)
Daniele Zonta – University of Strathclyde (UK), University of Trento (Italy)

UNIVERSITY OF TRENTO - Italy
Department of Civil, Environmental and Mechanical Engineering

02nd – 03rd March 2017
Zagreb, Croatia

Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

MOTIVATION

- SHM requires to make inference under uncertain information
- Bayesian inference is a powerful tool, but requires knowledge of statistics and experience in its application.
- Structural engineers often don't have a solid background in Bayesian statistics, prefer to make inference using heuristics
- Yet structural engineers are very familiar with structural mechanics
- Introducing a method for easy logical inference based on a formal analogy between mechanics and Bayesian logic
- Single- and multi-parameter cases are investigated

SINGLE-PARAMETER ESTIMATION

Goal

To estimate a parameter θ based on a set of uncertain information expressed by the vector

$$\mathbf{y} = \left\{ y_1, \ldots, y_i, \ldots, y_D \right\}^{\mathrm{T}}$$

Hypotheses

- All the uncertain quantities have Gaussian distribution.
- The relationship between information and parameter is linear.

With those hypotheses we can solve any logical inference problem using two fundamental inference rules.

EXAMPLE

Estimate the mean value of concrete strenght $\theta = f_{cm}$, given 3 samples y from different locations. Assume (as per EC2) concrete strenght variability with $\sigma_y = 5\ \text{MPa}$ and no prior information.

Samples

Sample 1

$y_1 = 36.5\ \text{MPa}$

Sample 2

$y_2 = 32.8\ \text{MPa}$

Sample 3

$y_3 = 38.0\ \text{MPa}$

$$\frac{1}{\sigma_\theta^2} = \sum_{i=1}^{n} \frac{1}{\sigma_i^2} = \frac{3}{\sigma_y^2} \quad \Rightarrow \sigma_\theta = \frac{\sigma_y}{\sqrt{3}} = 2.88\ \text{MPa}$$

$$\mu_\theta = \frac{\sum_{i=1}^{n} \frac{y_i}{\sigma_i^2}}{\sum_{i=1}^{n} \frac{1}{\sigma_i^2}} = \frac{y_1 + y_2 + y_3}{3} = 35.77\ \text{MPa}$$

INFERENCE RULES

First inference rule

- n observations y_i
- σ_i^2 variance of each observation

$$\frac{1}{\sigma_\theta^2} = \sum_{i=1}^{n} \frac{1}{\sigma_i^2}, \qquad \mu_\theta = \frac{\sum_{i=1}^{n} \frac{y_i}{\sigma_i^2}}{\sum_{i=1}^{n} \frac{1}{\sigma_i^2}}$$

Second inference rule

- m arguments x_j
- σ_i^2 variance of each argument
- $y = x_1 + \ldots + x_m$

$$\sigma_y^2 = \sum_{j=1}^{m} \sigma_j^2, \qquad \mu_y = \sum_{j=1}^{m} x_j$$

Accuracy

The closeness to the true value of the parameter/variable.

$$w = \sigma^{-2} = \frac{1}{\sigma^2}$$

Using the accuracy, we can reformulate the two basic inference rules.

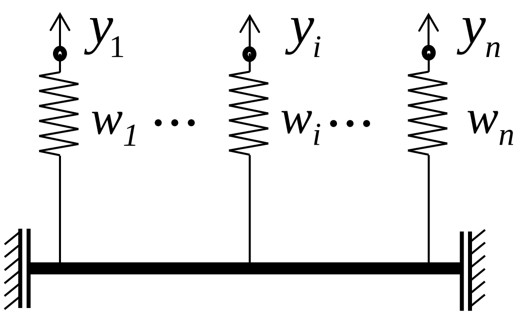
INFERENCE RULES

First inference rule

- n observations y_i
- σ_i^2 variance of each observation

$$\underbrace{w_\theta = \sum_{i=1}^{n} w_i}, \quad \mu_\theta = \frac{\sum_{i=1}^{n} y_i\, w_i}{w_\theta}$$

Equivalent stiffness of a set of springs in parallel

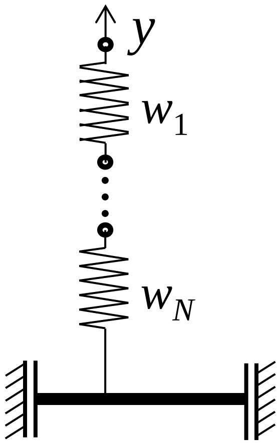


Second inference rule

- m arguments x_j
- σ_i^2 variance of each observation
- $y = x_1 + \ldots + x_m$

$$\underbrace{\frac{1}{w_y} = \sum_{j=1}^{m} \frac{1}{w_j}}, \quad \mu_y = \sum_{j=1}^{m} x_j$$

Equivalent stiffness of a set of springs in series



MECHANICAL EQUIVALENCE

Multiple sources of uncertainty

Series of springs

w_{yi} stiffness, accuracy

y pre-stretch, observation

Uncertain information

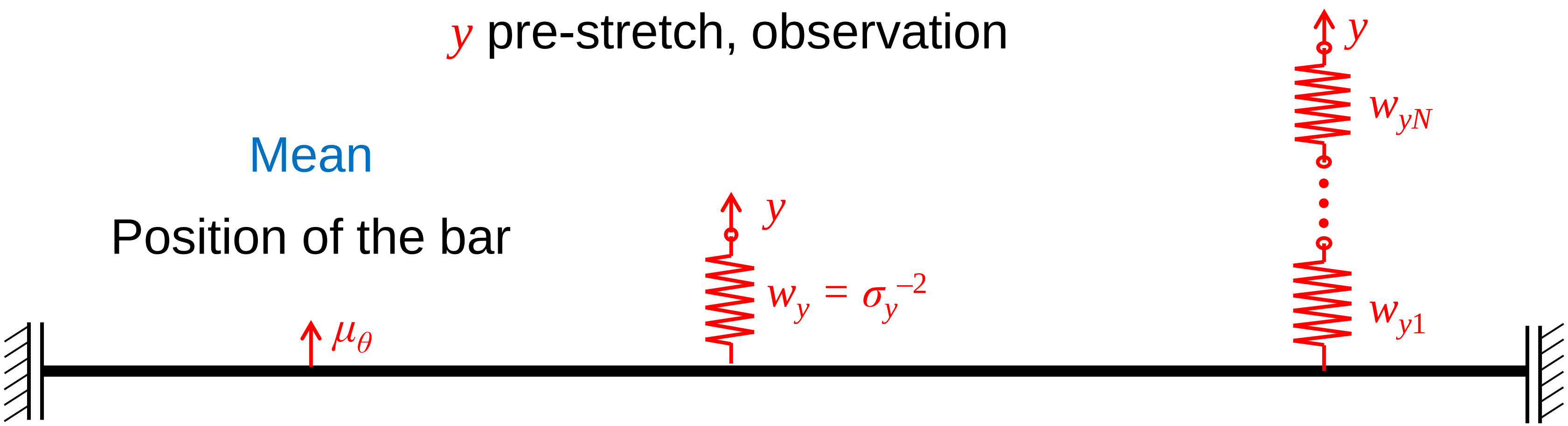
Linear spring

w_y stiffness, accuracy

y pre-stretch, observation

Mean

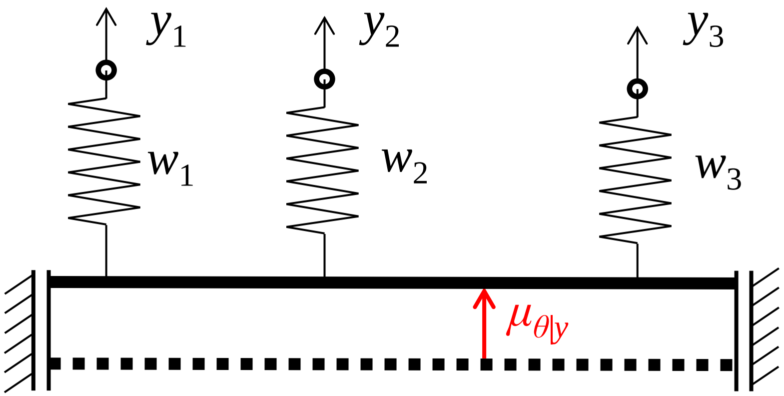
Position of the bar



INFERENCE

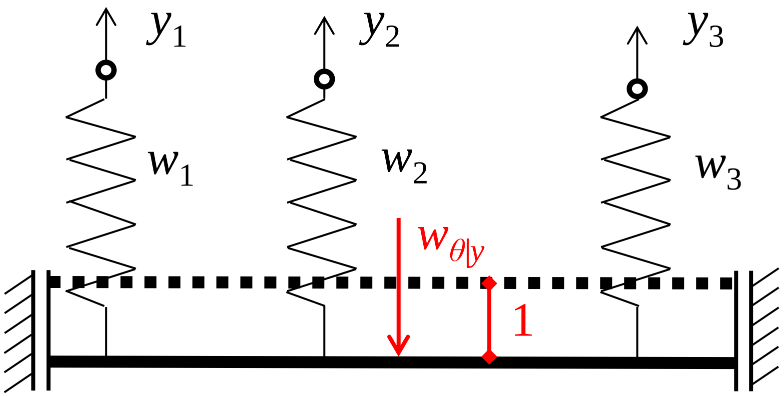
Expected value *a posteriori*

$\mu_{\theta|y}$ position of the bar in equilibrium



Variance *a posteriori*

Accuracy $w_{\theta|y}$ is the resulting stiffness,
$\sigma_{\theta|y}{}^2$ is the resulting flexibility



MECHANICAL ANALOGY

Symbol	World of logic	World of Mchanics
w, σ^{-2}	accuracy, inverse variance	stiffness
σ^{2}	variance	flexibility
y	observation	pre-stretch
μ	expected value	position at equilibrium

MECHANICAL ANALOGY

Symbol	World of logic	World of Mchanics
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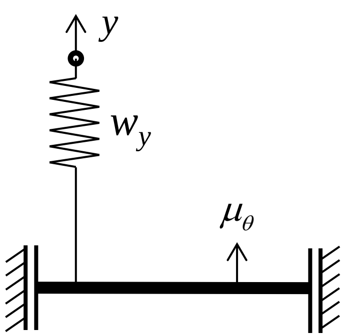
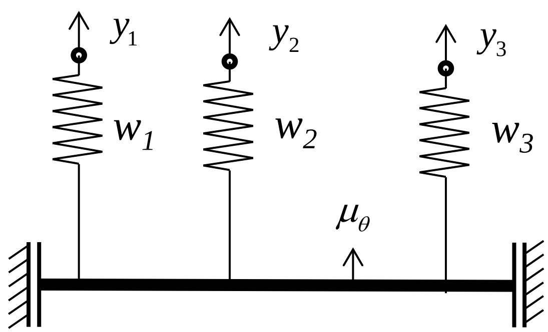
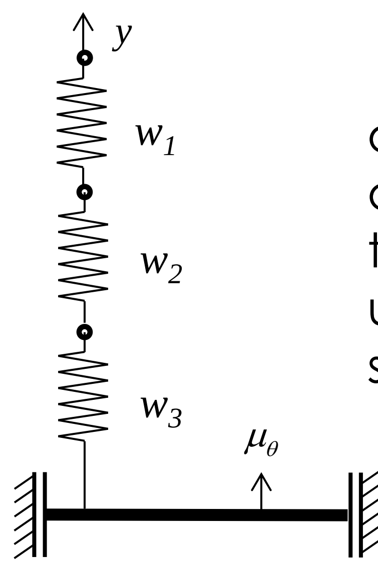


one observation



three uncorrelated observations

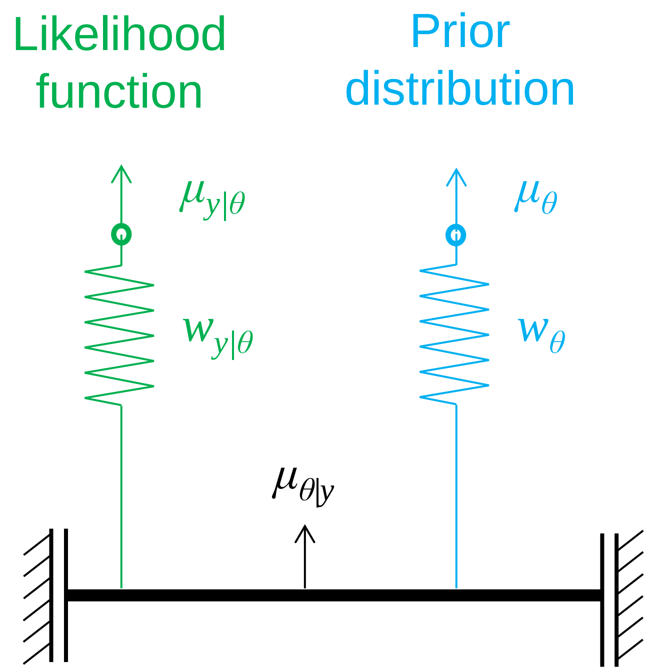


observation affected by three uncorrelated sources

BAYES' THEOREM

Given:

- a unknown parameter θ
- a set of observations $\mathbf{y} = \{y_1, \ldots, y_N\}^{\mathrm{T}}$



$$\underbrace{\mathrm{pdf}(\theta\,|\,\mathbf{y})}_{\text{Posterior d. of }\theta} = \frac{\overbrace{\mathrm{pdf}(\mathbf{y}\,|\,\theta)}^{\text{Likelihood function}}\cdot\overbrace{\mathrm{pdf}(\theta)}^{\text{Prior distribution}}}{\underbrace{\mathrm{pdf}(\mathbf{y})}_{\text{Evidence}}}$$

Evidence: $\mathrm{pdf}\left(\mathbf{y}\right) = \int_{D\theta} \mathrm{pdf}(\mathbf{y}\,|\,\theta)\cdot\mathrm{pdf}(\theta)\cdot\mathrm{d}\theta$

- Likelihood function, prior and posterior distribution are functions of θ
- The observations are given values, $\mathbf{y} = \{y_1, \ldots, y_N\}^{\mathrm{T}}$

SINGLE-PARAMETER: AN APPLICATION

Problem

To estimate the mean strenght of concrete $\theta = f_{cm}$, given 3 samples $\mathbf{y}$ from different locations and prior information.

World of logic

Prior information

Initial guess from design documentation.

Measurements

Sample 1

Sample 2

Sample 3

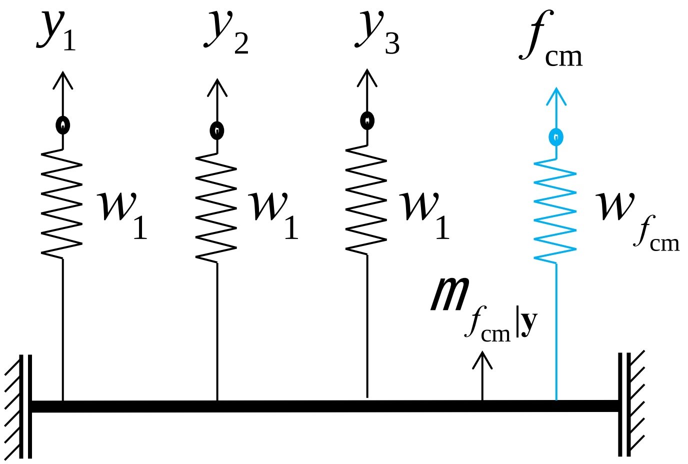
$$y_1 = 36.50 \text{ MPa}$$

$$y_2 = 32.80 \text{ MPa}$$

$$y_3 = 38.00 \text{ MPa}$$

$$\mathbf{y} = \{y_1, y_2, y_3\}^T \qquad \sigma_y = \sigma_1 = 5 \text{ MPa}$$

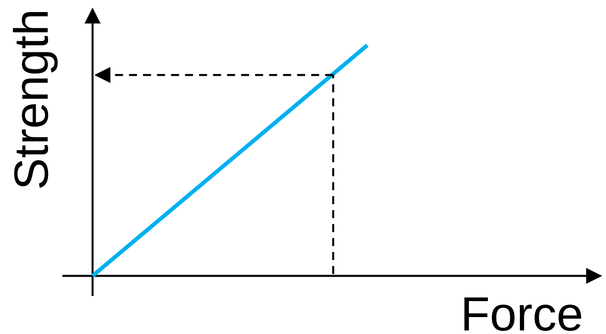
World of mechanics



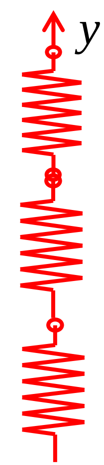
APPLICATION TO INDIRECT TESTS

Pull-out tests

Extraction force of an expansion bolt
→ compressive strength of concrete.



Sources of uncertainty



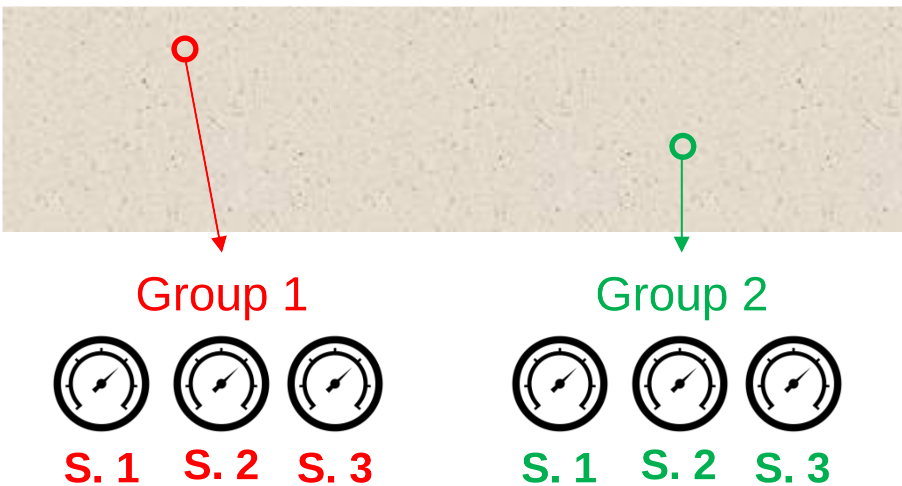
Inherent variability of concrete $w_1 = \sigma_1^{-2}$

Bias of the indirect estimation $w_3 = \sigma_3^{-2}$

Random scatter of test $w_4 = \sigma_4^{-2}$

INDIRECT TESTS: AN APPLICATION

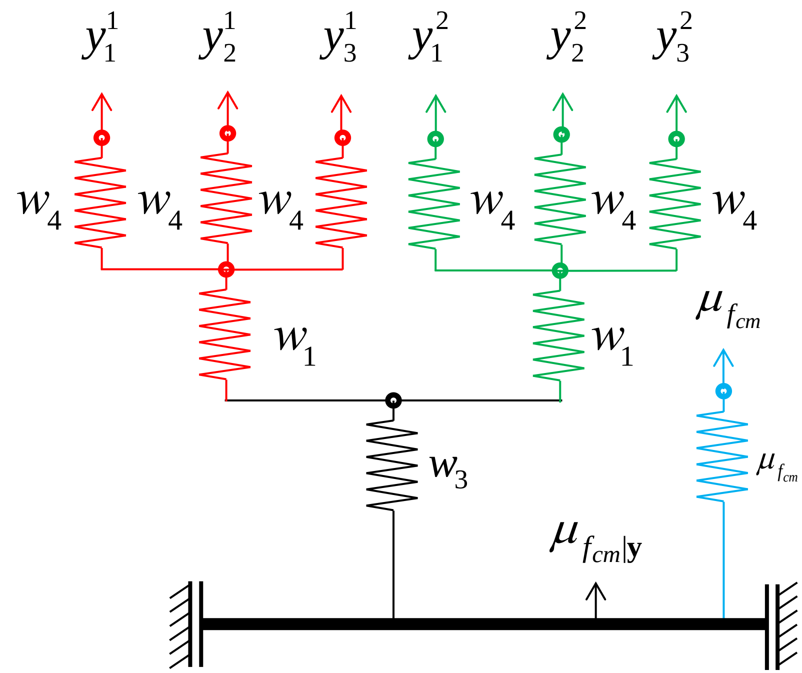
World of logic



Samples

Position 1 [MPa]		Position 2 [MPa]	
y_1^1	33.10	y_1^2	42.60
y_2^1	36.90	y_2^2	35.70
y_3^1	31.70	y_3^2	29.80

World of mechanics



Posterior distribution

$$\sigma_{f_{cm}|\mathbf{y}} = \sqrt{\frac{1}{w_{f_{cm}|\mathbf{y}}}} = 3.65\,\text{MPa} \quad \mu_{f_{cm}|\mathbf{y}} = 35.41\,\text{MPa}$$

MULTI-PARAMETER ESTIMATION

N unknown parameters, a set of D uncertain observations $\mathbf{y} = \{y_1, \dots, y_N\}^T$

$$\mathrm{pdf}\left(\boldsymbol{\mu}, \boldsymbol{\Sigma}; \boldsymbol{\theta}\right) = \frac{1}{(2\pi)^{\frac{N}{2}} \left|\boldsymbol{\Sigma}\right|^{\frac{1}{2}}} \exp\underbrace{\left\{-\frac{1}{2}\left(\boldsymbol{\theta}-\boldsymbol{\mu}\right)^T \boldsymbol{\Sigma}^{-1}\left(\boldsymbol{\theta}-\boldsymbol{\mu}\right)\right\}}_{\text{analogue of 'potential energy'!}}$$

MULTI-PARAMETER ESTIMATION

N unknown parameters, a set of D uncertain observations $\mathbf{y} = \{y_1, \ldots, y_N\}^T$

$$\mathrm{pdf}\left(\boldsymbol{\mu}, \boldsymbol{\Sigma}; \boldsymbol{\theta}\right) = \frac{1}{(2\pi)^{\frac{N}{2}} \left|\boldsymbol{\Sigma}\right|^{\frac{1}{2}}} \exp \underbrace{\left\{ -\frac{1}{2}\left(\boldsymbol{\theta} - \boldsymbol{\mu}\right)^T \boldsymbol{\Sigma}^{-1} \left(\boldsymbol{\theta} - \boldsymbol{\mu}\right) \right\}}_{\text{'potential energy'}}$$

Symbol	World of logic	World of mechanics
$\mathrm{E_p}(\boldsymbol{\theta})$	$-\log\left[\mathrm{pdf}\left(\boldsymbol{\mu}, \boldsymbol{\Sigma}; \boldsymbol{\theta}\right)\right]$	Potential energy
$\boldsymbol{\theta}$	Parameters	Degrees of freedom
$\boldsymbol{\Sigma}$	Covariance matrix [N x N]	Flexibility matrix [N x N]
$\boldsymbol{\Lambda}$	Accuracy matrix [N x N]	Stiffness matrix [N x N]
$\mathbf{y}$	observations	linear springs

$$\mathrm{E_p}(\boldsymbol{\theta}) = \frac{1}{2} w_{\theta_1} (\theta_1 - \mu_{\theta_1})^2 + \ldots + \frac{1}{2} w_{\theta_N} (\theta_N - \mu_{\theta_N})^2 + \frac{1}{2} w_y \sum_{j=1}^{D} (f_j(\boldsymbol{\theta}) - y_j)^2$$

MULTI-PARAMETER: INFERENCE

Accuracy matrix Λ → Hessian of p. energy

$$\Lambda = \begin{bmatrix} \frac{\partial^2 E_p(\theta)}{\partial \theta_1^2} & \cdots & \frac{\partial^2 E_p(\theta)}{\partial \theta_1 \partial \theta_N} \\ \vdots & \ddots & \vdots \\ \frac{\partial^2 E_p(\theta)}{\partial \theta_N \partial \theta_1} & \cdots & \frac{\partial^2 E_p(\theta)}{\partial \theta_N^2} \end{bmatrix} = \begin{bmatrix} w_{\theta_1|\mathbf{y}} & \cdots & \cdots \\ \vdots & \ddots & \vdots \\ \cdots & \cdots & w_{\theta_N|\mathbf{y}} \end{bmatrix}$$

Posterior accuracy of the *i*-th parameter

Covariance matrix Σ

→ inverse of Λ

$$\Sigma = \Lambda^{-1} = \begin{bmatrix} \sigma^2_{\theta_1|\mathbf{y}} & \cdots & \cdots \\ \vdots & \ddots & \vdots \\ \cdots & \cdots & \sigma^2_{\theta_N|\mathbf{y}} \end{bmatrix}$$

Expected values *a posteriori*

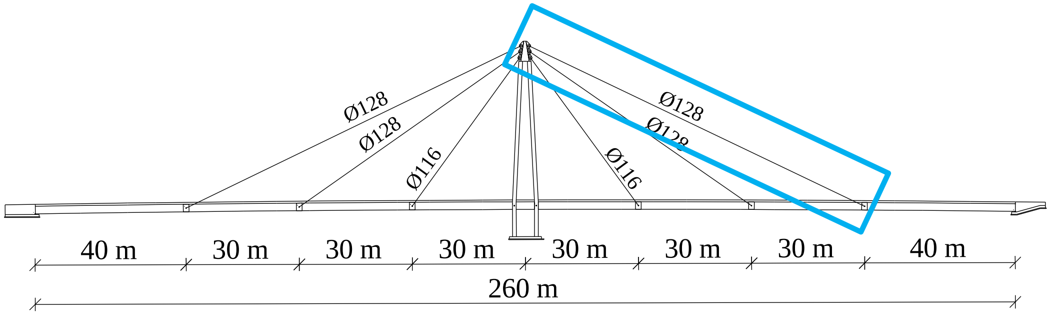
→ position in equilibrium

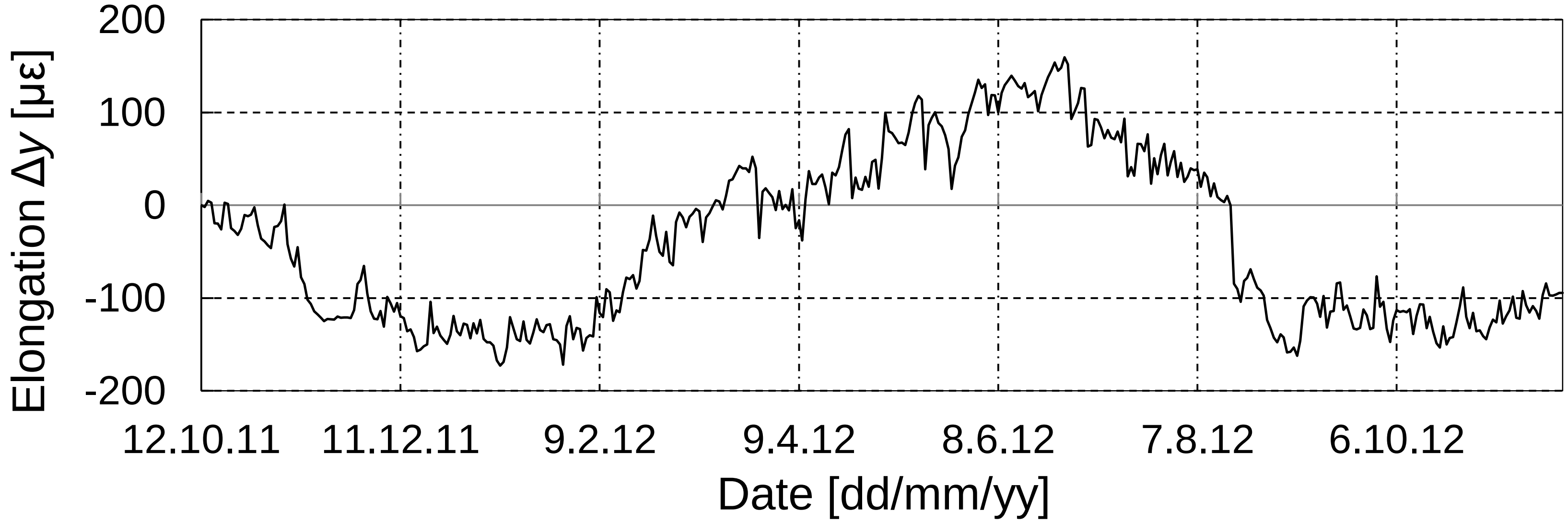
$$\begin{cases} \frac{\partial E_p(\theta)}{\partial \theta_1} = 0 \quad \rightarrow \quad \mu_{\theta_1|y} \\ \quad \cdots \\ \frac{\partial E_p(\theta)}{\partial \theta_N} = 0 \quad \rightarrow \quad \mu_{\theta_N|y} \end{cases}$$

MULTI-PARAMETER: AN APPLICATION

Problem: to estimate the elongation trend of a cable

Case study: Adige Bridge, a cable-stay bridge instrumented with fiber optic sensors





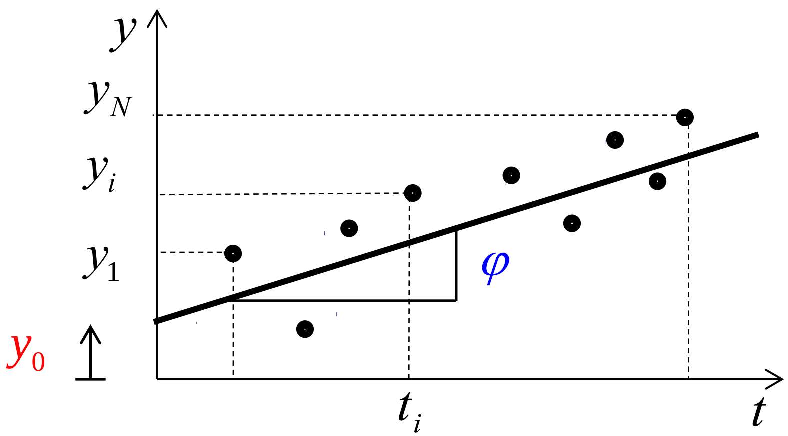
SOLUTION

World of logic

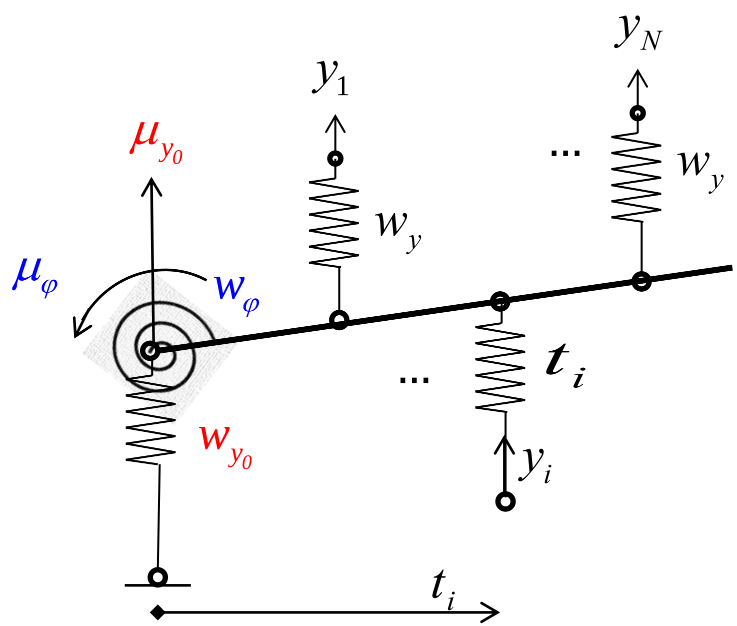
Measurements $\mathbf{y} = \{y_1, \ldots, y_N\}^T$

Model $y = y_0 + \varphi \cdot t$

Parameters $\boldsymbol{\theta} = \{y_0, \varphi\}$

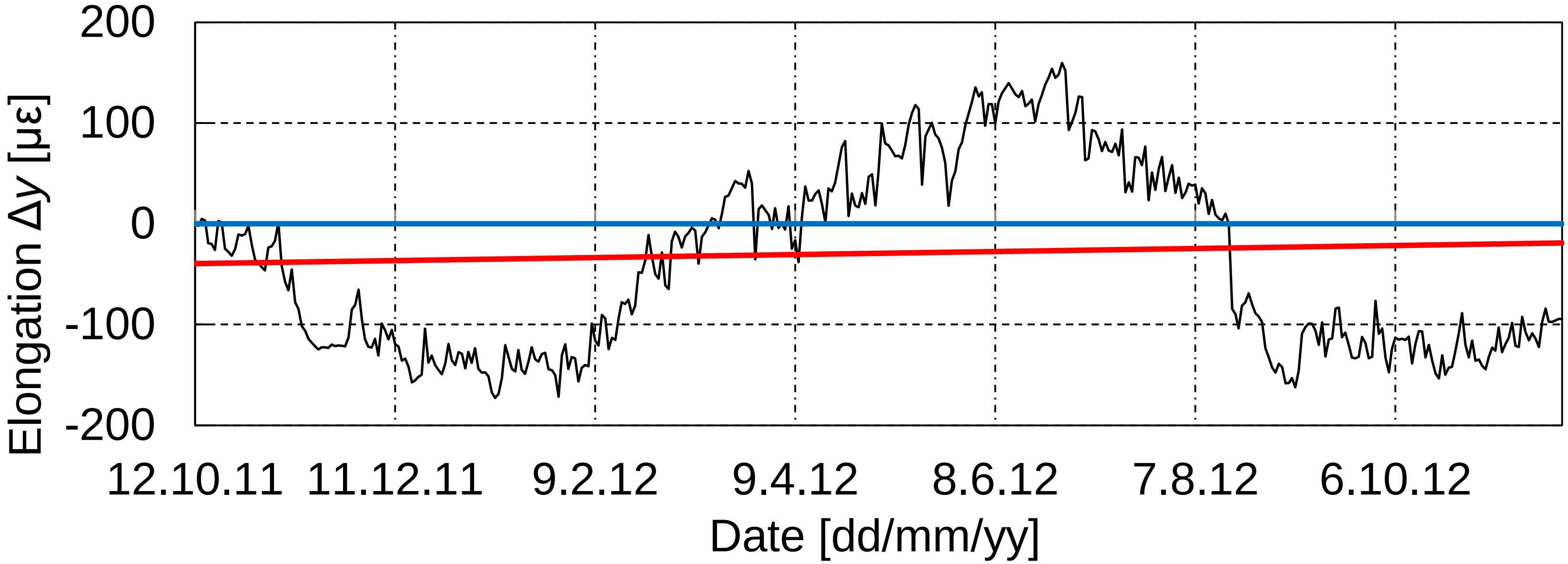


World of mechanics



SOLUTION

Prior distribution				Posterior distribution			
Parameter y_0		Parameter φ		Parameter y_0		Parameter φ	
w_{y0} [$\mu\varepsilon^{-2}$]	0.0025	w_φ [$\mu\varepsilon^{-2}$day^2]	1	w_{y0} [$\mu\varepsilon^{-2}$]	0.6601	w_φ [$\mu\varepsilon^{-2}$day^2]	36893
σ_{y0} [$\mu\varepsilon$]	20.000	σ_φ [$\mu\varepsilon^{-2}$day^{-1}]	1	σ_{y0} [$\mu\varepsilon$]	2.4400	σ_φ [$\mu\varepsilon$ day^{-1}]	0.0103
μ_{y0} [$\mu\varepsilon$]	0.0000	μ_φ [$\mu\varepsilon^{-2}$day^{-1}]	0	μ_{y0} [$\mu\varepsilon$]	–49.07	μ_φ [$\mu\varepsilon$ day^{-1}]	0.0473



UNCERTAIN INFORMATION - GAUSSIAN DISTRIBUTION

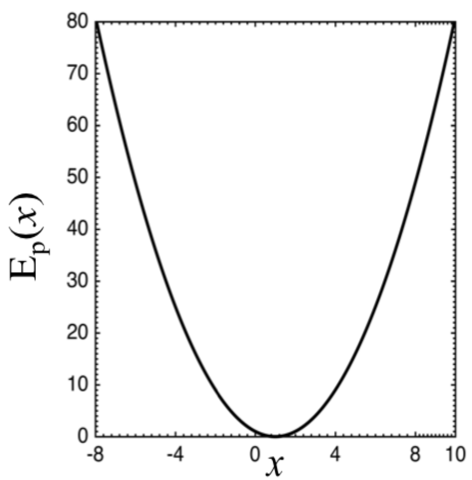
An uncertain info with Gaussian distribution

$$\mathrm{N}\left(\sigma,\mu;\theta\right)=\frac{1}{\sqrt{2\pi}}\frac{1}{\sigma}\exp\left\{-\frac{1}{2\sigma^2}\left(\theta-\mu\right)^2\right\}$$

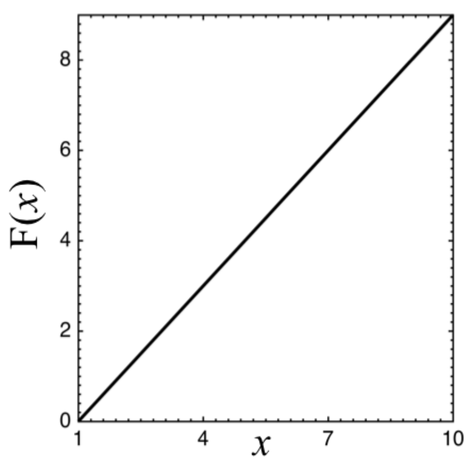
$$w_y=\sigma_y^{-2}$$

corresponds to a linear elastic spring with :

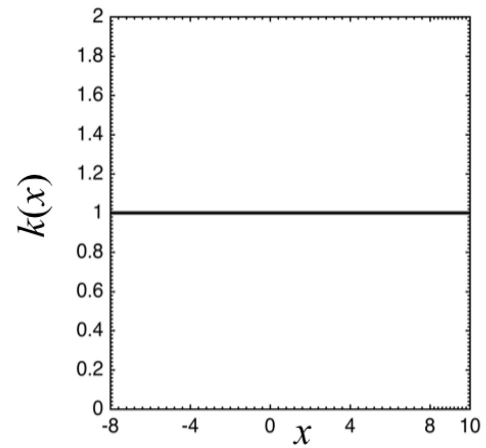
$$\mathrm{E}_p(\theta)=-\mathrm{In}(\mathrm{N}\left(\sigma,\mu;\theta\right))\qquad \mathrm{F}(\theta)=w\cdot\theta=\frac{d\mathrm{E}_p(\theta)}{d\theta}\qquad w=\frac{d^2\mathrm{E}_p(\theta)}{d\theta^2}$$



Potential energy



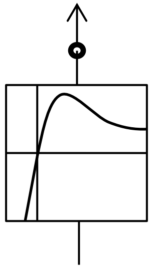
Restoring force



Stiffness

UNCERTAIN INFORMATION - NON GAUSSIAN DISTRIBUTION

Consider a generic information with **non-Gaussian** distribution ***f(θ,a,b).***



- ***θ*** parameter to estimate
- ***a,b*** parameters that characterize the distribution

The mechanical equivalent is a **non-linear** spring with:

$$E_p(\theta,a,b) = -\ln(f(\theta,a,b))$$

$$F(\theta,a,b) = \frac{dE_p(\theta)}{d\theta}$$

$$k(\theta,a,b) = \frac{d^2E_p(\theta)}{d\theta^2}$$

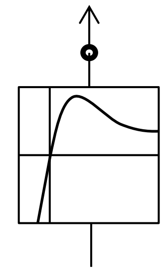
Potential energy of spring assiocated with *f(θ,a,b)*

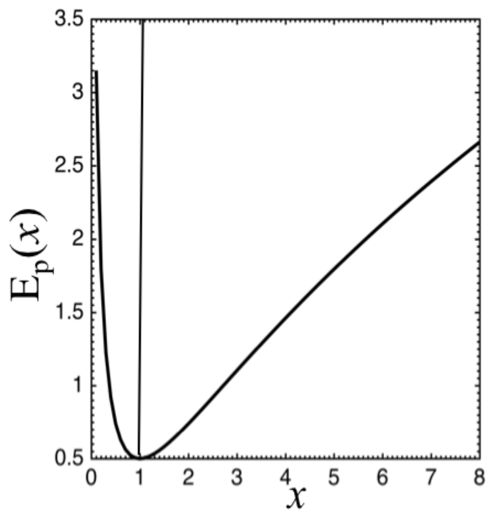
Restoring force of spring associated with *f(θ,a,b)*

Stiffness of spring associated with *f(θ,a,b)*

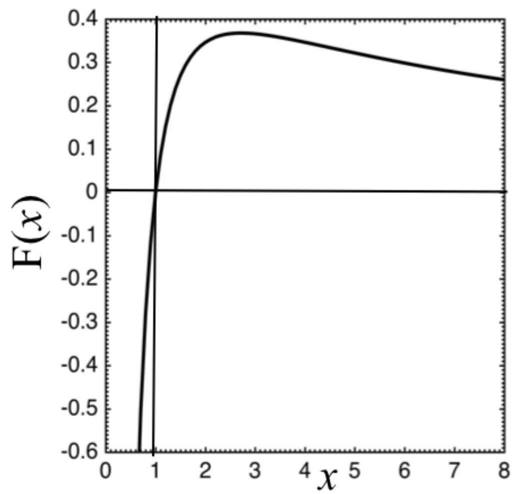
UNCERTAIN INFO WITH LOGNORM DISTRIBUTION

$$\mathrm{l}(\lambda,\varepsilon;x)=\frac{1}{x\,\varepsilon\sqrt{2\pi}}\,\mathrm{e}^{-\frac{1}{2\varepsilon^2}\left(\mathrm{In}(x)-\lambda\right)^2}\ \text{with}\ 0<x<+\infty$$

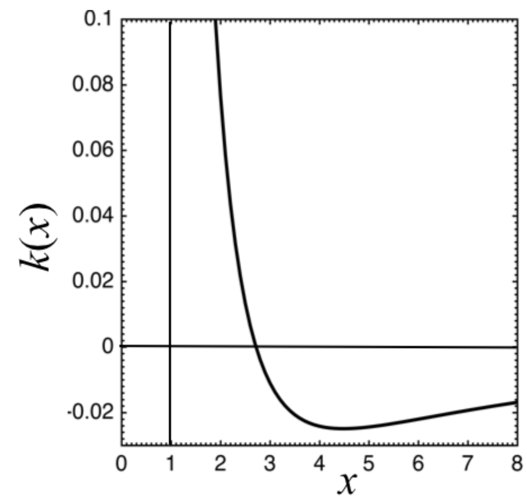




Potential energy



Restoring force



Stiffness

SINGLE-PARAMETER: AN APPLICATION

To estimate the mean strenght of concrete $\theta = f_{cm}$, given 3 samples $\mathbf{y}$ from different locations and prior information.

World of logic

Prior information

Initial guess from design documentation

Measurements

Sample 1 **Sample 2** **Sample 3**

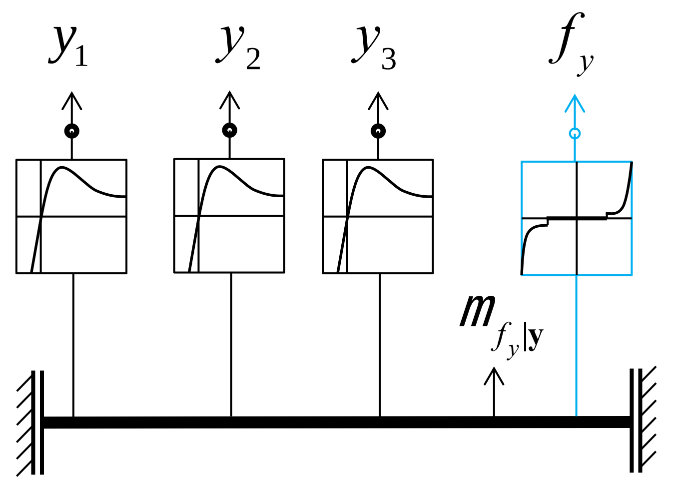
$$y_1 = 36.50 \text{ MPa}$$

$$y_2 = 32.80 \text{ MPa}$$

$$y_3 = 38.00 \text{ MPa}$$

$$\mathbf{y} = \{y_1, y_2, y_3\}^T \qquad \sigma_y = \sigma_1 = 5 \text{ MPa}$$

World of mechanics



CONCLUSIONS

- SHM requires to make inference under uncertain information
- structural engineers are often unfamiliar with Bayesian probability theory and prefer to make inference using heuristics
- proposed a quantitative method based on a formal analogy between logical inference and structural mechanics
- can solve any complex liner inference problem, even with correlated variables and non-Gaussian distributions
- solution easily found using the same methods of classical structural mechanics
- use ANSYS or ABACUS to make logical inference!

THANK YOU FOR YOUR ATTENTION!

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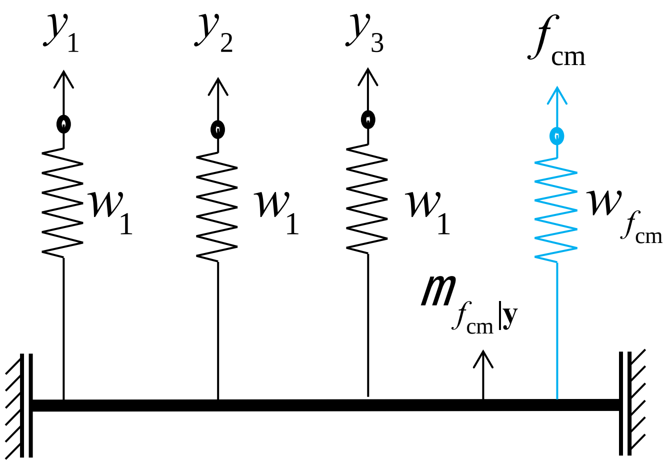
SOLUTION



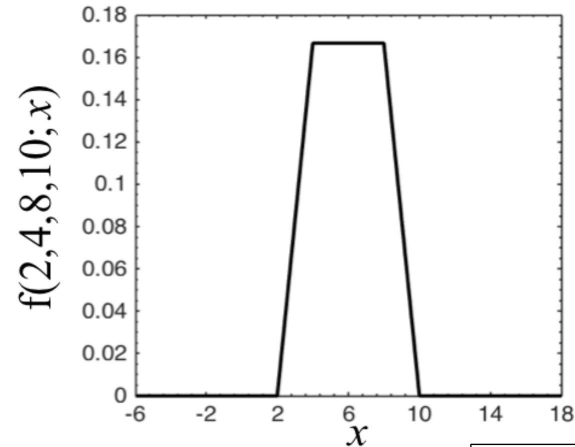
Prior distribution

$$\mu_{f_{cm}} = 35.50\,\mathrm{MPa}$$

$$\sigma_2 = \sigma_{f_{cm}} = 4\ \mathrm{MPa} \qquad \omega_2 = \sigma_2^{-2} = 0.06\ \mathrm{MPa}^{-2}$$

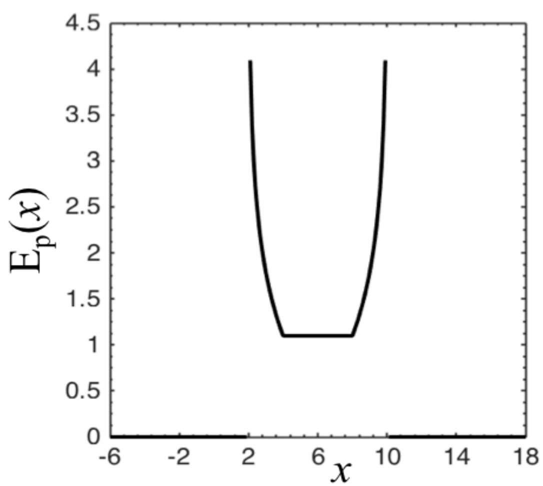
TRAPEZOIDAL DISTRIBUTION: *f(a,b,c,d;x)*

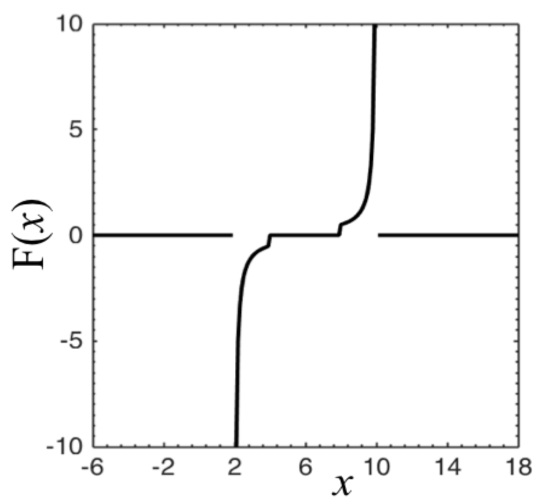
$$\mathrm{f}(a,b,c,d;x) = \begin{cases} u \cdot \left(\dfrac{x-a}{b-a}\right) & a \le x < b \\ u & \mathrm{b} \le x < c \\ u \cdot \left(\dfrac{d-x}{d-c}\right) & \mathrm{c} \le x < d \end{cases}, \quad \text{on } [a,d] \subset \mathbb{R}$$

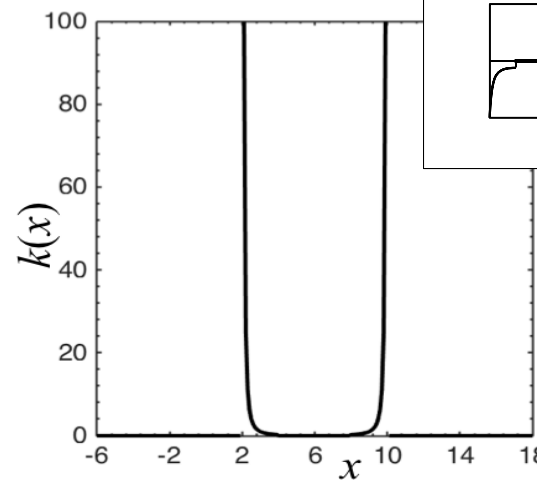


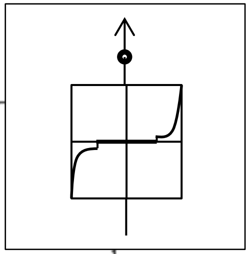
a=2 b=4 c=8 d=10

where $\mathrm{u} = \dfrac{2}{b+d-c-a}$









ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Integrated Structural Health Monitoring in steel arches bridges using continuous dynamic monitoring: two case studies in China

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Bruno BRISEGHELLA and Giuseppe Carlo MARANO - *College of Civil Engineering - Fuzhou University (China)*

College of Civil Engineering
Fuzhou University

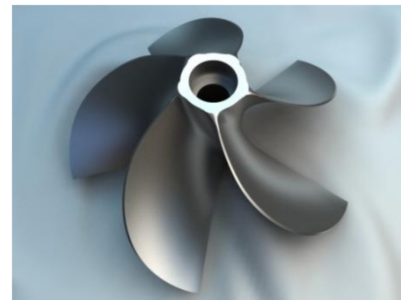
Xiamen University

02nd – 03rd March 2017
Zagreb, Croatia

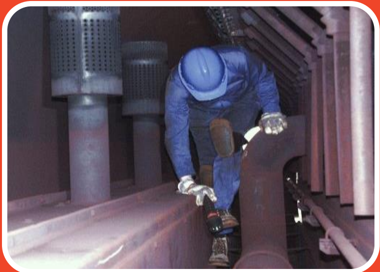
Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

Structural Health Monitoring system: a type of system that provides information on demand about any significant change or damage occurring in the structure (Mufti et al., 2007)

From the real word...



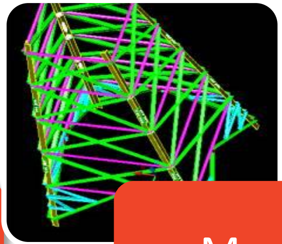
To improve the assessment of existing structures



Especially for ordinary structures, maintenance plans foresee a **systematic and repetitive visual checks** to identify the most damaged elements before failure. Actions are carried out according to the inspection results, and accounting for the observed deterioration rate.

A **monitoring network** would significantly reduce these problems, by simply reverting the approach to the problem: instead of looking at the possible causes, the monitoring would look at the effects on the structures produced by changes.

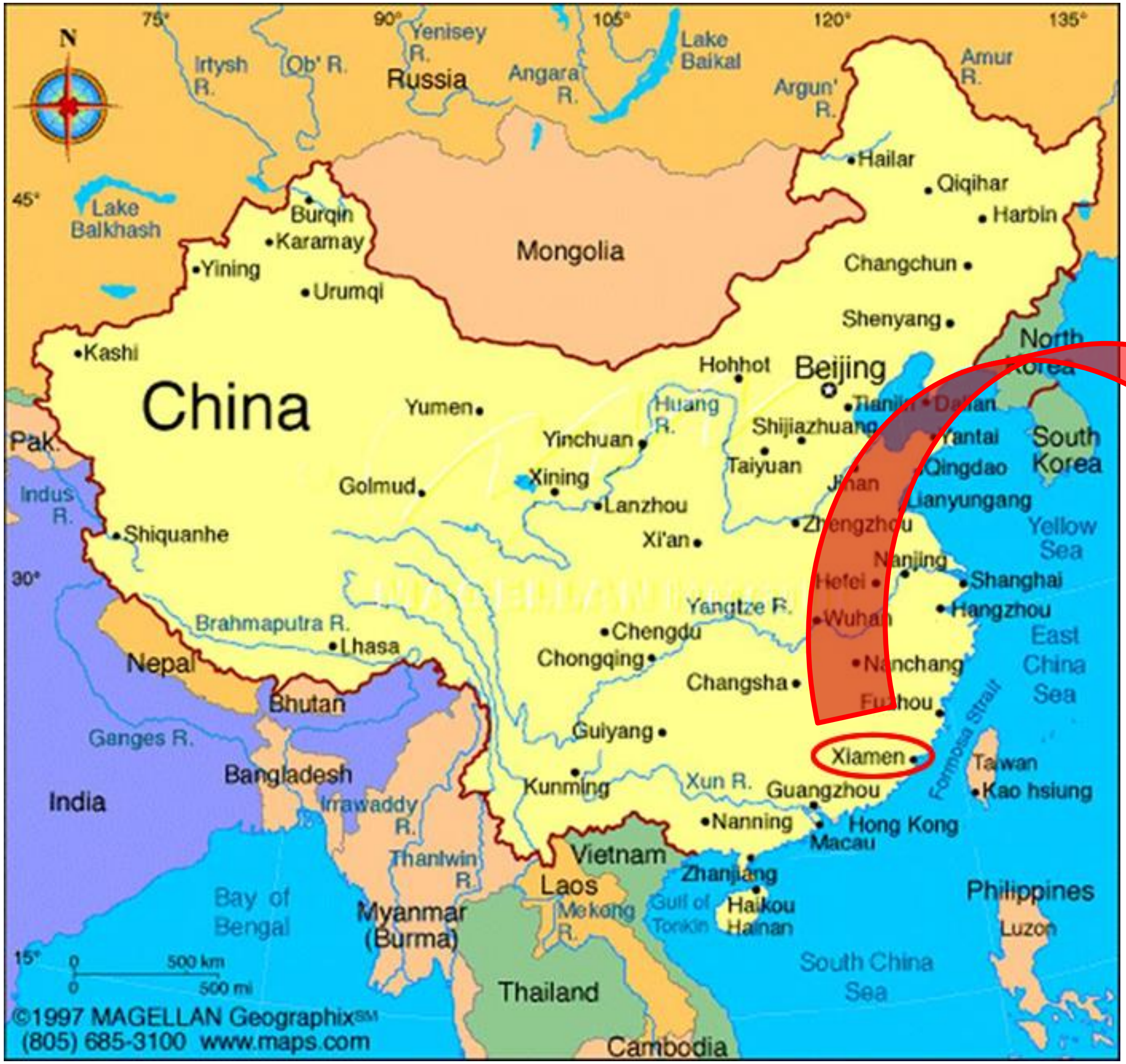
Data acquisition



Model updating

Structural assessment

Stand-alone analyses



Island of Xiamen
Fujian province

Island of Xiamen

Wuyuan Bridge

Tianyuan Bridge

Actions

- Increasing in traffic loads
- Climate and marine environmental conditions
- Extreme events

On average, 4~5 typhoons each year strike this area. The last severe typhoon strike Xiamen on 15.9. 2016, with the maximum wind speed 70m/s.

Tianyuan Bridge

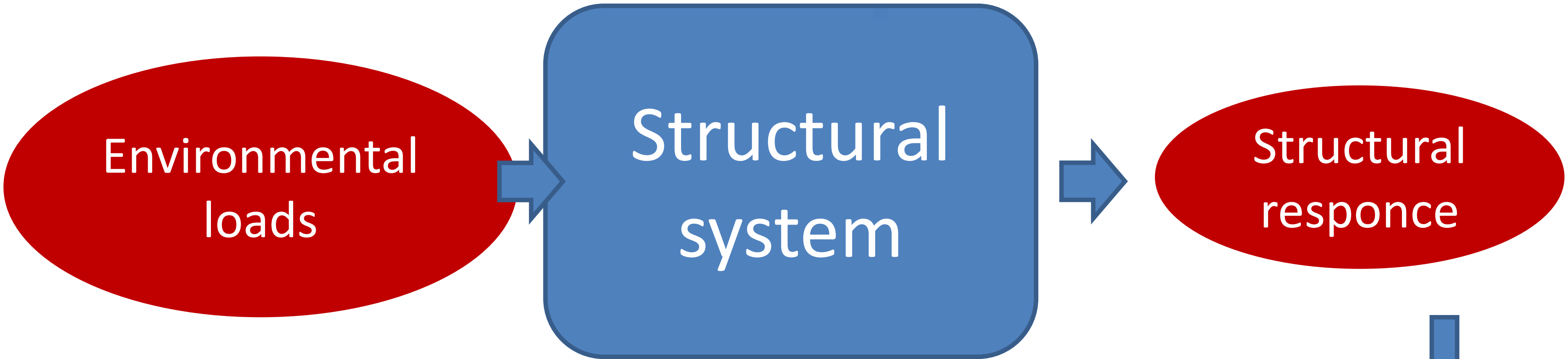
Tianyuan Bridge (behind is Wuyuan Bridge)

With span of 120m and width of 32m, with the arch rib by 14 rigid suspenders with a spacing of 6m

Wuyuan Bridge

with a main span of 208m and two side spans of 58m

SHM:



Monitoring

- Variation of behavior

Identify

- Set up and calibration of mechanical model

Damage detection

- How and where

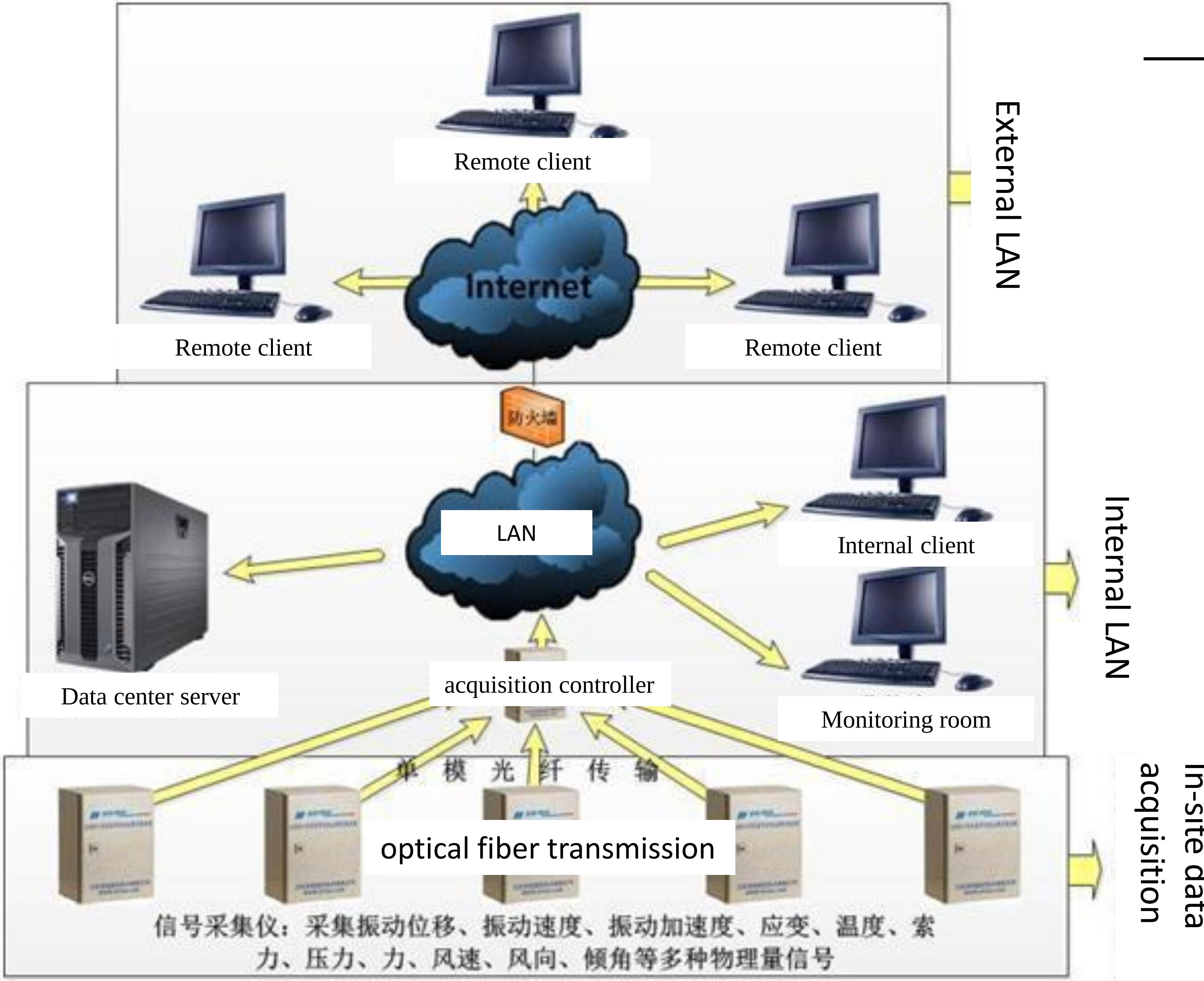
Arched strain meter

Welded strain meter

Temperature sensor

Humidity sensor

Data acquisition center

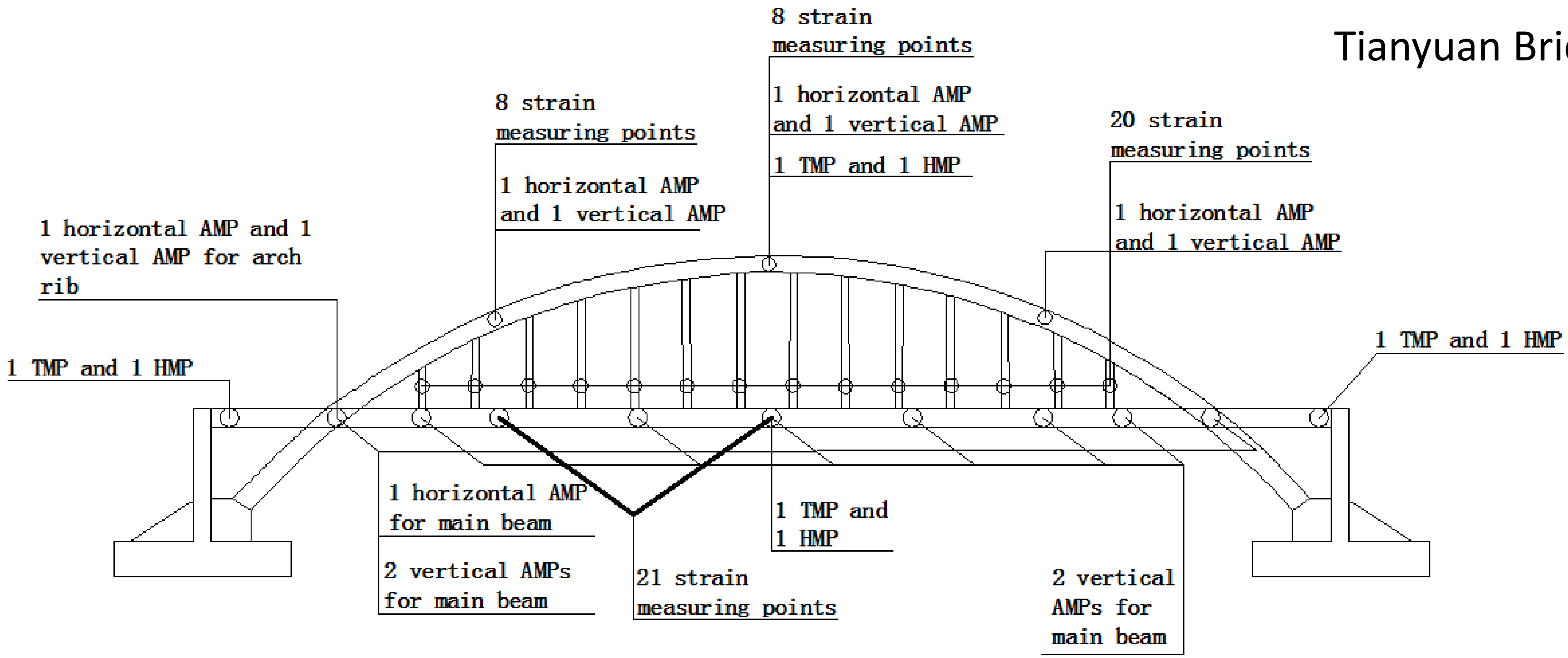


All types of sensors, including strain, temperature, cable force etc.

Tianyuan Bridge

No.	Parameter		Sensor type	Amount
1	Environmental effect	Temperature	Thermometer	4
		Humidity	Hygrometer	4
2	Loading sources	Weigh-in-motion system		6
		Traffic condition	Video Camera	2
3	Structural responses	Tension in hanger	Strain sensors	20
4		Deflection at arch crown	GPS	2
5		Vibration in deck girder	Acceleration sensor	30
6		Strain in arch rib	Dynamic strain sensor	16
7		Strain in deck girder	Dynamic strain sensor	42
Total				126

Tianyuan Bridge



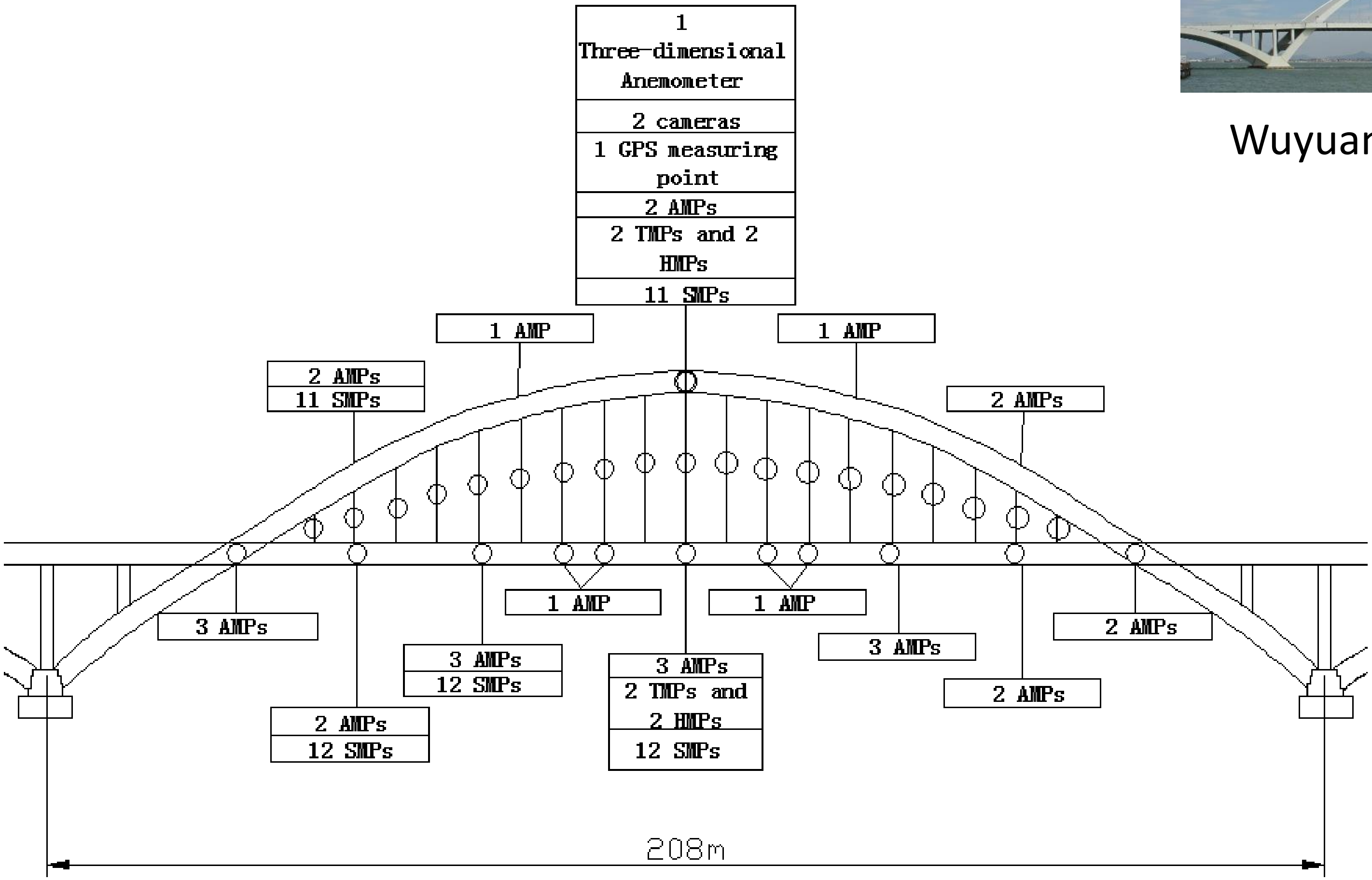
Arrangement of sensors.

(TMP - temperature measuring point; HMP - humidity measuring point; AMP - acceleration measuring point)

Wuyuan Bridge

No.	Parameter		Sensor type	Amount
1	Environmental effects	Wind speed and direction	Ultrasonic anemometer	1
		Temperature	Thermometer	4
		Humidity	Hygrometer	4
2	Loading sources	Traffic condition	Camera	2
4	Structural responses	Displacement at arch crown	GPS	1
5		Tension in hanger	Acceleration sensor	20
6		Vibration in arch rib and deck girder	Acceleration sensor	30
7		Strain in arch rib	Dynamic strain sensor	22
8		Strain in deck girder	Dynamic strain sensor	50
Total				134

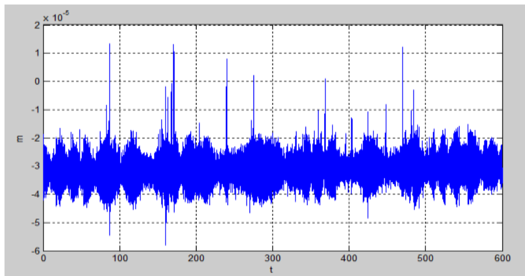
Wuyuan Bridge

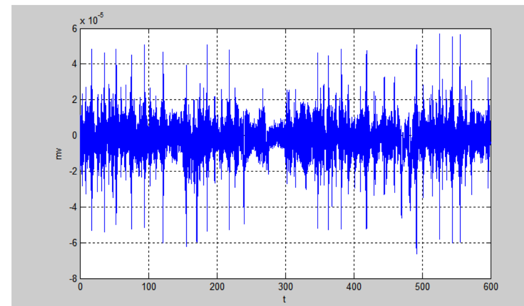


Sensor arrangement for Wuyuan Bridge

4 Preliminary results from dynamic monitoring

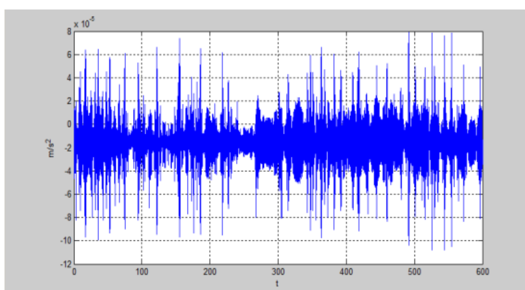
1) Strain response

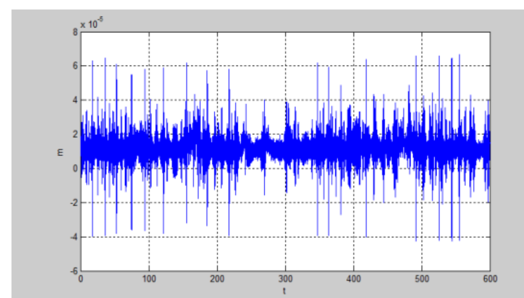




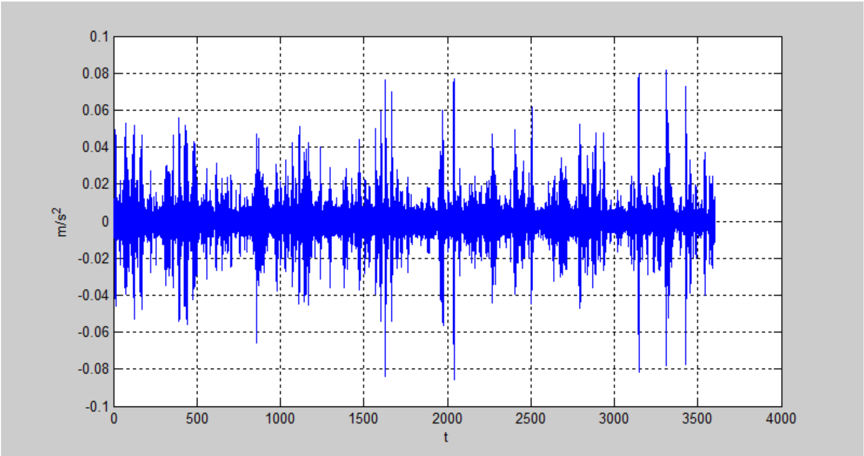
a) At mid-span of deck girder

b) At arch crown

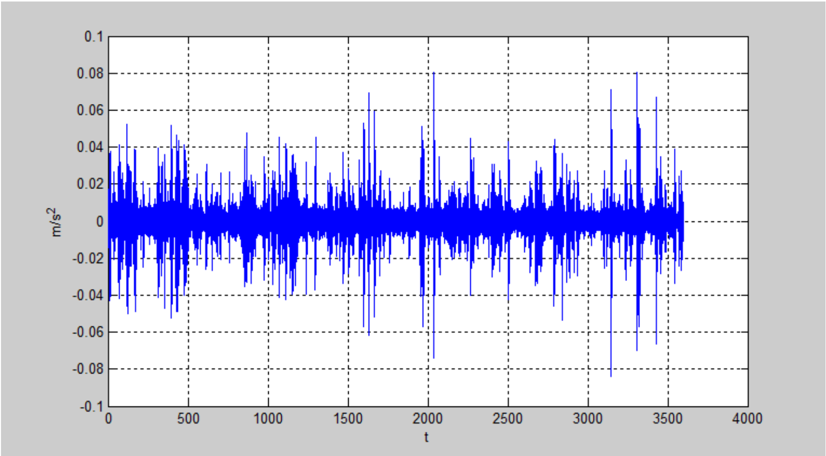




2) Acceleration response



a) Vertical



b) Transverse

Acceleration time history at mid-span of deck girder

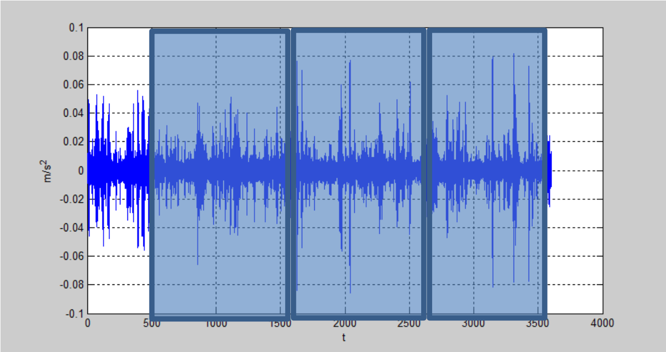
Monitoring

Synthetic indicators of system "status"

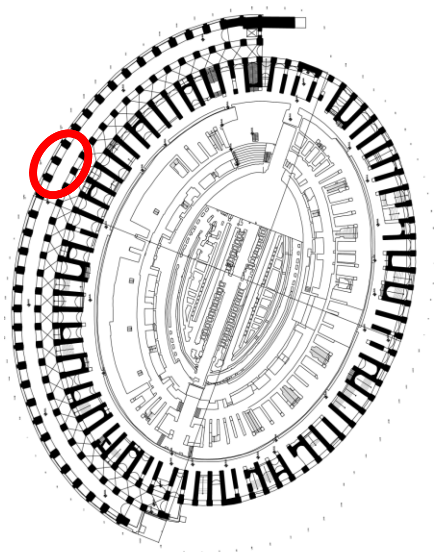
- PI to be used for decision support
- No a direct structural meaning (necessary)
- Easy meaning

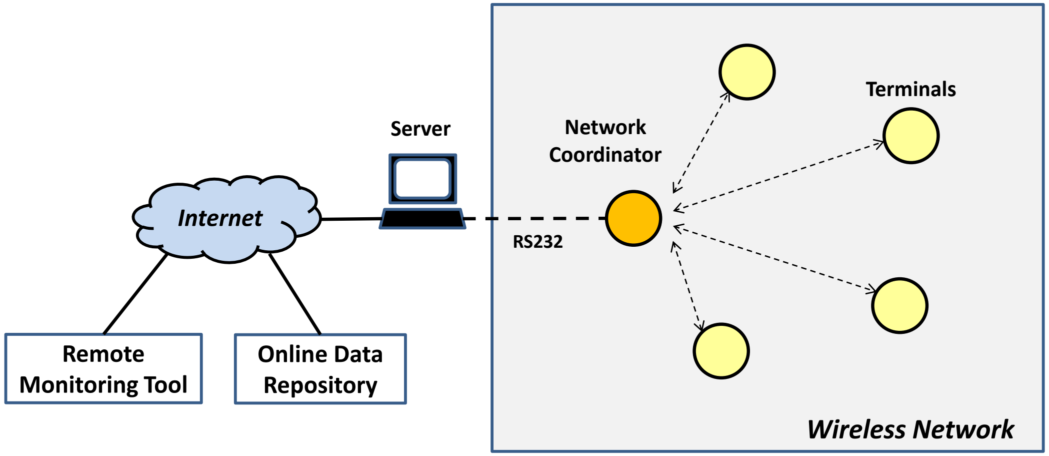
Signal extraction (granulation of data)

- Maximum
- Mean
- Variance
- etc



SAPIENZA
UNIVERSITÀ DI ROMA

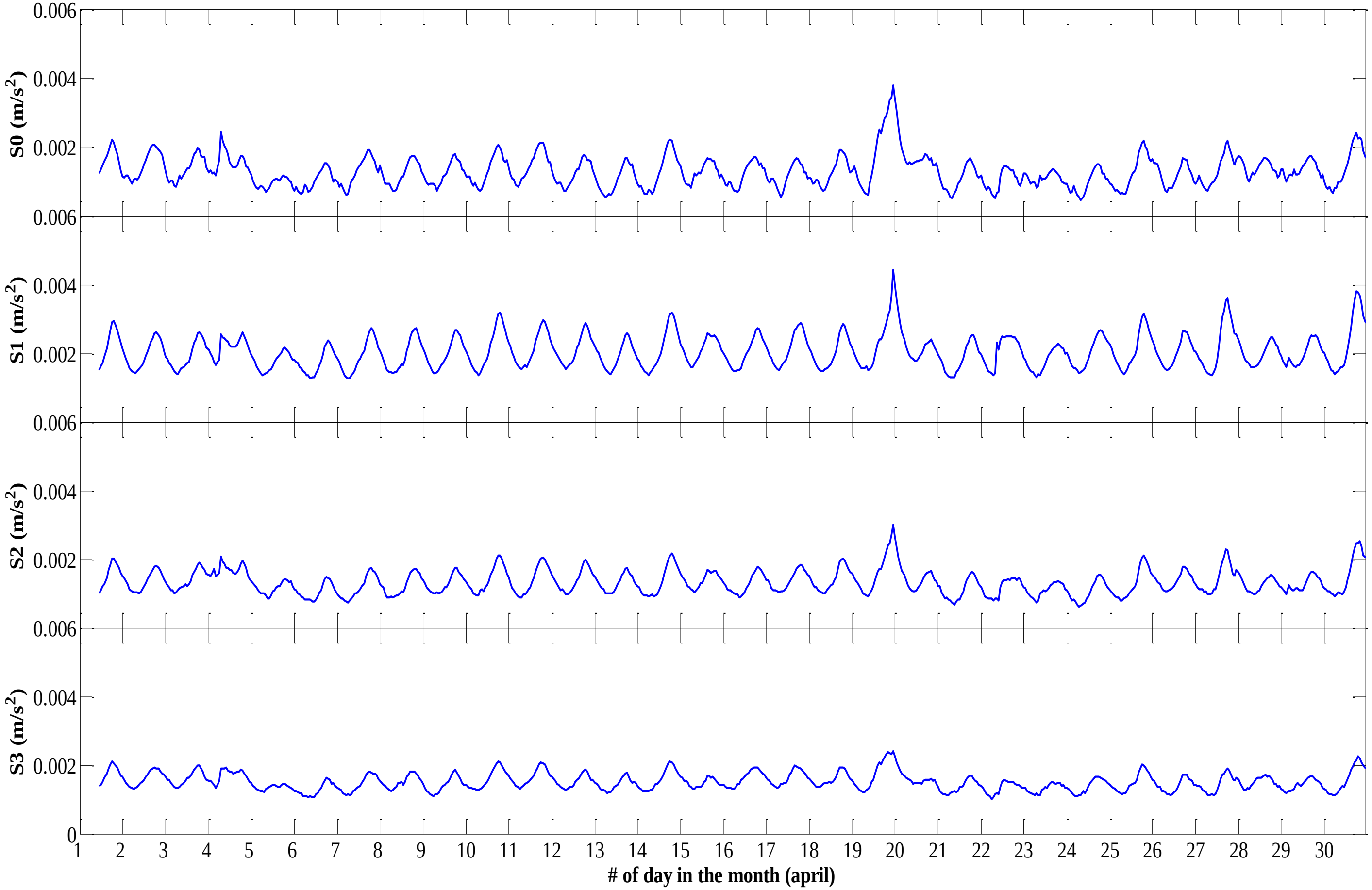




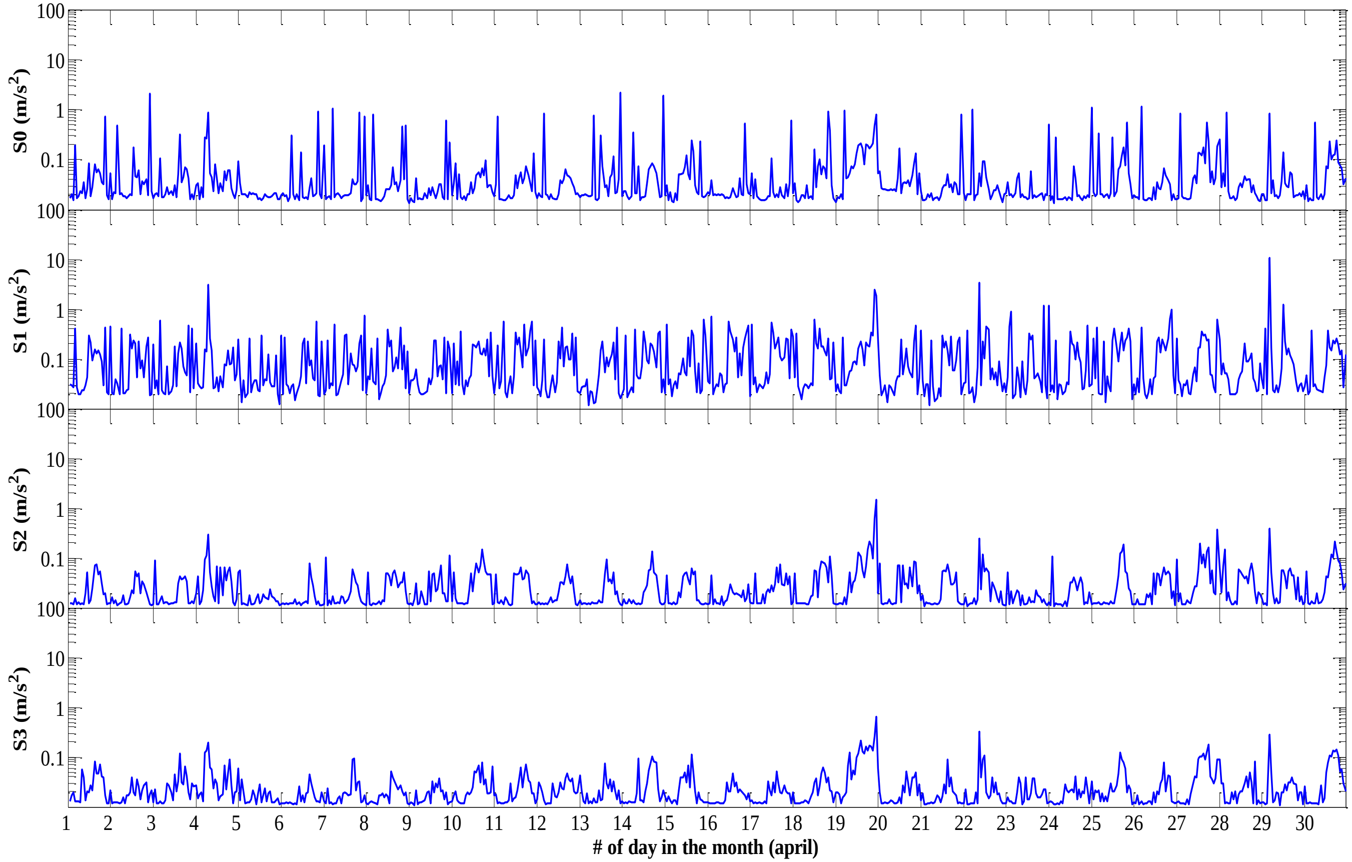
Allowable velocity for stone masonry by Chinese National Code

Level of preservation	Position of controlling points	Direction of controlling points	Allowable velocity for brick masonry [mm/s]		
			V_p < 2300 m/s	2300 m/s ≤ V_p ≤ 2900 m/s	V_p > 2900 m/s
National Level	The peak point of load-carrying structure	Horizontal	0.20	0.20-0. 25	0.25
Provincial Level	The peak point of load-carrying structure	Horizontal	0.36	0.36-0. 45	0.45
City and County Level	The peak point of load-carrying structure	Horizontal	0.60	0. 60-0. 75	0.75

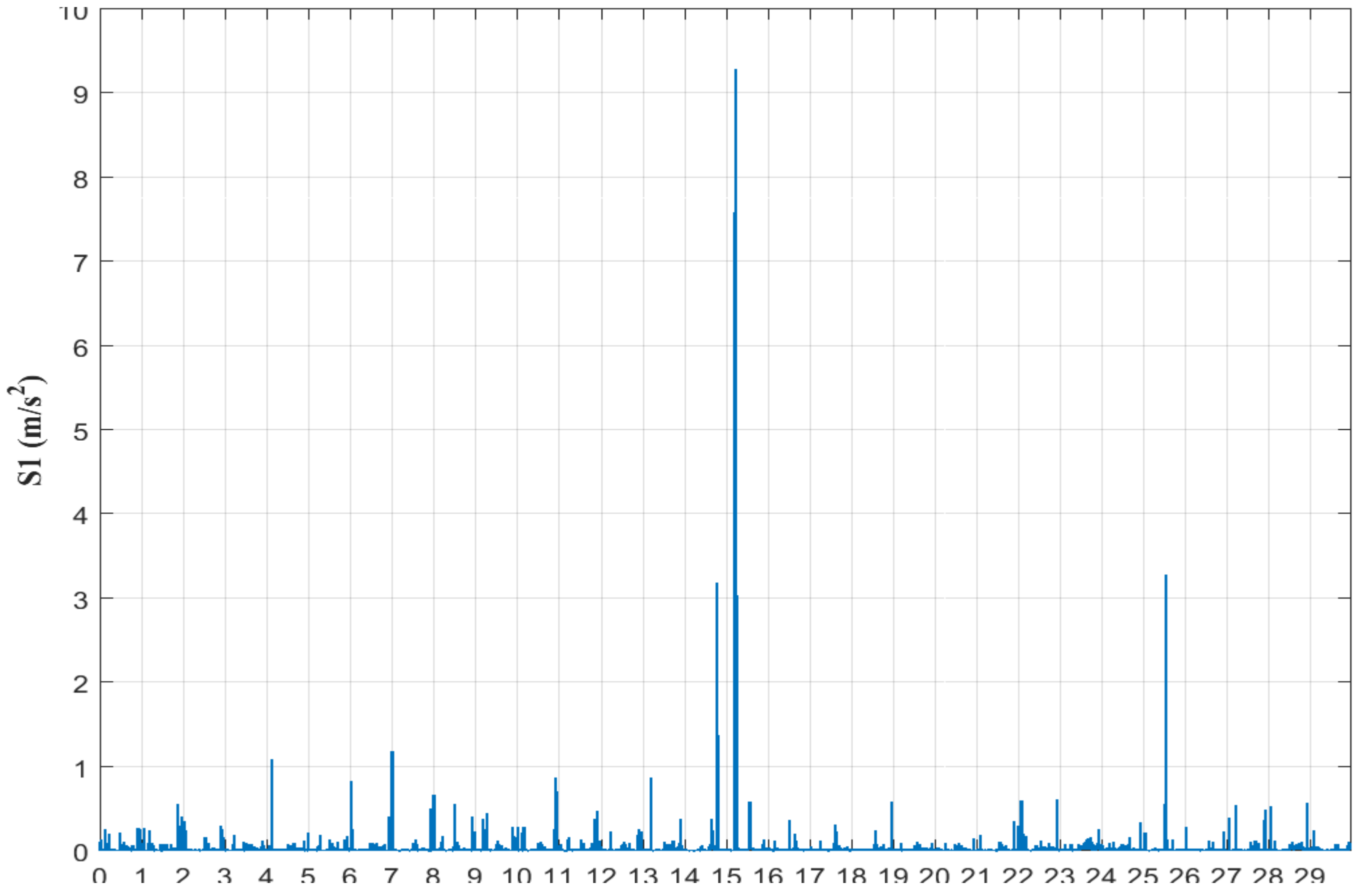
Mean acceleration values in the month with a EMA filtering



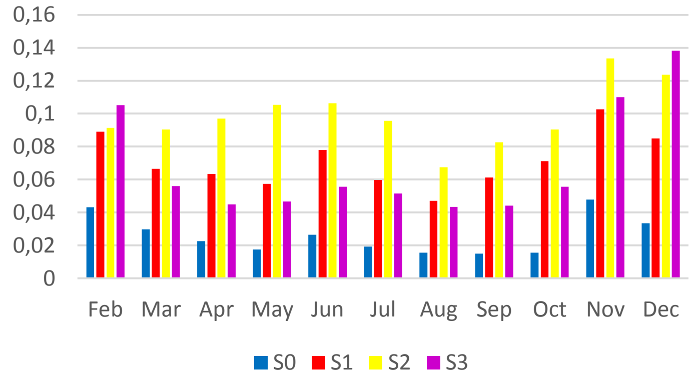
Time-history of the maximum values of acceleration maxima during the one month (logarithmic scale)



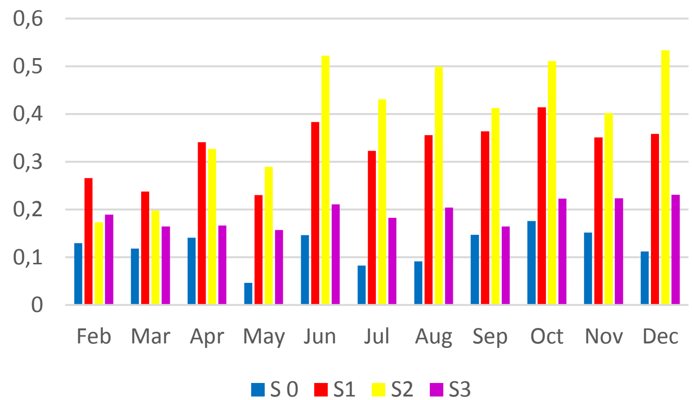
Maximum accelerations, sensor 1, June



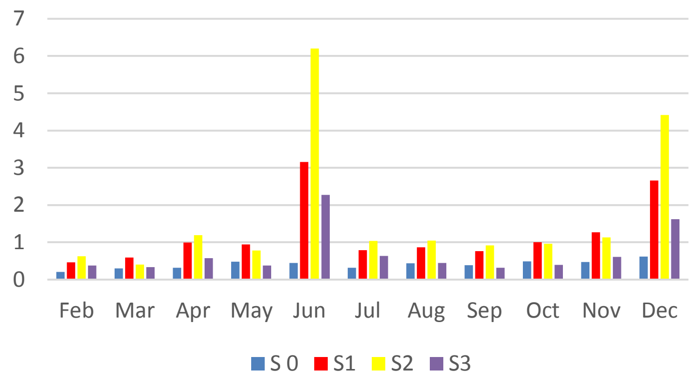
Monthly fractiles



99%



99,9%

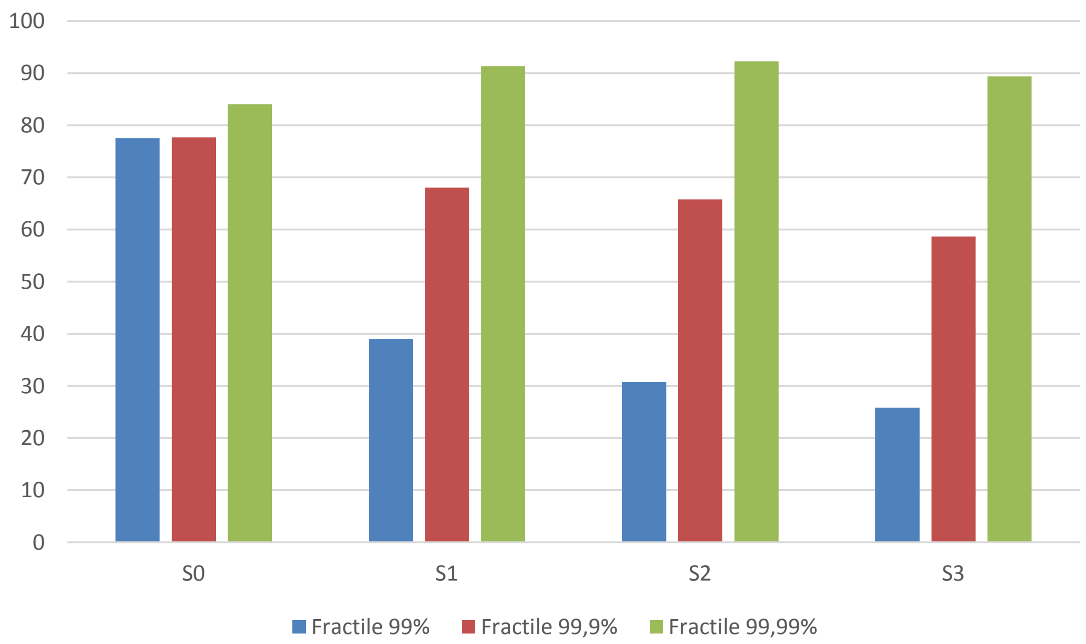


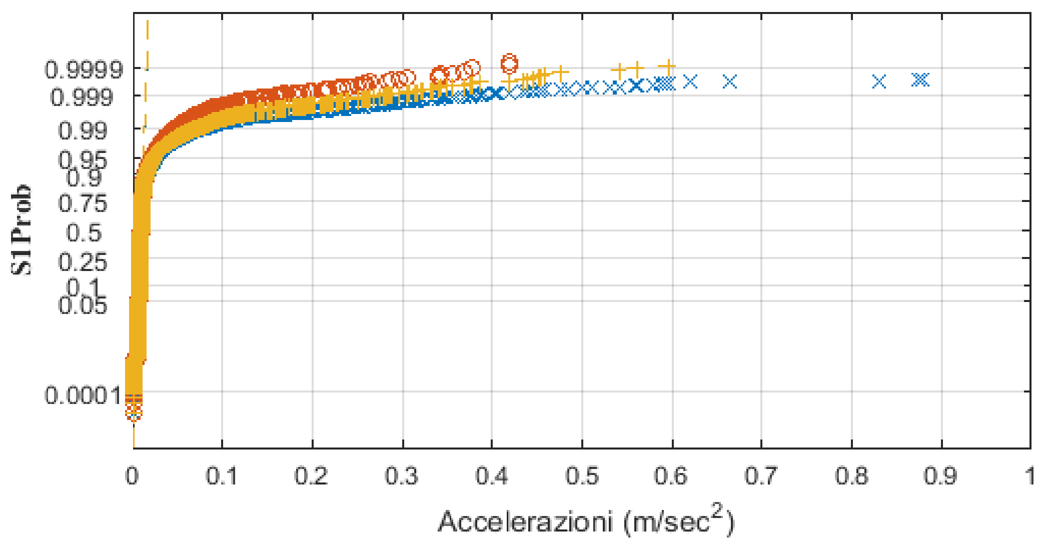
99,99 %

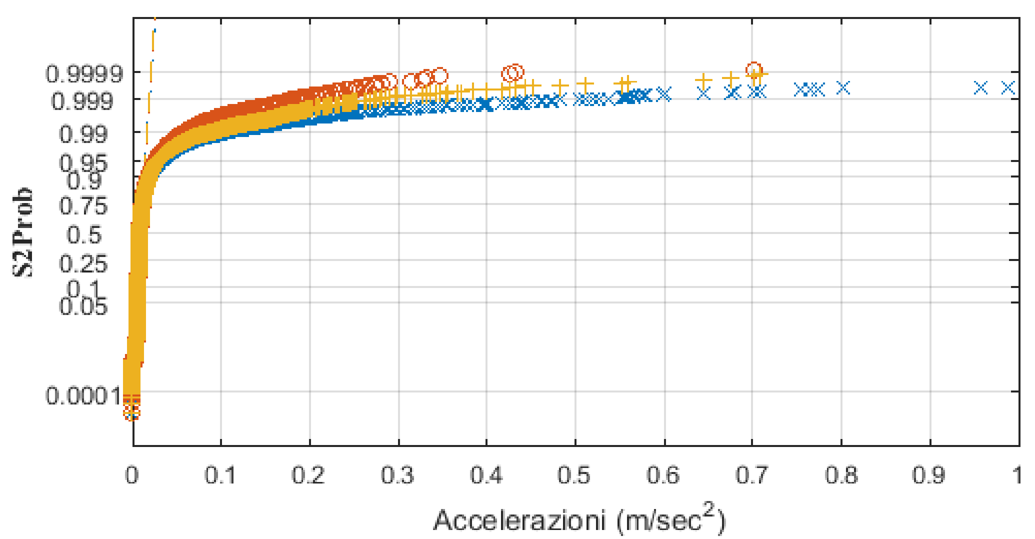
Filtering of maxima events

A filtering technique may consist in assigning a threshold value to the coefficient:

$$\psi = \frac{max_{acc}}{\sigma}$$




S1Prob
0.9999
0.999
0.99
0.95
0.9
0.75
0.5
0.25
0.1
0.05
0.0001
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Accelerazioni (m/sec^2)


S2Prob
0.9999
0.999
0.99
0.95
0.9
0.75
0.5
0.25
0.1
0.05
0.0001
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1
Accelerazioni (m/sec^2)

5 Conclusions

1) Structural health monitoring (SHM) system of two bridges (Tianyuan Bridge and Wuyuan Bridge) in China are developed for damage identification, structural safety evaluation, and maintenance decision making.

2) Through a long period of structural monitoring under different climate and load conditions, the SHM systems will generate time-specific status information such as bridge vibrations, strain distribution in several cross-sections, displacements, stresses in the hangers.

3) Based on data mining of SHM results, further study will be carried out to provide valuable information for bridge maintenance and decision making so as to reduce the maintenance cost and improve the technical level of long-term management. Extraction of PI based on time granulation and analysis of maxima along time should be a useful information for an intuitive management system

Thank you !

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College of Civil Engineering
Fuzhou University

Xiamen University

ZAGREB JOINT WORKSHOP

The Value of Structural Health Monitoring for the reliable bridge Management

Optimal sensor placement for Spatial Variability Assessment of Structures

TU 1402 WG3 (factsheets)

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+ to be share with Bernt Leira …. + DAAD (French-Germany project with D. Straub)

02nd – 03rd March 2017
Zagreb, Croatia

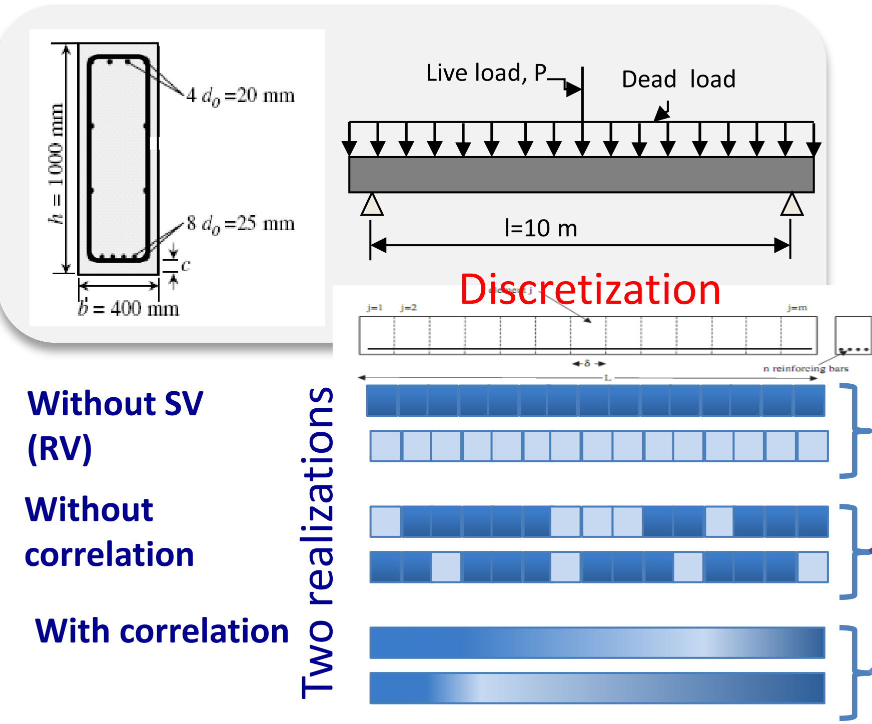
Sveučilište u Zagrebu
Građevinski fakultet
University of Zagreb
Faculty of Civil Engineering

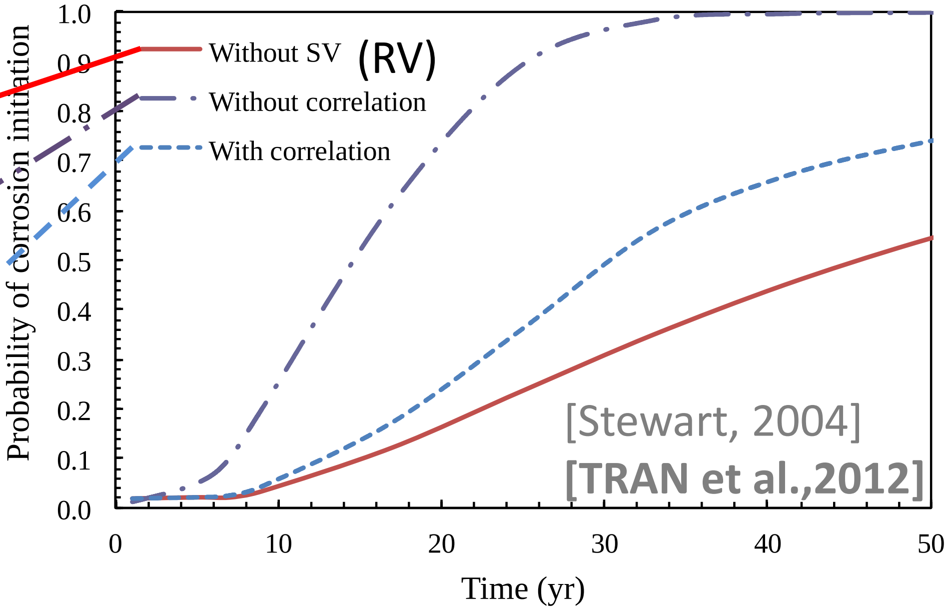
OUTLINE

1. Introduction
2. Methodology description
3. Numerical example
4. Real study cases
5. Conclusions

INTRODUCTION

Spatial variability plays a significant role on structural reliability





How to assess SV properties from SHM: how optimize the sensors design?

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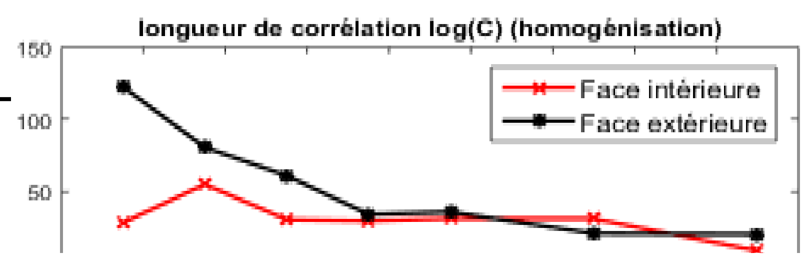
METHODOLOGY DESCRIPTION

- Simulations to produce reference cases

Main assumptions:

- Gaussian Stochastic field, second order stationary (statistically homogeneous and known marginal distribution)
- A larger number of sensors (20 to 60) can be placed over the same component to characterize both randomness and spatial variability
- Sensor measurements are considered as ‘perfect’ [Schoefs et al., 2009]
- Random field properties are not affected by loading or deterioration



METHODOLOGY DESCRIPTION
Random field modelling:

Karhunen-Loève decomposition

[Ghanem & Spanos, 2003]

$$Z(x,\theta) = \mu_Z + \sigma_Z \sum_{i=1}^{n} \sqrt{\lambda_i}\,\xi_i f_i(x)$$

where:

μ_Z: mean value

σ_Z: standard deviation

n: terms of the truncated decomposition

ξ_i: set of normal random variables

λ_i: Eigenvalues

$f_i(x)$: Eigenfunctions

Autocorrelation function

$$\rho_{ZZ}(\Delta x) = \exp\left(\frac{-\Delta x}{b}\right);$$

$$with\ b > 0 \text{ and } \Delta x \in [-a; a]$$

Expressions for λ_i and f_i

$$\lambda_i = \frac{2}{b\left(\frac{1}{b^2} + \omega_i^2\right)}$$

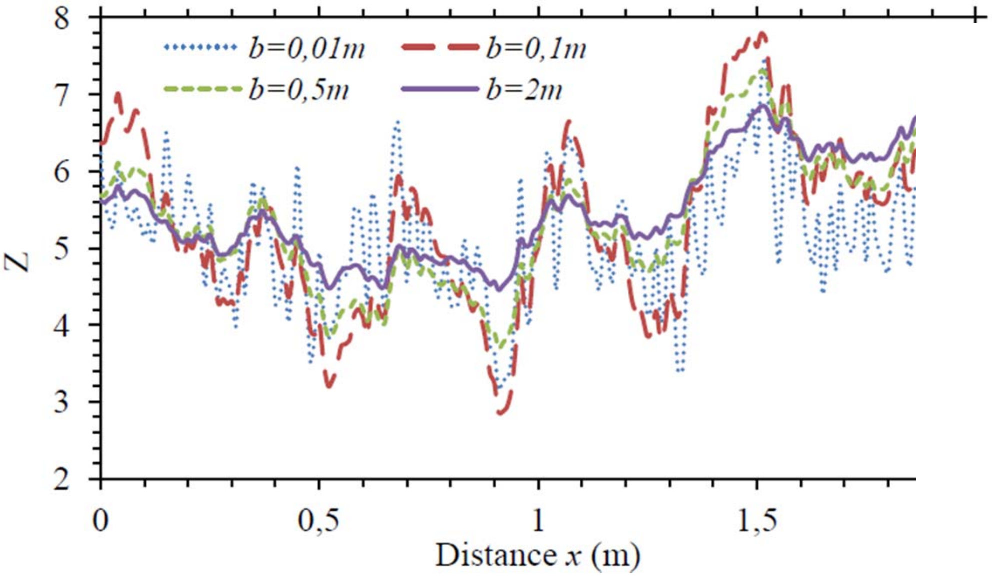
$$f_i(x) = \begin{cases} \dfrac{\cos(\omega_i x)}{\sqrt{a + \dfrac{\sin(2\omega_i a)}{2\omega_i}}} & \text{for } i \text{ even} \\ \dfrac{\sin(\omega_i x)}{\sqrt{a - \dfrac{\sin(2\omega_i a)}{2\omega_i}}} & \text{for } i \text{ odd} \end{cases}$$

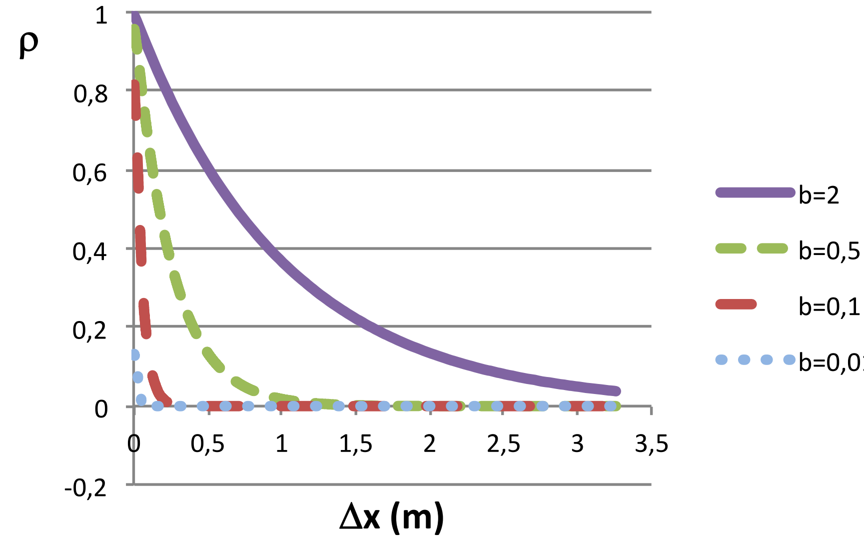
METHODOLOGY DESCRIPTION

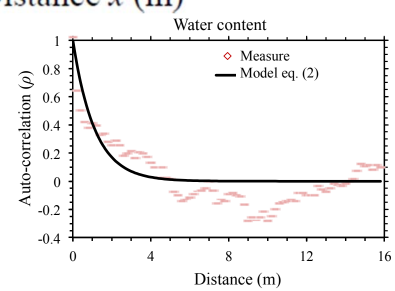
- Examples for several values of b

$$\rho(\Delta x) = exp\left(\frac{-\Delta x}{b}\right)$$

$$0 < b \quad \text{and} \quad \Delta x = x_1 - x_2$$







Fitting: - least squares
- max likelihood

METHODOLOGY DESCRIPTION

Spatial Correlation Threshold:

- To reduce monitoring costs: monitor some zones of a trajectory with sensors separated by "sufficiently short distance L_b"

- This "sufficiently short distance" is called Inspection Distance Threshold (IDT) and estimated as:

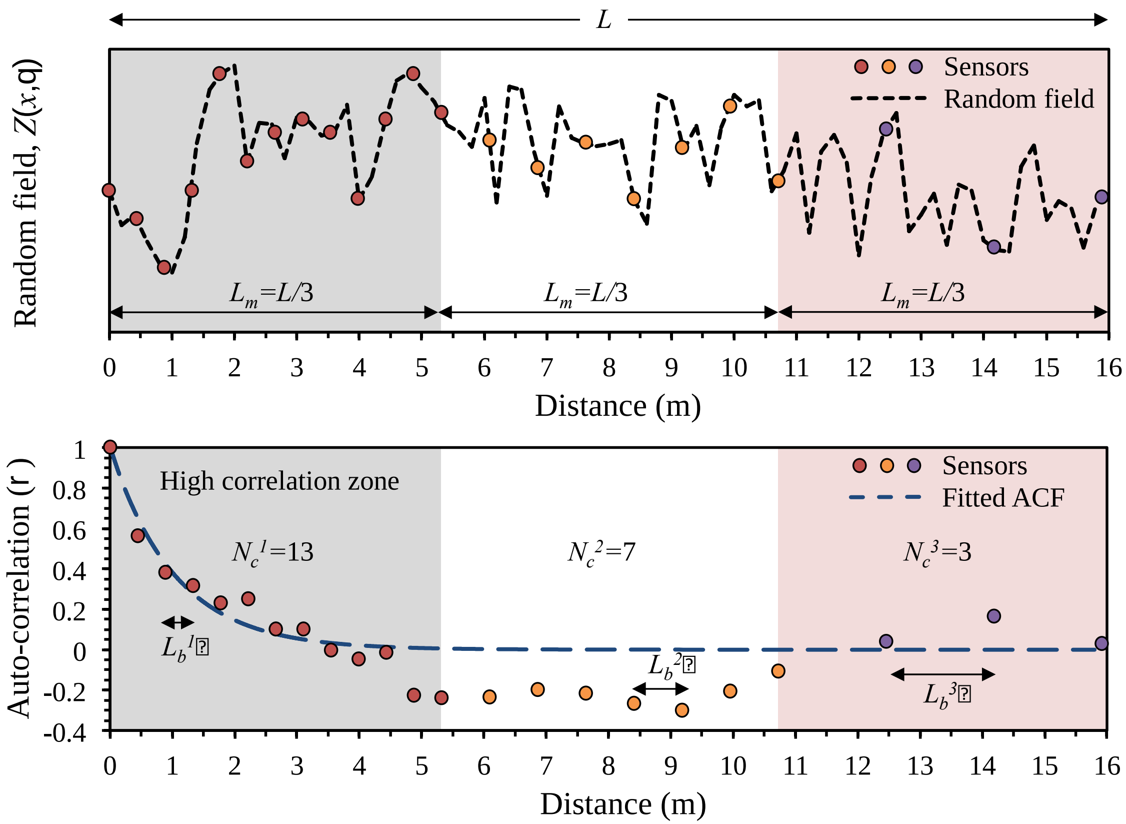
$$\mathrm{IDT} = -!\ \cdot !''\,(\mathrm{SCT})$$

- A Spatial Correlation Threshold (SCT) defines this weak correlation, e.g., [Schoefs et al. (2016)] proposed a value SCT < 0.3

- Thus the non-regular distances of sensors spacing L^i_b should satisfy: $L^i_b \in [0, \mathrm{IDT}]$ in the highly correlated zones.

METHODOLOGY DESCRIPTION
Parameterisation of non-regular spacing:

- The parameterization follows an octree approach.



where:

L : total length

L_m : length of each piece

n_p : number of pieces

N^i_c : number of sensors on the piece i

L^i_b : distance between sensors for the piece i

METHODOLOGY DESCRIPTION
Parameterisation of non-regular spacing:

- The number of sensors in the first piece is:

$$!_{!}^{!} = \mathrm{Round}\left(\frac{!_{!}}{1+\sum_{!!\,!}^{!\,!}\frac{1}{2(!-1)}}\right)$$

- The number of sensors for the pieces $N^{2}_{c}, \ldots, N^{n_p-1}_{c}$ is:

$$!_{!}^{!} = \mathrm{Round}\left(\frac{!_{!}^{!}}{2(!-1)}\right)$$

- The number of sensors for the last piece, $N^{n_p}_{c}$, is the remaining number of sensors

METHODOLOGY DESCRIPTION

Quality estimates and optimisation:

- Random field modelling is used for estimated the **optimal** placement of sensors that **minimises the error on the estimation of the autocorrelation parameter** *b*

- The quality of the estimate is estimated in terms of confidence interval expressed as a percentage Δ of the theoretical (true value) b^{th}:

$$! _{!,!} = ! \left(! _{\hat{!}} \in [(1-\Delta)!^{!!}, (1+\Delta)!^{!!}]\right)$$

where $!_{\hat{!}}$ is the mean value of $\hat{!}$ computed from random field simulations

- Or with the normalized quadratic error of the parameter $\hat{!}$: $!_{!} = \left(\frac{\hat{!} - !^{!!}}{!^{!!}}\right)^{!}$

METHODOLOGY DESCRIPTION

Quality estimates and optimisation:

- Thus, the optimal position and number n_p^{opt} of sensors is obtained by:

$$!_{!}^{"\#} = !"\#\max_{!_{!}}\{!_{!,!}(!_{!})\}$$

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NUMERICAL EXAMPLE
Basic considerations:

- Objective: to optimize the position of sensors in view to reach a good assessment of *b* for an error Δ=10%

- Set of 1D-components (beams) with a very large total length L>>b

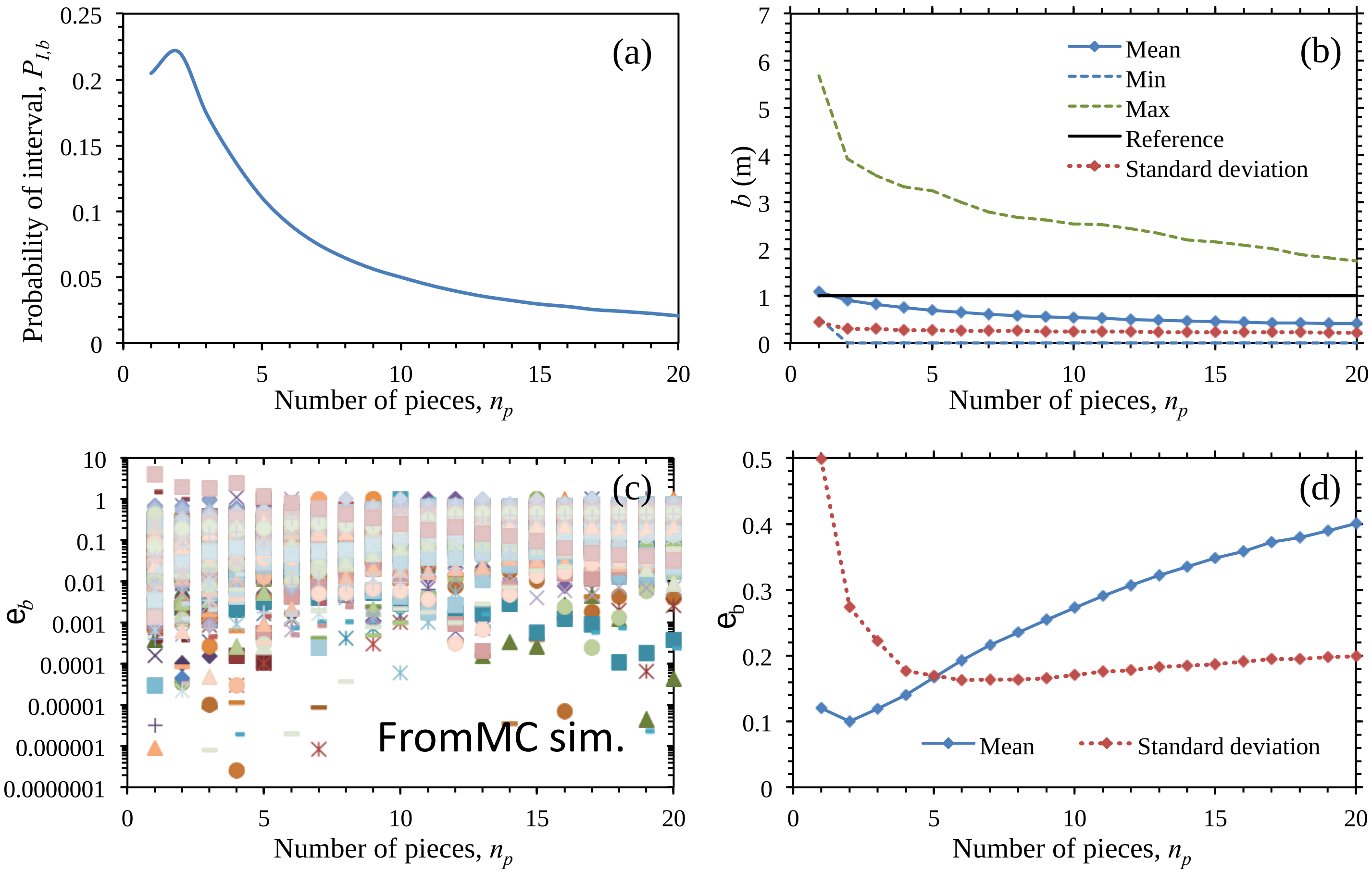
- The Gaussian stationary stochastic field is characterized by: b_{th}=1m, μ_Z=100 and σ_Z=20

- For SCT = 0.4 and b_{th}=1m, IDT = 0.91m

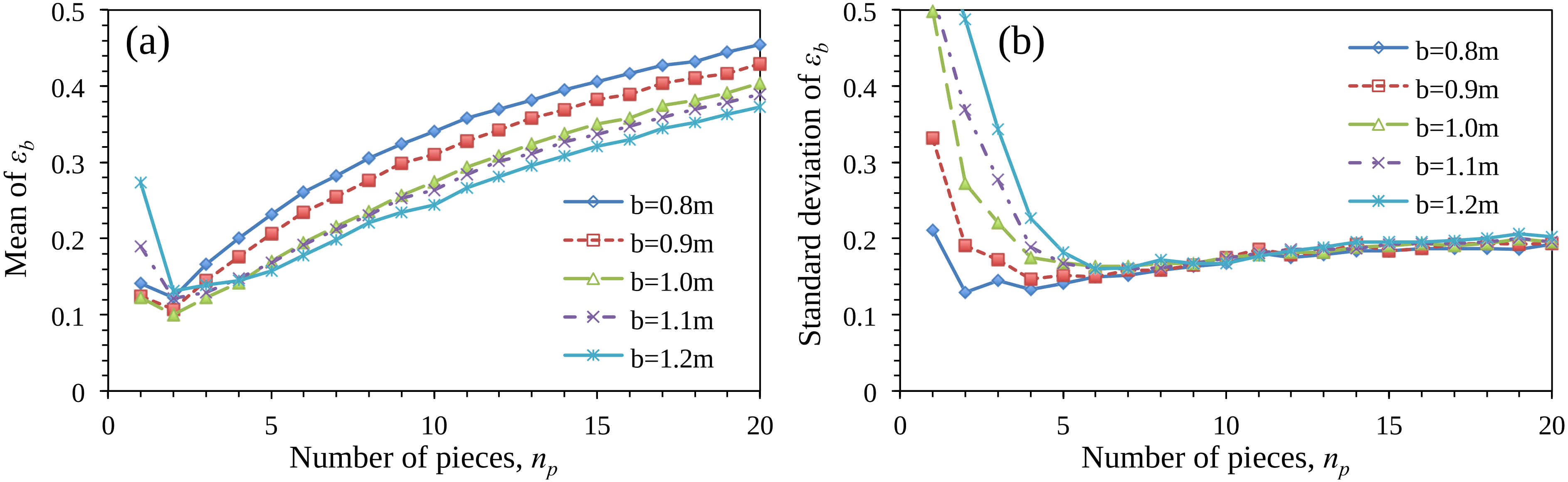
- Fixed values of N_s and L: N_s=200 sensors and L=100m

NUMERICAL EXAMPLE

Results: influence of the number of pieces on $P_{l,b}$, estimated values of b, and ε_b



NUMERICAL EXAMPLE
Results: sensitivity to *a priori* values of *b*



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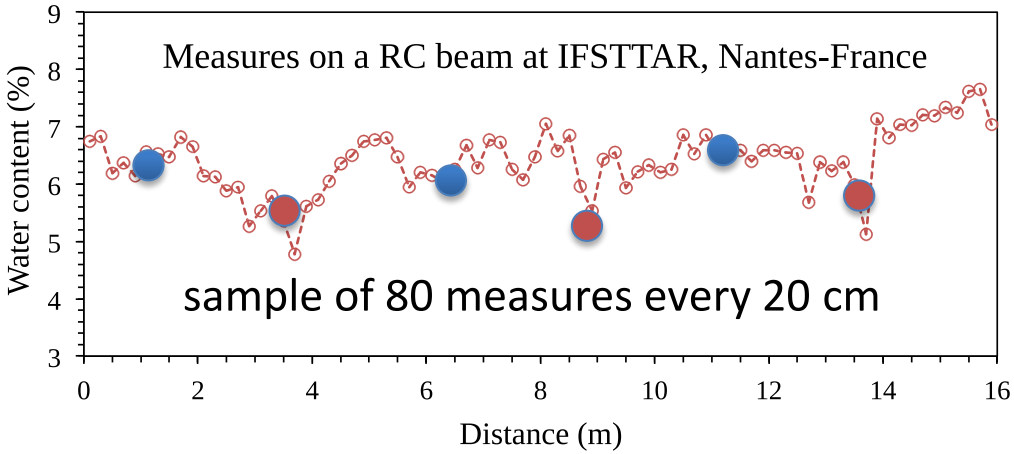
STUDY CASE 1: RC beam
Problem description:

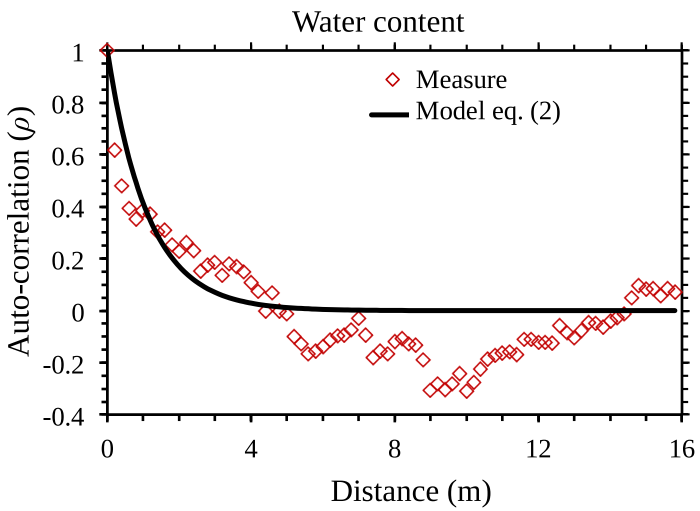
- Objective: to optimize the position of sensors in view to reach a good assessment of *b* for an error Δ=10%

- Data: water content along a **16m long** RC beam
- IFSTTAR Laboratory, Nantes, France [Schoefs et al., 2016].

- Measurements were carried out by using a capacitive NDT tool.

STUDY CASE 1: RC beam
Results: Spatial measurements and autocorrelation

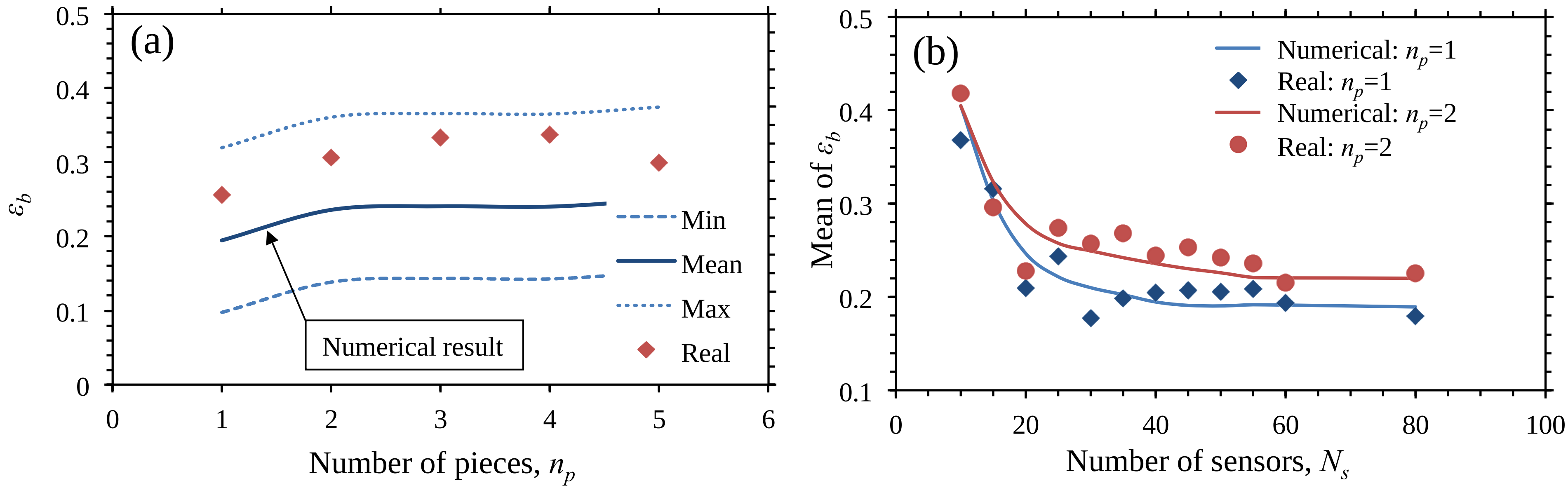




- Random field properties $\mu_W = 6.3\%$, $\sigma_W = 0.67\%$, and $b_W = 0.42$m
- In this case $\mathrm{IDT}_W = 0.5$m. Therefore: $\mathrm{IDT}_W/L = 0.4/16 > 1/40$

STUDY CASE 1: RC beam

Results: error of assessment of *b: from MC simulations of trajectories (numerical) or from experimental result (real)*



- ε_b is minimum for $n_p = 1$ confirming that for the case L < 40IDT the regular spacing is the optimal configuration

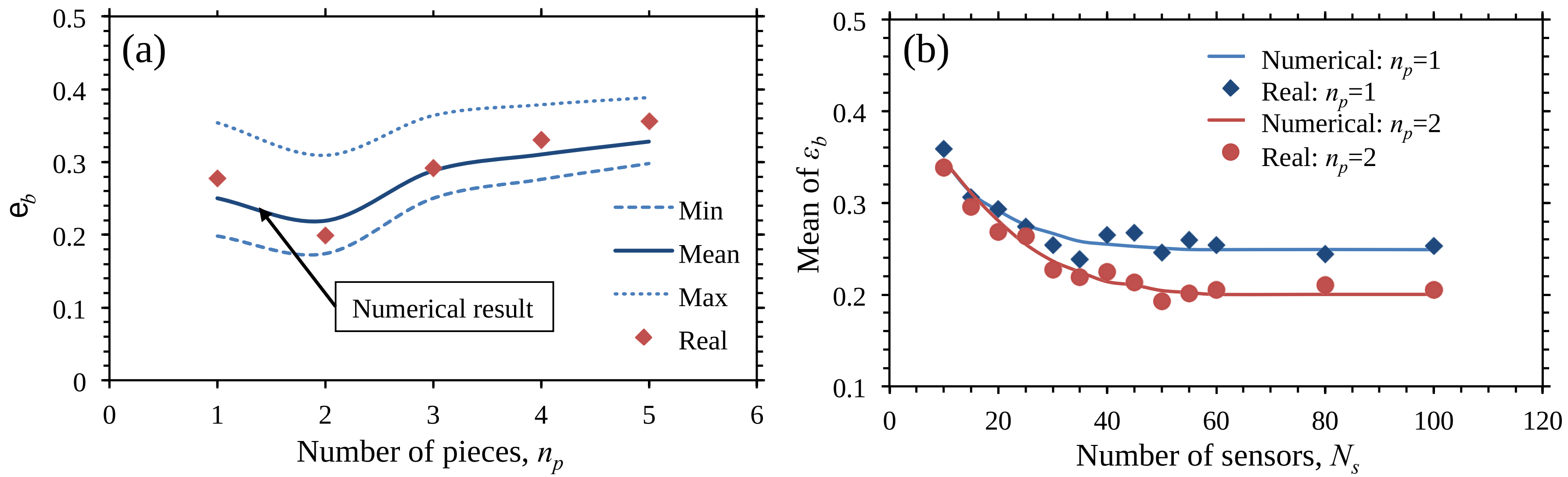
STUDY CASE 2: Steel sheet-pile
Problem description:

- Objective: to optimize the position of sensors in view to reach a good assessment of *b* for an error Δ=10%

- Data: Ultrasonic inspection of the corrosion depth on a corroded 12m length steel sheet-pile of gabion-type wharf in Boulogne, France.

STUDY CASE 2: Steel sheet-pile

Results: error of assessment of *b*



- ε_b is minimum for n_p = 2 confirming that for the case *L* > 40IDT the non-spacing is the optimal configuration

[Schoefs et al., under review]

CONLUSIONS

SV can be assessed and sensor placement can be optimized

Recent studies on chloride ingress from SDT and recent monitoring 22th feb, a wharf with our own SV system (geophysics based system).

Resilient or remaining issues

Identification improved for non stationary fields (SI3M and SURFFEOL regional project): full identification (pdf param, b)

Durability indicator from SHM data>> **need for models** (inv. Ana.l) >> are they able to propagate SV? Not their mathematical function [Rakanotovao, 2017]

VoI through pre-posterior analysis by considering scale of fluctuation (b) as a RV (**meta-RV**, model uncertainty)

Invitation

And position / MS

- Visits

Tomorrow's Megastructures

Preliminary Invitation and Call for Papers

Download the Preliminary Invitation here..

Important Dates

Deadline for receipt of abstracts:	September 15, 2017
Notification of acceptance of abstracts:	October 30, 2017
Deadline for receipt of full papers:	January 15, 2018
Deadline for early-bird registration:	May 31, 2018
IABSE Annual Meetings:	September 17-18, 2018
Symposium:	September 19-21, 2018

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