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COST Action TU1402: Quantifying the Value of Structural Health Monitoring

Editors: Jochen Köhler, Sebastian Thöns, October 2017

Summary

The COST Action TU1402 strives to enhance the benefit of Structural Health Monitoring (SHM) by novel utilization of 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.

The case study portfolio (Working Group 4) is the actual focus of the COST Action TU1402 activities. The case study portfolio buildsupon the decision theoretical framework (Working Group 1), a collection and classification of SHM strategies and structural performance models (Working Group 2) and methods and tools to perform the Value of Information analyses (Working Group 3).

The 9th TU1402 Workshop at the Trinity Colleague in Dublin has been organised in plenary sessions where the case study ideas and the input from Working Group 1, 2 and 3 are presented. Consecutively, the case studies have been worked on in parallel sessions. The developed case study plans and the approaches have been presented and discussed in a plenary session.

This report contains the factsheets describing the case study portfolio. The factsheets build upon the outcomes of the 8th Workshop on Classification at the Technical University of Munich, Germany. Additional documentation (e.g. presentations, videos) can be found on our website www.cost-tu1402.eu.

Part 1: WG4 Case Studies Portfolio

Part 1: WG4 Case Studies Portfolio

Case Study Fact Sheet No. WG4-1

“Digital Test Area Autobahn”

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I. Scope of the fact sheet

This fact sheet is presenting the working progress on a case study. It summarizes the character and context of the study and highlights the major steps and challenges for the implementation of a value of information assessment. Furthermore and outlook on the further development of the case study is given.

II. Abstract

The case study “Digital Test Area Autobahn” is presented. A new built (September 2016) pre-stressed concrete bridge is equipped with different monitoring systems for the detection of traffic load, climatic influence and the reaction of the bridge. Possibilities for the implementation of a VoI analyses are presented.

III. Description of the case study

In the project cluster “Smart Bridge” an adaptive system for information and holistic evaluation in real time is developed. The adaptive system consists of suitable sensors and sensor networks, suitable data analysis, and evaluation methods. The main benefit of the Smart Bridge are economic and safety improvements.
Within the scope of the research project “Digital Test Area Autobahn” “intelligent expansion joints and bearings”, a “monitoring of traffic loads” and “sensor networks” are presented at a bridge in the highway interchange Nuremberg A3/A8. The impacts of traffic loads and climatic influences on the bridge are measured. The reactions of the components with regard to the functionality of individual bridge components are detected locally. By using analytical bridge models and evaluation methods the condition and reliability of the bridge construction and its components can be determined.

IV. Implementation of a VoI analysis

a) Decision maker: The decision maker is the Federal Ministry of Transport and Digital Infrastructure and an additional stakeholder is the local read administration.

b) Regulative constrains: Over the whole lifetime the bridge should generate only small life cycle costs (predictive maintenance strategy). But most important are the aspects fulfill functionality and safety (code requirements). During the whole lifetime the structure must be available for the road users. To fulfill the aspects periodically visual inspections and maintenance and repair measures are implemented

c) System temporal and spatial boundaries:
    - Only one driving direction with two lanes is equipped with sensors.

- The monitoring system should be run on only for five years.

d) Events of interest and the corresponding representation: The bridge was built in 2016 and until now there are no events, which are important for the structural safety or the serviceability of the bridge. In the future heavy-transports can be important events.

1 Events consequences: Until now there are no events and consequences.

e) Indicators (to observe):

The following information is made available through different monitoring systems:

1. Data of the current traffic: The data of the current traffic are recorded by two different systems. The first system is the Roadtraffic Management System (RTMS). The RTMS is a modular system for determining the effects, resistances and the condition as well as the prognosis of the condition of a bridge structure. The following traffic data can be detected by the system: the number of vehicles, vehicle speeds, axle distances, dynamic axle loads per single axle and vehicle gross weight. The data are collected by using strain gauges (Freundt et al. 2014). The second system is the instrumented expansion joint. The following traffic data are detected by the expansion joint: number of vehicles, vehicle speeds, axle distances, dynamic axle loads per single axle and the vehicle gross weight. The vehicle speeds are detected due to the registration of the moving of lamellae by optical displacement sensors. The number of vehicle axle is detected by force sensors. Axle distances are derived from the vehicle speed. The dynamic axle loads are determined by the reaction force between the expansion joint and the frame construction.
2. Climate data: The parameters shadow temperature, wind speed and air moisture are detected at the bridge.
3. Prestressing force curve of the external tendons: The prestressing force curve of the external tendons is determined by accelation sensors.
4. Instrumented expansion joint: For the concrete bridge a modified expansion joint developed by Maurer SE was equipped with strain gauges and force sensors. Herewith the data of the current traffic can be determined as described above. Additionally a self-monitoring of the expansion joint in relation to the natural frequencies and the gap width takes place.
5. Instrumented bearing: The instrumented bearing monitors the range of functionality of the bearing by detecting the load and the sliding gaps. Additionally the distribution of the load, bearing rotation and displacement in dependence of temperature and traffic is determined.
6. Actual object-specific static traffic load and actual object-specific fatigue state: These aspects are detected by the RTMS. For the RTMS the bridge is equipped with strain gauges, inductive displacement transducers, and thermometers. With the measuring results of the sensors and within the framework of data processing, the degree of damage due to fatigue of the reinforced concrete of the bridge can be determined (Freundt et al., 2014).
7. Chronological sequence of the global stiffness: With the sensors of the RTMS and the framework of data processing the time course of the global stiffness of the bridge is determined (Freundt et al., 2014).

f) Decision alternatives – monitoring and or inspection options:

g) Decision alternatives – other measures, repairs, replacement, etc.:

V. Critical appraisal, necessary simplifications

A wide range of parameters are monitored at the concrete bridge. For the implementation of the VoI a reducing of one aspect can be useful. We decided only to analyze the monitoring of the RTMS, because the project has already started and data are available.

VI. Further steps and resources required

- Clarification of the current project status of the different monitoring systems
- Final decision which aspect of the case study should be used for the VoI analyzes.
- Searching for help by performing the VoI analysis. Until now no implementation of a VoI analysis has started. Within this fact sheet the possibilities of the implementation of the VoI should be clarified.

VII. Summary and conclusion

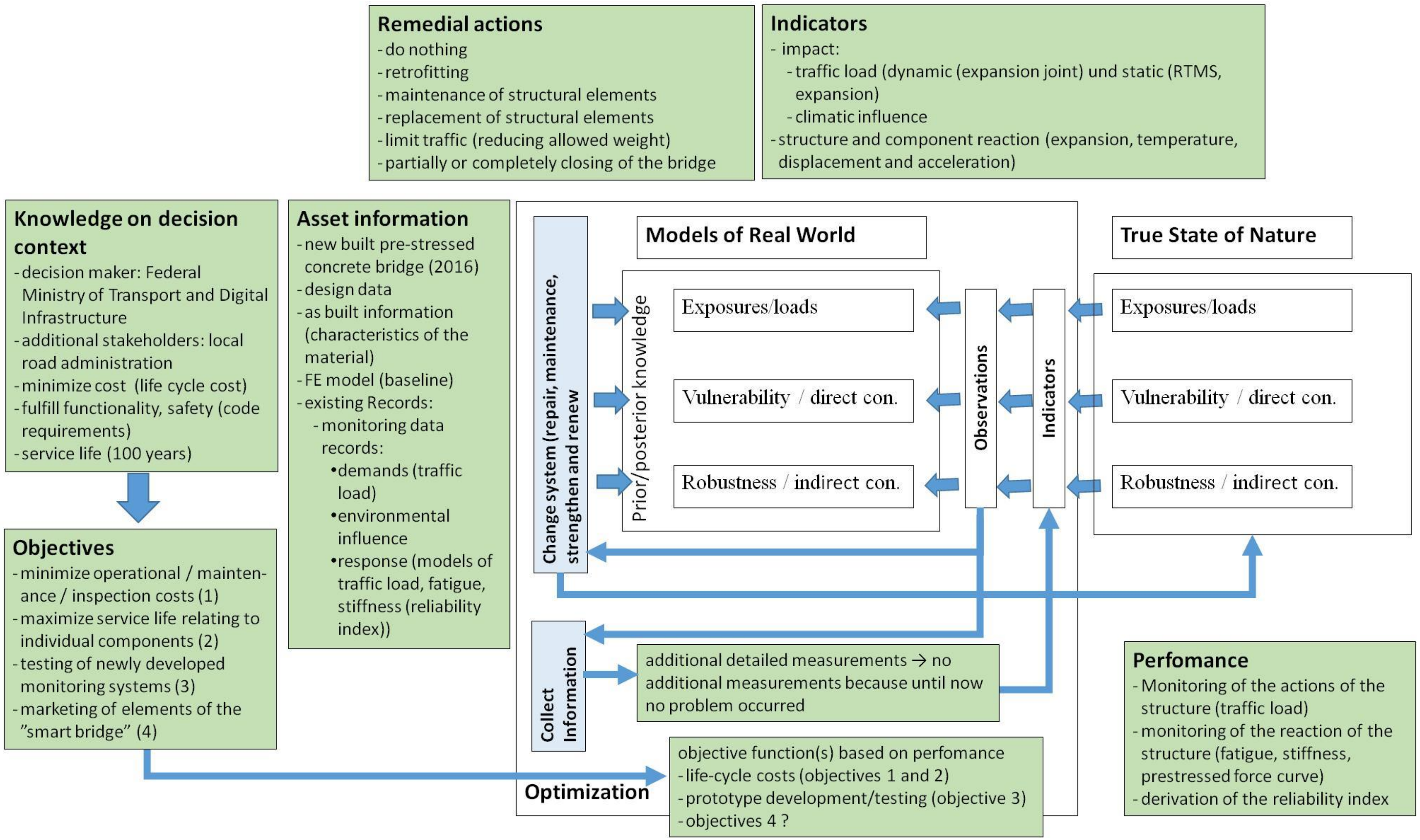

Figure 1: The VoI flow chart of the case study “Digital Test field Autobahn”

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References

Freundt, U., Vogt, R., Böning, S., Pierson, C. and Ehrle, P. (2014). Roadtraffic Management System. Bundesanstalt für Straßenwesen, Heft B 100.

Case Study Fact Sheet No. WG4-2

Instrumentation and monitoring equipment of a harbor concrete structure for optimized maintenance and design.

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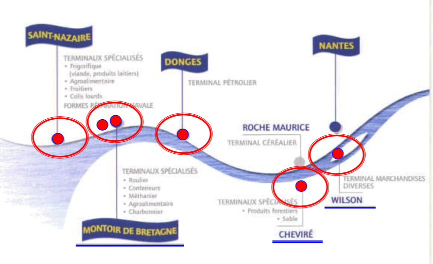
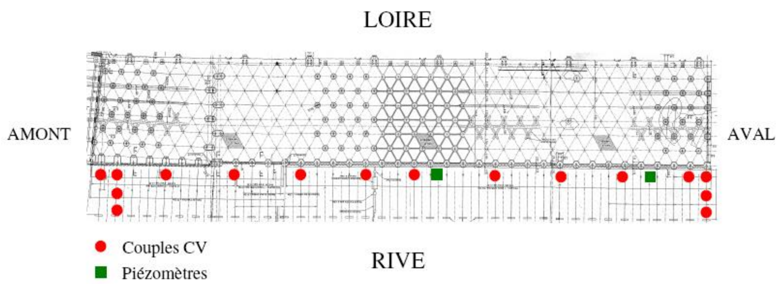
I. Scope of the fact sheet

The scope is to present the study case of an on-pile wharf and the potential dissemination through a movie. Harbor concrete structures are designed to fulfill their function during a long time period (50 to 70 years). Nevertheless, they are submitted to severe conditions (loading and environment) and they are extremely sensitive to damage. They are strategic and often link to highly competitive market. As a consequence, the design and maintenance has to be optimized considering a life cycle management approach.

Since 15 years, TRUST group of University of Nantes has monitored 6 wharves along the Loire River to assess:

- loading in tie-rods
- corrosion of rebars
- chloride ingress





The last one was monitored on february 22th 2017 (picture upon) and is the main focus of the factsheet.

Availability : data are properties of Université de Nantes
FEM models are available as well as degradation models.
\+ Research since 9 years (see related paper in the last section)

II. Abstract

With 80% of European trade exchange and a key role for European defense, harbours play a key socio-economic role. Inspection is very expensive due to the relative inaccessibility of components

(need of a boat, team …) and the economical impact (service breakdown). Thus the added value of SHM and accelerated tests in laboratory can be proved by optimizing the time between inspection and have an optimized planning of maintenance by accounting for the indirect cost impact of service breakdown. The case study proposed here, deals with the instrumentation and the monitoring equipment of a new harbor quay ordered by the Nantes Saint-Nazaire Port's corporate. It was built in 2016-2017. Samples were provided by the manufacturer for characterizing the material (durability indicators). Prior distributions are given by MRGenCi report submitted to JCSS.

To assess the added value of monitoring in this context, several models and methods are required. First existing inspection, maintenance and repair with a matrix of costs is provided. Second, the modeling of stochastic degradation processes governed by chloride ingress is performed at two levels:

- the benchmarking of 7 existing simplified models in terms of (i) uncertainty propagation (ii) ability to be updated from inspection/SHM is provided from French national project EVADEOS (Rakotovao et al., submitted to European Journal of Environmental and Civil Engineering).

- Stochastic Models of chloride ingress were developed and the added value of accelerated tests was proven (see factsheets).

The monitoring suggested here is composed of a set of sensors (humidity, temperature, and 2 for chlorides) and the related costs of installation / device / sensor are known. This redundancy of information will help to measure the added value of this design in comparison with a basic one (chloride only) knowing that stochastic models account for all these measurements and can be updated. The service life time of sensors is under investigation to analyze the complementary information from monitoring / inspection especially when sensors stop functioning.

A final and refined optimization is then suggested by added the knowledge of spatial variability. Recently, spatial variability of chloride ingress was measured and modeled and the topic of spatial assessment of SHM was pointed out (Zagreb workshop of Cost action 1402-March 2017). The objective is to analyze the added value of more sensors for assessing spatial variability in view to compute more accurately the structural reliability.

III. Description of the case study

This concrete structure of 350 m in length and 50 m wide is equipped with sensors for continuous monitoring with the objective of recording the structure performance in time for enhanced safety and optimized maintenance. Two outcomes are expected:

(i) The first one is to provide optimal solutions to analyse the chloride penetration into concrete in marine environment and the corrosion risk assessment

(ii) The second objective is to analyse the real behaviour of the structural parts of the quay: the beams, in order to extend their useful lives, as regards with the calculation standards and to optimize the design of future structures, especially for special components (short beams) that are not considered in Eucocodes.

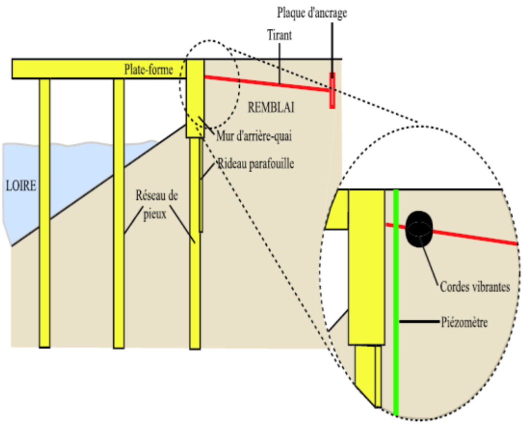
In the context, of the quantification of the value of SHM information, the sets of sensor can be used to detect different level of damage corresponding to different remedial actions:

(i) Resistivity, chloride, humidity and temperature sensors can be used to detect concrete contamination. The remedial action could be to repair concrete.

(ii) Ag electrode can be used to detect the beginning of the corrosion process. The remedial action could be concrete and armature repair

(iii) Fiber sensors (with Brillouin Rayleigh interrogation device) can be used to detect crack propagation. The remedial action could be concrete and armature repair

(iv) Strain sensors allow preventing the risk of complete failure of the structure. It could lead to take the decision to destruct the structure.



According to discussion after Dublin meeting, this factsheet focuses on (i) only.

Two beams have been instrumented for the monitoring of chloride penetration in concrete using resistivity sensors, chlorides sensors, humidity sensors, temperature sensors and optical fiber for the monitoring of strain and the detection of crack. Additionally, extensometers have been embedded within a beam in order measure the strain field inside the structure. Until now, we are able to present the result of the monitoring of the two first months i.e. the concrete process monitoring. The new sensor of UoN aims to reduce the cost when assessing spatial variability. Its added value will be one of the tangible outputs.
We will able to provide accelerated tests during the project to show the added value of these tests when monitoring a new structure knowing that the sensors inside will have a limited life time.
The ageing models will be provided as well as data from other projects (existing chloride profile of old structures in the same area are available to simulate inspection with the same material).
Another key model require for maintenance optimization is the uncertainty of NDT measurement and sensors. They are both available from preliminary studies on chloride ingress measurement.
Finally, technical data about, indicators, measurements, degradation, repair is available from EU project Interreg Duratinet (open web technical platform).

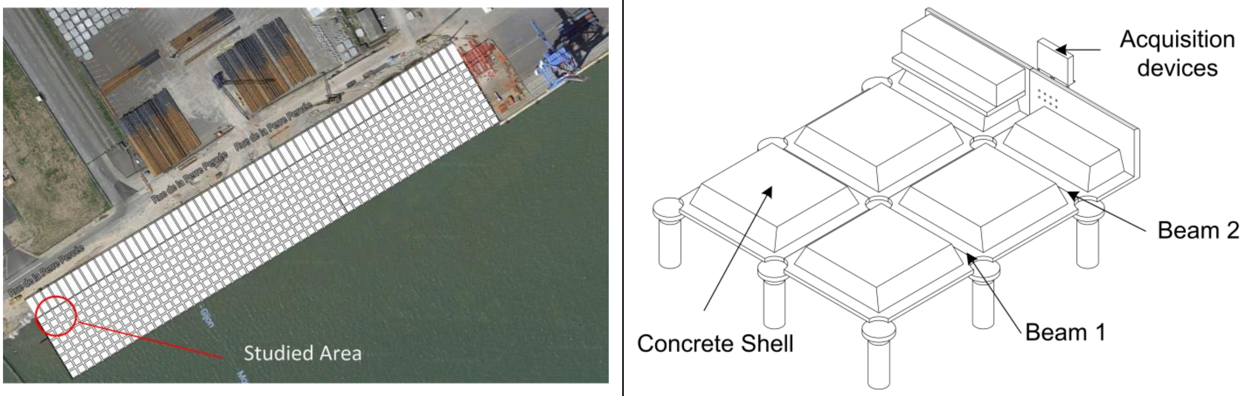


Fig 1 : Structure configuration

Fig 2 : Pictures of the structure

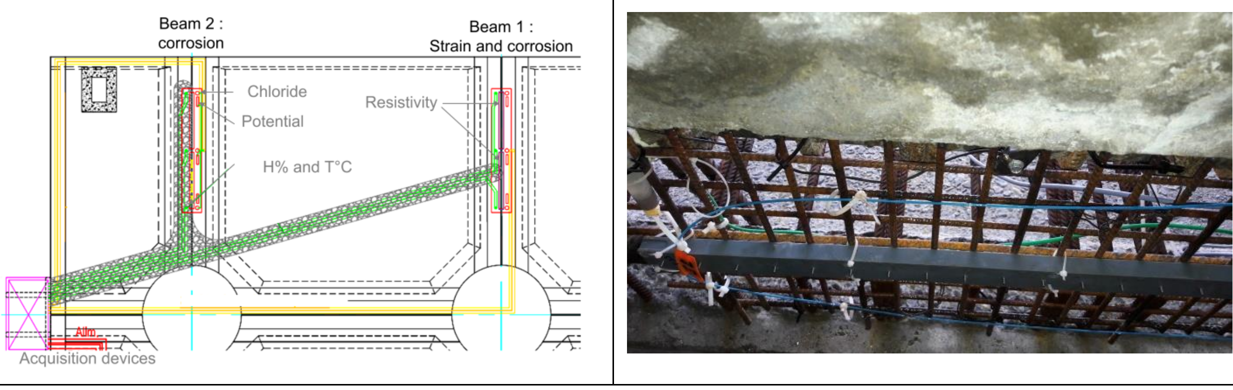


Fig 3 : Instrumentation for resistivity measurement

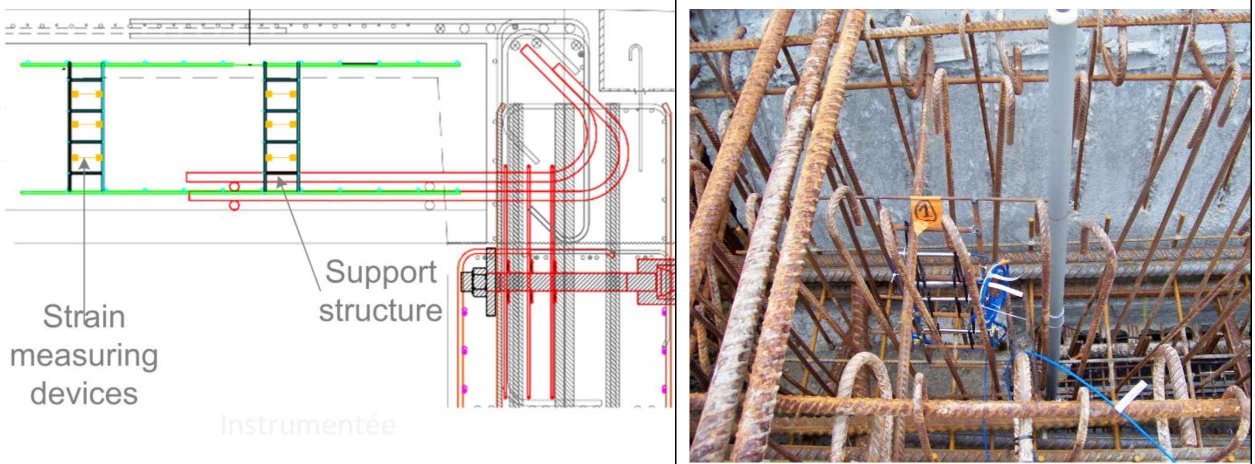


Fig 4 : Instrumentation for strain measurement

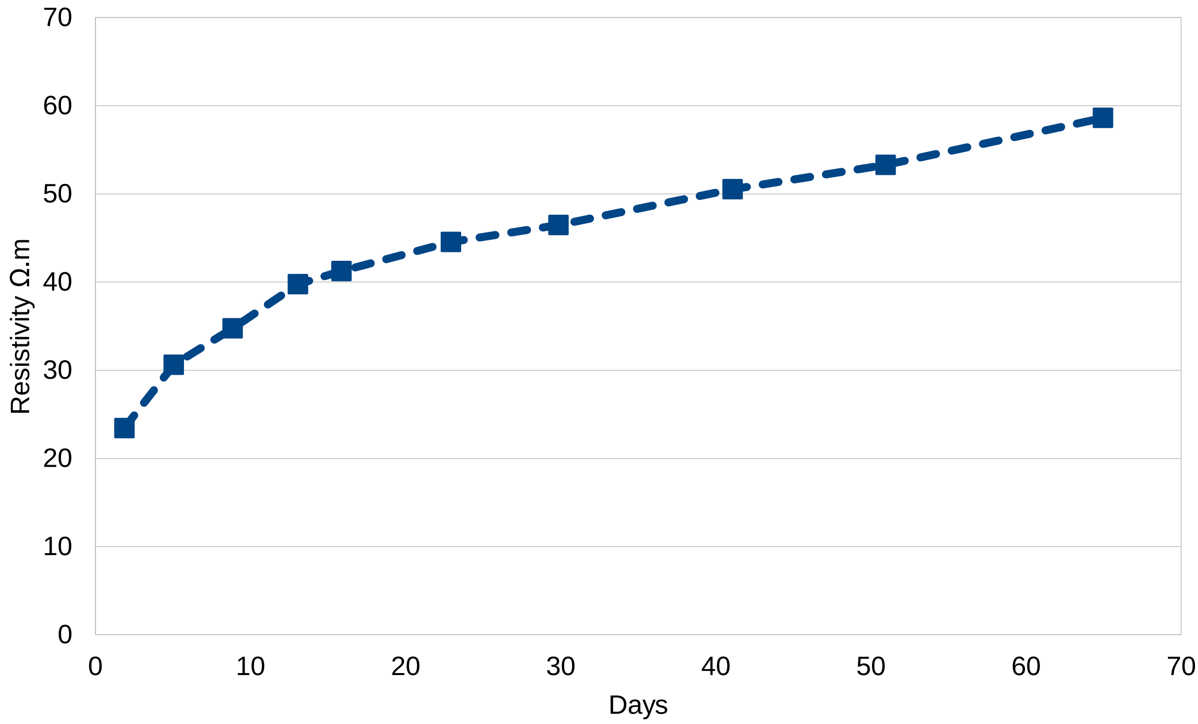
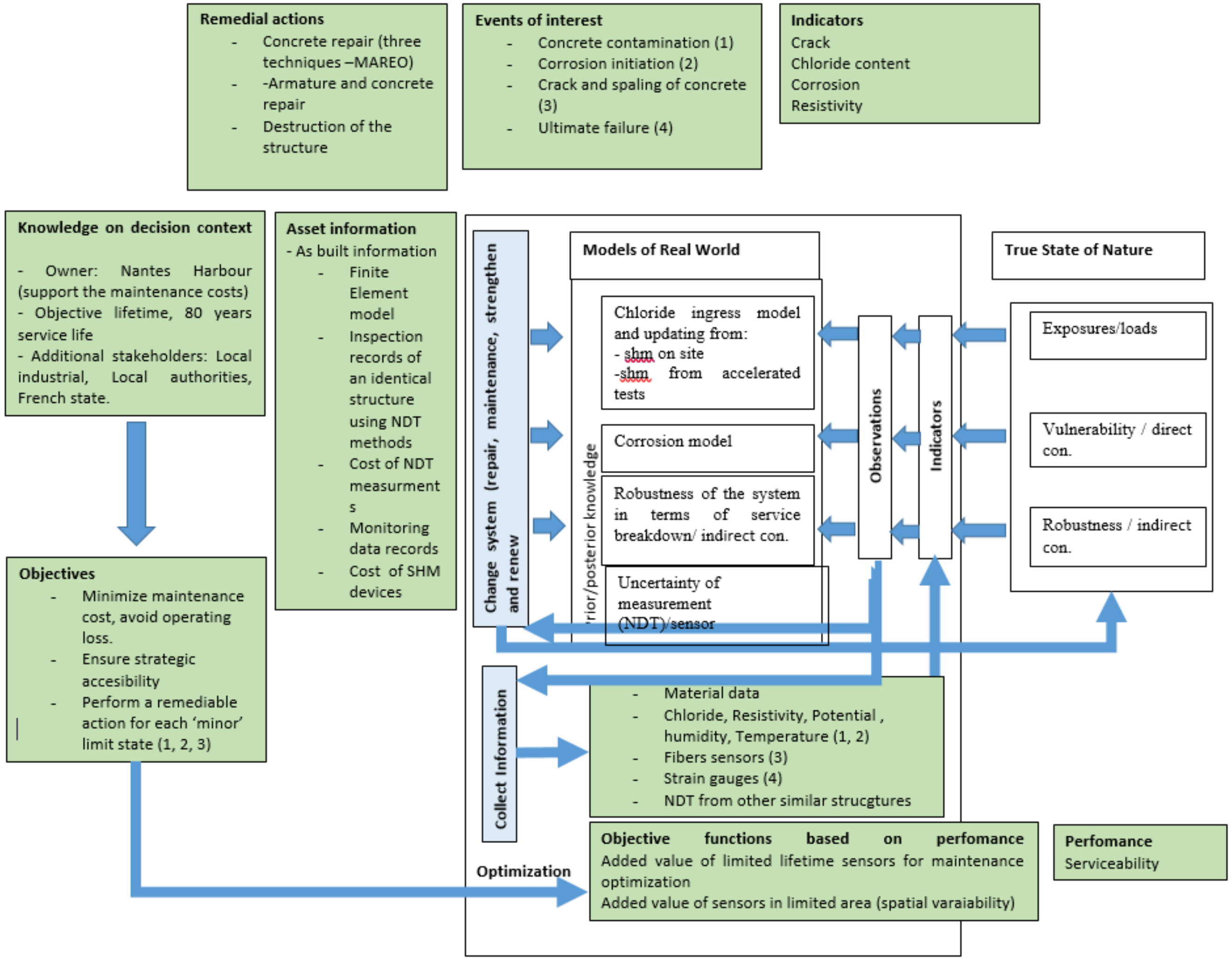


Fig 5 : First result, evolution of the in-situ resistivity within concrete

IV. Implementation of a VoI analysis



V. Collaborations in the COST Action

Need collaboration for estimation of the Value of information (NDT methods vs SHM instrumentation)

Associated Publication :

Yann Lecieux, Cyril Lupi, Virginie Gaillard, Romain Guyard, Dominique Leduc, Emmanuel Rozière, Michel Roche and Franck Schoefs, Instrumentation d'un quai portuaire pour le suivi de vieillissement de l'ouvrage : présentation des objectifs, des protocoles et des résultats de mesure issus du suivi de fabrication, AUGC 2017.

Associated Topic Research: detection of chloride in concrete using DC electrical resistivity measurement

Corrosion of steel reinforcements is the main cause of deterioration of reinforced concrete structures in marine environment. It is mainly due to the penetration of chloride ions in the concrete porosity. The severity of the pathology increases with the content of NaCl ions. It is not easy to measure directly the initiation of corrosion. Consequently we often prefer to observe a phenomenon linked to the corrosion. The main problem of the resistivity measurements in concrete is the dispersion of results mainly linked to the material heterogeneity and electrical contact limitations. To take into account the uncertainty of the measurement, we used a measurement device and sensors based on Geoelectrical Imaging methods. It gives richer information than the one delivered by conventional resistivity measurement sensors. Thus, it allows the use of statistical analysis and quality assessment methods. In this structure we have placed several types of sensors to have redundancy in view to analyse the effect of less information on the decision.

Collaborations: NTNU for spatial variability of a bridge in chloride environment.

During the last 9 years: 25 related publications and 3 patents of the group directly in link with the case study of wharves (collaboration with U. Los Andes, UCD, UCC and TCD)

PUBLICATIONS

[1] M.L.M. François, Y. Lecieux, D. Leduc, C. Lupi and E. Rozière, An Embedded 3D Strain Tensor Sensor Based on the Eshelby's Inclusion, Experimental Mechanics, 2017, DOI 10.1007/s11340-017-0266-2

[2] Romain Guyard, Dominique Leduc, Yann Lecieux, and Cyril Lupi, Superposition of fiber Bragg and LPG gratings for embedded strain measurement.Compte rendus de l'académie des sciences, 2016.

[3] Romain Guyard, Dominique Leduc, Yann Lecieux, and Cyril Lupi, Global overview of the sensitivity of LPG to strain, Optic and Laser Technology, 2015.

[4] Yann Lecieux and Franck Schoefs and Stéphanie Bonnet and Trystan Lecieux and Sergio Palma Lopes, Quantification and uncertainty analysis of a structural monitoring device : detection of chloride in concrete using DC electrical resistivity measurement, Nondestructive Testing and Evaluation, 2014.

[5] D. Leduc, Y. Lecieux, P.-A. Morvan, and C. Lupi. Architecture of optical fiber sensor for the simultaneous measurement of axial and radial strains. Smart Material Structures, 22(7) :075002, July 2013.

[9] El Hajj B., Castanier B., Schoefs F., Yeung T., “A condition-based deterioration model for the stochastic dependency of corrosion rate and crack propagation in corroded concrete structure", Computer Aided Civil and Infrastructure Engineering, Vol. 32, Issue 1/Jan 2017, 18-33; doi: 10.1111/mice.12208- 2017
[10] Tran T.B., Bastidas-Arteaga E., Schoefs F., “Improved Bayesian network configurations for probabilistic identification of degradation mechanisms: application to chloride ingress”, Structure and Infrastructure Engineering, Published on line Oct. 2, 2015, Vol. 12, Issue: 9, 1162-1176, doi: 10.1080/15732479.2015.1086387 - 2016
[11] Tran T.B., Bastidas-Arteaga E., Schoefs F., “Improved Bayesian network configurations for random variable identification of concrete chlorination models”, Material and Stuctures, published online: 04 Feb 2016, Volume: 49 issue: 11, 4705-4718, NOV 2016, doi: 10.1617/s11527-016-0818-4 – 2016
[12] Bastidas-Arteaga E., Schoefs F., ”Sustainable Maintenance and Repair of RC Coastal Structures”, Proceedings of the Institution of Civil Engineers - Maritime Engineering Journal, Volume 168, Issue 4 Dec 2015, 162-173 doi: 10.1680/jmaen.14.00018- 2015
[13] Torres-Luque M., Bastidas-Arteaga E., Schoefs F., Sanchez-Silva M., Osma J.F., “Non-destructive methods for measuring chloride ingress into concrete: State-of-the-art and future challenges”, Construction and Building Materials (CBM), Published on line 12-JULY-2014, 68/15 oct.2014, 68-81, doi: 10.1016/j.conbuildmat.2014.06.009 – 2014
[14] Schoefs F., Le K.T., Lanata F., "Response surface meta-models for the assessment of embankment frictional angle stochastic properties from monitoring: application to harbour structures", Computers and Geotechnics, 53, Issue Sept. 2013, 122-132, doi: 10.1016/j.compgeo.2013.05.005 – 2013.
[15] O’Connor A., Sheils E., Breysse D., Schoefs F., “Markovian Bridge Maintenance Planning Incorporating Corrosion Initiation and Non-Linear Deterioration”, ASCE Journal of Bridge Engineering Vol. 18, Issue 3/March 2013, 189-199, doi: 10.1061/(ASCE)BE.1943-5592.0000342– 2013.
[16] Boéro J., Schoefs F., Yáñez-Godoy H., Capra B., ” Time-function reliability of harbour infrastructures from stochastic modelling of corrosion”, European Journal of Environmental and Civil Engineering, 16:10/Nov. 2012, 1187-1201, doi: 10.1080/19648189.2012.688611– 2012
[17] Lanata F., Schoefs F., “Multi-algorithm approach for identification of structural behaviour of complex structures under cyclic environmental loading”, Structural Health Monitoring (SAGE), 11:1/January 2012, pp. 51-67, doi: 10.1177/1475921710397711
[18] Bastidas-Arteaga E., Schoefs F., “Stochastic improvement of inspection and maintenance of corroding reinforced concrete structures”, Engineering Structures, Vol. 41/August2012, pp. 50–62, doi: 10.1016/10.1016/j.engstruct.2012.03.011 – 2012.
[19] Bastidas-Arteaga E., Chateauneuf A., Sanchez-Silva M., Bressolette P. and Schoefs F., “A comprehensive probabilistic model of chloride ingress in unsaturated concrete”, Engineering Structures, Available online 23 December 2010, 33 (2011) 720–730, doi: 10.1016/j.engstruct.2010.11.008
[20] Schoefs F., Yáñez-Godoy H., Lanata F., “Polynomial Chaos Representation for Identification of Mechanical Characteristics of Instrumented Structures: Application to a Pile Supported Wharf”, Computer Aided Civil And Infrastructure Engineering, spec. Issue “Structural Health Monitoring”, First published online 2nd November 2010, Vol. 26, Issue 3, pages 173–189, April 2011, doi: 10.1111/j.1467-8667.2010.00683.x
[21] Bastidas-Arteaga E., Schoefs F., Chateauneuf A., Sanchez-Silva M., Capra B., “Probabilistic evaluation of the sustainability of maintenance strategies for RC structures exposed to chloride ingress”, International Journal of Engineering Under Uncertainty: Hazards, Assessment and Mitigation (IJEUU:HAM), Special Issue, Vol. 2, pages 61-74, 2010.
[22] Pakrashi V., Schoefs F., Memet J.B., O’Connor A., "ROC dependent event isolation method for image processing based assessment of corroded harbour structures", Structure and Infrastructure

Engineering: Maintenance, Management, Life-Cycle Design and performance (NSIE), First Published on line: 30 April 2008, Vol. 6, Issue 3, (6(03)), pp. 365-378, doi: dx.doi.org/ 10.1080/15732470701718072 – 2010
[23] Boéro J., Schoefs F., Capra B., Rouxel N., ”Risk management of French harbour structures. Part 1: Description of built assets”, PARALIA, http://www.paralia.fr/revue/rpa0906.pdf, Published on line December 21th 2009, Vol. 2, p 6.1-6.11.
[24] Boéro J., Schoefs F., Capra B., Rouxel N., ”Risk management of French harbour structures. Part 2: Current practices, needs– Experience feedback of owners”, PARALIA, http://www.paralia.fr/revue/rpa0907.pdf, Published on line December 21th 2009, Vol. 2, pp 6.13-6.24.
[25] Yáñez-Godoy H., Schoefs F., Casari P., “Statistical Analysis of the Effects of Building Conditions on the Initial Loadings of On-piles Quays”, Structural Health Monitoring (SAGE), Published on line April 1st 2008; Published September 1st 2008, Vol. 7, N°3, pp. 245-263, doi: dx.doi.org/10.1177/1475921708090563

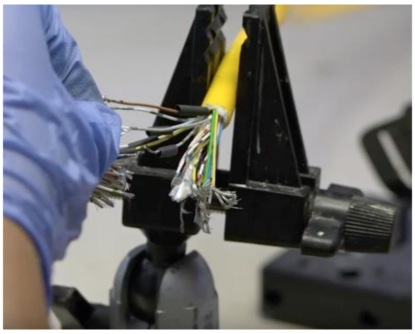
PATENTS

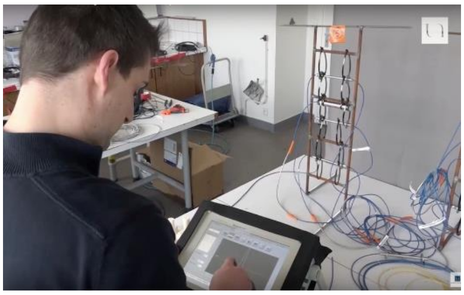
M. François, Y. Lecieux, D. Leduc, and C. Lupi. Dispositif de mesure de contraintes. patent n° WO 2014140496 (A1), 2014.

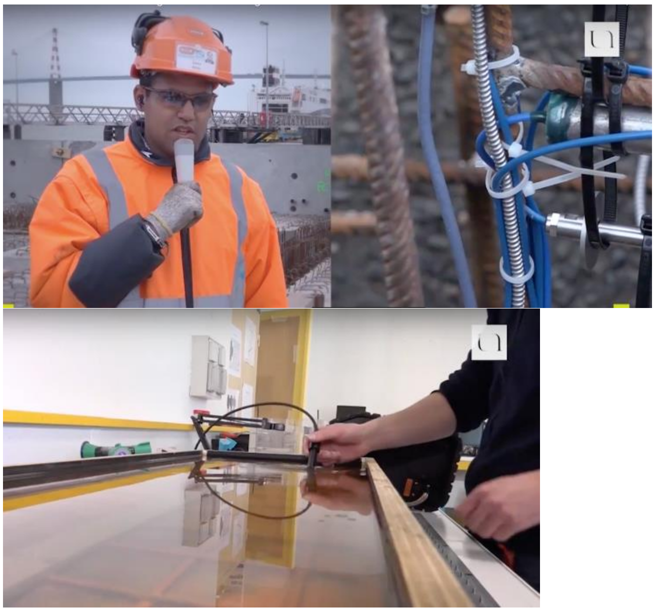
M.M. Torres Luque (U Los Andes), J.F. Osma Cruz (U Los Andes), E.M. Sanchez Silva (U Los Andes), E. Bastidas Arteaga, F. Schoefs « ChorDetect: System for assessing chloride concentration and corresponding method and sensor », 9 juillet 2015, N° 219515EP00, 2015

VI. Dissemination from a movie (cost 6 000 euros – support from Cost Action: 3000 euros).

To increase the impact of the case studies a movie was prepared from the lab to the implementation on site. Supplementary sequences are needed (data transfer, treatment and analysis). The making of the movie and the translation in english of the comments are required. Université de Nantes already spent 3000 euros and we need complementary contributions from COST action to end this dissemination.
Few pictures from the film:







Case Study Fact Sheet No. WG4-3

Optimising in-situ testing for historic masonry structures

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Dimitris Diamantidis, OTH Regensburg
Maria Giovanna Masciotta, University of Minho, ISISE

Scope of the fact sheet

The working progress of the case study focused on optimising the use of destructive and non-destructive testing (DT and NDT) for heritage masonry structures is presented. The character, context of the study, key steps and challenges for the implementation of a value of information assessment are outlined.

I. Abstract

Masonry constructions have a long history in Central Europe. Such structures were built by means of various techniques, using materials, the properties of which exhibit a considerable scatter dependent on periods of construction, region-specific soil resources and manufacturing procedures. Moreover, a significant spatial variability within the structural system is often observed due to various deterioration mechanisms affecting the properties of masonry constituents. Therefore, it is of crucial importance to obtain case-specific information on the properties of existing masonry structures.

Focusing on heritage structures, various non-destructive tests (NDTs) – such as hardness tests – and minor-destructive techniques – using cores of small diameter – are commonly applied, while application of destructive testing is minimized. The use of NDTs is inherently associated with measurement uncertainty that affects the estimates of material properties. Annex I of ISO 13822 (2010), focused on heritage structures, indicates that "in some cases, destructive tests may be necessary to calibrate NDT". Such statement is rather weak since the prevalent practice of structural health monitoring is to calibrate NDTs by destructive tests to provide sufficient input data for structural reliability analysis. This is in agreement with the scientific findings for concrete and stone masonry, Pucinotti (2015) and Kahraman (2001).

The case study considers 14 historic stone and brick masonry structures that were built in what is now the territory of the Czech Republic in the period from the 17th to the 20th century. For each building, the results obtained by two types of NDTs are verified through destructive tests of masonry units, while a well-calibrated NDT is used to estimate mortar strength. The ultimate goal of the case study is to optimise the number of DTs for spot monitoring to support reliability verification at a time instant by credible input data with minimum destructive interventions. The study encompasses a database of historic masonry constructions, aiming at providing general guidance for the survey and assessment of such structures.

II. Description of the case study

Table 1 provides basic information on the experimental database related to the strength of masonry units in historic structures. All the data are from the database of the authors; some of them were published previously as indicated in the note below Table 1. The NDT results have been obtained either by a Schmidt rebound hammer or by a certified drill supplemented by an indenter, a revolution counter, a load gauge and by a 6mm bit, Witzany et al. (2016b). Figure 1 indicates the variation of test uncertainty – ratios of DT to NDT masonry unit strength, $\theta = f_{b,DT} / f_{b,NDT}$ – with $f_{b,DT}$. The test uncertainty for both the NDTs exhibits a clear pattern with $f_{b,DT}$, highlighting the need for re-calibration.

Table 1: Basic information about the experimental database for strength of masonry units.

No.	Use of building	Built in	Masonry units	Number of measurements		
				DT	Schmidt*	drill*
1	vicarage	17th	sandstone	3	3	3
2	church**	17th	sandstone	11	11	11
			bricks	6	6	6
			pudding stone	1	1	1
3	printing works***	1930s	bricks	18	18	17
4	residential	end of 19th	bricks	4	4	4
5	offices, storage	1890	bricks	6	6	3
6	monastery, barrack	1638	bricks	11	10	8
			marlstone	3	3	3
7	offices, archive	early 20th	bricks	4	4	2
			marlstone	2	2	0
8	textile mill	second half of 19th	bricks	6	6	4
9	boiler house	1959	bricks	4	4	1
			unspecified stone	1	1	1
10	water mill	1930	bricks	4	4	4
			unspecified stone	1	1	0
11	residential	1867	bricks	6	6	3
			granite	1	1	0
12	engineering works	1870	bricks	5	5	5
13	residential	1890	bricks	2	2	0
			marlstone	1	1	0
14	residential	1871	bricks	6	6	0

*Number of measurements at locations verified by DT. **Witzany et al. (2016a). ***Witzany et al. (2016b).

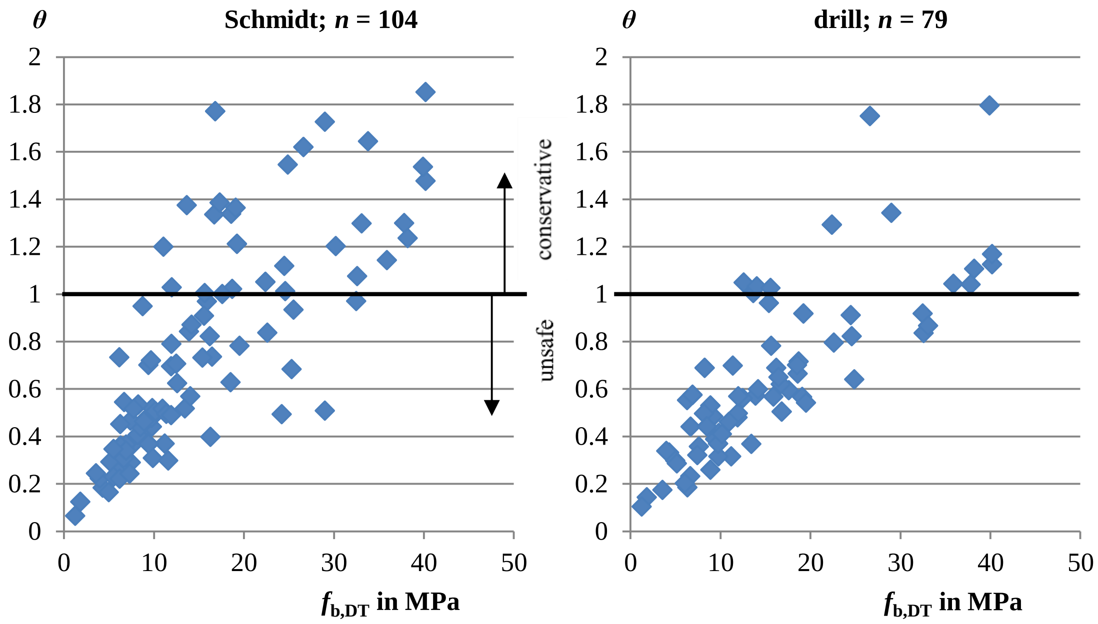


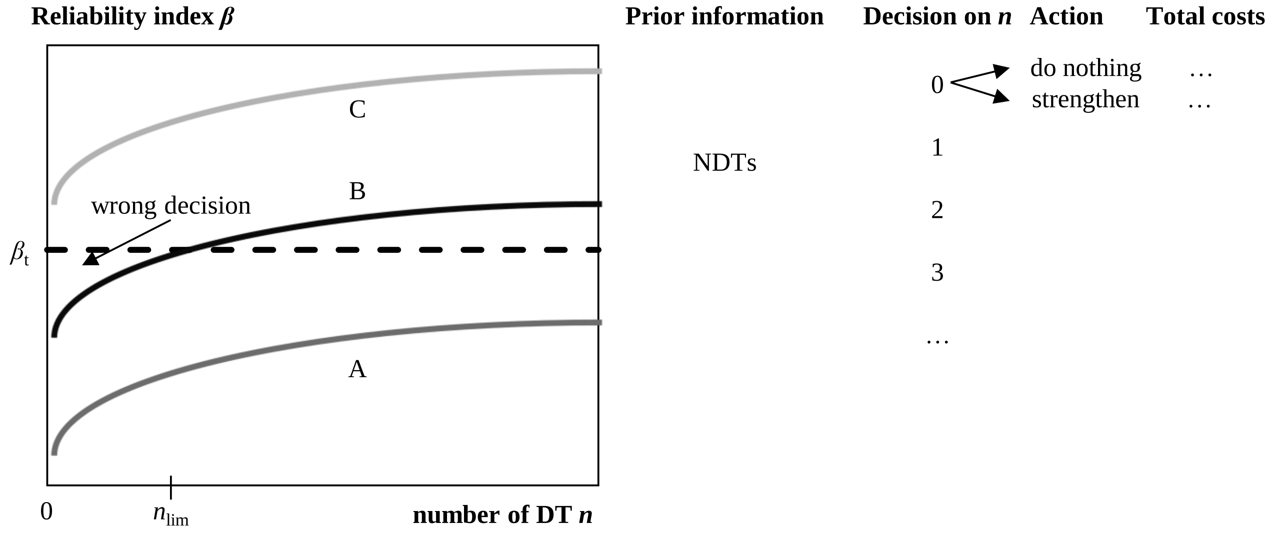
Figure 1: Variation of θ with $f_{b,DT}$

The basic assumption is that there is insufficient prior information regarding material properties of historic masonry and NDTs need to be made in all cases. Number of NDTs is not optimised in the following as the plan for NDTs is commonly based on the need to verify homogeneity of the material. The following steps have to be taken to achieve the envisaged aim – optimisation of DTs number, n:

1. Quantification of uncertainties related to the NDTs validated for structure-specific conditions by several DTs, n ranging from n = 0 to some reasonable upper limit, say, 10 for one structural member for which homogeneous material properties can be assumed.
2. Consideration of the test uncertainties in reliability analyses of typical masonry members exposed to imposed and climatic actions. Seismic actions are beyond the scope of this study.
3. Preliminary optimisation will be based on the simplifying assumption that the reliability of the structure under investigation should be verified for the target level β_t given in standards for a specified reference period. In this case, the utility function can be simplified to:

$$C_{DT}(n) + C_{strengthening} \times P_{wrong\ decision} \tag{1}$$

where C_{DT} = cost of n destructive tests including possible impact on the cultural heritage value; $C_{strengthening}$ = cost of strengthening to upgrade the structure to comply with β_t; and $P_{wrong\ decision}$ = probability of making a wrong decision on strengthening in case structural reliability can be proven to be above β_t if a sufficient number of DTs is taken. Figure 2a) shows the variation of the estimated reliability index β over the number of DTs. An increasing n leads to reduced uncertainty and, in general case, to increasing β. As for structure B, the assessment based on low n ($n < n_{lim}$) will lead to false indication of the need to strengthen. Indeed, structure A should be strengthened and increased n will help to select an optimum strategy for repair. In what concern structure C, NDTs already indicate sufficient reliability and no further action is needed. It is expected that, in accordance with ISO 13822 (2010), a two-phase survey will be recommended – a preliminary assessment based on NDTs followed by a detailed assessment based on the optimum DTs number n_{opt}. An adaptive scheme to assess n_{opt} on the basis of the preliminary assessment should be proposed.



a) b)
Figure 2: a) Variation of β with n; b) Scheme of the decision tree for full risk optimisation.

4. A detailed, full risk pre-posterior analysis could later improve the results of the preliminary optimisation. This would, however, require specification of the failure consequences over a

reference period (both inputs are difficult to assess for heritage structures). Scheme of the decision tree is outlined in Figure 2b).

III. Implementation of a VoI analysis

The most important attributes of the VoI analysis for the case study under consideration are presented in Figure 3.

IV. Critical appraisal, necessary simplifications

1. As a full risk optimisation is difficult due to the need of estimating failure consequences over a reference period, the analysis can be simplified by accepting a certain target level as described in Section III.
2. As this study attempts to provide guidance for a general case (not for a selected particular structure), the optimisation needs to take into account that reliability of an investigated structure is either low or high. Effect of this on n_{opt} will be investigated by a parametric study and an adaptive scheme to optimise n on the basis of preliminary assessment will be proposed.

V. Further steps and resources required

The issues for the support from the COST TU1402 network are listed below:

1. What target reliability level and what reference period are appropriate for historic structures with cultural heritage value?
2. How to estimate failure consequences? Lower bound estimates can be based on insurance estimates, structural costs of replica or on inverse cost optimisation corresponding to target reliability accepted by national cultural heritage authorities.

VI. Summary and preliminary conclusions

Masonry structures have been built by various techniques using different materials, properties of which exhibit a considerable scatter dependent on periods of construction and region-specific conditions. This is why it is essential to obtain case-specific information on masonry properties. The submitted study, which is focused on the estimation of masonry mechanical properties by destructive and non-destructive techniques (DT and NDT), leads to the following conclusions:

- The crude NDT estimates can be improved by three different approaches: (1) Calibration using DT of a particular structure, in which the strength of masonry units is expected to vary within a reasonable range. (2) Use of an updated NDT calibration relationship by considering a large database for historic masonry units. (3) Implementation of hardness-based techniques combined with ultrasonic verifications.
- Considering approach (1), the use of NDT results without calibration through DT cannot be recommended for historic masonry.

Acknowledgements

This study has been supported by the Ministry of Education, Youth and Sports of the Czech Republic under Grant LD15037 and by the Czech Science Foundation under Grant 16-11378S.

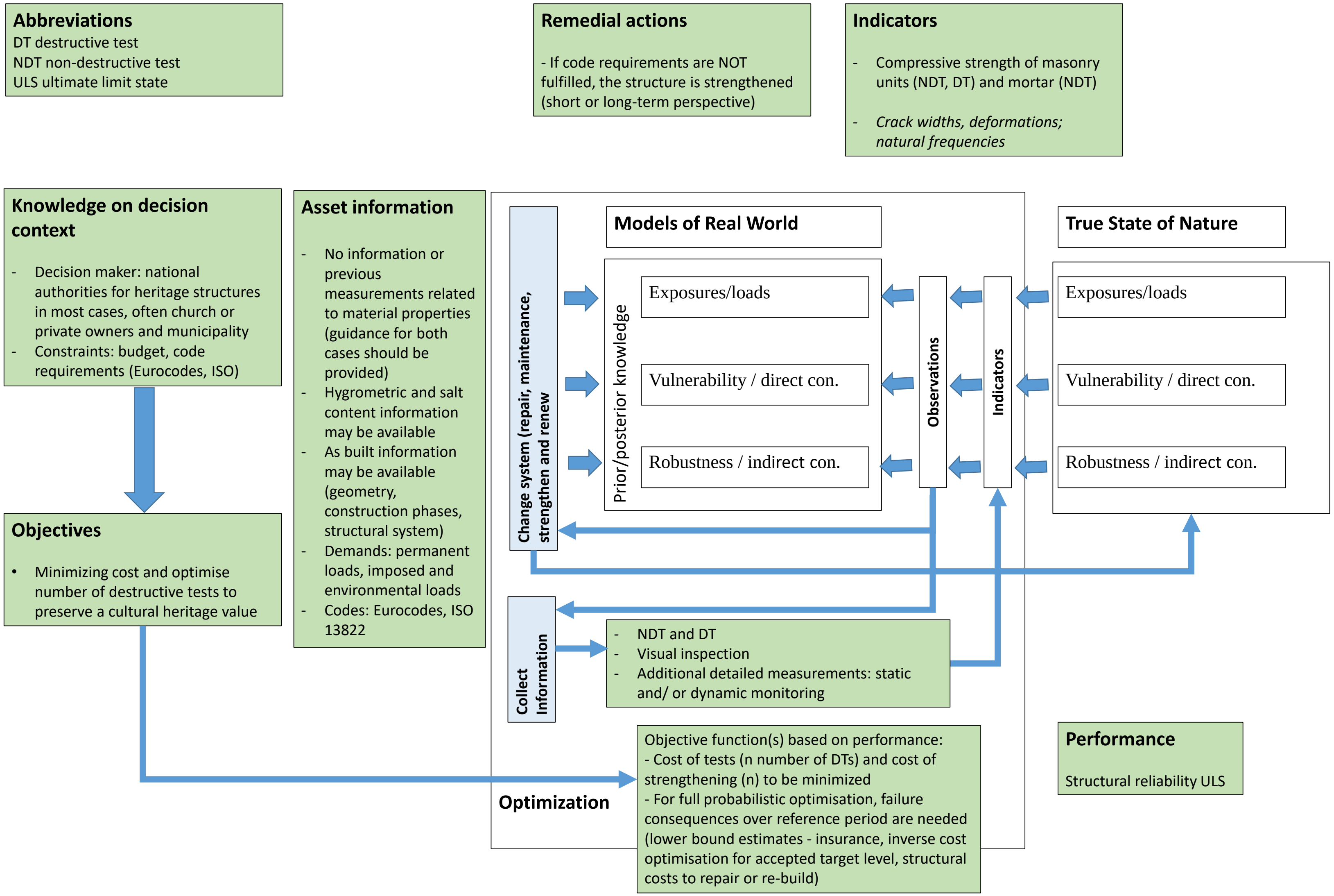


Figure 3: VoI flow chart.

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References

ISO 13822 2010. *Bases for design of structures - Assessment of existing structures.* Geneve, Switzerland: ISO TC98/SC2.

Kahraman, S. 2001. Evaluation of simple methods for assessing the uniaxial compressive strength of rock. *International Journal of Rock Mechanics and Mining Sciences* 38(7): 981-994.

Pucinotti, R. 2015. Reinforced concrete structure: Non destructive in situ strength assessment of concrete. *Construction and Building Materials* 75(-): 331-341.

Witzany, J., Cejka, T., Sykora, M. & Holicky, M. 2016a. Assessment of Compressive Strength of Historic Mixed Masonry. *Journal of Civil Engineering and Management* 22(3): 391-400.

Witzany, J., Cejka, T., Sykora, M. & Holicky, M. 2016b. Strength Assessment of Historic Brick Masonry. *Journal of Civil Engineering and Management* 22(2): 224-233.

Case Study Fact Sheet No. WG4-4

Structural Health Monitoring for Historical Kościuszko Mound in Cracow

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I. Scope of the fact sheet

In this fact sheet the working progress on a case study relating to the monitoring designed for historical Kościuszko Mound located in Cracow is presented and shortly discussed. Both the character and context of the study are summarized as well. The major steps and challenges for the implementation of a value of information assessment are highlighted. Furthermore, an outlook on the further development of the study is given.

II. Abstract

The monumental Kościuszko Mound in Cracow (Fig. 1) is an artificial hill of 34.1m relative height, heaped up by Poles in the years 1820-1823 in order to commemorate the national hero – Tadeusz Kościuszko, in recognition of his merits. It is a geotechnical structure especially sensitive to deformation, mainly due to the unfavourable structural material and very steep slopes.

Figure 1. Kościuszko Mound in Cracow seen from above, current state.

In the past it failed several times. The most significant structural failure occurred after the flood of 1997. Then the state of the mound reached the level of building disaster, and this in turn resulted in closing of the mound to the public for about 3 years. The intense rainfall induced significant erosion of the slopes triggered the almost total destruction of footpaths leading from the bottom to the top of the mound (Fig. 2). Furthermore, the condition of the mound seriously worsened after the rainfall in the year 2010.

Figure 2. Destruction of the mound in the year 1997.

The complex, very expensive renovation did not yield the expected result, and did not stop the destructive forces of nature in their tracks. Due to that fact, in the year 2012, the automatic monitoring system, based mostly on the string sensors, was installed on the structure. The main purposes of this system were the identification of ground layers behaviour and observation of the temporal changes in the mound deformations, depending on the weather conditions. It was undoubtedly very costly, but the decision was made because of the historical significance of the object and its value to the national heritage. The important from the economical point of view problem of assuring the year round availability of the mound to the tourist traffic also played a role. It is assumed that the monitoring will constitute the basic, reliable and sufficiently precise source of information on all abnormalities occurring in artificially made soil mass, and thus will allow for immediate corrective action and will prevent serious damage.

Figure 3. Piłsudski Mound in Cracow - current state.

In addition the detailed recognition of soil behaviour in the man-made mound of this type may in the future allow for preparation of an efficient strategy to save the similar Piłsudski Mound (Fig. 3) located in Cracow as well. This mound has the relative height of 35.0 m and has been heaped up in the years of 1934-1937. Because of very acute erosion affecting approximately 65% of the slopes it is in general excluded from commercial use.

III. Description of the case study

Kościuszko Mound is an artificial, man-made geotechnical structure erected on a sloped rock bed of indigenous limestone (Fig. 4). It was mainly made of loess type soils, generally in the form of silty clays and silts with a substantial addition of man-made dump ground. Soils of this type proved to be extremely susceptible to the environmental actions during the service time, and especially sensitive to the rainwater infiltration and multiple frieze thaw cycles. The way the mound was heaped up determined extremely unfavourable layered structure of the tambour, exhibiting heterogeneous and locally variable layer thicknesses. Extreme slope of the tambour side, reaching up to 46-51^{o} (Fig. 4) significantly affects the behaviour of the structure as well, as it substantially exceeds the angle of internal friction of soils used.

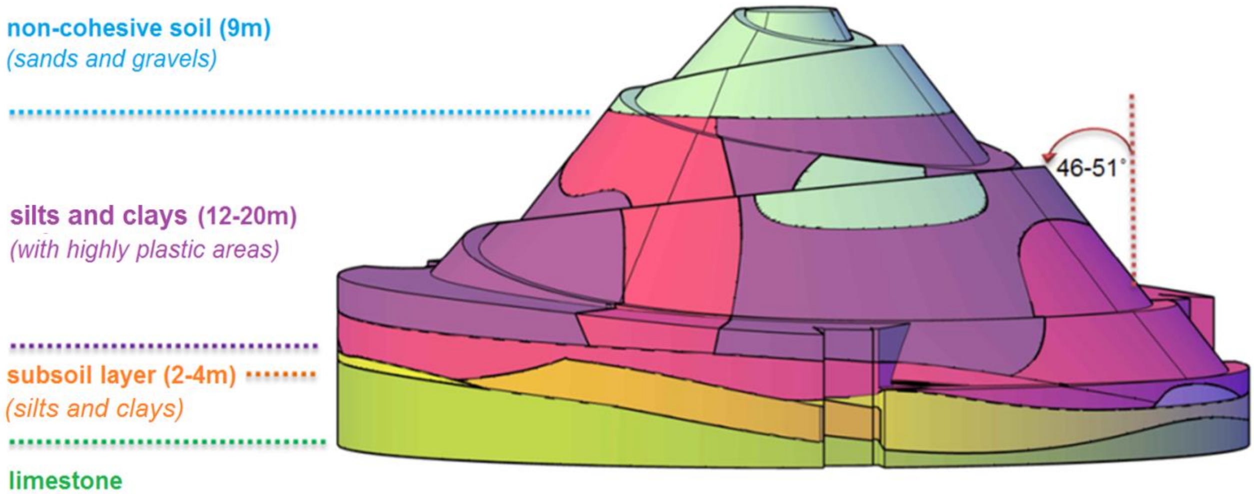

Figure 4. Internal soil structure of the Kościuszko Mound.

The monitoring system applied in 2012 yields detailed information on the vertical displacements and hoop as well as radial deformations. Location of sensors used to perform such measurements is depicted in Fig. 5. Vertical displacements are measured by the vertically oriented string sensors (Fig. 6) installed at five locations. The horizontal deformations are measured by another group of string sensors installed at the perimeter of the mound and oriented circumferentially and radially towards the inside of the mound structure (Fig. 7). The pore water pressure within the interior of the structure as well as the humidity of the top layer are monitored as well by piezometric sensors located in four measuring stations, at three different depths (Fig. 8). All the string sensors have been equipped with thermistors in order to measure the soil temperature at respective measurement stations. The obtained results are related to the geodesic measurements and measurements of inclination as well. They are correlated with meteorological data provided by a weather station, constituting a part of the monitoring system. Detailed geological analyses resulted in the installation of a permanent monitoring system. This action allowing for continuous observation, within the whole mound volume, of the very heterogeneous and mostly man-made, layered and water permeated soil structure was presumed to be the only possible technical solution warranting future trouble free service of the mound.

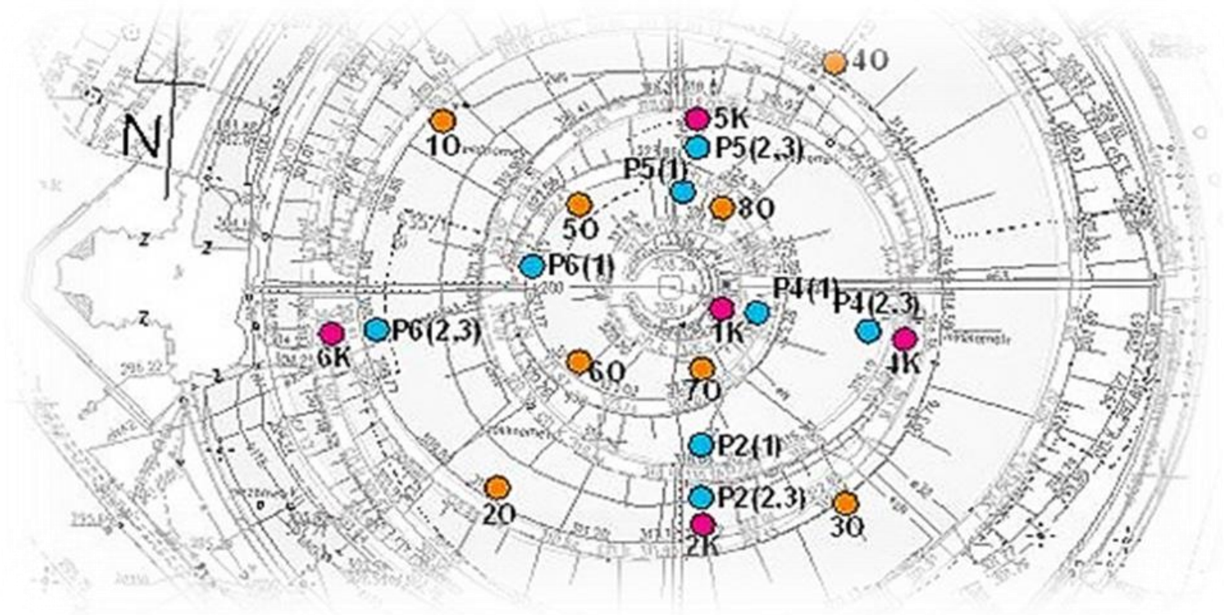

Figure 5. Horizontal projection of the mound with indicated locations of measurement stations (denotations: K - sensors measuring vertical displacements, O - sensors measuring horizontal displacements (radial and circumferential) and soil humidity, P - sensors measuring pore water pressure and humidity of the external soil layer.

Figure 6. Details of vertical displacement measurements.

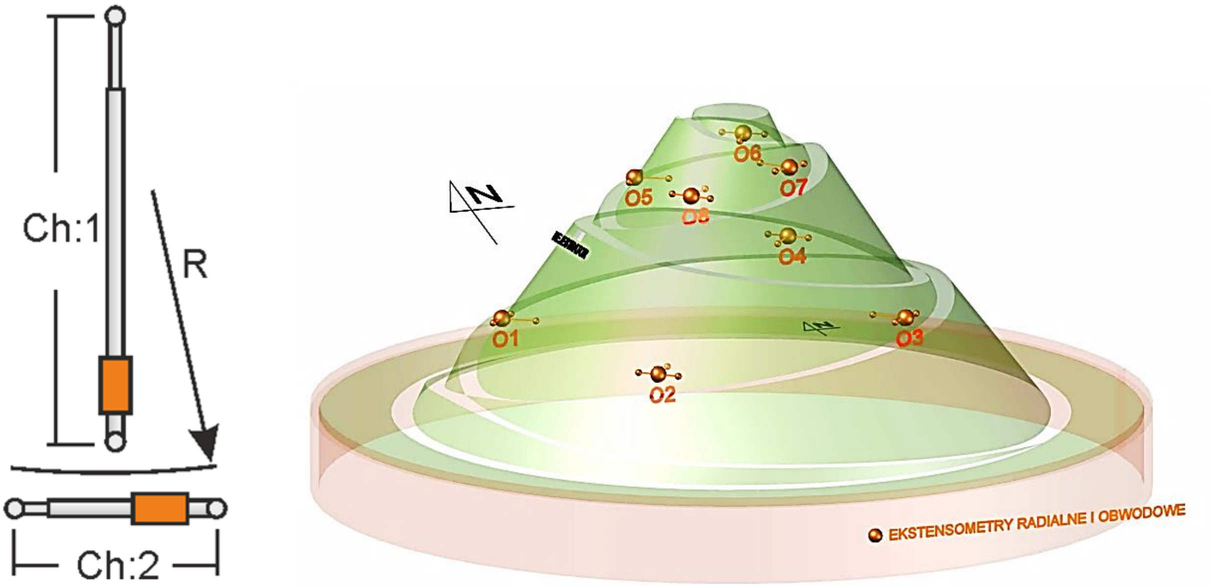

Figure 7. Details of horizontal deformation measurements (along the radius and circumference).

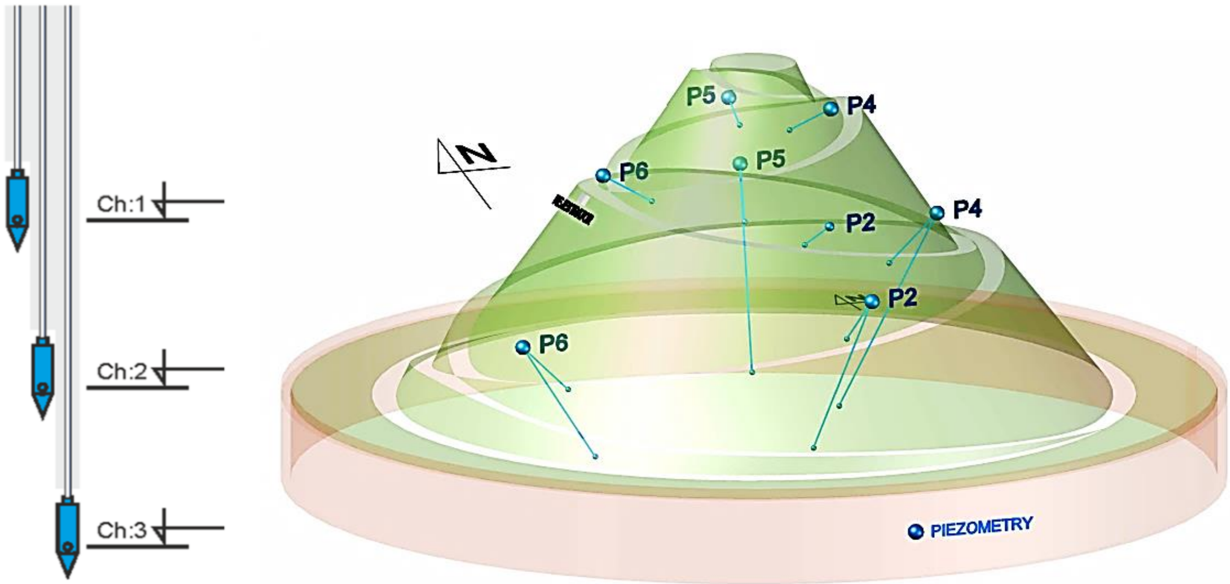

Figure 8. Details of the pore water pressure and external layer humidity measurement stations.

IV. Implementation of VoI analysis

In order to more or less reliably estimate the Value of Information for the considered mound and the monitoring system installed on it one should at first estimate the value of the mound itself, treated as a historical monument. It is difficult to come up with an unambiguous value, as one must take into account above all its historical value related to the national cultural heritage. Therefore it is not only the value of the structure as such, built from a specific material at given expenditure of work effort. One should take into account contexts of various types, even the unquantifiable ones such as emotional value. If such approach is used this object may turn out to be priceless for many. One may also estimate the value based on the insurance premium the local and/or state authorities were willing to pay to insure the structure against complete loss. Whichever method one would prefer to use, these values would not be objective. It is also not the actual cost of installing the monitoring system described above on the mound in question and for its socio-economic determinants in order to see if such an operation would be profitable for society. This monitoring already exists, and one may only think about its improvement or exstension. In our analysis we suggest that the VoI analysis should be treated somehow conditionally, subject to the condition, that the results of monitoring applied are available to the potential user, but some expenditures need to be borne to get these results. Thus the first stage in the procedure proposed by us would be the specification of various failure classes of considered structure, differentiated by the consequences of occurrence. Calculations of this type need to be accompanied by the qualitative and quantitative risk analysis. We propose at first to define and quantitatively describe a local failure mode, occuring rather on the surface and extending over a limited area. A small, but unequivocally determined failure consequence shall be assigned to such phenomenon. Subsequent failure types, defining increasing threshold consequence coefficients would denote increasingly global character of the observed destruction. These would be not only surface damage or deformations but also deformations extending inwards, such which may result in, for instance, loss of stability of the whole slope. At the end there remains the identification of global failure mode for the whole structure as an incident having the highest failure consequences by value. The monitoring results allow for calculation of time dependent stress levels in the soil at every measurement station based on the measured data. The time dependent mechanical properties of soil located in the vicinity of monitoring stations, and thus indirectly the effective soil strength, depend on the parameters measured at the same stations. A proper limit state condition (for instance pertaining to the slope stability) may be set and verified at any measurement station via the comparison of stress state determined at such point in the given moment in time with the corresponding value of material strength. Thus one may determine the probability of achieving such limit state or alternatively exceeding or not reaching it. At any moment of measurement, with proper data processing, such approach will yield a risk map specified in principle for the whole mound volume. Of course such 3D map will change in the subsequent measurement moment, as it will be built based on the updated monitoring data. Thus one deals in the proposed procedure with various failure levels, such that for each of these levels the cost of failure may be attributed. One has also at his disposal the probabilities of the occurrence of such failures at given measurement moment accompanied by the affected part of the total mound volume or only local surface area determined for considered probability. Such data set constitutes a sufficient basis for formulating the classical decision problem. Since one is able to attribute the probability of occurrence to each level of failure and one knows the costs of more or less comprehensive use of the available monitoring system, potential alternative scenarios for future strategies, including, but not limited to the two listed below, may be considered.

- Strategy no 1: at the determined and relatively low risk level it is suggested not to undertake any remedial actions as long as the monitoring would not indicate the local or global increase of this level to at least the threshold level arbitrarily assumed at a relatively high value. This strategy, if applied, results in the mound owner having to bear relatively high repair costs, but rather infrequently. Failures are infrequent but if do happen are of serious consequences.
- Strategy no 2: often initiated remedial actions of limited scope are preferred, i.e. actions are initiated every time the acceptable threshold risk level defined by the owner at relatively low level is identified as reached or exceeded even over a relatively small and insignificant area of the mound.

Subsequent possible strategies would in this case constitute combinations of the strategies presented above. The computational procedure proposed would allow the owner for, in our opinion, sufficiently justified planning of the future remedial actions, to be compatible with the real state of danger efficiently monitored by the professional measuring system installed on the mound.

Link diagram specific to the analysed system and underlying the classical VoI analysis is depicted in the Fig. 9.

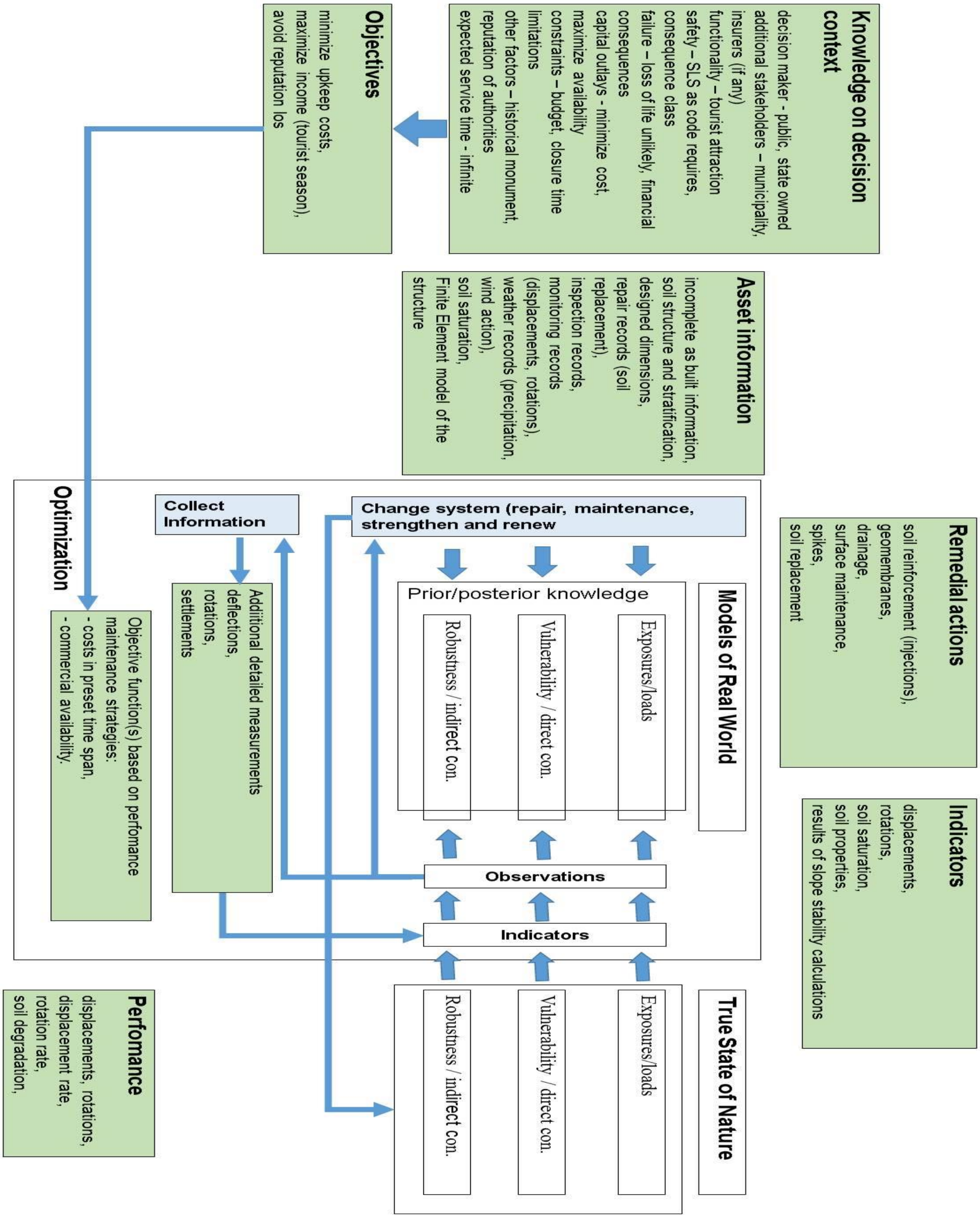

Figure 9. Preliminary flow chart for the VoI analysis relating to the historical Kościuszko Mound in Cracow.

V. Critical appraisal, necessary simplifications

The Authors of this paper consider the monitoring system existing on the mound in the middle of 2017 to be sufficient. Until 2014 this system was undoubtedly incomplete. It allowed for observation and measurement of various tambour deformations, however the coincidence of these measurements with weather conditions, and especially with insolation changing in a daily cycle and with wind directions predominant in Cracow was completely lost. Therefore the installation of the system was completed only when the independent weather station comprising of pluviometer, anemometer, air temperature sensor, relative air humidity sensor, and pyranometer measuring the amount of sun energy reaching the mound was added to the sensor set. The sample of data changing in time and recorded by the monitoring system is depicted in Fig. 10 through 15.

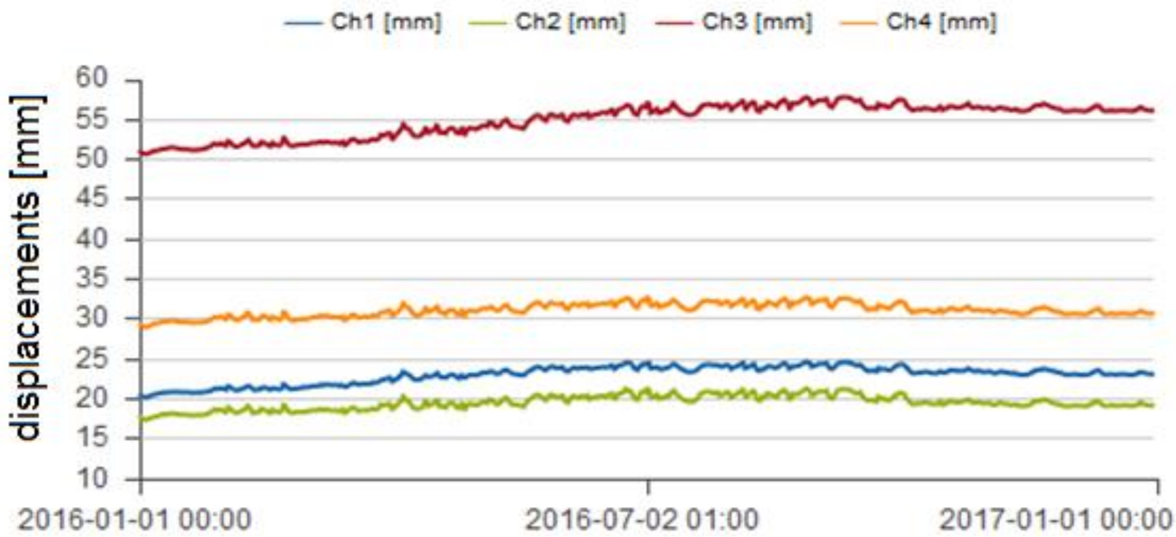

Figure 10. A segment of the graph depicting monitored vertical displacements in the 1K measurement station at the boundaries of subsequent soil layers.

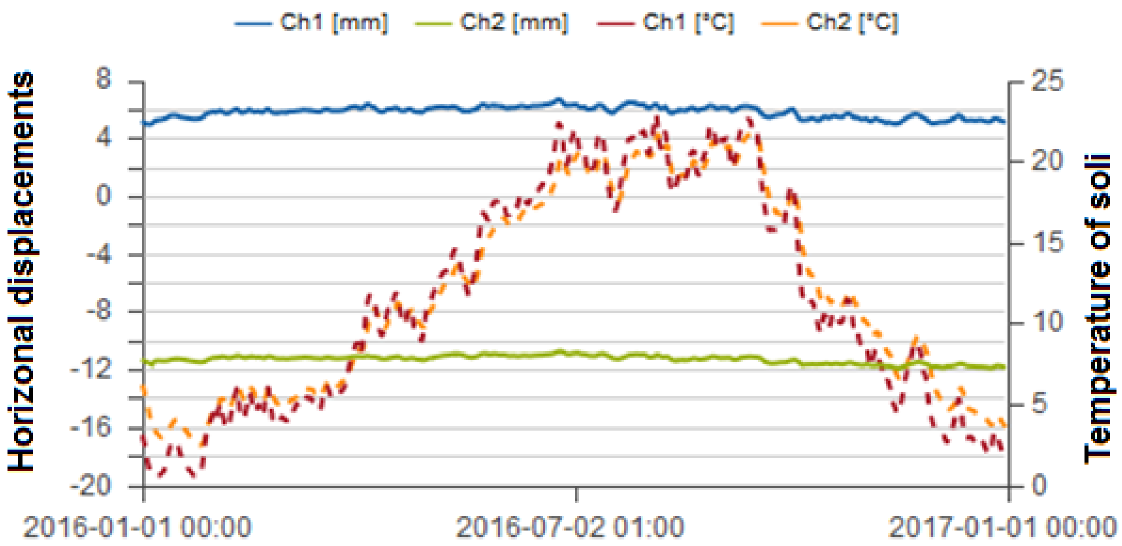

Figure 11. A segment of the graph depicting monitored horizontal displacements in the 2O measurement station at the boundaries of subsequent soil layers accompanied by the soil temperature graph (yearly fluctuations of the soil temperature are clearly visible).

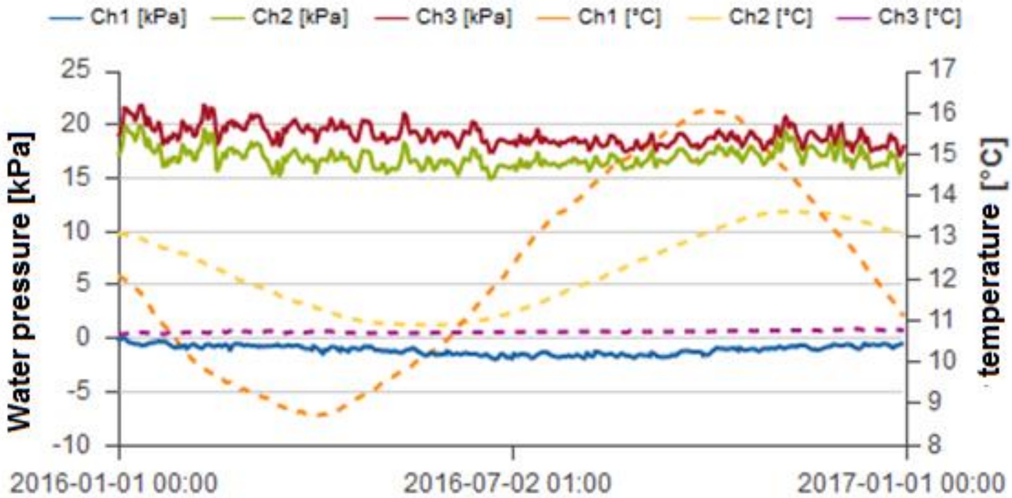

Figure 12. A segment of the graph depicting monitored pore water pressure and corresponding soil temperature values measured at the 2P station (yearly fluctuations of the soil temperature are clearly visible).

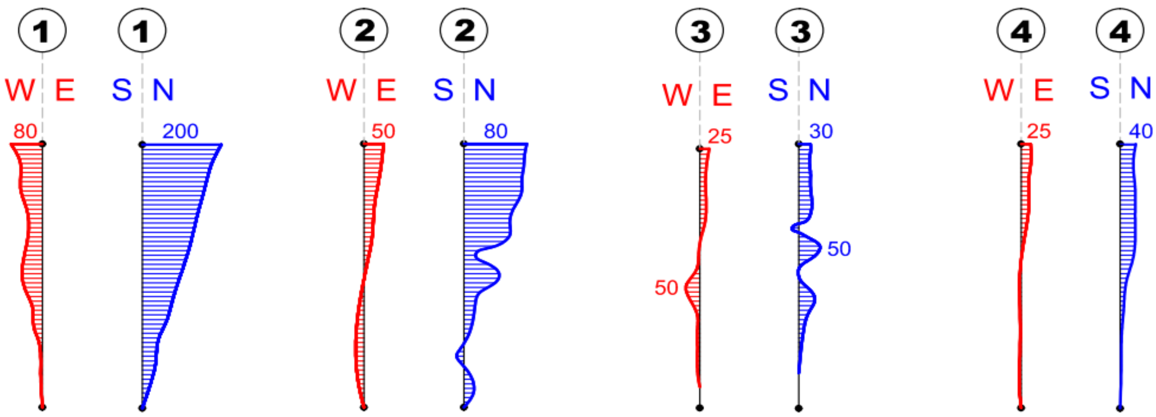

Figure 13. Results of the inclination measurements at a given moment in time.

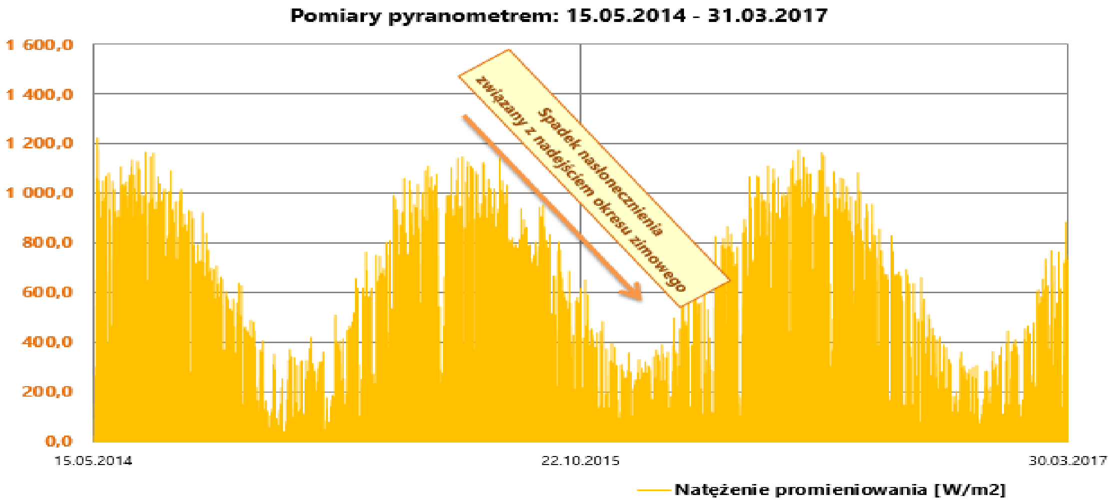

Figure 14. Changes in the mound insolation in a yearly cycle observed by pyrometer.

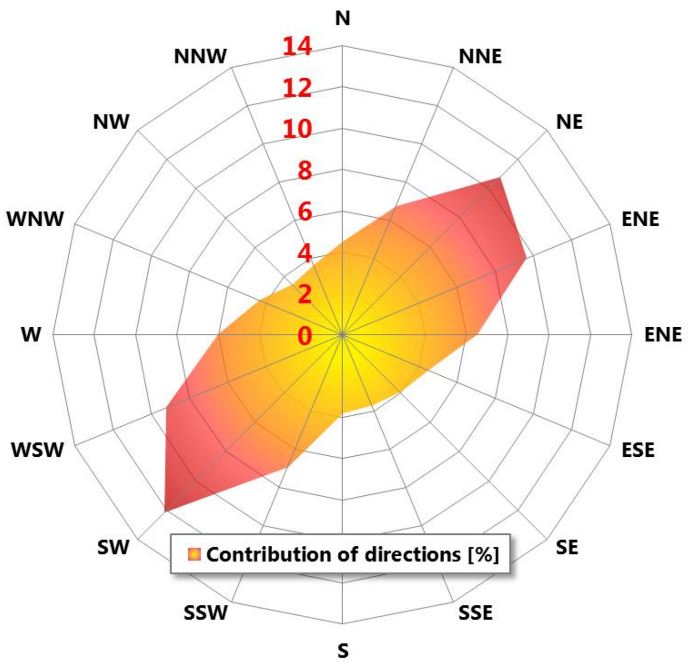

Figure 15. Monitoring obtained wind rose for the mound location.

VI. Further steps and resources required

Until now the emphasis was laid upon the development of the monitoring system described by additional sensors and enrichment of the available data set. It seems, that the preparation of an optimised and unequivocal procedure allowing for efficient use of the gathered data constitutes the subsequent step, and this is undoubtedly expected by the mound owner. Implementation of such procedure in the case of mound analysed in this case study constitutes the research problem undertaken by the authors of this study. The point is that the various data types should be analysed in a complex manner in a single procedure taking into account the inherent intrinsic interactions and correlations present within the whole data set, instead of piecewise analyses performed on chunks of available data. It is also postulated, that the results obtained a posteriori, based on the measurements performed by the monitoring system sensors, should be used to improve the a priori assumed theoretical model. An extension of the user interface, and especially of the part on system "output", would also constitute a significant improvement of the system as a whole according to the authors. As noted above, the ultimate objective is to deliver to the evaluator a 3D risk map, showing the risk of soil reaching the ultimate limit state of bearing capacity or the mound slope stability, reproducing the whole volume of the monitored structure and accompanied by the risk level specified for each point on the map coincident with sensor location.

VII. Summary and conclusion

The man-made artificial mound described in this fact sheet is a very peculiar structure of very high historical value. Preservation of this mound in a state allowing for a year round tourist traffic calls for nonstandard actions assuring safe and reliable service in the future. Analysis of a structure of this type combines the problems of classical civil engineering (especially geotechnics), with problems of safety evaluation and systems engineering. The data available to the person appraising the technical condition of the structure of this type are characterized by the high variability of not only the statistical type but also due to the daily and yearly changes in the climate and weather conditions. The basic source of uncertainty is in this mound the layered structure of the hill, with very heterogeneous soil structure in all layers. Furthermore, the man made ground has its origins in many locations, thus its parameters fluctuate substantially. Finally, the soil properties to a large extent depend on its humidity, and this in turn is affected by the seasonally changing weather conditions. This raises questions about the representativeness and reliability of the data collected. The limited accuracy of the measurements, due to the precision of the sensors used and their sensitivity to the weather conditions, is an additional source of uncertainty. The limitation of this type is especially important, as there is a risk of accumulating measurement errors, especially in the case of slope measurements. The uncertainty due to the limited credibility and random nature of the measurement data is aggravated by the inference uncertainty, based on the simplified model of analysis applied in practice. So far the inference based on the existing monitoring is supported by the statistical processing of the measured data and the analysis of the random processes describing the temporal evolution of the measured parameters. However, this variability is interpreted only locally in order to identify the possible trends and record the cases when the limit values are exceeded. It seems, that in order to qualitatively and quantitatively describe the deformation and mechanical properties of the mound structure, which change during the service time, it would be desirable to apply the measurement data gathered at discrete locations to develop and calibrate the 4D (three spatial dimensions and the temporal one) random field describing the soil behaviour in the whole volume of the mound. The 3D numerical model should also take into account the climate loads correlated with this field, and having the statistically determined values, based on the measurements supplied by the weather station.

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References

Howiacki T., Mieszczak M. and Jedliński S. (2015). Structural Health Monitoring System for the Kościuszko Mound in Cracow (in Polish). Proceedings of XXVII Scientific - Technical Conference "Structural Failures", Międzyzdroje, May 20-23, 2015, pp. 327-334.

Case Study Fact Sheet No. WG4-5

CONCRETE BRIDGE AFTER IMPACT

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I. Scope of the fact sheet

This fact sheet is presenting the working progress on a case study related to impact damage of a prestressed bridge in Ireland and the monitoring of the bridge during the entire repair process (Pakrashi et al., 2013a). This is the first full-scale monitoring of this kind of repair and the challenge lies in the uncertainty around the damaged bridge. A two-span continuous slab-girder bridge consisting of six precast prestressed U8 simply supported concrete beams (27.35m span, no skew) connected by a continuity diaphragm was damaged following an impact to its soffit from a low-loader carrying an excavator passing underneath the bridge. The edge of the outer beam was damaged benignly but one of the tendons in the lower row snapped. An internal beam was significantly damaged where the tendons remained intact but the concrete was crushed from the impact. It was difficult to estimate the existing stresses within the beam and continuous monitoring was required during the entire period of rehabilitation. While the value of the monitoring during the commercial work was justified by the need to investigate the changing responses of the structure throughout a detailed monitoring process, further investigation led to the understanding and insight of its value even further. Currently, while the case study has demonstrable value through monitoring and related information, it is difficult to formalize it in the same manner as visual inspections or inspections with standard SHM mainly due to the context and complexity of the work, along with uncertainties at several levels. The location of the bridge was on a major motorway, so it had a significant geographical importance given the fact that the repair was carried out on a bank holiday weekend by involving traffic calming measures on a main motorway between the two largest cities in the country, leading to traffic delays. This factsheet is an ongoing work investigating how the value of monitoring of this bridge can be linked to a formalized process.

II. Abstract

The objective here are as follows: a) to present the methodological details of repair and monitoring; b) to present the uncertainties around the work; c) to present the apparent value of the work during monitoring and context around further opportunities; d) to attempt to provide information to a structured and uniform format to assess its value;

Technical Information on Monitoring: A total of 19 vibrating wire strain gauges providing strain data once every minute at five preselected monitoring points were provided. The monitoring points were strategically chosen so that the interaction of the damaged and the undamaged beams, including the behaviour of gauges at, near and away from the damage, can be probed. This ensured the many scenarios assessed by models could be checked and predictions related to various stages could be assessed.

Repair Method: The repair was carried out by preloading the damaged location with 120t, releasing some strain in the damage part and repairing it with a material that gained strength overnight and attempted to match the existing concrete strength. The 120t load was then released with the expectation that some of this pre-strain is retained within the bridge. This was the first monitoring of its kind globally for this type of rehabilitation and provided guidance in relation to uncertainty in modelling, understanding of various phases of rehabilitation, correlation between damaged and undamaged beams (Pakrashi et al., 2013c), redistribution of loading in damaged and repaired bridges, effects of hydrodemolition and the diurnal thermal effects. Without monitoring, it was not possible to choose the correct model or have an insight into the real behaviour of the structure in a damaged, rehabilitating and repaired state.

III. Description of the case study

A 3D Laser scan of the impact damaged bridge is presented in Figure 1, while Figure 2 depicts the damage on the bridge from visual inspection. The repair was carried out at different stages and a schematic is provided in Figure 3. Installation of the gauges (Figure 4) ensured that while static responses are measured, the sampling rate of one per minute over several days provided a pseudo-dynamic response of what was going on with the bridge. There was uncertainty on the amount of stress redistribution that took place following the impact damage and thus several scenarios were created using first principles and Finite Element modelling to assess the range of stresses in different beams of the bridge. The damaged location was loaded with 120t on the deck equally on either side of the damage and in 20t intervals on each side. This reduced compressive stress at the bottom of the prestressed beam and there was some release of locked-in stresses in damaged concrete, as evidenced through monitoring. Hydrodemolition (cutting of concrete with water jet) was carried out to provide access for repair and exposed tendons were instrumented with further gauges. Repair material was applied which hardened overnight and reached a reasonable strength in relation to the existing concrete. Following the hardening, the 120t preload was removed (Figure 5) and it was expected that due to the preload removal, some prestressing could be reintroduced to the bridge. However, there is uncertainty around the extent to which this was possible. Following preload removal, further strength gain took place over the next few days.

IV. Implementation of a VoI analysis

Covering the different attributes of the analysis according to the VoI flowchart, most importantly:

a) Decision maker: County council, engineering consulting firm working on behalf on council
b) Regulative constraints: Limit states, traffic calming and safety, high-risk activity in the form of hydrodemolition, time constraint for repair, insurance aspects for work
c) System temporal and spatial boundaries: Short time period within which repair was to be done; spatial boundaries obtained from as-designed drawings and visual inspection (laser scan and impact echo providing additional information)
d) Events of interest and the corresponding representation: i) Thermal diurnal effect before and after repair, ii) hydrodemolition intensity, iii) applying of loads, iv) release of locked strain in damaged concrete, v) hardening of repair material, vi) removal of preload, vii) strength gain over time – all represented primarily by instrumentation (see Figure 3)
e) Event consequences: i) understanding of true thermal effects, ii) effectiveness of hydrodemolition as a low impact tool, iii) tensile strain development at the bottom of beam, iv) understaning of the extend of locked-in stresses in damaged concrete; v) efficiency of repair material and understanding shrinkage properties, vi) redistribution of prestrain as prestress in beams after hardening of repair materials, vii) effectiveness of repair
f) Indicators (to observe): i) strain, ii) correlation of strain at different locations (Figure 6), iii) change in strain as predicted by models and scenarios (Figure7), iv) Hurst exponents of strain records (Pakrashi et al., 2013b) (Figure 8), v) Delay Vector Variance (DVV) marker (Jaksic et al., 2016) for strain time series;
g) Decision alternatives – monitoring and/or inspection options: i) accelerometer based monitoring; ii) monitoring using fibre optics; iii) video monitoring [but not as effective often as the one adopted due to stability and manageability over time in terms of data and the robustness of the sensors]
h) Decision alternatives – other measures, repair, replacement, etc.: i) rebuild, ii) carry out repair without preloading, iii) carry out composite fibre based strengthening, [all with or without the model scenarios presented and uncertainties related to them]

V. Critical appraisal, necessary simplifications

The VoI methods for visual inspection (Quirk et al., 2017) or SHM (Memarzadeh and Pozzi, 2016) do not lend themselves easily to this problem. The monitoring was important but effective over a shorter time period and uncertainties around various parameters and models required significant engineering judgement and assimilation of information at different hierarchical levels. Under such circumstances and based on the match between certain aspects of modelling and monitoring (e.g. the strain change predicted by model for preloading was very closely observed to have been followed during monitoring, indicating that the structure was still behaving in a linear elastic fashion, as assumed by the model etc.), a simplication can be in using the monitoring against a simplified working model from the first principles. Also, while there are several ways of assessing the impact of the work, aligning it to standard practice of computing the values for some options and assessing levels or 'what-if' situations of traffic diversion costs may provide some idea around how the VoI evolves as we start considering different criteria, like examples of other bridges in Ireland (Pakrashi et al., 2011). Keeping in mind that the VoI is representative of some utility obtained from the monitoring, it is still better to present it in some sort of financial unit for comparability and uniformity, but with caveats around interpretation of such financial numbers.

VI. Further steps and resources required

While it is difficult to even reasonably come to a number for VoI, the first step should be to align the case study to a common format, like Figure 9. This can map the information and methodology to a more manageable and comparable situation. However, resources are required in terms of numerical guidelines or best practice to come up with a number, a utility curve or a value derived thereof which can reasonably and faithfully represent the impact of monitoring through VoI, while preserving the commonality of following the template in Figure 9. This should be taken up in the later stages of COST TU1402 through meetings and workshops.

VII. Summary and conclusion

The monitoring of an impact damaged prestressed bridge and its consequent repair is presented to lead towards a common VoI format, which is presented in Figure 9. The context, method, key assumptions and results are presented, along with challenges and difficulties around adapting the case study to a common VoI format. The mapping indicates promise of being able to adapt the study to a common format and leads to the next steps of computation, comparison and interpretation of the VoI obtained from this case study and in relation to other case studies.

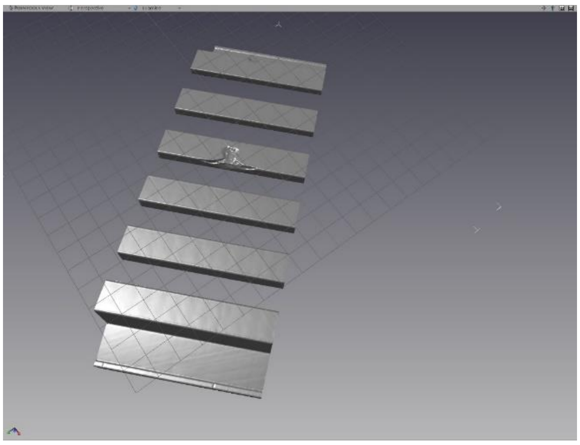

Figure 2: Damage within the beam following hydrodemolition.

Figure 1: A 3D Laser scan of the bridge showing damage.

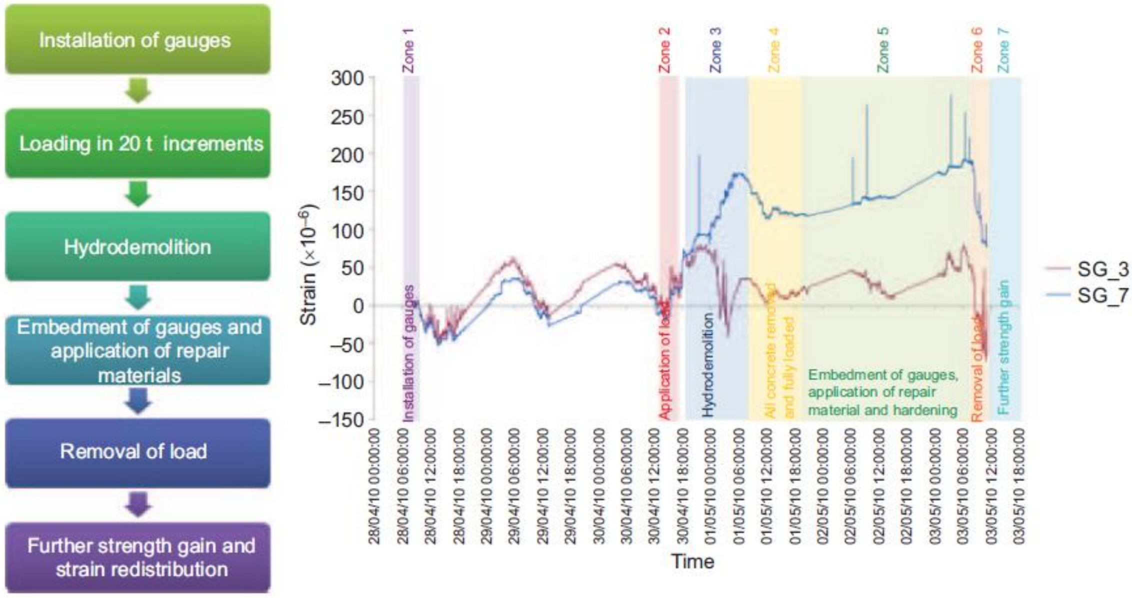

Figure 3: Schematic of various repair stages using representative strain gauges SG_3 and SG_7.

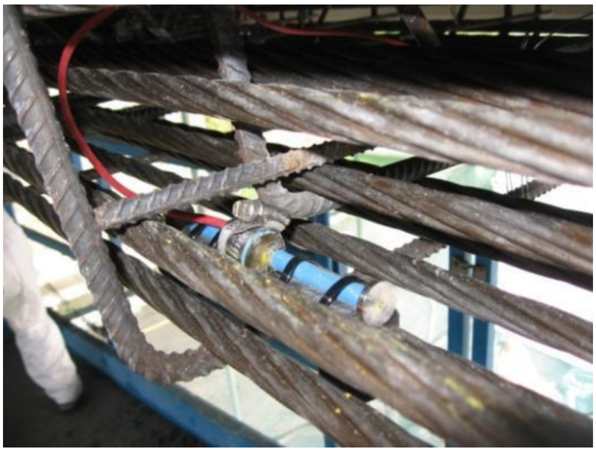

Figure 4: Installation of gauges to prestressed tendons following hydrodemolition.

Figure 5: Removal of preload following hardening of epair material.

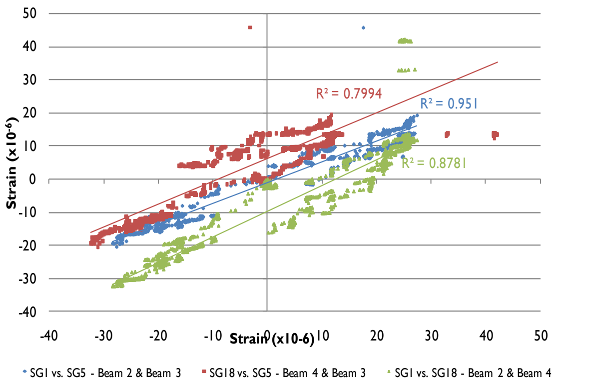

Figure 6: Strain correlation between damaged and undamaged beams.

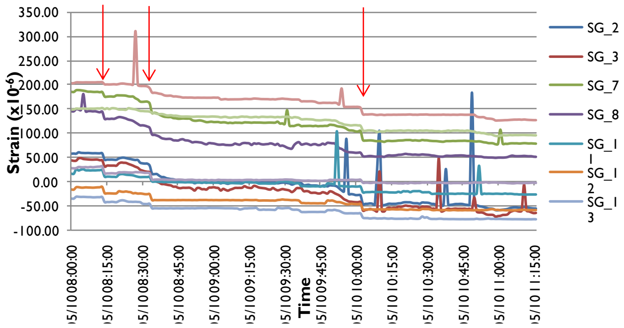

Figure 7: Removal of preload following hardening of repair material.

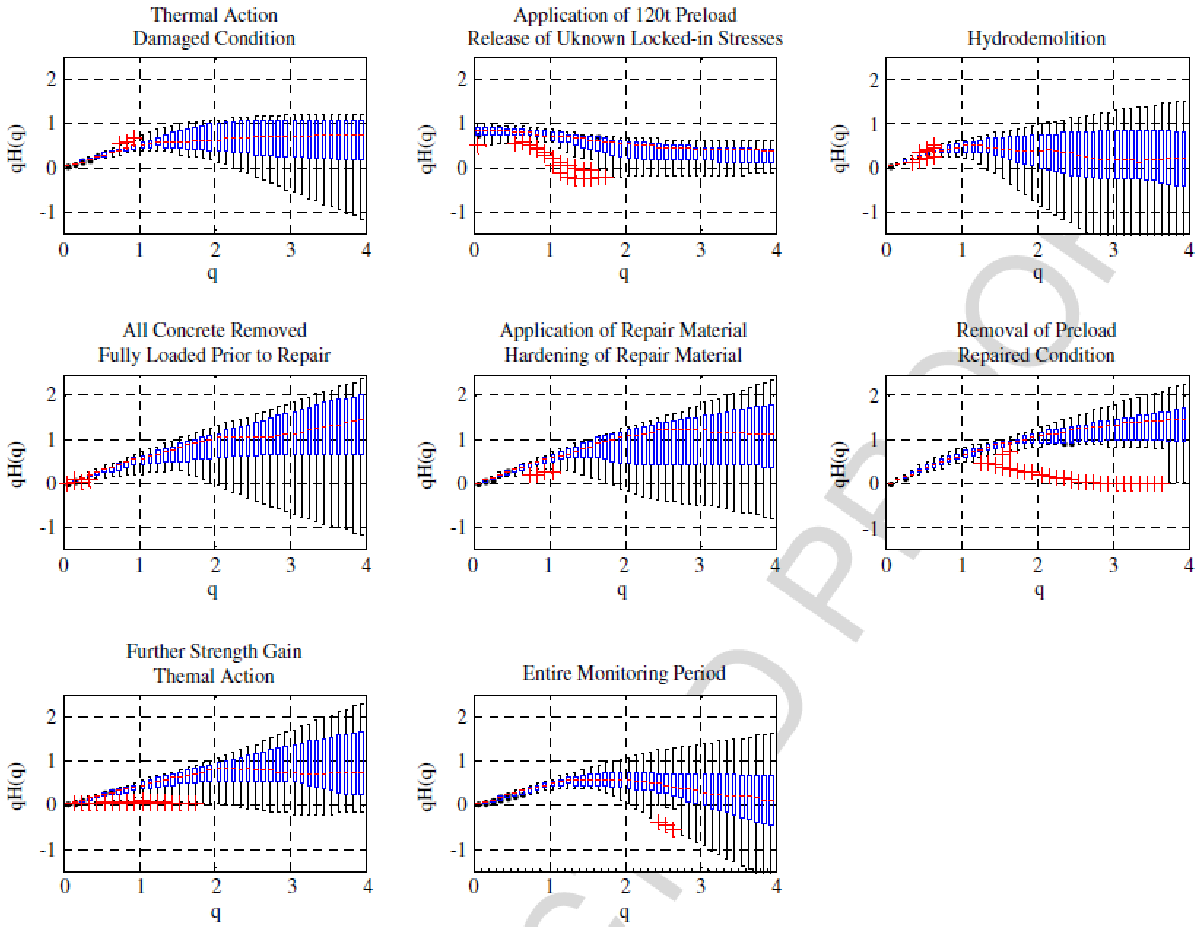

Figure 8. Hurst exponent signatures for different repair stage.

Contact information

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References

Jaksic V, Mandic DP, Karoumi R, Basu B and Pakrashi V. (2016). Estimation of nonlinearities from pseudodynamic and dynamic responses of bridge structures using the Delay Vector Variance method. Physica A: Statistical Mehcanics and its Applications 441: 100-120.

Memarzadeh, M. and Pozzi, M., (2016). Value of information in sequential decision making: Component inspection, permanent monitoring and system-level scheduling. Reliability Engineering and System Safety 154: 137–151.

Pakrashi, V., Harkin, J., Kelly, J., Farrell, A. and Nanukuttan, S. (2013)[a]. Monitoring and Repair of an Impact Damaged Prestressed Concrete Bridge, Proceedings of the Institute of Civil Engineers, Journal of Bridge Engineering 166(1): 16-29.

Pakrashi, V., Kelly, J., Harkin, J. and Farrell A. (2013)[b]. Hurst Exponent Footprints from Activities on a Large Structural System, Physica A: : Statistical Mehcanics and its Applications 392(8): 1803 – 1817.

Pakrashi, V., Kelly, J., Harkin, J. and Farrell, A. (2013)[c]. Rapid Calibration of a Damaged Beam Undergoing Rehabilitation, International IABSE Conference, Rotterdam, The Netherlands.

Pakrashi, V., Kelly, J., and Ghosh, B. (2011). Sustainable Prioritisation of Bridge Rehabilitation Comparing Road User Cost, Transportation Research Board Annual Meeting, Washington DC, USA.

Quirk, L., Matos, J., Murphy, J. and Pakrashi, V. (2017). Visual Inspection and Bridge Management. Journal of Structure and Infrastructure Engineering, In Press, DOI: 10.1080/15732479.2017.1352000

Case Study Fact Sheet No. WG4-6

Assessment of fatigue detail of the Söderström Bridge

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I. Scope of the fact sheet

This fact sheet is presenting the working progress on a case study focusing on the assessment of a fatigue detail of the Söderström Bridge in Stockholm, Sweden. It summarizes the character and context of the study and highlights the major steps and challenges for the implementation of a value of the information (VoI) assessment. Furthermore and outlook on the further development of the case study is given.

II. Abstract

Condition assessment of bridges includes several decisions based on imperfect information. One such decision is related to collecting new information that will help to select subsequent actions maximizing the life cycle benefits of the bridge. A rational decision on whether collecting new information or not, selecting the most appropriate method(s), choosing suitable spatial and temporal resolution of the observations etc. requires (some kind of) quantification the value the information might bring before it is actually obtained. In practice the quantification of this value is often carried out intuitively rather than in a structured way. Since, in a general sense, this evaluation could be done for any assessment option that might improve our overall judgement about structural condition a rigorous approach is recommended to avoid suboptimal decisions.
The current fact sheet provides a preliminary description of a case study aiming to utilize the VoI analysis and help identifying appropriate condition assessment options in general, and inspection methods in particular, for the detailed condition assessment of a steel bridge focusing on a fatigue critical detail.

III. Description of the case study

The Söderström Bridge is located in in the city centre of Stockholm, Sweden (Fig. 1a). Its functionality is crucial for the railway traffic in the region and in some extent for the entire country.

Figure 1a: A photo of the Söderström Bridge

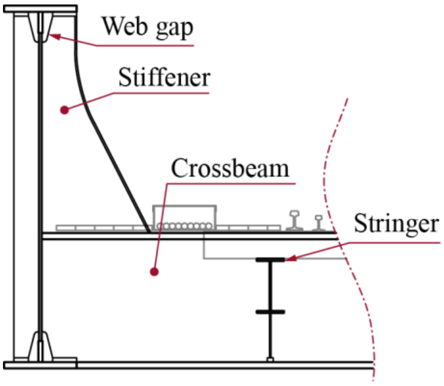

Figure 1b: Fatigue detail

More than 500 trains pass the bridge every day, mainly commuter trains but also intercity trains and freight trains. It is a continuous steel girder bridge, inaugurated in 1954, built up by a grid of main beams, crossbeams and stringer beams. The bridge has 6 spans with a total length of 190 m. Due to its high traffic and central position within the city, the bridge is critical for the transportation network i.e. disruption would lead to high economic consequences.

The bridge contains many fatigue critical details at several locations and at the connection between the crossbeams and the main beams, cracks have been found in the webs of the main beams. The cause is well known and often referred to as web gap cracking. Until 2008, a total of 90 similar cracks have been found and repaired. Another critical detail, which is the focus of the current case study, is the connection of the bracing to the stringer beams (Fig. 1b). Theoretical assessments and results from strain measurements have indicated an exhausted fatigue life, however, no sign of cracks have been found during inspections at these locations.

It is assumed that with further assessment the expected remaining service life can be extended. Thus the main question is if further assessments should be carried out and at what cost. This leads then to several additional questions e.g.: Which of the available inspection methods should be selected? How often should inspection be carried out?

IV. Implementation of a VoI analysis

Knowledge on the decision context and objectives

The *decision maker* in this case is a public authority, i.e. the Swedish Transport Administration (Trafikverket) responsible for a large portfolio of bridges (and other transport infrastructure assets). Additional stakeholders affected by the decisions include the municipality of Stockholm, the operators of interdependent infrastructure services (e.g. railway and utility operators) and the users of the bridge and infrastructure networks affected by restricted use of the bridge. For the sake of simplicity we focus on traffic only as the main service provided by the bridge and the preferences of the additional stakeholders could be included in the utility function of the operating authority as it acts on behalf of the entire society.

Constraints of the decision problem are principally given by the budget available for the maintenance of the bridge. Since the bridge is part of a larger portfolio of bridges and the budget for maintenance of the portfolio is assigned for a given period (1 year), the principles of prioritization for distributing the budget should be known, i.e. current decision making process involving organizational aspects. This is especially true if maintenance and upgrading/strengthening fall under the responsibility of different departments within the organisation using different budgets. Furthermore, buying services for interventions might be subject to public procurement process, which might need to be taken into account. To manage its bridge (and tunnel) portfolio Trafikverket uses an asset management system, called BaTMan, where they collect and store various data about the bridges, required to make decisions about their maintenance. Other important constraints and requirements are related to structural safety and relevant codes and the current Swedish guidelines for inspection and assessment of existing bridges.

Concerning *system temporal and spatial boundaries,* to make the case study as simple as possible, the analysis is restricted to a single detail (connection of the bracing to the stringer beams) and a single failure and deterioration mode (cracking due to fatigue). The *events of interest and the corresponding representation* focus on the fatigue reliability of the selected detail with various levels of modelling sophistication including e.g. linear damage accumulation and linear elastic fracture mechanics. The limit state function is related to fatigue life, i.e. number of fatigue cycles.
The *consequences* of *events* are represented as monetary costs due to inspection, repair and failure of the detail. However, a more sophisticated representation would be possible considering human

(e.g. increased accident rate), environmental (e.g. increased CO_2 emissions) and socio-economic costs (e.g. traffic delay costs). Possible *indicators* to observe on the bridge due to inspection or monitoring are strains, deflections, accelerations and cracks. Here we focus on fatigue cracks of the selected detail.

The primary *decision alternatives* are related to the (further) condition assessment of the bridge, i.e. 1) no further assessment is done (and base further decisions on current level of knowledge), 2) carry out assessment according to a certain method (and base further decisions on the improved outcomes). Subsequent *decision alternatives* could be related to interventions based the results of the (detailed) condition assessment and include: 1) do nothing, 2) repair (if crack exists), 3) strengthen, 4) restrict traffic and 5) close down bridge.

The *overall decision objective* of the bridge opertator is to minimize inspection and maintenance costs with the consideration of environmental and social impacts, i.e. aspects of sustainabilty and resilience of the transportation network.

Information about the asset

Data from the bridge is available in the Swedish bridge (and tunnel) management system (BaTMan) together with inspection records. Furthermore data from previous monitoring campaigns and traffic information are available. Numerical models for the critical detail exist. Relevant design standards and assessment guidelines for existing bridges are available.

Methods of collecting new information

Visual inspection to detect cracks is carried our regularly (based on previous inspection results, normally every 6 years). To improve the detection non-destructive testing (NDT) methods can be utilized e.g. Ultrasonic testing (UT), Eddy current testing (ET), Magnetic particle testing (MT) or Alternating current field measurement (ACFM). The different methods have different probability of detection (PoD) curves and costs that needs to be taken into account in the VoI analysis. Further possibilities for inspection could include monitoring of e.g. strains, dynamic properties and loads.

V. Critical appraisal, necessary simplifications

The decision problem is relatively complex, thus several simplifications are required to set up the VoI analysis, which can be further extended later.

- The context in short: public decision maker, existing structure, assessment of critical element of the structure. (It is interesting to note that several such elements exist!)
- The overall objective would include the calculation of the overall life-cycle benefits, which might be quite tedious, thus here we consider minimizing the cost of the condition assessment itself. The cost of the condition assessment includes the costs of inspection, repair (of detected cracks) and replacement of the failed detail (not the failure of the bridge!). For all these, first we use tentative values.
- Only inspection of fatigue cracks is considered.
- VoI is calculated for different inspection methods with known PoD.

As a starting point a generic influence diagram is created for the case of a bridge in which there are doubts about the condition of the bridge. These doubts may be based on a simplified assessment of the bridge and the decision to be made in light of this information is whether to take some invasive action, e.g. to repair the bridge, or to perform an enhanced assessment in hopes of obtaining more favorable results. This decision problem can be formulated as an influence diagram as shown in the Figure 2.

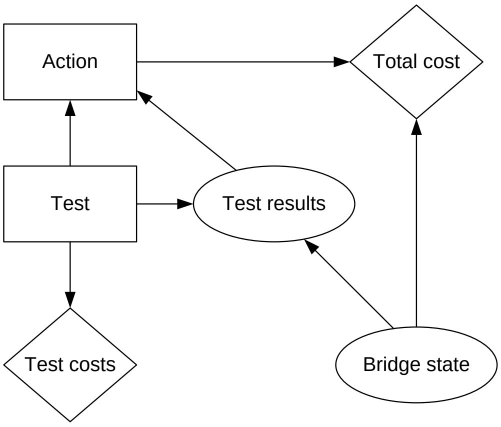

Figure 2: Influence diagram for the case study

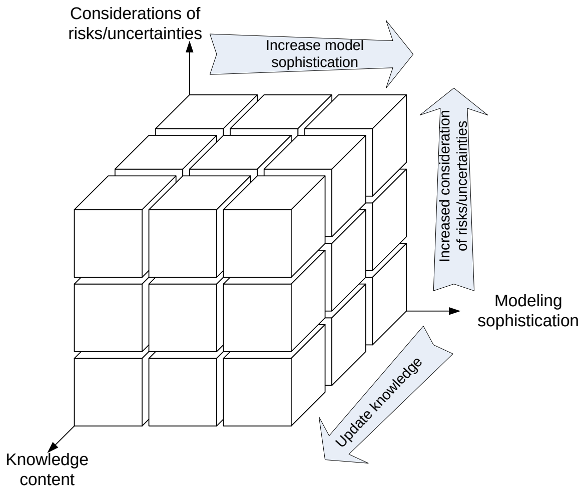

Figure 3: Directions of enhanced assessment

This diagram has been constructed considering two different decisions which are dependent; i.e. to make an enhanced assessment or not and to repair or not. In this example the state space for the decision nodes are *Action* = {Do nothing, Repair}, and *Test* = {No inspection, Inspection method A, Inspection method B}. In addition, there are also two chance nodes; the first reflects the actual state of the bridge while a second chance node represents the possible results obtained from a possible enhanced assessment. In this example the state space for the chance nodes are *Bridge state* = {No damage, Damaged detail}, and *Test results* = {No improvement, Improvement, No inspection}. The actual state of the bridge is independent of the other decisions and outcomes in this case and thus only the prior probabilities are needed as input. The *Test results* on the other hand are dependent on which *Test* is chosen as well as on actual *Bridge state*. Finally, there are two utility nodes: *Test costs* and *Total costs*. The prior includes the costs associated with outcomes defined within the *Test* decision node. The *Total costs* are dependent on two outcomes; the first is the *Bridge state* and the second is the decision regarding which *Action* is taken.

VI. Further steps and resources required

The generic influence diagram (Figure 2) will be applied for the specific case of the Söderström Bridge using realistic values where possible and the VoI will be calculated for various inspection methods. The calculation will be illustrated with the relevant decision trees as well.

The influence diagram in Figure 2 is presented as a problem of making observations in forms of “tests” to support decision making. In a general sense, assessing the condition of a structure as a basis for decision making can be done in multiple ways and it might be convenient to classify them based the specific aspects of these approaches as presented in Figure 3. The decision model is planned to be extended to include successive actions of enhanced condition assessment to guide moving along the three principal axes of the cube presented in Figure 3.

In parallel the generic model will be further developed to include more complex cases and integrated in the generic influence diagram to be developed within the Action. The alteration/integration of model will be supported by the existing frameworks from other COST members and might be carried out as a joint effort within an STSM. It is strongly proposed to work closely with similar case studies, e.g. *Asset integrity management of supports structures in an offshore wind park*.

VII. Summary and conclusion

An initial VoI flowchart has been created, see Figure 4, and will be continuously updated as the case study progresses.

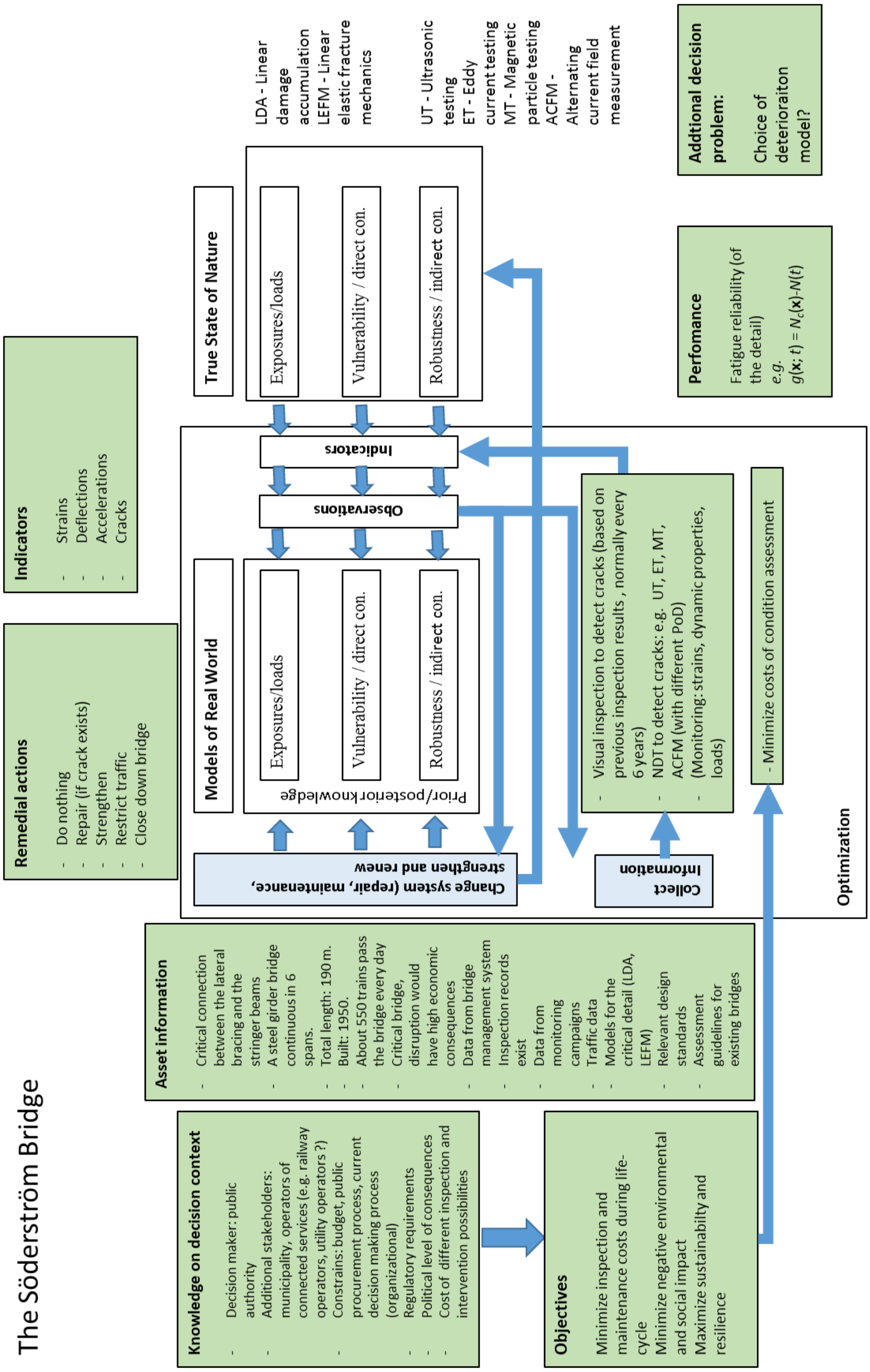

Figure 4: Preliminary VoI flowchart

In the current study VoI analysis is used to select the most appropriate inspection method to increase the fatigue life of a critical detail of a steel railway bridge. With the help of VoI analysis besides focusing on the possibilities of how to collect new information it is also possible, to some extent, to consider the effect of other types of improvements in the detailed condition assessment related to the modelling sophistication (i.e. structural analysis level) and the consideration of uncertainties. Furthermore the Bayesian decision analysis can help to decide which further actions should be taken regarding interventions to lengthen the service life of the bridge.

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References

Leander J., Andersson A., Karoumi R. (2010). Monitoring and enhanced fatigue evaluation of a steel railway bridge. *Engineering Structures*, Vol 32(3), pp. 854-863.
Leander J. and Karoumi R., (2012). Quality assurance of measured response intended for fatigue life prediction. *Journal of Bridge Engineering*, Vol. 17(4), pp. 711-719.
Leander J., Norlin B., Karoumi R., (2015). Reliability-based calibration of fatigue safety factors for existing steel bridges. *Journal of Bridge Engineering*, Vol. 20(10).
Leander J. and Selén E. (2017) The applicability of reliability-based inspection planning for steel bridges based on fatigue crack detection, *Proc. ICOSSAR 2017*, Vienna, Austria, pp. 3155-3164.
Leander J., Honfi D., Börnsson I. (2017). Risk-based planning of assessment actions for fatigue life prediction, *Procedia Structural Integrity,* Vol. 5, pp. 1221–1228.
Honfi D., Leander J., Björnsson Í. (2017) Decision support for bridge condition assessment. *4th International Conference on Smart Monitoring, Assessment and Rehabilitation of Civil Structures,* SMAR 2017, 13-15 September, 2017, Zürich, Switzerland.

Case Study Fact Sheet No. WG4-7

Optimizing monitoring: application to assessment of roof snow load risks

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I. Scope of the fact sheet

This fact sheet is presenting the working progress on the case study dealing with the monitoring of snow on a stadium roof and summarizes preliminary results regarding the risk related to the future use of the structure.

II. Abstract

A methodology to obtain cost-optimal decisions is implemented on the basis of limit state design, monitoring results, probabilistic reliability analysis and cost estimates. The implementation of the risk-based approach is illustrated in a case study dealing with a roof of a stadium located in Northern Italy. As the roof fails to comply with the requirements of the Eurocodes, the installation of a permanent monitoring system is recommended to allow for its real time reliability assessment. The results demonstrate the potential of the use of monitoring systems and probabilistic reliability analysis in order to support decisions regarding safety measures such as snow removal or temporary closure of the stadium and reflect the need for implementation of the discussed procedures in future standards.

III. Description of the case study

The case study is focused on a stadium erected at the beginning of the 1990s and located in Northern Italy, at an altitude of 190 m. The roof of the stadium consists of a cantilever steel beam IPE450 (Figure 1). The spacing between adjacent beams is 5 m. The inclination of the steel beam is negligible (α ≈ 0°).

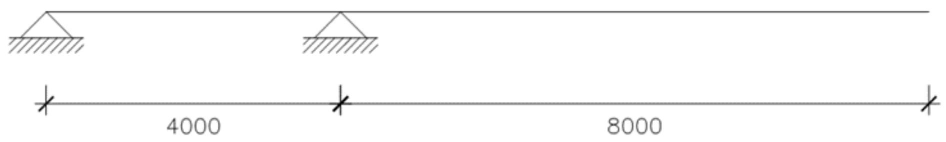

Figure 1. Scheme of the roof beam

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, the assessment of its actual structural reliability became an important issue after the recent roof collapses and the related studies. It is noted that the case study is focused on the Ultimate Limit State verification and serviceability aspects are not addressed herein.
The 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) results for the stadium under consideration, a snow load of 0.9 kN/m^2 was thus assumed. The current code consistent with EN 1991-1-3 (2003) – leads to a value of 1.2 kN/m^2. The obtained values indicate that the present snow loads exceed those considered in design and many existing structures, reliability of which is dominated by snow loads, do not comply with the requirements of the Eurocodes. The resistance of the roof is about 90% of the design value required by the Eurocodes.

In order to keep the reliability level of the stadium classified in the highest consequence class CC3 according to EN 1990 (2002) acceptable, the reliability of the roof is analysed by probabilistic methods to support the decision regarding the use of the stadium and the implementation of a permanent online monitoring system. Thereby the monitoring of snow depth or alternatively the monitoring of the snow load on the roof of the stadium are critically compared with the alternative to update structural reliability using ground snow load measurements from a nearest meteorological station. The purpose in this study is to select an optimal and immediate safety measure, i.e. close the stadium or clean-up the roof when the snow load reaches a threshold value and the reliability of the roof cantilevers drops below a specified target level.

IV. Implementation of a VoI analysis

Reliability analysis using prior information

Component reliability: The probabilistic reliability analysis of one cantilever is based on the limit state function for the section of the beam subjected to the maximum bending moment due to permanent actions and annual maxima of snow load for the region under consideration. Notation and probabilistic models of the basic variables are based on the recommendations of the JCSS Probabilistic Model Code (2017). Since the roof is flat without any obstacles, the snow load is dominating and the wind effects are neglected.

The obtained annual reliability index of 3.83 is significantly below the annual target level of 5.2 given in EN 1990 (2002) for CC3. Note that the EN 1990 level may be deemed to be too strict for verifications of existing structures; lower levels are proposed for instance in JCSS (2001), which can be further reduced for existing structures.

The FORM analysis indicates the dominating role of uncertainties in snow load, the small influence of steel yield strength and the model uncertainties and the negligible influence of the other random variables included in limit state function.

System reliability: The roof is supported by N = 40 identical cantilever beams. For simplicity reasons, the limited contribution of horizontal stiffening members to the system resistance is neglected. Reliability of the series system consisting of components with equal reliability indices and equal correlation coefficients is then analysed. The computed system annual reliability index of 3.55 suggests that the inclusion of system aspects has a moderate effect on a reliability level. However, this value should be understood as a lower bound estimate as horizontal stiffening members and other secondary beams will likely provide some redundancy in the system in case of failure of a cantilever. Consequently, the system reliability index may be comparable with that obtained for a component; detailed analysis is, however, beyond the scope of this study. As EN 1990 (2002) provides the target levels for structural members, the following analysis is exclusively focused on verification of components.

Updated reliability: The reliability is then updated considering the satisfactory past performance – the principle supported by ISO 13822 for assessment of existing structures (2010) and disputed also in Diamantidis (1987) – to improve this estimate. The updated component reliability index results as 3.83 and is negligibly higher than the prior index. Obviously, the updating for the Ultimate Limit State with the survival load around its characteristic value provides no improvements in reliability.

The obtained results show that the roof does not comply with the target reliability level in EN 1990 (2002) for a CC3 structure. The ground snow dominates structural reliability based on prior information. Therefore, it is proposed to apply continuous monitoring of snow loads in order to update information about this dominating variable. When a specified limiting value of the monitored parameter is exceeded, either snow on the roof can be removed or the stadium can be temporarily closed. This is in full agreement with the concept of a safety plan provided in ISO 2394 (2015). Such plan specifies "the performance objectives, the scenarios to be considered for the structure, and all present and future measures (design, construction, or operation, - e.g. monitoring) to ensure the safety of the structure." Table 1 illustrates the considered alternatives of monitoring.

Table 1. Alternatives of monitoring.

Alternative	**Description**	**Monitoring cost**	**Uncertainty in observed variable**
M1: Information on snow load on the ground at a nearest meteorological station	The decision maker is informed by the meteorological service about the actual ground snow load.	No charges related (negligible personal costs).	The major drawback is that considerable uncertainty in the shape factor due to conversion from ground to roof snow loads is not reduced.
M2: Measurements of snow depth on the roof	Following the recommendations of the provider of the sensor technology, snow depths are monitored at two locations. Note that a number of sensors on the roof could be optimised. However, this is beyond the scope of this study.	Acquisition cost: 7000 € for two sensors Annual operational cost including periodical inspections in winter periods and replacement each 10 years on average – 800 €/year	The predicted roof snow load is highly uncertain due to the significantly variable estimate of snow density. The uncertainty in the shape factor is reduced as no conversion from ground to roof snow loads is needed.
M3: Measurements of snow load on the roof	See the description for M2.	Acquisition cost: 14000 € for two sensors Annual operational cost including periodical inspections in winter periods and replacement each 20 years on average – 800 €/year	Accurate estimate of the roof snow load and reduced uncertainty in the shape factor.

V. Critical appraisal, necessary simplifications

When a target reliability level is specified, a limiting value of the monitored parameter above which reliability becomes unacceptable can be specified and a safety measure must be implemented. Target levels for such temporary situations are typically not provided in standards. As a first and conservative approximation, the annual target level of 5.2 given in EN 1990 (2002) can be adopted to specify the limits. Alternatively, a cost-benefit analysis can be conducted to decide about the use of the stadium on the basis of the balance between safety measure cost and the expected failure consequences. This strategy has been approved as it is supported by the reliability management in EN 1990: "The choice of the levels of reliability for a particular structure should take account of the relevant factors, including the possible cause and /or mode of attaining a limit state, the possible consequences of failure in terms of risk to life, injury, potential economic losses, public aversion to failure and the expense and procedures necessary to reduce the risk of failure."

Snow on the roof is removed whenever the risk – the probability of failure depending on the observed parameter (snow load) multiplied by the consequences of failure (including human, economical and environmental) – exceeds the safety measure costs. The risk-based criterion builds the basis for the VoI analysis. The variability of component reliability indices with the observed variables for M1, M2 and M3, respectively, is computed. When the target level of 5.2 is accepted and heavy snowfall is expected, the results indicate that the immediate cleaning is needed to guarantee sufficient reliability. When the target level is reduced according to the aforementioned cost-benefit considerations to 3.7, respective snow load limits for the three monitoring alternatives are identified.

VI. Further steps and resources required

An optimum monitoring strategy can be selected on the basis of the total costs of monitoring and of safety measures over a specified reference period. For consistency, the present and future expenses need to be expressed on a common basis. The results demonstrate the optimal monitoring strategy to be chosen.

Further development primarily concerns with a more detailed specification of failure consequences considering system aspects. In this respect assistance of the COST TU1402 members is sought. Further, extended datasets of snow depths and snow loads are needed to improve the decision making process.

VII. Summary and conclusion

Methodological procedures are implemented to assess snow load risks of a stadium roof structure that fails to comply with the requirements of the Eurocodes. The case study is illustrated in a flow chart of Figure 2. The study is based on the considerations of reliability acceptance and a general methodology to support cost-optimal decisions using limit state design and probabilistic reliability analysis and demonstrates that:

1. The required information to be obtained by SHM needs to be clearly specified before the monitoring system is installed.
2. Design of SHM is a complex issue that commonly needs to include the following steps:
    a) Component and/ or system structural and probabilistic reliability analysis providing insights into structural reliability and importance of basic variables; experience with previous performance of the structure can be utilised by probabilistic updating
    b) Identification of possible monitoring strategies, estimates of acquisition and running costs and quantification of uncertainties related to measurements
    c) Specification of threshold values for observed variables, which is commonly based on cost-benefit analysis that needs to take into account costs of safety measures and failure consequences
    d) Selection of a most appropriate monitoring strategy based on total cost optimisation for which detailed information on loads is needed to predict a number of exceedances of threshold values over a specified period.
3. SHM systems allow for a real time evaluation of the reliability level and support decisions regarding safety measures that in the case study comprise the snow removal or the temporary closure of the stadium.

Applications of the presented methodology 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.

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References

Diamantidis, D. (1987). Reliability assessment of existing structures. Engineering Structures 9(3): 177-182.

D.M. 12.02.82. (1982). Criteri generali per la verifica di sicurezza delle costruzioni, dei carichi e dei sovraccarichi (in Italian). Italy.

EN 1990 (2002). Eurocode - Basis of structural design. Brussels: CEN.

EN 1991-1-3 (2003) Eurocode 1: Actions on structures - Part 1-3: General actions; Snow loads. Brussels: CEN.

Joint Committee on Structural Safety (JCSS) (2017). JCSS Probabilistic Model Code (periodically updated, online publication).

Joint Committee on Structural Safety (JCSS) (2001). Probabilistic Assessment of Existing Structures, RILEM Publications S.A.R.L., edited by D. Diamantidis.

ISO 2394 (2015). General Principles on Reliability for Structures. 4th ed. Geneve, Switzerland.

ISO 13822 (2010). Bases for design of structures - Assessment of existing structures. Geneve, Switzerland: ISO TC98/SC2.

Case Study Fact Sheet No. WG4-8

Solar Molten Salt Concrete thermocline tank Monitoring: Introduction and preliminary framework for VoI analysis implementation

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I. Scope of the fact sheet

This fact sheet is presenting the working progress on a case study. It summarizes the character and context of the study and highlights the major steps and challenges for the implementation of a value of information assessment. Furthermore, outlook on further developments of the case study are given.

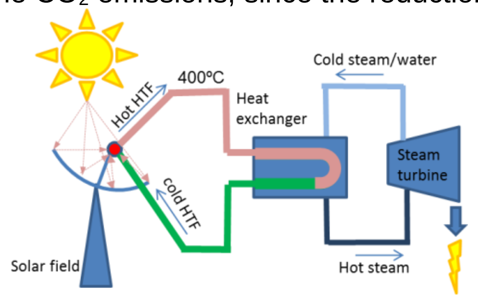
II. Abstract

Efficient thermal energy storage is one of the main challenges to help the widespread adoption of Centralized Solar Power (CSP). Currently we are working in and advanced solution, a solar molten salt concrete thermocline tank, to increase the efficiency and reduce the cost of thermal storage in CSP. To assure the performance of the tank and its structural integrity a monitoring system, based on fiber optic sensors is being developed and in order to quantify the value of this monitoring system a Value of Information (VoI) analysis is proposed as a case study. In this document, the preliminary framework for VoI analysis implementation is set.

III. Description of the case study

i. Introduction to thermal energy storage in CSP

To address the global challenges such as clean energy, climate change and sustainable development, there is a strong need for development of advanced energy technologies. Concentrating solar power (CSP) plants have grown considerably worldwide since 2010 as a clean renewable energy source. In fact, it is solar energy, one of the most suited to be the energy of the 21st century[1,2], where CSP's is about to play a major role in the Global Energy Mix, as it is expected to increase from 4GW in 2014 to 1000GW in 2050. This transition will have a big impact in lowering the CO_2 emissions, since the reduction is estimated at 2.1Gton[1].



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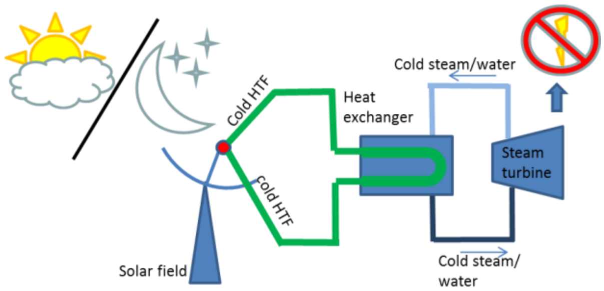

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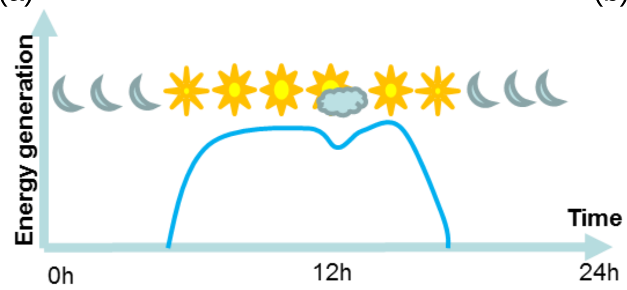
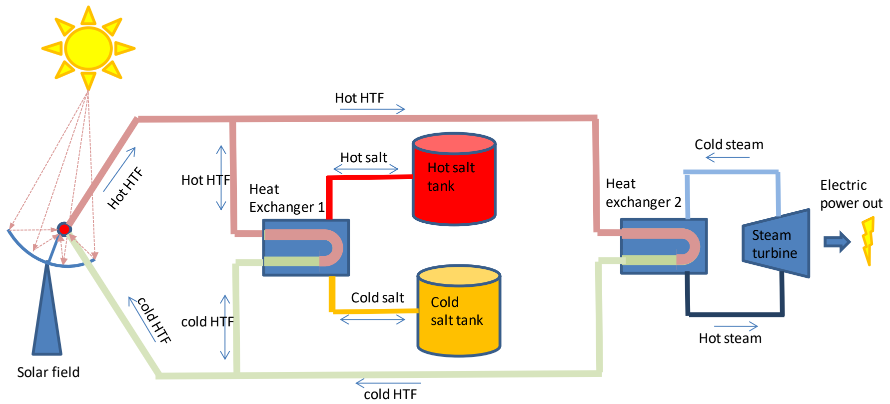


(c)

Figure 1: Schematic of a CSP using HTF (a) with sunlight (b) without sunlight (c) and energy generation throughout the day.



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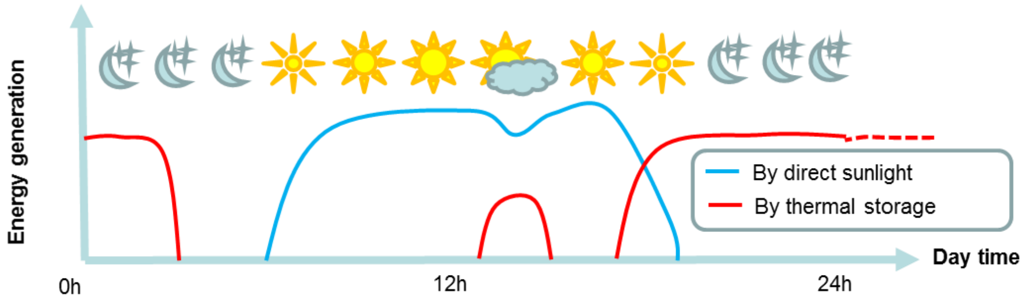

(b)

Figure 2: (a) Schematic of a CSP using SMS (b) and energy generation throughout the day.

Although the technical benefits from CSP in the energy challenge field are evident, the service life of actual CSP infra-structures are widely questioned, given that the durability of functional materials used to operate in the severe conditions are far from the optimized ratio of Efficiency/Durability/Cost. One solution to overcome this limitation is to use thermal storage materials. These materials should keep temperature levels that guarantee the working regime of heat exchange and turbines. Therefore, the solar resource intermittency is eliminated, by increasing the performance of the turbines due to constant steam production. Solar molten salts (SMS), composed of KNO_3 and $NaNO_3$, have been proven as a solution for this matter, since they can keep a plant working during a complete night[3]. SMS are however very corrosive and have high melting temperature (usually above 200ºC depending on the mixture used), they can operate currently up to 600ºC. CSP plants can use SMS in two different architectures. It can be used as an intermediate fluid just to store heat and oil as the main heat transfer fluid (HTF) or as the only HTF fluid. In figure 1 a parabolic trough solar facility is explained using SMS as intermediate fluid and oil as main HTF. The HTF is heated in the receiver with daylight and then diverted, toward two heat exchangers. In the heat exchanger, cold SMS, (at 300ºC) coming from the cold salt tank is heated up to 385ºC and sent to the hot salt tank. The cold HTF is sent back to the solar field to be heated. In heat exchanger 2, the heat is transferred to the steam, so the HTF is cooled down while the steam is heated up. The cold HTF is sent back to the solar field and mixed up with the cold HTF coming from heat exchanger 1. When the system is discharged (i.e. at night) the SMS and HTF flow in the opposite direction in heat exchanger 1. Therefore, the heat is transferred back to the HTF from the SMS. The hot HTF is sent to heat exchanger 2, where saturated steam is generated, allowing the generation of power in the turbine even at nighttime. In the case of using SMS as the only HTF, the salt is sent to the solar field from the cold tank at daytime to collect heat from the solar parabolic through colletors (or a solar concentrating tower). After heating, the hot SMS, at about 550ºC, is sent to the hot molten salt tank, where it is stored until it is directed to the heat exchanger. It is in the heat exchanger where the heat from the SMS is transferred to the steam, so, the SMS gives away energy coming down to a temperature around 300ºCwhile the steam is heated up. The hot steam heads to the steam turbine, where the electrical energy is generated. The steam is then sent to the turbine, cooled down after the turbine, and directed to the heat exchanger again in a cyclic operation. The cold salt after exiting the heat exchanger, is stored in the cold salt tank before being returned to the tower for heating again. If enough hot SMS is stored, the plant may operate during nighttime, reducing intermittency of CSP generation, since the salt in the hot salt tank can remain several hours hot enough to keep the plant running. When oil is used as HTFs the temperature at which the steam is heated is lower, so the efficiency of the plant is lower. However, when HTF and SMS are simultaneously used there is a great advantage: if a problem occurs with the SMS heat storage system, the plant can continue running, although only at daytime. As we have already discussed, SMS are at high temperatures and are very corrosive, having a big impact in the long-term stability of the materials used in the storage tanks and pipelines. Therefore, care must be taken while choosing the materials and design of the tanks, heat exchangers and pipelines. Especially since they must withstand the high temperatures of the salt and avoid corrosion. Also, note that SMS salt should not solidify, which could be specially harmfull for the pipelines, so they must be kept well above their melting point, >290ºC. Therefore, they impose several limitations coming from their high temperature and corrosive nature, which make their deployment and application a challenge[3].

ii. Advanced SMS tanks

Currently, SMS tanks are made of steel and must be carefully insulated to avoid any heat loss. Also, the current configurations need the deployment of two tanks: one for hot SMS and another one for cold SMS storage and great volumes of SMS. Therefore, a lot of research effort is conducted to optimize de Efficiency/Durability/Cost in the SMS storage tanks. Currently, in NEWSOL project[4], a solution based in novel materials for tank heat storage, advanced SMS, insulating materials, filler materials within the tank and advanced monitoring systems is being developed and demonstrated in a novel configuration: the thermocline tank. Figure 3 shows a CSP plant running only on SMS as HTF and storage fluid and the thermocline tank. The SMS, after heated in the solar field, is stored in the thermocline tank. The hot SMS, from the top of the tank is directed to the heat exchanger, where the steam is heated. Note the power block is the same as in the two-tank configuration. The cold SMS is then directed to the thermocline tank, where it is stored until is heated in the solar field again. Note that the SMS input and output to and from the tank are at different levels in the figure. This is to ensure that the fluid with different temperatures are placed at different parts of the tank to reduce heat transfer between them and isolate one from another. So, in the thermocline tank a vertical temperature gradient is observed, with an abrupt change in temperature between the hot and cold salt, called the thermocline. Therefore, when the tank is fully charged, the tank will be filled with hot SMS only, and when it is discharged, only cold SMS will be inside the tank. This way a lower volume of SMS is needed and a single tank is needed for storage, reducing the cost of the system. The design consists of a single thermocline tank with concrete walls, instead of the classic 2–tank system with steel walls, containing filler materials inside the tank for sensible heat storage from 290ºC up to 550ºC. The new thermocline concrete tank will take advantage of the thermocline effect to combine both, the hot and the cold molten salt in the same tank by separating them through a thin layer of a high-temperature gradient. This thermocline tank will be filled with low cost solid filler materials that displace the more expensive molten salts and act as the primary thermal storage medium. These filler materials will be arranged in several layers along the tank height.

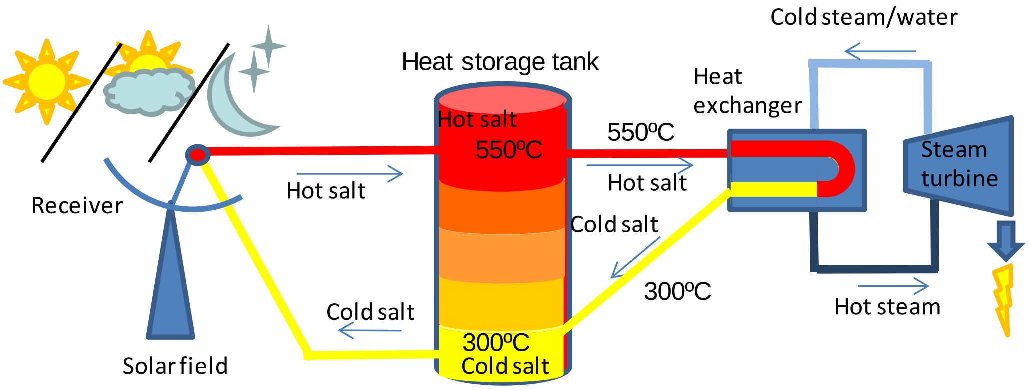

Figure 3: Thermocline SMS tank configuration on a CSP

iii. Monitoring system and sensors for molten advanced SMS concrete tanks

A solution to help widespread the adoption of SMS technology and thermocline concrete tanks, alongside with their benefits, is predictive maintenance of the storage tanks. Measuring and storing temperature data of the salt or the structural health of the tank is an important input and the sensing technologies applied are the key element for feasible and stable measurements. The most widely employed transducers are thermocouples for temperature monitoring, strain gages for strain monitoring and embedded or surface-bonded piezoelectric sensors for vibrations or acoustic emission (AE). However, these technologies present serious drawbacks for monitoring large structures at high temperature or corrosive environments. Thermocouples and strain gages are hard to multiplex and are subject to electric and magnetic disturbances. In the case of piezoelectric sensors, they lose the piezoelectric effect above temperature of Curie (300ºC) and exhibit degradation due to thermal cycling[5]. Furthermore, they do not offer very good multiplexing capacities, and resistance to corrosion.

Fiber optic sensors (FOS) are a promising alternative, since they can measure strain, temperature, vibrations, AE… and their properties make them well suited for applications in harsh environments such as CSPs[6]. Also, they are completely passive, so no power source is needed in the communication channel or sensor itself. Also, they are immune to electric and magnetic fields. The diameter of the cladding of the fiber is near 125um, so they are very small in size and light in weight. Furthermore, FOS can be easily multiplexed, so a complete CSP plant could be monitored with a simple optical network, especially in the case of Fiber Bragg Grattings (FBG)[7]. Some applications have been studied using commercial FOS in SMS, however they have been discarded, mainly because the packaging used for the sensor protection is corroded by the SMS[7]. A solution based on FBGs was presented by Grandal et al.[8], where a corrosion resistant package was employed to protect the FBG sensor from the salt and the temperature between 290 and 550ºC was monitored. In NEWSOL this solution will be further developed[4] for monitoring at different locations of the tank within two scopes: (1) structural health monitoring of the tank and (2) assessment of thermal performance of the system. So, the monitoring system for the thermocline tank will consist on a fiber optic sensor network that will monitor input and outlet temperatures of salt, molten salt temperature, concrete wall temperature, lateral wall temperature, base concrete temperature, cover temperature. Therefore, there will be two main groups of sensors: embedded sensors in the different concrete layers and temperature sensors immersed in molten salt. The sensors embedded in concrete layers will measure temperature and strain. The output of this monitoring system will consist on temperatures and strain in the different materials, structural parts and locations where the sensors are placed or embedded. So, the VoI analysis while rely in the information provided by this monitoring system.

IV. Implementation of a VoI analysis

As it has been explained, the information provided by monitoring system developed for the tank will assess the structural health and the thermal performance of the system. Therefore, the VoI analysis will be indicated for both cases from now on when necessary, and in section V the necessary simplifications will be discussed.

i) Decision maker
The decision maker will be the power plant operator, usually a private company.

j) Regulative constraints
In the case of the concrete SMS thermocline tank developed regulatory constraints are neglected, since the actual tank to be monitored is a demonstrator to proof the concept of the technology and material performance to a TRL level[9] equivalent of 7.

k) System temporal and spatial boundaries
A predesign of the demonstrator tank has already been performed, - However, since the tank will be built in 2019 and some of the material solutions have not been completely developed yet, changes may be made to obtain the final design. Also note that the tank will operate during a three-month period, from September 2019 to January 2019, so the conclusions will be only available after this time.

l) Events of interest and the corresponding representation
Structural health:
- crack is generated or growing: the apparition of cracks is an event that must be detected and evaluated, since it can be an indicator for spalling or excessive load.
- thermal stress of materials is detected: Thermal stress can undermine properties of the materials and risk the structural health of the system.
- mechanical load is a given value: when applied loads are higher than the ultimate loads the structure was designed for, the structural integrity of the system is endangered.

Thermal performance:
- temperature of the SMS at inlet or outlet is a given value: the temperature at inlets and outlets of the tank is an important parameter to estimate the charge and discharge rate of the tank.
- The temperature gradients on tank depth is at a given height: will indicate the volume and place of hot and cold salt.
- The tank is charged with a given capacity.
- The electrical power demand is at a given value

m) Event consequences
Structural health:
- bad performance of the thermocline tank;
- collapse of the tank;
- No thermal energy stored;
- no electrical power generation

Thermal performance:
- More/less efficient energy generation;
- Energy cost decrease/increase

n) Indicators (to observe)
Structural health:
- Loads, strains and cracks in the concrete
- Temperature in materials and positions

Thermal performance:
- Current stored thermal energy in the tank
- Electricity demand (not by monitoring system)
- Sun radiation (not by monitoring system)

o) Decision alternatives – monitoring and/or inspection options
- Cracks can also be detected by visual inspection but only if the cracks are in the outer part of the concrete tank, or if in the inner part during an outage (planned or emergency) and the tank is empty.
- Temperatures can also be measured with thermocouples but only at some given locations.

p) Decision alternatives – other measures, repair, replacement, etc.
Structural health:
- Do nothing
- Repair
- Replace

Thermal performance:
- do nothing
- charge tank at a given rate
- discharge tank at a given rate

V. Critical appraisal, necessary simplifications

The case study presented offers the possibility to perform a previous analysis and a model and to check it with a demonstrator and at the current state and timeline there is still room for changes that may ease the VoI analysis[8]. However, note that the demonstrator tank will not be constructed until September 2019, so the case study related to COST action TU1402 will not be completed until the action is already over. However, the first part, with the previous analysis and model should be ready before COST action TU1402 is finished.
Being the tank constructed a demonstrator that will not be connected to a real CSP plant, there are some constraints related to the thermal performance VoI analysis. Only some tests will be run during the operation of the tank, and these tests may not be indicative of the actual performance of a real tank. Also, there are factors related to energy generation cost that may change during the timeline of the project that would affect the VoI analysis related to thermal performance. So, from now on we will concentrate only in the structural part and we will leave the thermal performance VoI analysis as a future work.

VI. Further steps and resources required

Within the NEWSOL project[4], where the main developments of material solutions and monitoring system are developed and the tank is constructed. So, the following timeline is set:
June 2017 – framework to start project.
- From this moment on the monitoring system will be developed. This is already taking place in AIMEN under the author of this fact sheet's coordination. This task will finish in December 2019.

- The model that will allow VoI analysis will be set. Here collaboration from the cost network will be necessary, since the main strength of AIMEN is on monitoring system evaluation as a tool for SHM, but not so much in SHM and VoI. A STSM could help with this part. This could take place during the first semester of 2018.
- Preliminary VoI analysis. Fall 2018.

January 2019 – Monitoring system developed and tested in lab demonstrator
September 2019 – Tank and module operating
January 2020 – Evaluation after 3 month of tank operating. Re-evaluate VoI.

VII. Summary and conclusion

The VoI flowchart for concrete SMS thermocline tanks monitoring system is presented in figure 6. Note that although the thermal performance related VoI has been discarded, it is still shown (in grey) for completeness.

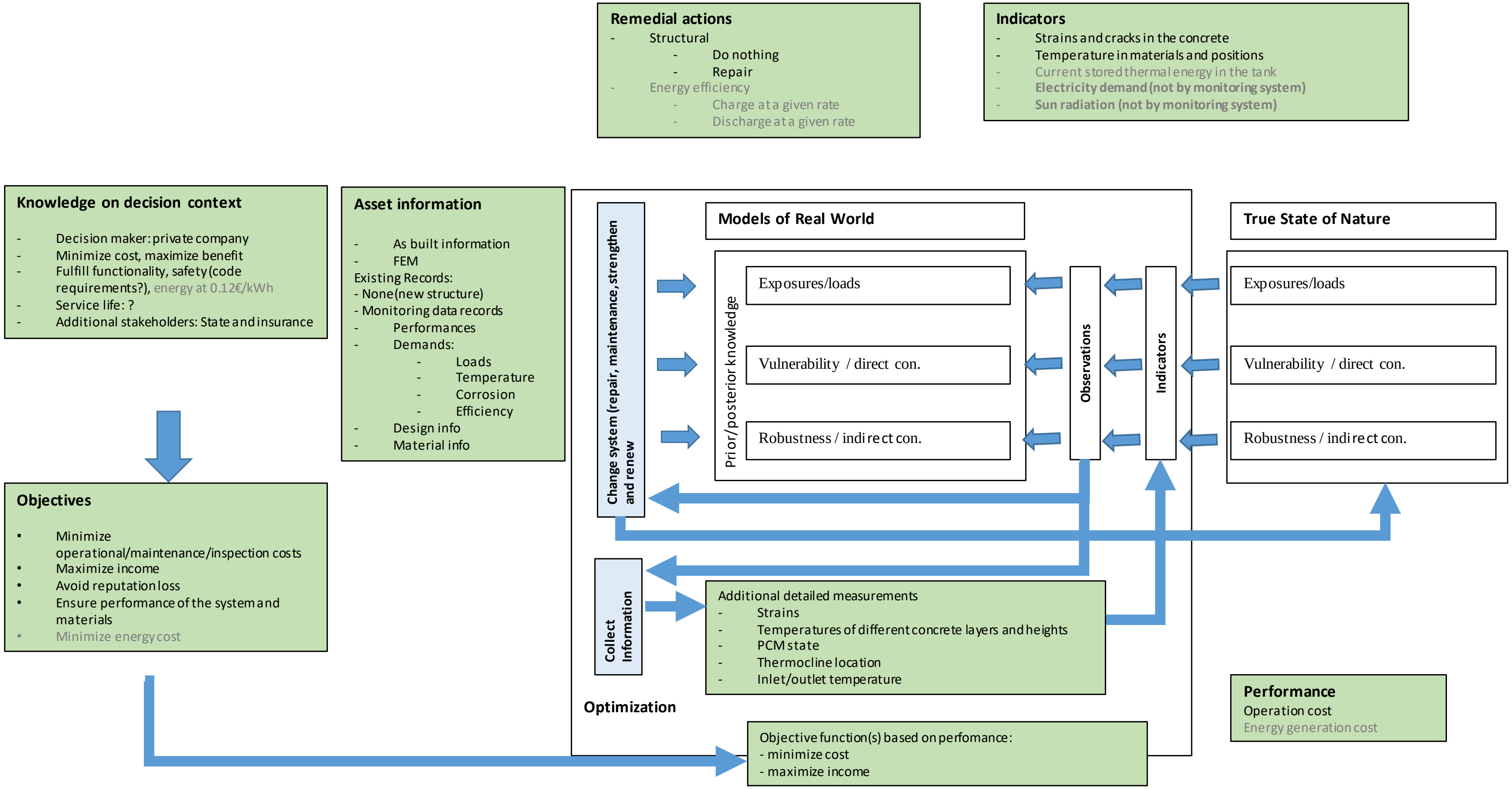

Figure 6: FlowChart for SMS concrete thermocline tanks.

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References

1. IEA. Technology Roadmap Solar Thermal Electricity, 2014 edition.
2. ESTELA. Solar Thermal Electricity. Strategic Research Agenda 2020-2025, (2012).
3. S. Kuravi, J. Trahan, D.Y. Goswami, M. M. Rahman, E.K. Stefanakos, "Thermal energy storage technologies and systems for concentrating solar power plants", Progress in Energy and Combustion Science, Vol. 39 (4), pp. 285-319, August 2013.
4. http://www.newsol.uevora.pt/
5. Santos J.L. and Farahi F., [Handbook of Optical Sensors], CRC Press (2014).
6. S.J. Mihailov, "Fiber Bragg Grating Sensors for Harsh Environments", Sensors, vol. 12, pp. 1898-1918, 2012.
7. Gill D., Kolb W., Briggs R., "An Evaluation of Pressure Measurement Technology and Operating Performance Using Sandia's Molten Salt Test Loop". Proceedings of the SolarPACES 2013 International Conference, Energy Procedia, Volume 49, 2014, Pages 800–809 (2013).
8. T. Grandal, Sergio Fraga and Ander Zornoza, "Solar molten salt temperature monitoring with fiber optic sensors", accepted for publication at Advanced Photonics congress, Control Number: 2526009, 2016, Vancouver (Canada).
9. HORIZON 2020 – WORK PROGRAMME, General Annex G: "Technology readiness levels".

Case Study Fact Sheet No. WG 4-9

Case Study Bridge proposed for further VoI analysis

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Ana Mandić Ivanković, University of Zagreb, Faculty of Civil Engineering, Croatia
Aleš Žnidarič, Slovenian National Building and Civil Engineering Institute, Slovenia

I. Scope of the fact sheet

This fact sheet is presenting the working progress on a case study. It summarizes the character and context of the study and highlights the major steps and challenges for the implementation of a value of information assessment. Furthermore and outlook on the further development of the case study is given.

II. Abstract

Highway bridge evaluated using monitoring data is proposed as a Case Study to be implemented in further VoI analysis. Focus of the proposed Case Study is to prove that implementation of monitoring data in decision making process can result in extended bridge service life and overall more sustainable bridge management. Monitoring data is obtained with Bridge Weigh-in-Motion (B-WIM) measurements, and applied in the load carrying capacity assessment, as bridge is designed accordingly to older codes applicable to time of its construction.

III. Description of the case study

Proposed bridge was assessed using monitoring data during the COST TU1402 supported STSM [1] at ZAG in Slovenia. Original design plans and built in reinforcement are available from the archives, along with monitoring data obtained with B-WIM measurements.
Case Study is simply supported highway bridge with a single span of 24,8 meters, with superstructure composed of five prefabricated I-type prestressed concrete girders connected with monolithic deck (Figure 1). Visual inspection and load carrying capacity assessment (both deterministic and probabilistic) of the bridge were conducted during the STSM.

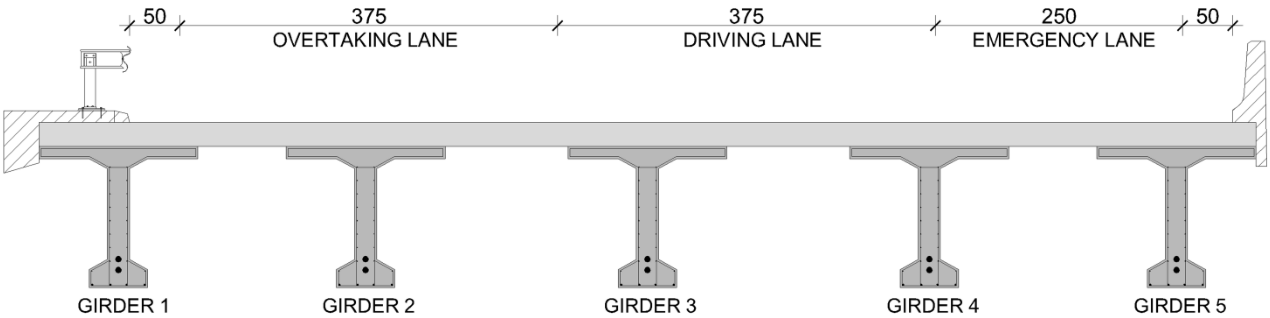

Figure 1 - Cross section of the Case Study Bridge

Results of the bridge evaluation clearly show the quantification of B-WIM measurements as a part of Structural Health Monitoring, and therefore provide good foundation for further research of a Case Study trough the VoI analysis. Results are presented in the Figure 2 as calculated reliability indexes with and or without implementation of monitoring data [1]. Use of such probabilistic procedures has shown in many cases that a bridge that does not meet usual safety requirements can in fact safely carry actual service loads without requiring any strengthening or traffic restrictions [2,3].

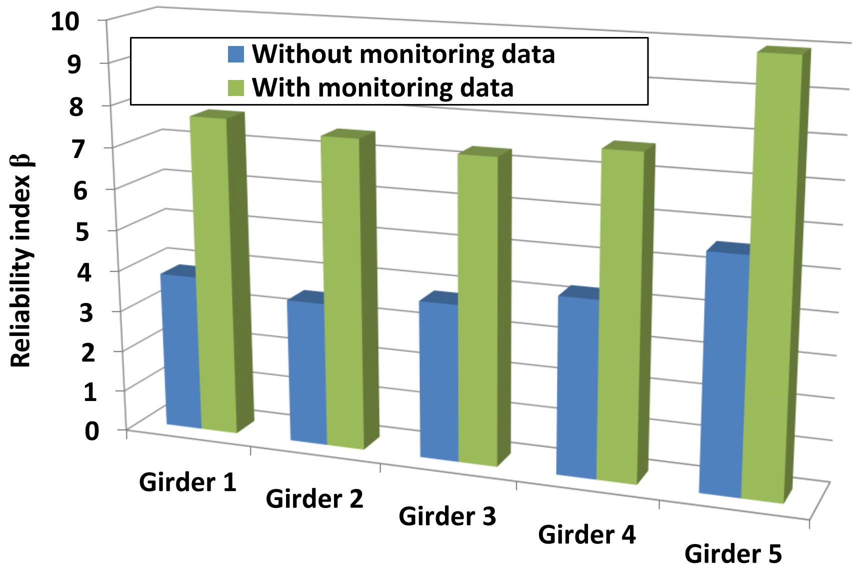

Figure 2 – Comparison of probabilistic assessment results - Case Study Bridge

IV. Implementation of a VoI analysis

q) Decision maker

Owner and operator of proposed bridge is National Road Directorate, meaning that there are no additional stakeholders influencing the decision making process. Main interest from owner's perspective is optimization of their bridge management system, based on priority ranking of bridge maintenance. General objective is to ensure normal and steady traffic flow, which is achieved with extending of designed bridge service life, thus avoiding bridge strengthening or imposing a traffic restrictions. Additional investments in SHM tools and advanced calculation procedures can be justified by fulfilling these objectives, and by that, minimizing the cost of bridge management.

r) Regulative constraints

Initial investments in SHM tools and advanced calculation procedures can increase cost of the bridge maintenance at first, but in time it will be justified by fulfilling objectives listed above, and by that, minimizing the overall cost of bridge management.
Additional major constraint regarding highway bridges is closing the bridge for traffic, which result in owner's income and reputation loss, along with traffic jams on alternative road directions. Unlike many other SHM tools, B-WIM system has minimum interference with the traffic on the bridge, as all sensors are applied on the bottom side of the bridge, and the bridge should be closed for minimum time during calibration. Visual inspection can also be conducted during calibration process, and bridge can be re – opened in just few hours.
If the bridge does not fulfil Eurocode threshold requirements for ULS and SLS on initial assessment level, re-evaluation is conducted using monitoring data, as shown on Figure 2.

s) System temporal and spatial boundaries

B-WIM system can be used on variety of road bridges, regardless of their dimensions and type, but it requires qualified personnel for installation and for data post-processing. Furthermore, advanced calculation procedures using probabilistic approach also requires additional knowledge and computational software.

t) Events of interest and the corresponding representation

Owner's decision on further actions regarding the bridge is based on:

i. condition assessment only,
ii. assessment according to valid codes;
iii. assessment according to data gained from short-time B-WIM measurements;
iv. assessment according to data gained from long-time of B-WIM measurements.

u) Event consequences

Consequences based on whether SHM data is used or not:

i. Unnecessary vs. necessary bridge strengthening.
ii. Appropriate vs. unsuitable bridge strengthening.
iii. Unnecessary vs. necessary traffic restriction.
iv. Minor or no action vs. medium or major measures on bridge before next assessment (e.g. in 5 years).
v. Money loss vs. money saving.
vi. Unmanaged waste of resources vs. appropriate allocation of resources.

v) Indicators (to observe)

Data gathered with B-WIM systems also provides realistic structural response of the bridge, which can be used to improve numerical models used in analysis. This structural response data can be defined by indicators such as: realistic influence lines, girder distribution factors and dynamic response of the bridge (DAF – dynamic amplification factor – which can be used to determine pavement condition). Furthermore, site specific traffic load model derived from B-WIM data is also defined as important performance indicator [4].
Reliability index is defined as a resulting performance indicator, based on which decision on further actions regarding the bridge will be made.

w) Decision alternatives – monitoring and/or inspection options

Based on the requirements, B-WIM monitoring can be set – up for different time periods, as some indicators listed above require less data than the others. Requirements and appropriate measurement periods can be defined based on detailed visual inspection and preliminary assessment results. Threshold values for different indicators are recommended with number of required vehicle passages [5].

x) Decision alternatives – other measures, repair, replacement, etc.

Evaluation of the bridge using B-WIM data can be conducted on multiple levels, with each substantial level requiring more data and advanced methods of calculation, and therefore resulting in increased bridge reliability. In case where a bridge does not fulfil any requirements (both ULS and/or SLS), recommendations for further actions are, based on the results and state of the bridge [6]:

i. Redefine the use of the bridge
ii. Impose a traffic weight restriction on the bridge
iii. Strengthening of the bridge
iv. Demolition and total replacement of the bridge

V. Critical appraisal, necessary simplifications

In order to set up VoI analysis, complete assessment of the bridge on each level is required, with results and substantial costs included. It is also required to evaluate the bridge importance in the network level define alternative routes and consequences of eventual bridge closing.
Development and calculation of bridge numerical models can be simplified, for example by reducing the bridge 3D model to 2D model of critical bridge girder. Also, bridge evaluation on the network level can be assumed based on similar bridges if enough data from the owner is available.
Advantage of proposed case Study Bridge is that results of complete assessment and numerical models are available from the work carried out during the STSM [1].

VI. Further steps and resources required

As contribution of b-WIM measurements to optimized bridge assessment is already proven (Figure 2), further research on proposed Case Study would include detailed cost analysis and feasibility study of all the parameters. COST TU 1402 network could contribute to further steps by allowing STSM to an institution with knowledge and experience in VoI analysis.

VII. Summary and conclusion

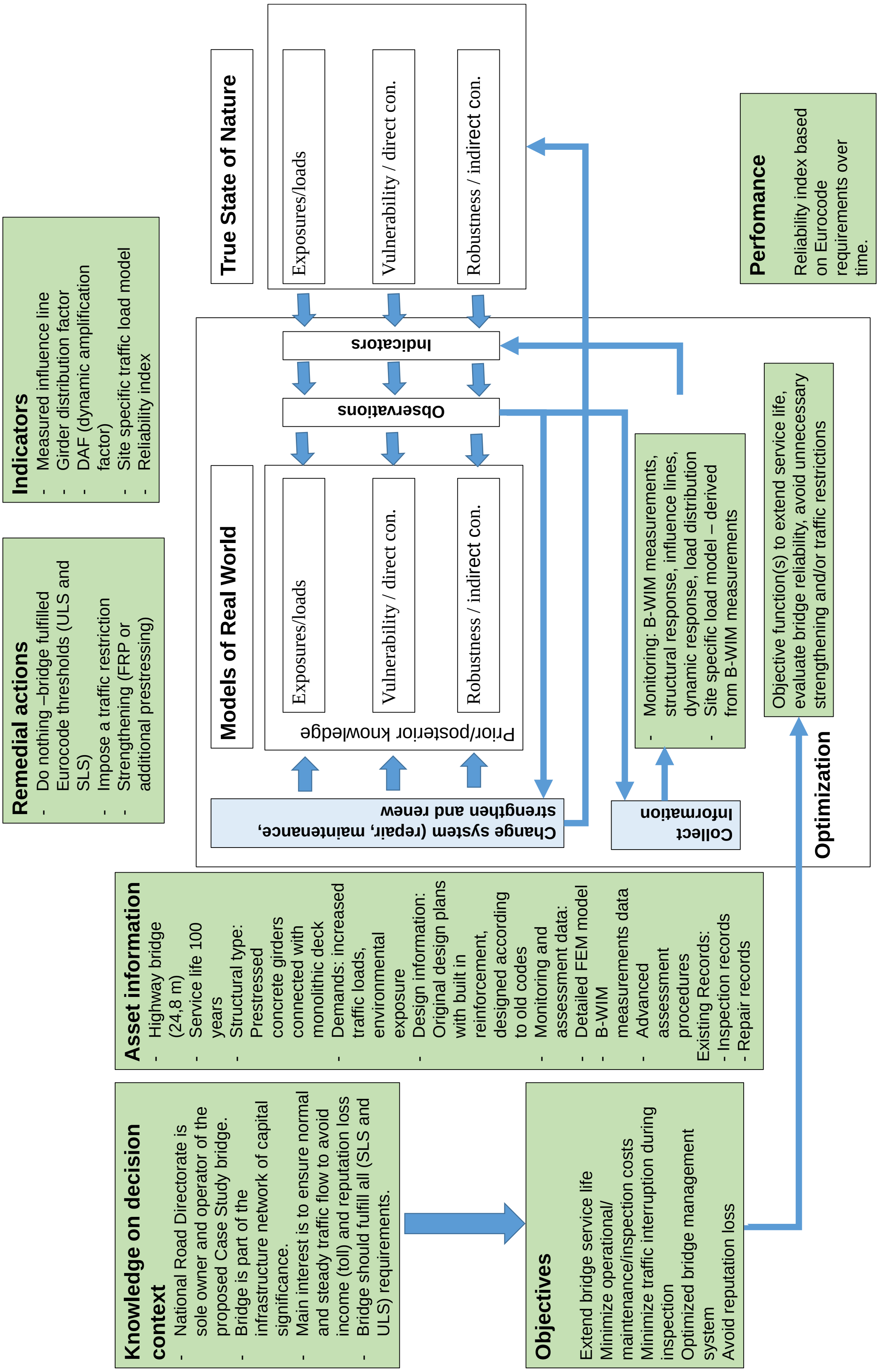

Figure 3 - Flow chart for VoI analysis of Case Study Bridge

Presented Case Study bridge is proposed for further research as it was already used to evaluate quantification of B-WIM as a part of SHM during COST Action TU 1402, and therefore all the data and information regarding the bridge are available, along with numerical models and assessment results. Therefore, we find it very suitable for further research including cost and VoI analysis to define real value of initial investments in bridge monitoring system against optimization of its maintenance and management over bridge service life.

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References

[1] Skokandić D., STSM Report – Application of B-WIM measurements in assessment of existing road bridges, *Report of Short Term Scientific Mission carried at ZAG and supported by COST Action TU 1402*, 2016.

[2] Enevoldsen, I.: Experience with Probabilistic-based Assessment of Bridges, *Struct. Eng. Int*., 2001; 11(4): 251-260

[3] Lauridsen, J., Jensen, J.S., Enevoldsen, I.: Bridge owner's benefits from probabilistic approaches, *Struct. Infrastruct. Eng*., 2007; 3(4): 281-302,

[4] Skokandić D., Žnidarič A., Mandić – Ivanković, A., Kreslin M.: – Application of B-WIM measurements in assessment of existing road bridges, *In Proceedings of Joint COST – IABSE Workshop,* Zagreb, 2017.

[5] Žnidarič A., Influence of number and quality of weigh-in-moiton data on evaluation of load effects on bridges*, Doctoral disertation,* Ljubljana: University of Ljubljana, 2017.

Part 2: General

Assessment of resilience indicators for structures and infrastructures

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Scope of the fact sheet

The fact sheet focuses on the description of the different approaches to define and group resilience indicators. Challenges for future developments are outlined.

Abstract

This fact sheet analyses and classifies the different resilience indicators available in literature, since there is no a widely accepted type of indicator that should be used to measure resilience. A list of existing resilience indicators is provided together with different classification methods, which are based on the hazard type, the temporal scale, the measurement method etc.

1 Why Resilience indicators?

The vagueness of the concept of resilience makes it difficult to define, but it becomes even more problematic when trying to measure it. The motivations and goals of resilience measurement are as different as the proponents advocating for them. Most researchers in the field emphasize that research on measuring community resilience is still in the early stages of development. There is no single or widely accepted method to the measurement issue as the landscape of resilience indicators is con- fusing and increasingly hard to navigate Cutter et al (2014). This is particularly the case for community resilience to disasters, since this concept raises not only questions related to the measurement of resilience, but also related to the definition and conceptualizations of communities. Since communities are interconnected systems whose indicators may apply to different scales and policy realms and also address different types of shocks. Resilience indicators can help to characterize the basic elements of the targeted system or unit of analysis and thus help to raise community awareness, because whenever there is a benchmarking, weak and strong points are identified and so it is easier to know where to address the funds to enhance the system. Being explicit about the objectives and motivations of measuring resilience is of critical importance for choosing the right approaches that integrate current conceptualizations and operationalisations of resilience.

2 Type of assessments

Three main categories are defined for the different resilience assessment approaches:

- indices
- scorecards
- tools and models

Indices are those quantifiable that represent a selected characteristic of resilience and these individual indicators are combined to create an index. The relationship between the indicators and the phenomenon they are measuring may be more or less direct. Indices are a statistical approach that summarizes observations or measurements by aggregating multiple indicators into a single value.

Scorecards provide an evaluation of performance or progress toward a goal. A vastly used method of this kind are the checklist, a series of questions related to presence or absence of resilience-related items and actions. A score is then produced based on how often the items are present, used, and so forth. Scorecards can have numerical values (110), letter "grades" such as (AF), or descriptors such as "excellent to poor". Scorecards are normally based on qualitative assessments and then converted to scores, while indices mostly use quantitative data to derive the index value.

Tools and models. *Models* create simplified representations of processes using mathematical formulas to approximate and understand the relationships and the interactions in the real world. Models can characterize economic resilience or resilience of a specific place. Models can be used to characterize economic resilience (Rose and Liao, 2005) in a computational way or to characterize the resilience of specific places (Renschler et al, 2010). *Tools* have been developed to provide a guidance for assessing resilience with sample procedures and survey instruments, or data for use in compilation of indices or scorecards.

3 Methodological approaches

There are two main different types of approaches. The first one is an *idiographic measurements or bottom-up*, which are locally generated and customized to particular places (Pfefferbaum et al, 2014). Typically use qualitative methodology and stress the resilience using highly localized data that may not be widely available. Due to the local knowledge and information these kind of case studies are rich and detailed, but the ability to compare across places is difficult because of the variability of the data and the different contexts and meanings of resilience.

On the other hand there are the *nomothetic or top-down* type assessments, which strive toward comparisons across varying units of analysis. They tend to use larger spatial units such as states or nations. This allows comparing units of analysis using standardized data, which make these types of resilience indices more amenable for examining spatial variability, allocating resources, and/or monitoring progress-all done at state, national, or international scales.

4 List of existing indicators

Many frameworks are available in literature. Different frameworks propose similar indicators and most of them overlap each other. After an extensive comparison between different frameworks, an exaustive list of resilience metrics, which is mainly based on the work of Mileti (1999), Renschler et al (2010), Cutter et al (2014) and Burton (2015), has been identified. The metrics are grouped according to the following five domains: *social, economic, community, institutional, environmental*.

5 Classification of indicators

An indicator, as can be inferred, simply "indicates" something or communicates information about a phenomenon of interest, which is called the *indicandum*. This phenomenon is sometimes difficult to analyze, difficult to measure or even it may not be measurable at all (Meyer, 2011). Since resilience is difficult to define and analyze, there are several different ways to classify the indicators of resilience. During a classification process different methods, such as spatial scale, temporal scales, hazard type etc. can be considered. The majority of the indicators are time and spatial dependent and are difficult to be transferred from one scale to another. So it is important to distinguish between indicators which are specific to the case study considered and the ones that can be generalized and extended to different hazards, communities etc. (Weichselgartner and Kelman, 2014). Another important characteristic of the indicators is their *relation to the phenomenon and resilience*, because it is a prerequisite for measuring resilience in quantitative terms. So it is possible to distinguish between indicators which cannot be *ordered* or *ranked* (e.g. gender or hazard type), the ones that can only be ranked (e.g. education level) and the ones that can be ranked and ordered by quantifying the interval between classes (e.g. net income in Euro/year).

6 State of art on classification methods

The first comprehensive work on classification about resilience metrics have been performed in the European project EMBRACE (Embrace, 2013) which proposed the following categories:

1) *Inherent or adaptive*
2) *Outcome or process*
3) *Domain*
4) *Relation with the phenomenon*
5) *Composite indicators*
6) *Scale of applications*
7) *Level of measurements*
8) *Resources & Capacities, Actions and Learnings*
9) *Generalization*
10) *Relation to resilience*
11) *General importance*
12) *Pre/Post-hazard event phase*
13) *Qualitative or quantitative*

However, the classification proposed in Embrace presents some limitations, because some of these categories overlap each other and they are not integrated in a useful manner, but they have the advantage of listing a series of characteristics of the indicators.

7 Proposed classification method

After reviewing the state of the art on classification methods, a new classification method is presented. Through this classification, it is possible to help decision makers in selecting the proper indicators for their problem at hand. This classification will allow them to assess resilience quantification properly and select the optimal resilience strategy. Based on these considerations, the resilience metrics have been classified according to 7 categories (or classification methods):

1) *Hazard Type*
2) *Temporal scale*
3) *Spatial scale*
4) *Building type*
5) *Level of Development*
6) *Domain*
7) *Measurement method*

7.1 Hazard type

In literature can be found resilience indicators that are just defined for a specific hazard corresponding to the specific case study presented. For example, Kafle (2012) developed a method (CRI) for measuring community resilience using process and outcome indicators in 43 coastal communities in Indonesia. He emphasized that community resilience can be measured, but in each measurement both location and hazard should be specified.

7.2 Temporal scale

Resilience, means the ability to recover from (or to resist being affected by) some shock, insult or disturbance. Recovery is a concept which is intertwined with time.in this case, resilience can be considered as a dynamic quantity that changes over time. Three temporal levels can be defined:

- Pre-hazard event phase (Preparedness);
- Short-term post-hazard event (Emergency response phase);
- Long-term post-hazard event (Reconstruction phase);

The indicators within the Pre-hazard event phase evaluate how much the system is ready to face unpredictable event. Indicators related to this phase mainly address the reduction of risks and vulnerabilities. The indicators within the emergency response phase describe the ability and the speed of a system in responding the initial needs after an extreme event.

7.3 Spatial scale

This classification emphasizes the importance of quantifying place-specific indicators. In fact, the resilience indicators may refer to a small unit of analysis (e.g. single building unit), or can be related to a whole city or nation. This classification divide indicators according to five categories which are:

- Building unit ;
- Building block;
- City/state;
- Region;
- Country;

The indicators in each category (neighborhood/city/region/country) are subsets of a larger group. The classification has been made just to facilitate resilience quantification in a proper scale.

7.4 Building type

This classification can be split in three groups:

- Critical facilities (e.g. hospitals, city-hall, etc);
- Residential buildings;
- No building type;

Critical/essential facilities are those facilities that provide services to the community and should be operative after a hazard. They include hospitals, police stations, fire stations, schools etc. Examples of indicators which belong to the first group are for example the accessibility and the special needs for disabled

7.5 Level of development

Two categories can be determined within this classification:

- Developed countries;
- Underdeveloped countries;

This classification is important because some indicators for example, lifelines such as communication, transportation, etc. all depend on the country‘s infrastructures condition, which is different in developed and underdeveloped countries. So this classification affects the resilience assessment, because some indicators might not be applied in underdeveloped countries.

7.6 Domain

Indicators can be also classified according to their domains or perspectives. For example there are indicators referring to ecological and social-ecological resilience, psychological resilience, critical infrastructural resilience or organizational and institutional resilience (Birkmann et al, 2012). The categories which belong to this type of classification are:

- Social;
- Economic;
- Ecological/environmental;
- Governmental/welfare/institutional;
- Physical/infrastructural;

Social resilience: the ability of groups or communities to cope with external stresses and disturbances, such as Child and Elderly Services, Community Participation etc. *Economic resilience*: the ability of the economy to cope, recover, and reconstruct. *Ecological / Environmental resilience*: is the capacity of an ecosystem to respond to a perturbation or disturbance by resisting damage and recovering quickly, such as biodiversity, water and air quality etc. *Governmental / welfare / institutional* resilience: In contrast to the more or less spontaneous individual and neighborhood responses to extreme Events, governmental services are designed to allow an orderly response. *Physical / infrastructural resilience*: this dimension focuses on a community's infrastructures, such as transportation, facilities, health care, etc.

7.7 Measurement method

They belong to this classification two categories:

- qualitative indicators;
- quantitative indicators.

Whenever a description is made, qualitative or quantitative assessments are necessary because some aspects in life cannot be measured and shall be described without a scale. These indicators can be used to identify the important constituent characteristics that shape community resilience. The use of these indicators is due to the fact that community resilience may be understood as a multi-faceted concept that goes beyond isolated capacities and views communities not only in spatial terms, but recognizes also common interests, values and social structures (Twigg, 2009).

Qualitative indicator-based approaches take into account that resilience is a dynamic and multi-faceted concept that relates to multiple levels. It is important to highlight that is possible to make qualitative indicators “quantifiable”. There are several examples published like to use a “structured subjective” method (Forrester et al, 2015), coding schemes, to derive proxies or to use rating scale. However, it has to be noted that despite transferring qualitative indicators into quantitative metrics, the underlying information remains still subjective.

On the other hand, quantitative indicators provide concrete metrics that are provided with data sources, justifications and sometimes the relationship to resilience. A good example of quantitative indicator is the proposed by Cimellaro et al (2015) to obtain a new resilience index for urban water distribution networks. This study proposes an index based on the product of three indicators. These indicators help planners and engineers to evaluate the functionality of a water distribution system which consists in delivering a certain demand of water with an acceptable level of pressure and quality. The quantitative indicator-based approach provide concrete metrics which are able to cover different perspectives of community resilience.

8 Aggregation of indicators

Typically the result of an analysis with different indicators, no matter the classification, is a composite indicator rather than numerous discrete indicators, since is easier to comprehend to the general public and to the policymakers as well (OECD, 2008). Therefore decision makers request the aggregated results in most cases. The step of aggregation is combined often with a weighting factor for each indicator. Different weights might influence the aggregation results to a smaller or larger extent, generating a loss of underlying information.

Whenever an aggregation is carried out it is absolutely necessary to make transparent which methodology has been applied and with which weight each individual indicator has contributed to the overall result. It is also highly recommended to keep hold of the information of the underlying individual components to be able to explain the reasons behind aggregating results.

9 Selection of key-indicators for specific case study

The classification methods given in Session 7 may be used to select the *key-indicators* for a specific case study (e.g. building, community, etc.) from the list given in session 4. Then in detail, the *key-indicators* are defined as indicators that:

- are rated with a high importance by the case studies;
- are universally applicable;
- show a clear relation to resilience;
- were mentioned by more than one case study.

Once the characteristics are established, a list with all the important indicators is ready to be composed. Some important indicators might not be considered in this list due to the applied criteria (especially the criteria “mentioned by more than one case study” reduces the list significantly), but this way of filtering allows communities to create a list of indicators that is concise and substantive.

10 Potential challenges of community resilience assessment using indicators

There exist potential and advantages of *indicator-based* approaches for assessing community resilience. These indicators can be classified or systematized in different groups for an easy understanding. Indicator-based approaches for assessing resilience are promising tools, because they allow when evaluated at regular intervals monitoring changes over time in both magnitude, direction and space (Cutter et al, 2010). They allow identifying the major weaknesses or drawbacks of resilience. The use of qualitative indicators for constant comparison and evaluation of changes in the spatial and temporal domain is more difficult than with quantitative indicators because the data are subjective.

11 Examples of measurement methods

In table 1 is given a list of the different measurement methods to assess resilience categorized according to its type, spatial scale and method.

Table 1: Methodologies to evaluate resilience

Measure Name	Type	Spatial	Focus	Method
APIRE	Tool	Country	Whole community	Top down
BRIC	Index	USA Counties	Whole community	Top down
CART	Tool	Community	Whole community	Botton up
CC RAM	Tool	Community	Whole community	Botton up
CDRI	Index	USA Coastal Counties	Whole community	Top down
Coastal Resilience index	Score-card	Community	Whole community	Botton up
CoBRA	Tool	Community	Whole community	Botton up

Community Resilience system	Tool	Community	Whole community	Botton up
Community Resilience index	Index	Community	Asset	Top down
CREAT	Tool	Infrastructures	Whole community	Top down
DFID Resilience	Tool	Country	Whole community	Botton up
FAO Livelihoods	Index	Community	Asset	Botton up
Financial system Resilience	Index	Infrastructures	Asset	Top down
FM Global Resilience	Index	Infrastructures	Whole community	Top down
NIST	Tool	Infrastructures	Whole community	Top down
Oxfam GB	Index	Community	Whole community	Botton up
PEOPLES	Tool	Community	Whole community	Top down
RCI	Index	USA metro areas	Asset	Top down
ResilUS	Tool	City	Asset	Top down
RMI	Index/Tool	Infrastructures	Whole community	Top down
Rockfeller 100 Resilience cities	Tool	Community	Whole community	Botton up
RRI	Index	Community	Whole community	Botton up
SPUR	Score-card	Community	Asset	Botton up
Surging Seas	Tool	USA coastal countries	Whole community	Top down
TNC Coastal Resilience	Tool	Coastal areas	Whole community	Top down
UNISDR Resilient Cities	Tool	Cities	Whole community	Botton up
USAID Resilience	Tool	Countries	Whole community	Botton up

12 Remarks and conclusions

Measurement tools and indicators cannot create a resilient community, but they can assess resilience and provide a community the directions for becoming safer, stronger, and more vibrant in the face of unanticipated events. This chapter emphasizes that it is important to decide and establish a norm to classify the indicators, but this classification can vary throughout the different case studies and/or projects. Several classifications are presented showing that there are many options available, so the more suitable classification should be determined based on the parameters that are going to be analyzed

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Bank W (2015) Female labor force participation rate. URL http://data.worldbank.org/indicator/SL.TLF.CACT.FE.ZS

Berke PR, Campanella TJ (2006) Planning for postdisaster resiliency. The Annals of the American Academy of Political and Social Science 604(1):192–207

Burby RJ, Deyle RE, Godschalk DR, Olshansky RB (2000) Creating hazard resilient communities through landuse planning. Natural hazards review 1(2):99–106

Burton CG (2015) Validation of metrics for community resilience to natural hazards and disasters using the recovery from hurricane katrina as a case study. Annals of the Association of American Geographers 105(1):67–86, DOI 10.1080/00045608.2014.960039

Center H (2002) Human links to coastal disasters. Report, Heinz Center for Science, Economics, and the Environment

Cimellaro GP, Tinebra A, Renschler C, Fragiadakis M (2015) New resilience index for urban wa- ter distribution networks. Journal of Structural Engineering, ASCE 10.1061/(ASCE)ST.1943- 541X.0001433, DOI 10.1061/(ASCE)ST.1943-541X.0001433
Colten CE, Kates RW, Laska SB (2008) Three years after katrina: Lessons for community resilience. Environment: Science and Policy for Sustainable Development 50(5):36–47
Cutter S, Boruff B, Shirley W (2003) Social vulnerability to environmental hazards. Social Science Quarterly 84(2):242–261
Cutter S, Barnes L, Berry M, Burton C, Evans E, Tate E, Webb J (2008) A place-based model for understanding community resilience to natural disasters. Global Environmental Change 18(4):598–606
Cutter S, Burton C, Emrich C (2010) Disaster resilience indicators for benchmarking baseline con- ditions. Homeland Security and Emergency Management, 7(1), DOI 10.2202/1547-7355.1732 Cutter S, Ash K, Emrich C (2014) The geographies of community disaster resilience. Global Environmental Change 29:65–77
Enarson E (2007) Identifying and Addressing Social Vulnerabilities, ICMA Press, Washington, p pp. 258378
Forrester J, Cook B, Bracken L, S C, Donaldson A (2015) Combining participatory mapping with q-methodology to map stakeholder perceptions of complex environmental problems. Applied Geography 56:199–208
Godschalk DR (2003) Urban hazard mitigation: creating resilient cities. Natural hazards review 4(3):136–143
Godschalk DR (2007) Mitigation. Emergency management: Principles and practice for local gov- ernment pp 89–112
IFRC (2012) Characteristics of a safe and resilient community. community based disaster risk reduction study. Report, International Federation of Red Cross and Red Crescent Societies
Kafle S (2012) Measuring disaster-resilient communities: a case study of coastal communities in indonesia. Journal of business continuity & emergency planning 5(4):316–326
Meyer W (2011) Measurement: Indicators scales indices interpretations
Mileti D (1999) Disasters by Design: A Reassessment of Natural Hazards in the United States. Joseph Henry Press (May 18, 1999), Washington D.C.
Morrow B (2008) Community resilience: A social justice perspective (the community and regional resilience initiative research report 4). Oak Ridge, TN: Community and Regional Resilience Initiative
Murphy BL (2007) Locating social capital in resilient community-level emergency management. Natural Hazards 41(2):297–315
Norris FH, Stevens SP, Pfefferbaum B, Wyche KF, Pfefferbaum RL (2008) Community resilience as a metaphor, theory, set of capacities, and strategy for disaster readiness. Am J Community Psychol 41:127–150
OECD (2008) Handbook on constructing composite indicators: Methodology and user guide. Re- port, http://www.oecd.org/std/42495745.pdf
Pfefferbaum RL, Pfefferbaum B, Nitima P, Houston JB, Van Horn RL (2014) Assessing community resilience: An application of the expanded cart sur- vey instrument with affiliated volunteer responders. American Behavioral Sci- entist doi:10.1177/0002764214550295, DOI 10.1177/0002764214550295, URL http://abs.sagepub.com/content/early/2014/09/24/0002764214550295.abstract
Renschler C, Frazier A, Arendt L, Cimellaro GP, Reinhorn AM, Bruneau M (2010) A framework for defining and measuring resilience at the community scale: The peoples resilience frame- work. Report NIST Technical Report GCR 10-930, NIST - U.S. Department of Commerce & National Institute of Standards and Technology
Ronan K, Johnston D (2005) Promoting community resilience in disasters: The role for schools, youth, and families. Springer Science & Business Media
Rose A, Liao SY (2005) Modeling regional economic resilience to disasters: A computable general equilibrium analysis of water service disruptions. Journal of Regional Science 45(1):75–112, URL ¡Go to ISI¿://WOS:000226572800004, times Cited: 22
Sylves R (2007) Budgeting for local emergency management and homeland security.in emergency management: Principles and practice for local government, edited by w.l. waugh and k. tierney. Washington, DC: International City Managers Association pp 183–206
Tierney K (2009) Disaster responce: Research findings and their implications for resilience mea- sures. Report, CARRI Research Report 6, Community & Regional Resilience Institute
Twigg J (2009) Characteristics of a disaster-resilient community: A guidance note (version 1) for the dfid disaster risk reduction interagency coordination group. Report, Community Research and Development Information Service
UNISDR (2012) How to make cities more resilient: A handbook for local government leaders Vale LJ, Campanella TJ (2005) The resilient city: How modern cities recover from disaster. Oxford
University Press
Weichselgartner J, Kelman I (2014) Geographies of resilience: Challenges and opportunities of a descriptive concept. Progress in Human Geography DOI 10.1177/0309132513518834