TU1406
COST ACTION

COST ACTION TU1406
QUALITY SPECIFICATIONS FOR ROADWAY BRIDGES, STANDARDIZATION AT A EUROPEAN LEVEL

eBook for the 3rd Workshop Meeting

Delft, 20 – 21 October 2016

Editors : Irina Stipanovic Oslakovic, Giel Klanker, Jose Matos, Joan Casas, Rade Hajdin

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EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

EUROPEAN SCIENCE FOUNDATION
SETTING SCIENCE AGENDAS FOR EUROPE

Opening Note from the Chair

Prof. Jose Matos
Chair COST Action TU 1406

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

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

COST Action TU1406 Workshops aim to facilitate the exchange of ideas and experiences between active researchers and practitioners as well as to stimulate discussions on new and emerging issues in line with the conference topics. This second Workshop addresses the WG1, performance indicators, WG2, performance goals, and WG3, establishment of a Quality Control plan, developments. The main outcome, given in this eBook, is really important, not only for those directly involved in this Action, but also for the whole bridge engineering community.

[COST TU 1406 short video]

Note from the Vice Chair

The working group meetings and 3rd Workshop of COST Action TU1406 in Delft is the continuation of the work developed within WG1 and the second working sessions for WG2 and WG3. The state-of-the-art and the different approaches along Europe on the performance indicators used by the different owners and operators to meet the quality expectations of the users is close to its completion. A huge amount of information has been collected and the posterior processing will become a relevant input for the rest of the WG´s. Also the collecting of research performance indicators was presented. This will serve as the basis for the proposal of new indicators that will allow a more optimized definition of future quality control plans for highway bridges.

An important number of papers were also presented during the Workshop related to all WG´s. The key-note presentations explaining the experience from previous COST actions, on sustainability indicators and pavement performance indicators, will be very helpful for the Action in seeking the best methodology and approaches to gather the most relevant and representative data from the large data base that is in our hands by now.

Lively discussions after the presentations and in the WG´s meetings has made possible to get and agreement and deliver a clear route map among the different WG conforming the Action on how and what to focus in the coming years, looking at their specific goals and close interactions and avoiding overlapping of activities. In summary, looking to the success of this second workshop, and the future activities planned, we may be confident on the achievement of the required standardization of the quality specifications for highway bridges in Europe.

Prof. Joan Casas
Vice-Chair COST Action
TU 1406

Note from the Local Organizers

The third workshop of COST Action TU 1406 “Quality specifications for roadway bridges, standardization at a European level” (www.tu1406.eu), was held in Delft, the Netherlands, at TNO Institute on 20th and 21st of October 2016, after IALCCE conference. http://www.ialcce2016.org/

The working group meetings and 3rd Workshop of COST Action TU1406 in Delft is the continuation of the work developed within WG1 and the second working sessions for WG2 and WG3.

The COST Action TU1406 comprises members from nearly all European Countries, as well as countries outside Europe. Wide participation is an important feature of these actions, whose scope is to form a European research area across borders and interlink high-quality research and practice communities in Europe and worldwide. The location of workshop is Delft, which is located centrally in the heart of Holland known as the Randstad region, between Rotterdam and The Hague. Delft represents Europe at its best, with fast connections to anywhere in the world. Both physically, boasting Amsterdam and Rotterdam The Hague airports in close proximity and countless international railway connections available, as well as in terms of its role in many international knowledge networks, alliances and businesses.

With these words: It is a pleasure to welcome the WG Meetings and the 3rd Workshop of the COST TU1406 Action in Delft!

Dr. Irina Stipanovic,
University of Twente

Giel Klanker,
Rijkswaterstaat

Acknowledgement

The editors would like to thankfully acknowledge the contribution of those who supported the execution of this event:

TNO, the Netherlands Organization for applied scientific research

Rijkswaterstaat, Ministry of Infrastructure and the Environment

University of Twente, Faculty of Engineering Technology

and especially to:

Agnieszka Bigaj-van Vliet, TNO

Jos Wessels, TNO

Jaap Bakker, Rijkswaterstaat

Zaharah Allah Bukhsh, University of Twente

Eleni Chatzi, *Technical Secretariat of COST Action TU1406*

Sérgio Fernandes, *Technical Support of COST Action TU1406*

Lara Leite, *Administrative Secretariat of COST Action TU1406*

WG 2 AND WG 3 WORKSHOP BRIDGE PERFORMANCE GOALS AND QUALITY CONTROL PLANS

In the first year of the COST TU 1406 Action the main focus was on screening process of existing European documents and establishing a database for PIs. The next step in the project is to focus on technical and non-technical bridge performance goals, followed by the establishment of quality control plans. Therefore, the workshop is dedicated on following topics:

- Evaluation of bridge performance (criteria, requirements, goals, thresholds);
- Technical, sustainability and economic aspects of bridge performance;
- Life-cycle Assessment;
- Quality control plans;
- Reliability of testing/investigation techniques and methods;
- Qualification criteria for inspectors.

During the 2-day workshop selected manuscripts have been presented and discussed, followed by WG 2, WG 3 and WG5 meetings.

Dr. Irina Stipanovic, WG 2 leader

Prof. Rade Hajdin, WG 3 leader

Dr. Helmut Wenzel, WG 5 Vice leader

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Joint WG 2 and WG 3 meeting

Alfred Strauss	Categorization of indicators / findings
Irina Stipanovic	Performance goals and performance assessment
Rade Hajdin	From findings to performance goals

Workshop Contributions

TU1406
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WG2 and WG3 Workshop

20-21 October 2016
Delft, The Netherlands

J. Campos e Matos   Chair   University of Minho, Portugal

COST is supported by the EU Framework Programme

COST
EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

OUTLINE

1. Aim
2. Organization
3. Scientific Programme
4. Impacts
5. Representation
6. Members
7. Dissemination
8. Future Events

1. AIM

The overall intention of the Action is to

develop a guideline for the establishment of Quality Control (QC) plans in roadway bridges

Reachable by taking 5 specific steps:

(i) Systematize knowledge on QC plans for bridges, which will help to achieve a state-of-art report that includes performance indicators and respective goals;

(ii) Collect and contribute to up-to-date knowledge on performance indicators, including technical, environmental, economic and social indicators;

(iii) Establish a wide set of quality specifications through the definition of performance goals, aiming to assure an expected performance level;

(iv) Develop detailed examples for practicing engineers on the assessment of performance indicators as well as in the establishment of performance goals, to be integrated in the developed guideline;

(v) Create a database from COST countries with performance indicator values and respective goals, that can be useful for future purposes.

2. ORGANIZATION

Management Committee

Including:
- MC Chair
- MC Vice-Chair
- WG's Leaders and Vice-Leaders
- General Secretariat
- STSM Leader and Vice-Leader
- M&E Leader and Vice-Leader
- Innovation Leader and Vice-Leader
- R&D Leader and Vice-Leader

Core Group

- MC Chair
- MC Vice-Chair
- WG's Leaders
- General Secretariat
- STSM Leader
- M&E Leader *
- Innovation Leader *
- R&D Leader *

Advisory Board

- Industry/Owners/ Operators
- External Advisors (MC Observers)

MC Observers

- Australia
- Japan
- North America
- South America
- South Africa

An MC Observer per Continent

* under an "ad-hoc" basis

2. ORGANIZATION

WG5. Drafting of guidelines/recommendations

- Existing documentation (format and content)
- Document preparation
- Easy to use document

WG4. Implementation in a case study

- Benchmarking
- Validation
- Discussion

WG1. Performance indicators

- Technical indicators
- Environmental indicators
- Others

WG2. Performance goals

- Technical goals
- Environmental goals
- Others

WG3. Establishment of a QC plan

- Bayesian nets
- Procedure to develop a QC plan for a single bridge

3. SCIENTIFIC PROGRAMME

It is organized based on the division of tasks (and subtasks) allocated according to a timetable.

WG		Year 1				Year 2				Year 3				Year 4			
		Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4	Q1	Q2	Q3	Q4
WG1	Technical indicators	X	X	X	X												
	Environmental indicators	X	X	X	X												
	Other indicators	X	X	X	X												
WG2	Technical goals					X	X	x	x								
	Environmental goals					X	X	x	x								
	Other goals					X	X	x	x								
WG3	Survey of European roadway QC plans	X	X	X	X	X	X										
	Procedures for the establishment of a QC plan							x	x	x	x						
WG4	Selection of case studies							x	x	x							
	Benchmarking										x	x	x	x	x		
	Application on a QC plan											x	x	x	x		
WG5	Standardized performance indicators				X	X	X	x									
	Standardized goals								x	x	x	x					
	Standardized QC plan										x	x	x	x	x	x	x
WG6	Dissemination	X	X	X	X	X	X	x	x	x	x	x	x	x	x	x	x

3. SCIENTIFIC PROGRAMME

General plan

- One Workshop per year (end of each year);
- One Training School per year (end of each year):
- STSMs occur through out the entire project;
- WG meeting dates will usually coincide with Workshops and MC meetings.

Activity/Months	3	6	9	12	15	18	21	24	27	30	33	36	39	42	45	48
Meeting	X			X		X		X		X		X		X		X
Workshop				X				X				X				
Conference																X
Training school								X				X				X
STSM	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X	X
Website	X			X		X		X		X		X		X		X
Milestone				M1				M2		M3				M4		M5

3. SCIENTIFIC PROGRAMME

Milestones

Activity/Months	3	6	9	12	15	18	21	24	27	30	33	36	39	42	45	48
Milestone				M1				M2		M3				M4		M5

M1: WG1 – Performance indicators
Elaborate a report of performance indicators

M2: WG2 – Performance goals
Elaborate a report of performance goals

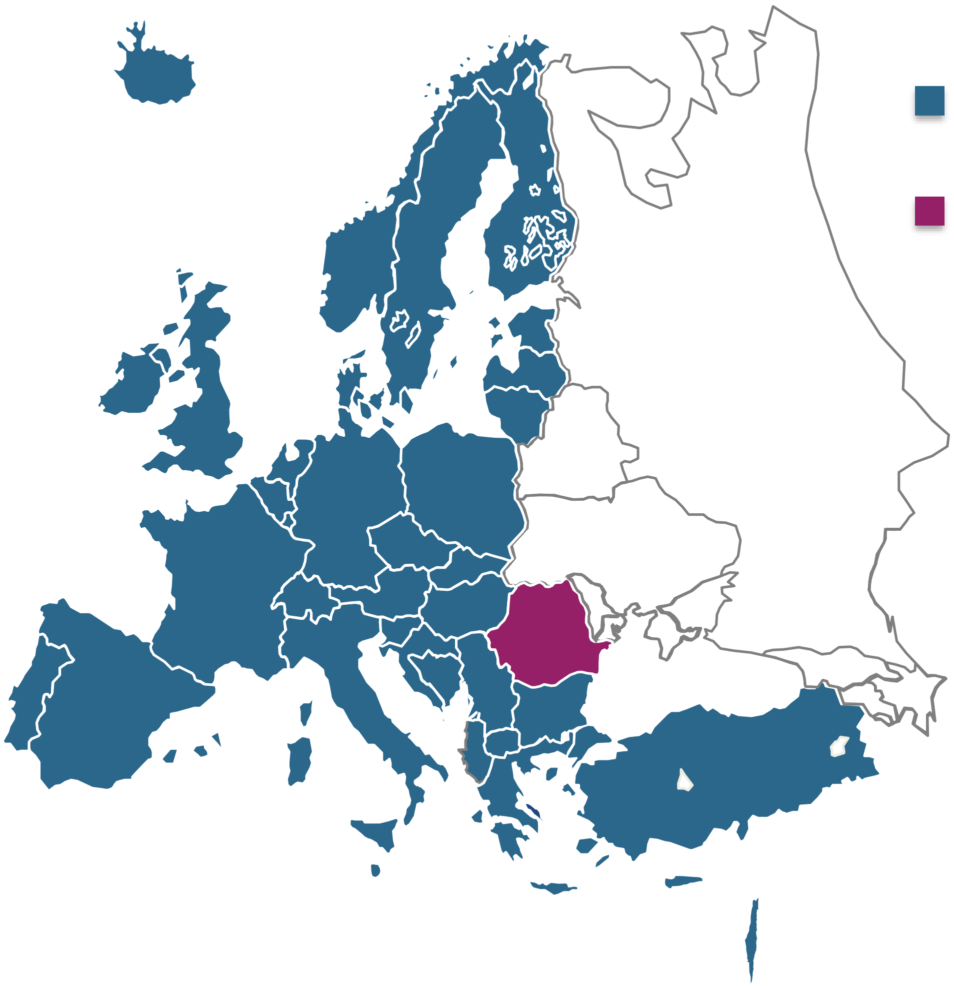
M3: WG3 – Establishment of a QC plan
Prepare recommendations for the establishment of Quality Control plan

M4: WG4 – Implementation in a Case Study
Prepare database from benchmarking

M5: WG5 – Drafting of guideline/recommendations
Prepare guideline/recommendations for the establishment of QC plan

5. REPRESENTATION

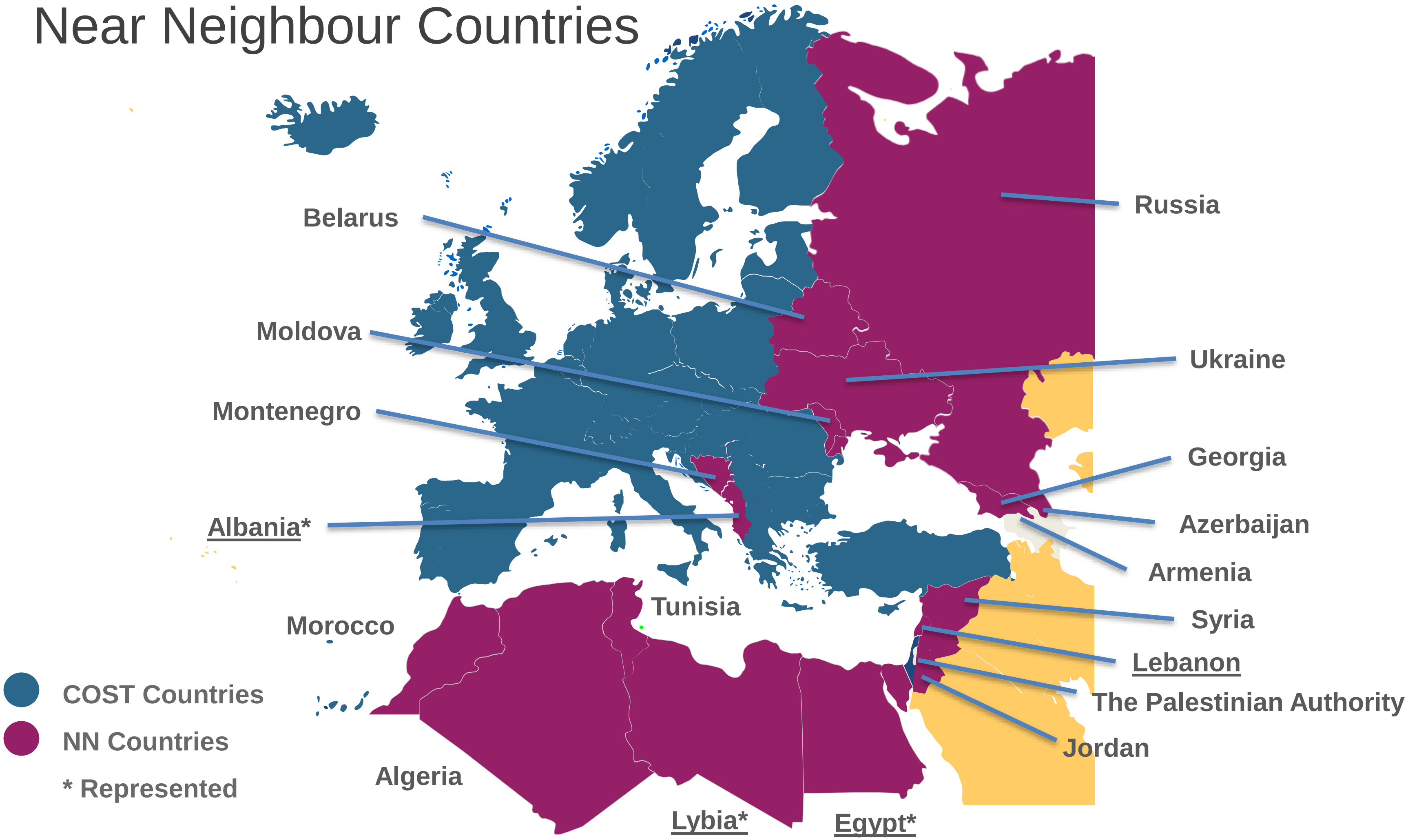
COST Countries



- **36 Action represented countries**
- **Missing Country**
  - Romania

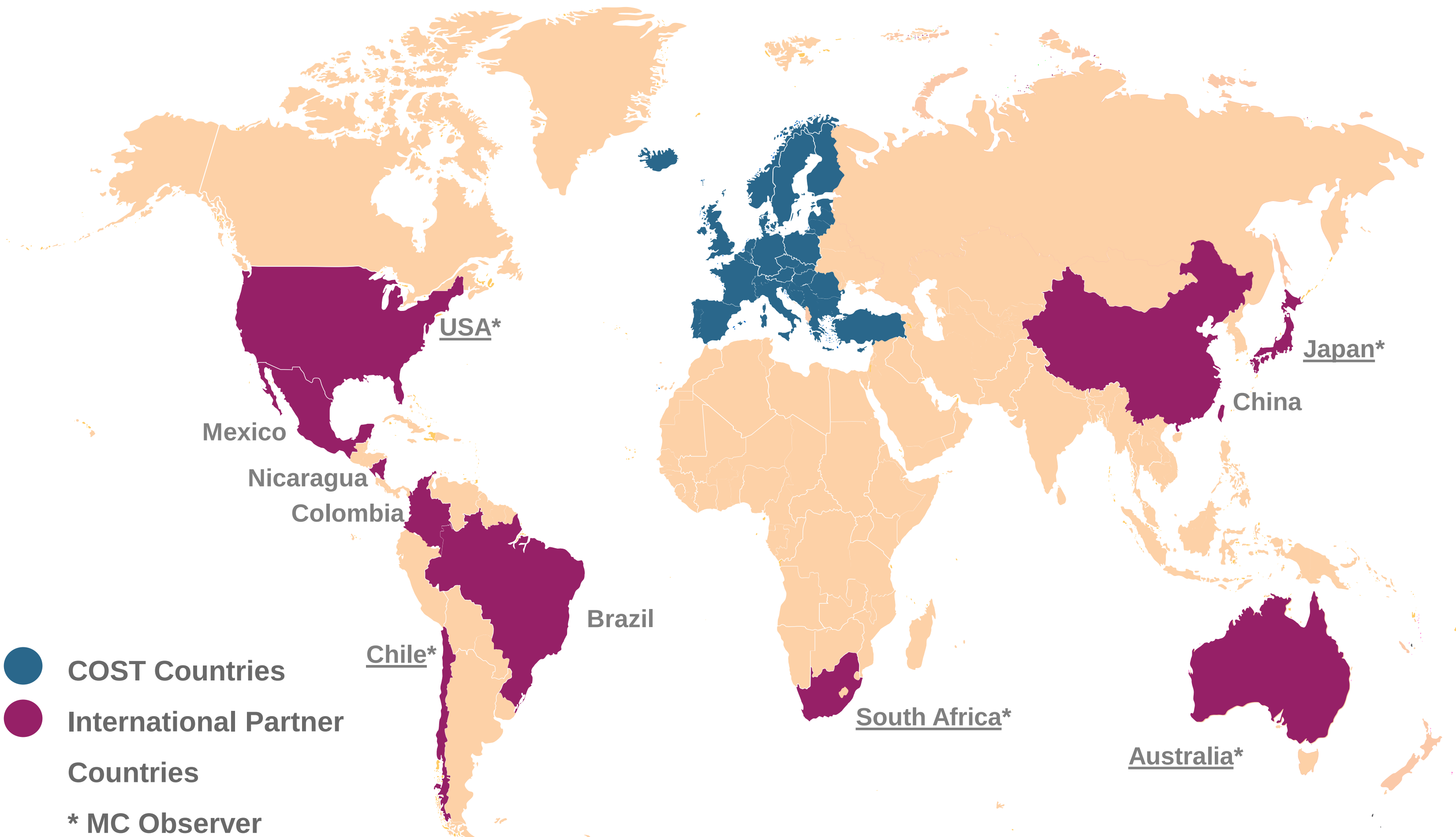
5. REPRESENTATION

Near Neighbour Countries

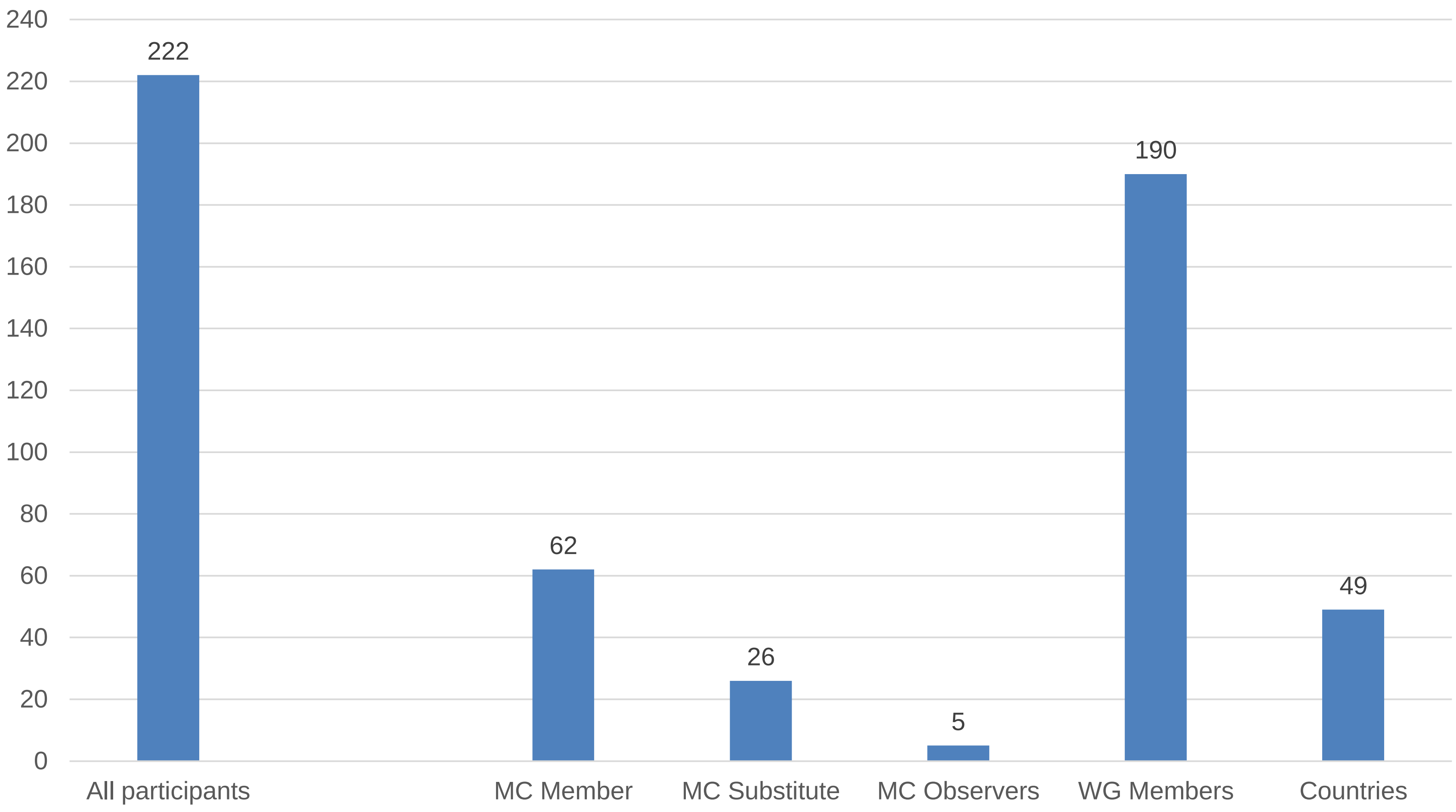


5. REPRESENTATION

International Partner Countries



6. MEMBERS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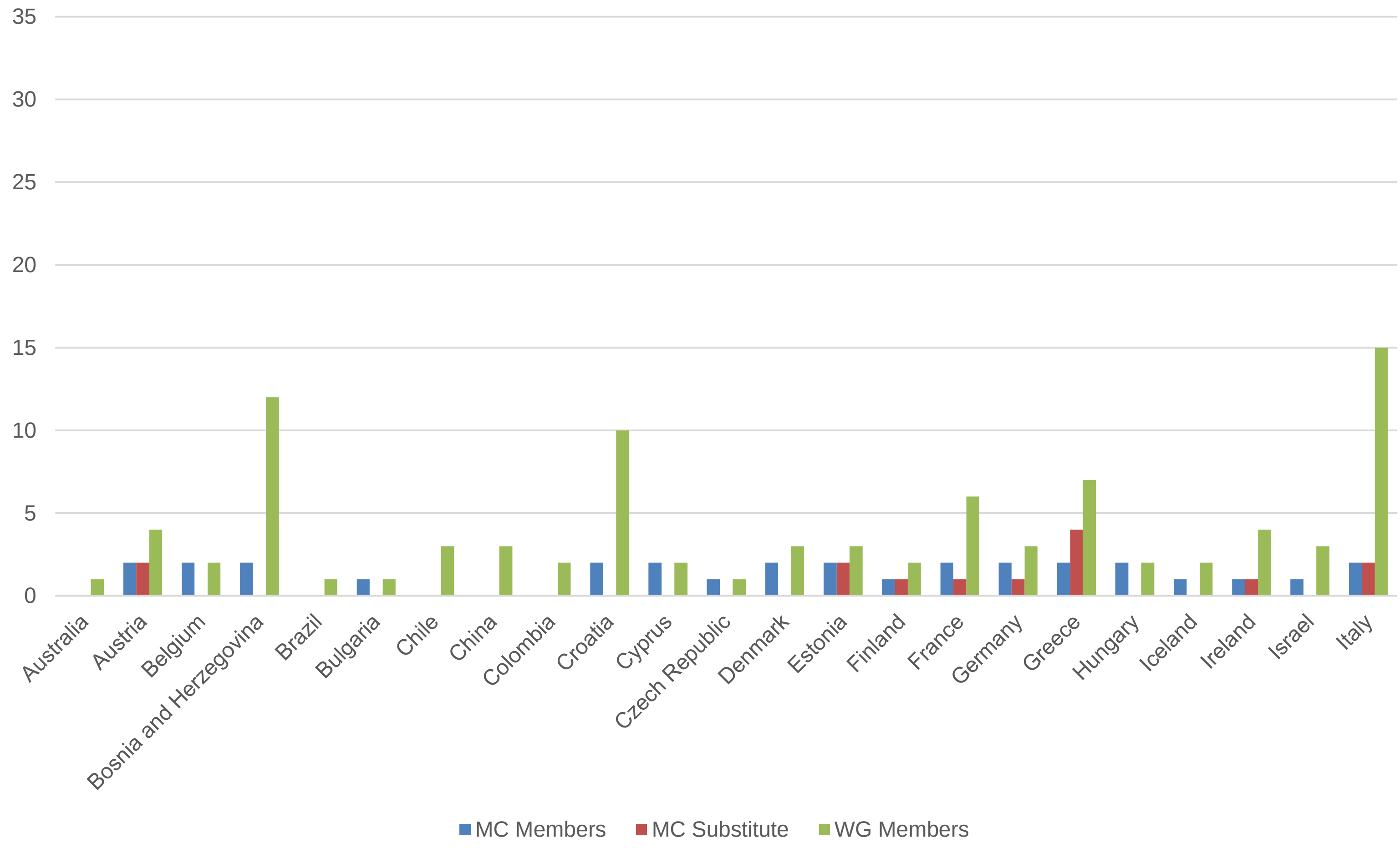
All participants
MC Member
MC Substitute
MC Observers
WG Members
Countries

6. MEMBERS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Brazil
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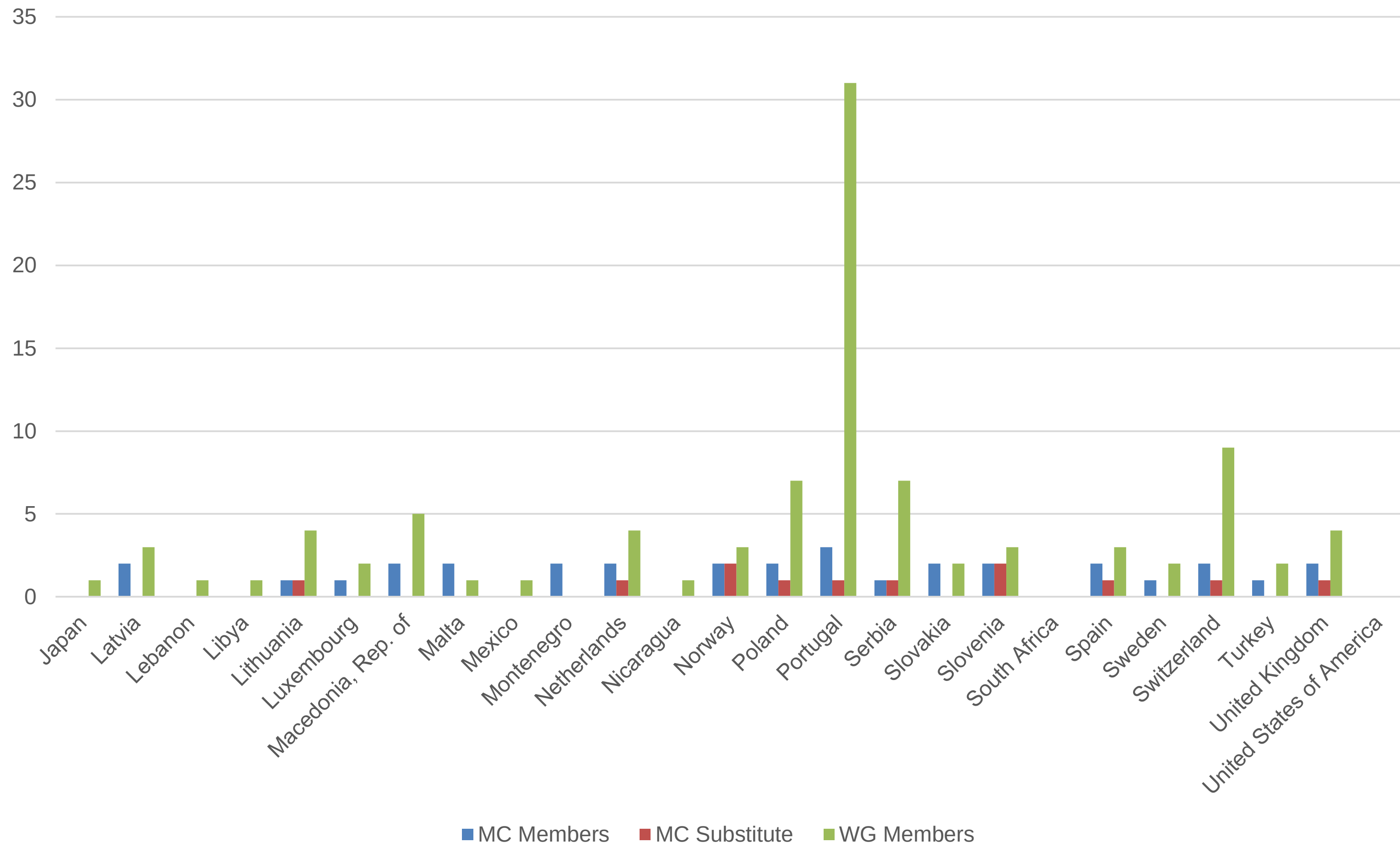
China
Colombia
Croatia
Cyprus
Czech Republic
Denmark
Estonia
Finland
France
Germany
Greece
Hungary
Iceland
Ireland
Israel
Italy
MC Members
MC Substitute
WG Members

6. MEMBERS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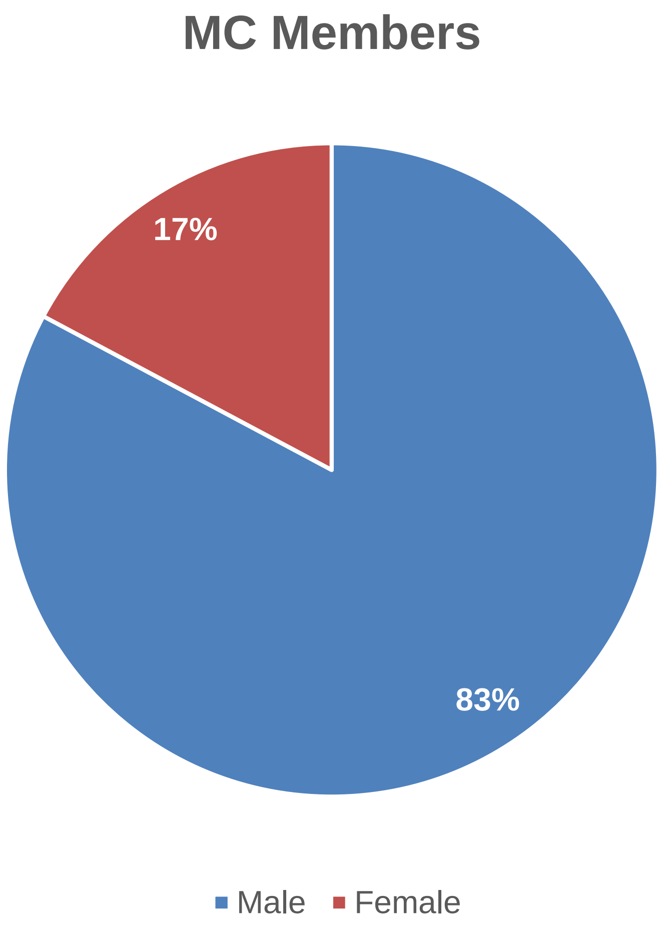
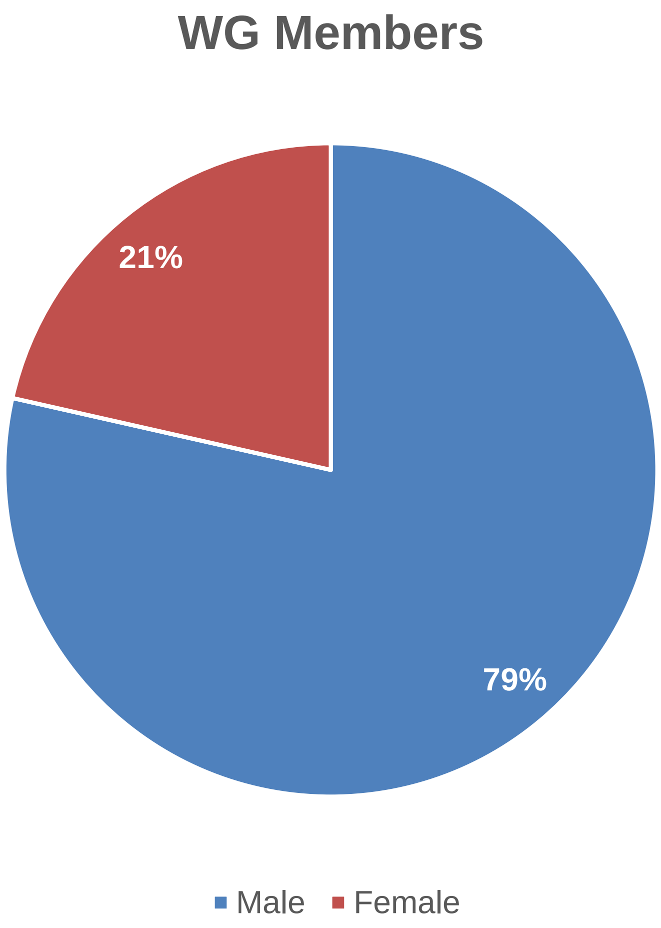
Luxembourg
Macedonia, Rep. of
Malta
Mexico
Montenegro
Netherlands
Nicaragua
Norway
Poland
Portugal
Serbia
Slovakia
Slovenia
South Africa
Spain
Sweden
Switzerland
Turkey
United Kingdom
United States of America
MC Members
MC Substitute
WG Members

6. MEMBERS





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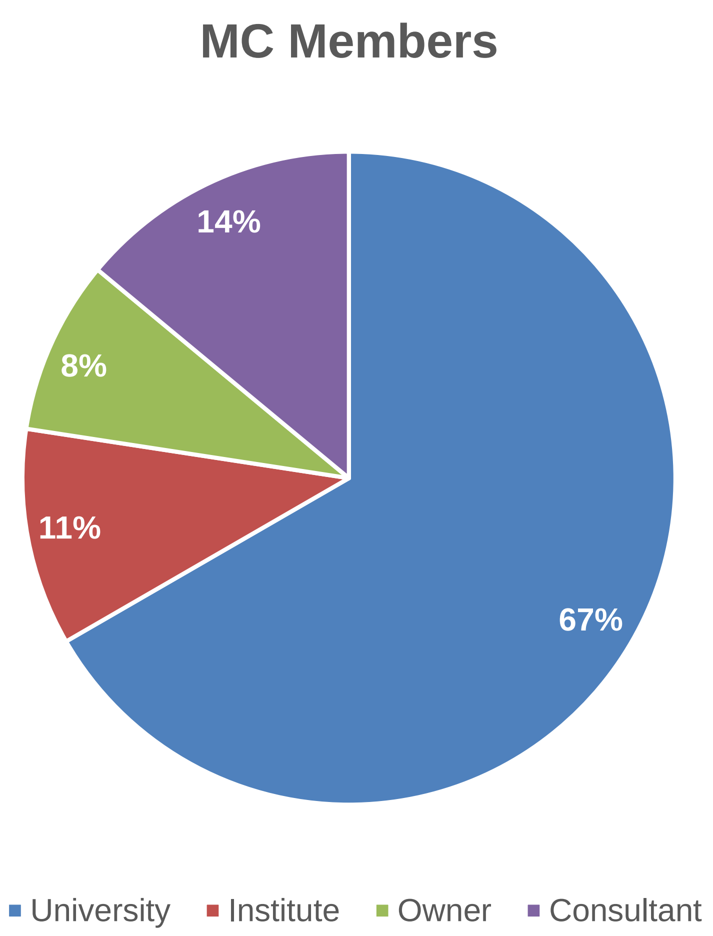
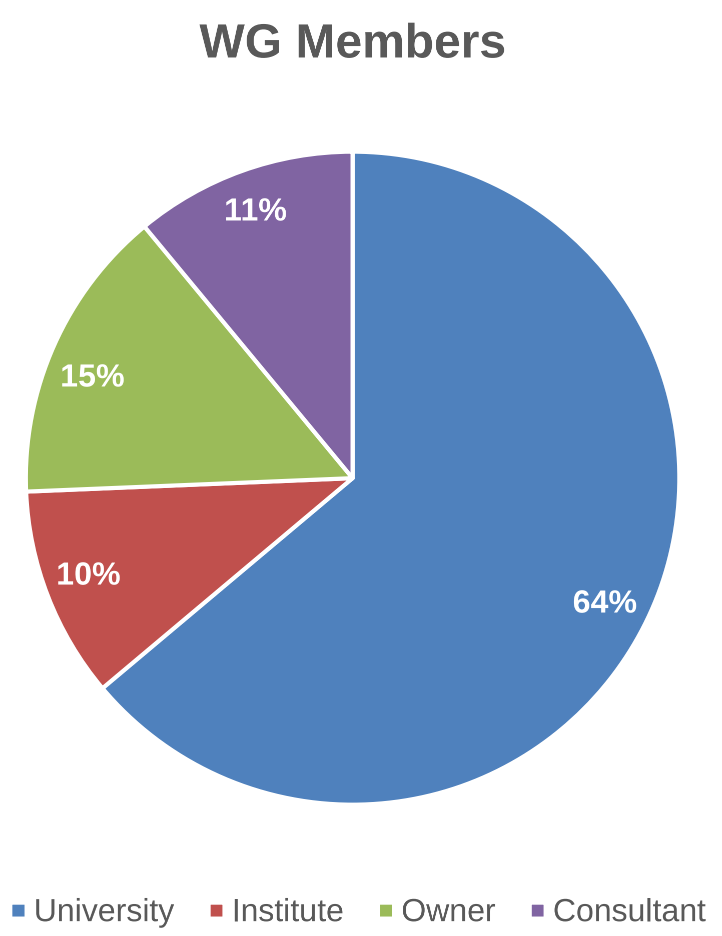

MC Members
14%
8%
11%
67%
University
Institute
Owner
Consultant


WG Members
11%
15%
10%
64%
University
Institute
Owner
Consultant

7. DISSEMINATION

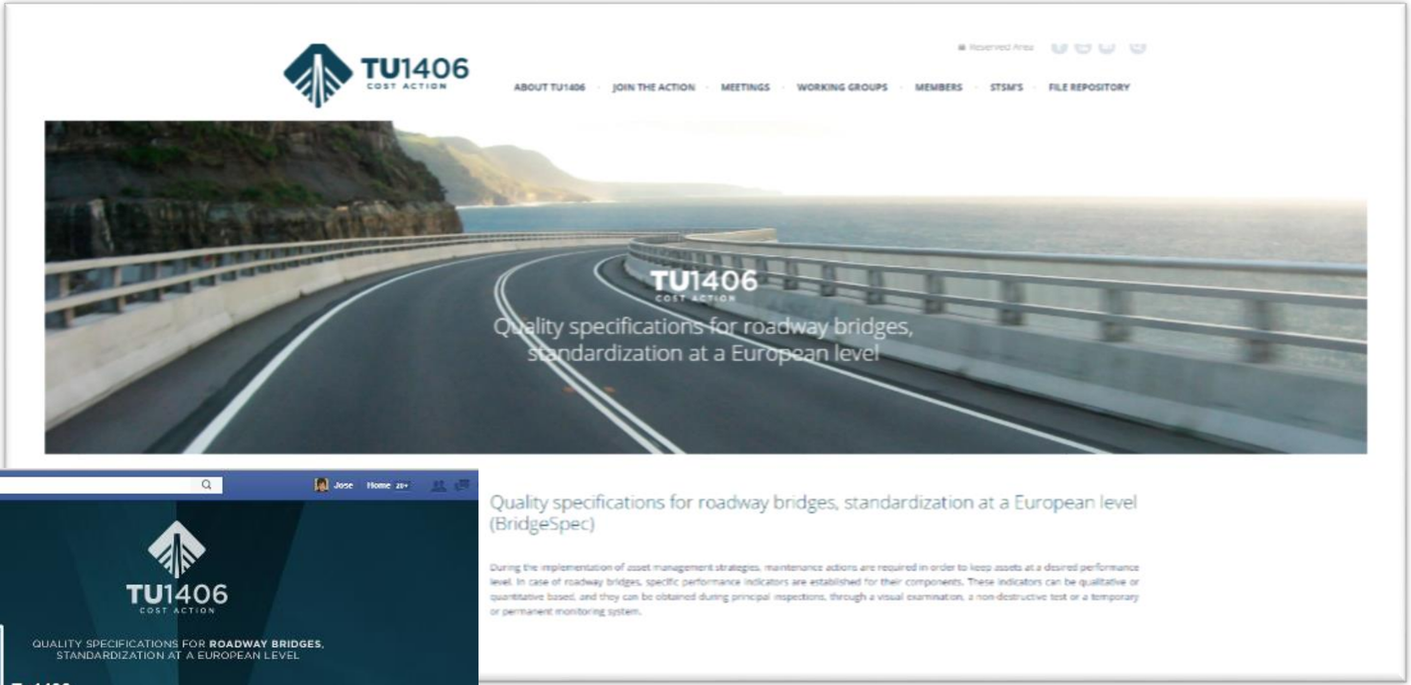
- Activities
  - Workshops/Conferences
  - STSM (especially promoted to early-stage researchers)
  - Dissemination meetings (e.g. conferences, etc.)
  - Training schools

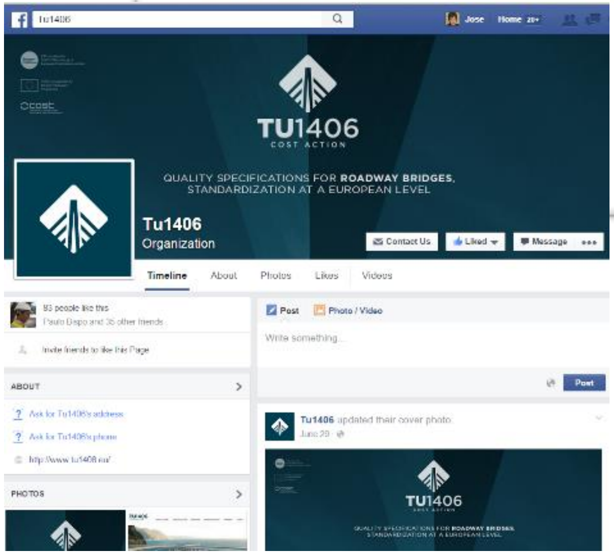
- Publications
  - Journal special issues
  - Peer-reviewed articles
  - Reports issued by the Action
  - E-book
  - Guideline and link to standardization

7. DISSEMINATION

Website

www.tu1406.eu





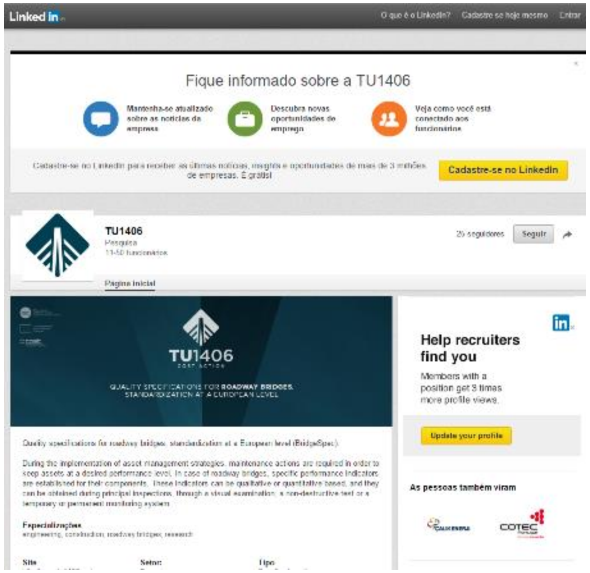
Facebook page

https://www.facebook.com/tu1406ca

7. DISSEMINATION

LinkedIn page

https://www.linkedin.com/company/tu1406

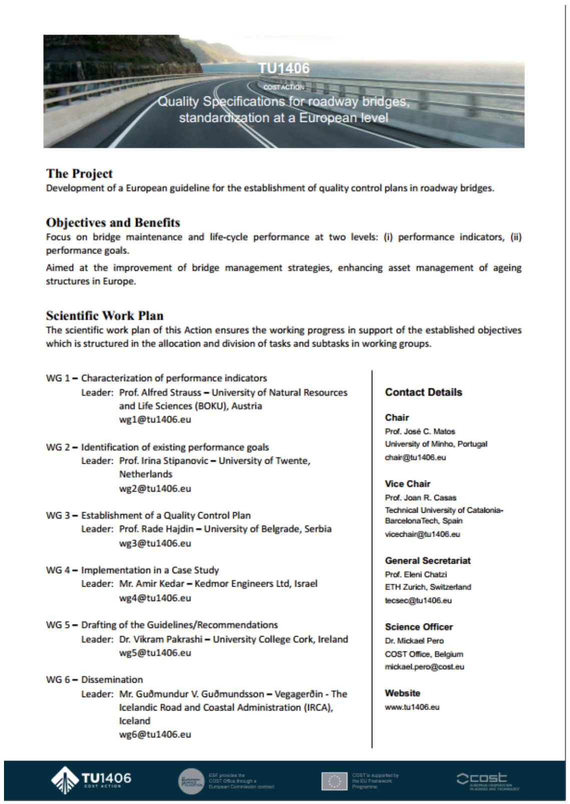


YouTube channel

https://www.youtube.com/channel/UCwxn7su9Bjm_D5xsuExRg8Q

7. DISSEMINATION

Poster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COST ACTION

Quality Specifications for roadway bridges, standardization at a European level

The Project

Development of a European guideline for the establishment of quality control plans in roadway bridges.

Objectives and Benefits

Focus on bridge maintenance and life-cycle performance at two levels: (i) performance indicators, (ii) performance goals.

Aimed at the improvement of bridge management strategies, enhancing asset management of ageing structures in Europe.

Scientific Work Plan

The scientific work plan of this Action ensures the working progress in support of the established objectives which is structured in the allocation and division of tasks and subtasks in working groups.

WG 1 – Characterization of performance indicators
Leader: Prof. Alfred Strauss – University of Natural Resources and Life Sciences (BOKU), Austria
wg1@tu1406.eu

WG 2 – Identification of existing performance goals
Leader: Prof. Irina Stipanovic – University of Twente, Netherlands
wg2@tu1406.eu

WG 3 – Establishment of a Quality Control Plan
Leader: Prof. Rade Hajdin – University of Belgrade, Serbia
wg3@tu1406.eu

WG 4 – Implementation in a Case Study
Leader: Mr. Amir Kedar – Kedmor Engineers Ltd, Israel
wg4@tu1406.eu

WG 5 – Drafting of the Guidelines/Recommendations
Leader: Dr. Vikram Pakrashi – University College Cork, Ireland
wg5@tu1406.eu

WG 6 – Dissemination
Leader: Mr. Guðmundur V. Guðmundsson – Vegagerðin - The Icelandic Road and Coastal Administration (IRCA), Iceland
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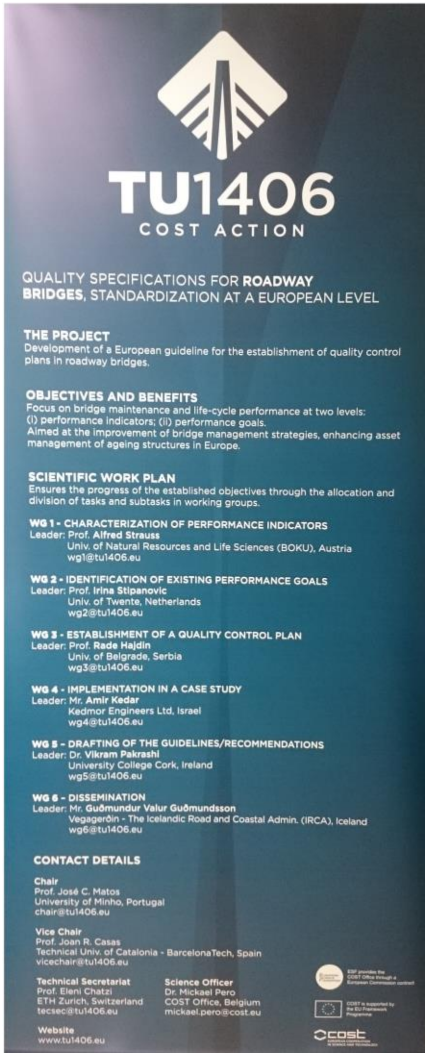
TU1406 COST ACTION

Leaflet/Brochure

Roll-up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COST ACTION

QUALITY SPECIFICATIONS FOR **ROADWAY BRIDGES**, STANDARDIZATION AT A EUROPEAN LEVEL

THE PROJECT

Development of a European guideline for the establishment of quality control plans in roadway bridges.

OBJECTIVES AND BENEFITS

Focus on bridge maintenance and life-cycle performance at two levels:
(i) performance indicators; (ii) performance goals.
Aimed at the improvement of bridge management strategies, enhancing asset management of ageing structures in Europe.

SCIENTIFIC WORK PLAN

Ensures the progress of the established objectives through the allocation and division of tasks and subtasks in working groups.

WG 1 - CHARACTERIZATION OF PERFORMANCE INDICATORS
Leader: Prof. **Alfred Strauss**
Univ. of Natural Resources and Life Sciences (BOKU), Austria
wg1@tu1406.eu

WG 2 - IDENTIFICATION OF EXISTING PERFORMANCE GOALS
Leader: Prof. **Irina Stipanovic**
Univ. of Twente, Netherlands
wg2@tu1406.eu

WG 3 - ESTABLISHMENT OF A QUALITY CONTROL PLAN
Leader: Prof. **Rade Hajdin**
Univ. of Belgrade, Serbia
wg3@tu1406.eu

WG 4 - IMPLEMENTATION IN A CASE STUDY
Leader: Mr. **Amir Kedar**
Kedmor Engineers Ltd, Israel
wg4@tu1406.eu

WG 5 - DRAFTING OF THE GUIDELINES/RECOMMENDATIONS
Leader: Dr. **Vikram Pakrashi**
University College Cork, Ireland
wg5@tu1406.eu

WG 6 - DISSEMINATION
Leader: Mr. **Guðmundur Valur Guðmundsson**
Vegagerðin - The Icelandic Road and Coastal Admin. (IRCA), Iceland
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Science Officer
Dr. Mickael Pero
COST Office, Belgium
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Website
www.tu1406.eu

8. FUTURE EVENTS

Title: **COST Action TU1406: Quality specifications for roadway bridges, standardization at a European level (BridgeSpec) – Performance indicators**

Date: **Wednesday, November 23rd, 2016**

Group: **Session 11 – Bridge Structures**

Note: **TU1406 Meeting will take place on Wednesday, November 23rd**

8. FUTURE EVENTS

Title:	**The Value of Structural Health Monitoring for the Reliable Bridge Management**		
Date:	**02-03 March 2017**		
Venue:	**Faculty of Civil Engineering, University of Zagreb**		
Organizers:	**COST 1402**	-	Quantifying the value of structural health monitoring
	COST 1406	-	Quality specifications for roadway bridges, standardization at EU level
	IABSE WC1	-	Structural performance, safety and analysis

8. FUTURE EVENTS

Topics: **Performance assessment of existing bridges for their reliable management** *(COST Action TU1406)*

Framework, strategies and tools towards the quantification of the value of structural health monitoring *(COST TU1402)*

Management and Performance-assessment of existing structures *(IABSE WC1)*

Call for short papers: **November 20th, 2016**

8. FUTURE EVENTS

Topic:	**Bridges**		
Venue:	**Eesti Näitused – Estonian Fairs Ltd**		
Call for abstracts:	**October 1st, 2016**	Scientific Committee notice:	**December 1st, 2016**
Paper submission:	**May 10th, 2017**		
Note:	**TU1406 Meeting will take place**		

8. FUTURE EVENTS

Title: **COST TU1406 – Quality Control for the efficient management of existing bridges**

Sponsoring Group: **WC4 + COST TU1406**

Symposium Theme: **Existing Structures into the Future**

Session Format: **Traditional paper + Discussion number of 90 minutes**

8. FUTURE EVENTS

Title: **BESTInfra – Building up Efficient and Sustainable Transport Infrastructure**

Date: **21-22 September 2017**

Venue: **Faculty of Civil Engineering, Czech Republic University**

Organizers: **Ministry of Transport**

COST 1406 – Quality specifications for roadway bridges, standardization at EU level

TA ČR – Technology Agency of the Czech Republic

8. FUTURE EVENTS

Ministry of Transport

TU1406 COST ACTION

TAČR Technology Agency of the Czech Republic

Abstract Submission Deadline: **November 30th, 2016**

Full Paper Submission Deadline: **February 28th, 2017**

Final Manuscript Submission Deadline: **May 21st, 2017**

Selected papers will be published in extended form in Scopus indexed journal Acta Polytechnica, Journal of Advanced Engineering published by the CTU in Prague.

8. FUTURE EVENTS

WORLD ROAD ASSOCIATION MONDIALE DE LA ROUTE AIPCR PIARC

Dirección de Vialidad Ministerio de Obras Públicas Gobierno de Chile

ACCT CHILE Asociación Chilena de Carreteras y Transporte

Title: **Design, construction and maintenance**

Organizers: **Chilean Association of Roads and Transport (ACCT & Chile PIARC National Committee)**

National Roads Administration of Chile

Sponsor: **COST TU1406**

Call for abstracts: **January 15th, 2017**

Note: **TU1406 Central and South American WG meeting will take place on October 17th, 2017**

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THANK YOU FOR YOUR ATTENTION!

WWW.TU1406.EU

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

TNO
Innovation for built environment

Agnieszka Bigaj-van Vliet - TNO, the Netherlands
Jos Wessels - TNO, the Netherlands

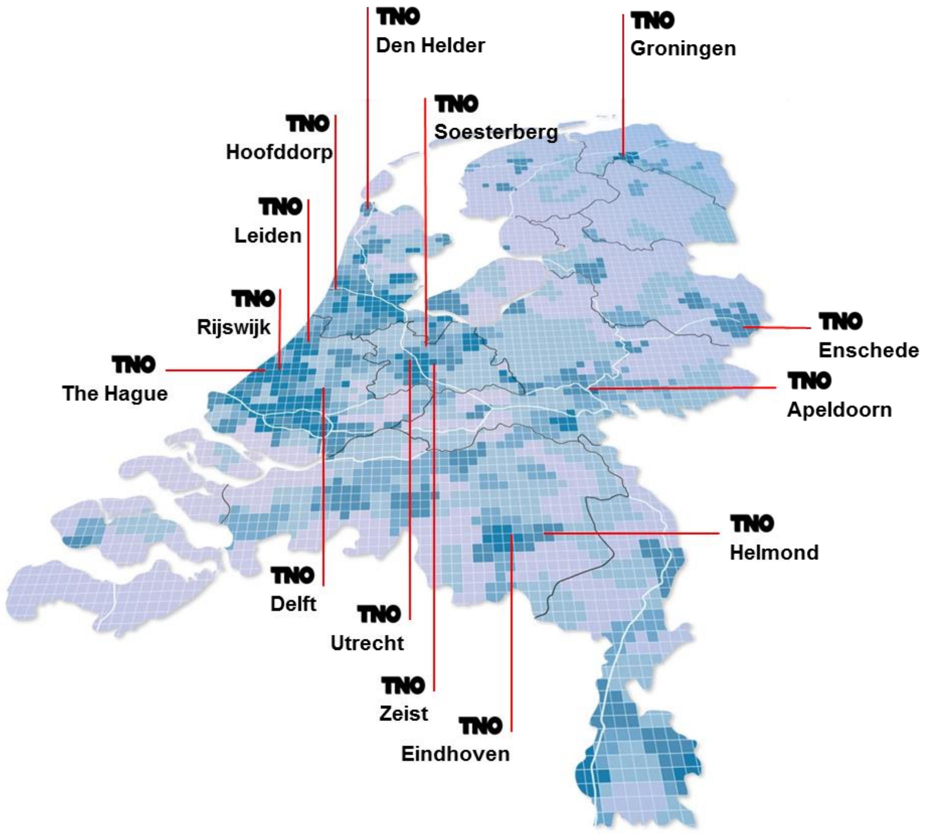
TNO innovation for life

20th 21st October 2016
Delft, Netherlands

TNO Dutch Organisation for Applied Scientific Rersearch

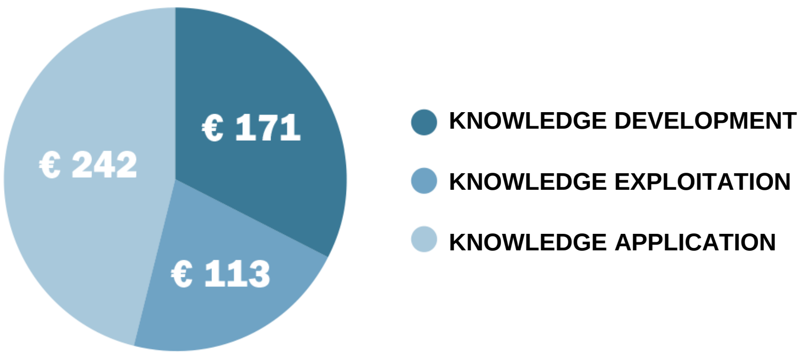
Status

- Founded by law in 1932
- Independent organisation regulated by public law: not part of any government, university or company
- Transitioned from largely government funded research institute to a modern Research & Technology Organisation (RTO) with hybrid funding

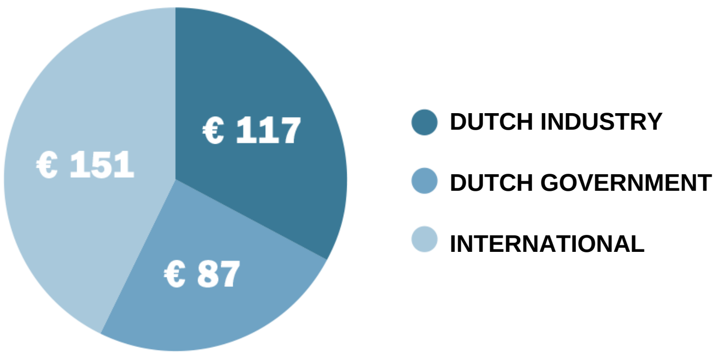


TNO in the Nerherlands

CONSOLIDATED TURNOVER 2014 (€ 526 million)



CONSOLIDATED MARKET TURNOVER 2014 (€ 355 million)



TNO research approach

DEVELOPMENT OF FUNDAMENTAL KNOWLEDGE

Cooperation with universities & knowledge institutes

INTEGRATION OF KNOWLEDGE

Public-private partnership, cooperation with industry, participation in EU research, etc.

APPLICATION OF KNOWLEDGE

Contract research for and with the customers, consultancy

EXPLOITATION OF KNOWLEDGE

Implementation in cooperation with the industry, testing & certification, licensing, TNO BVs

TNO scope and expertise

Focus areas

- Urbanisation
  - **Buildings & Infrastructures**
  - Mobility & Logistics
  - Environment & Sustainability
  - Smart Cities
- Industrial innovation
- Healthy living
- Defense, safety and security
- Energy

TNO Buildings & Infrastructures

Research focus

- Asset management for availability, reliability & sustainable growth
- Durable safety and service life extension of infrastructure
- Optimization of large scale systems & processes
- Energy and serviceability in built environment
- Future-proof infrastructure by knowledge exchange

Expertise in structural reliability

- Risk and reliability analysis
- Performance analysis, design and assessment of structures (concrete, steel, masonry, FRP etc.)
- Evaluation of damage, durability modelling and design of repair systems for structures
- Multi-scale numerical modeling of structures
- Development, optimisation and use of (novel) building materials for roads and (infra) structures
- Inspection and monitoring incl. SHM

TU1406

COST ACTION

TU1406
COST ACTION

WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Advancement of *fib* Model Code for structural concrete to incorporate assessment of existing structures

Agnieszka Bigaj-van Vliet - TNO, the Netherlands
Deputy-chair fib T10.1 Model Code 2020

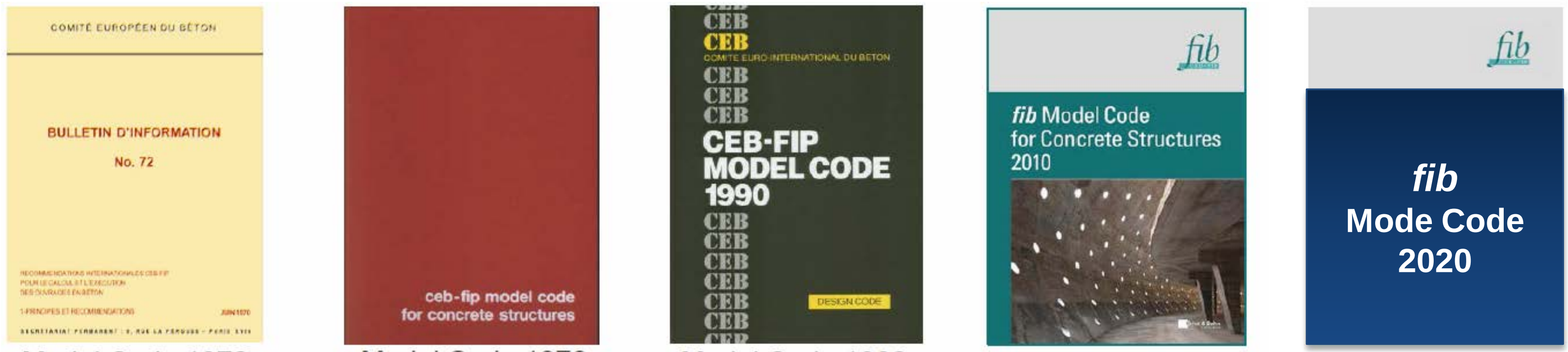
TNO innovation for life

20th – 21st October 2016
Delft, Netherlands

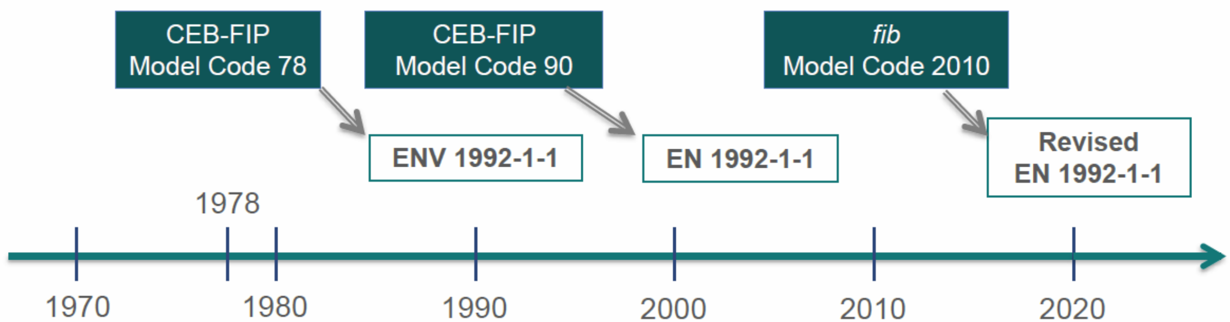
CONTENTS

- Achievements & experience with MC2010
- Need for Mode Code advancement
- Proposed updates and extensions for MC2020
    - General principles
    - Scope extension
    - KPI vs Mode Code approach
- Organization framework & timeline for MC2020

Evolution of Model Codes for structural concrete



Model Code 1970 | Model Code 1978 | Model Code 1990 | Model Code 2010

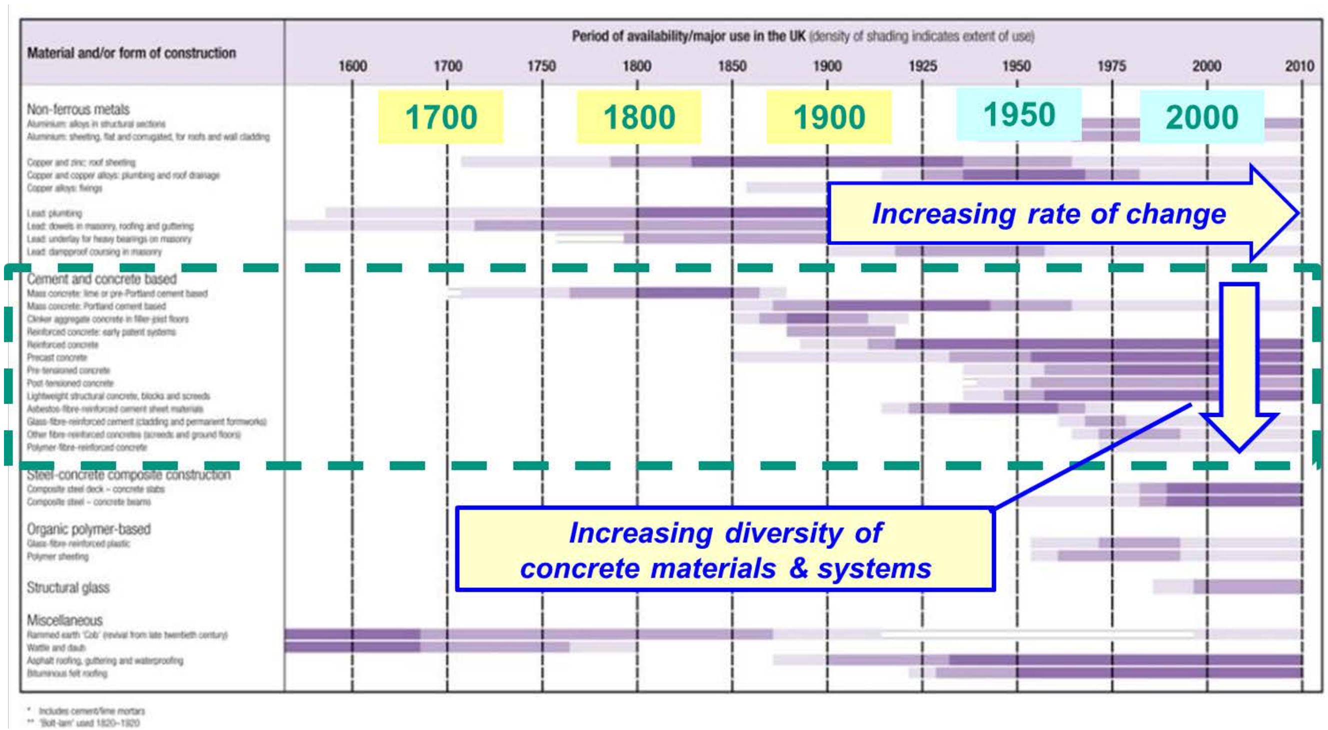


Looking back at *fib* MC2010

11th fib General Assembly voting on Model Code 2010, Lausanne, 29 October 2011

Looking back at *fib* MC2010

- Implications of improved materials and forms of construction



Looking back at *fib* MC2010

- Implications of improved materials and forms of construction
  - High performance materials offer amazing chances, but need new standards
  - Concrete strength until C120 in daily use
  - Defined performance design asks for uncoupling concrete strength and concrete properties

Test of prestressed slab, USA, 1954

Test of UHPC slab, The Netherlands, 2015

Looking back at *fib* MC2010

- Implication of dealing with old types of concrete and old forms of construction
  - Knowledge of old materials is decreasing
  - Insight in deterioration processes is required
  - Upgrading, repairing, refurbishment, retrofitting & strengthening are becoming daily business
  - Understanding of interaction between old and new materials is incomplete
  - Reliable determination of structural performance of existing structures is needed

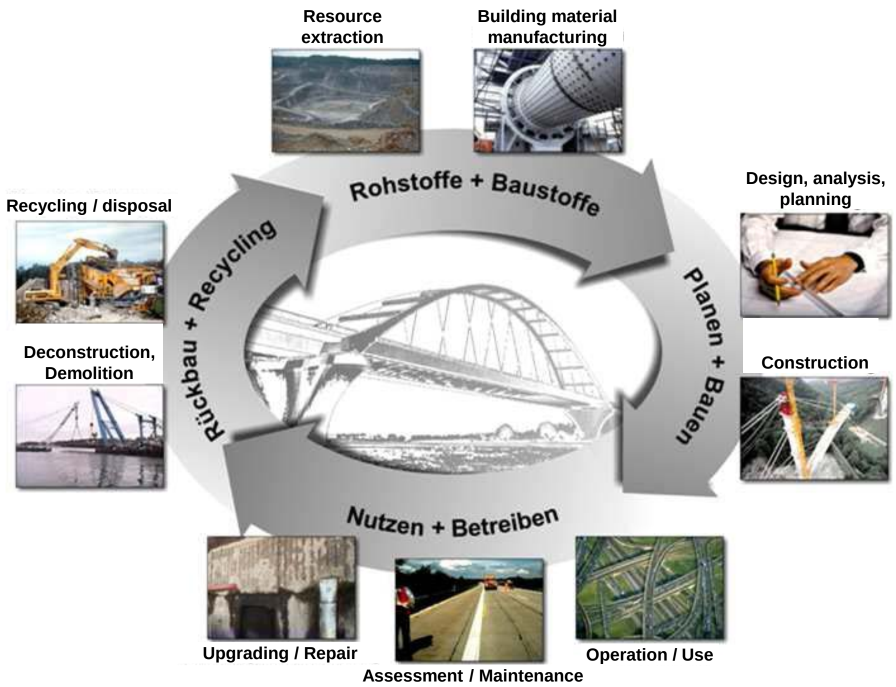
Impact damage

Freeze-thaw damage

Fire damage

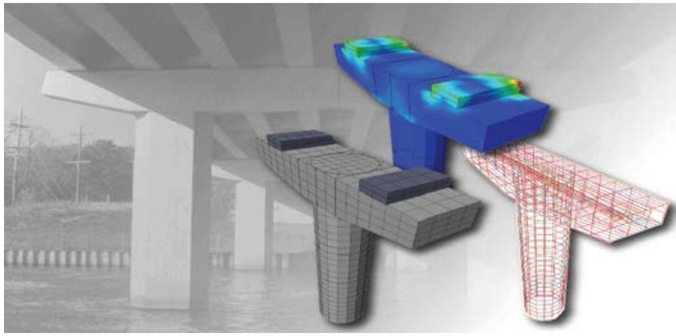
Looking back at *fib* MC2010

- Recognition of User Needs & Life-Cycle Perspective
  - New world-wide challenge
    - Dealing with increasing need for mobility
    - Dealing with aging of society
    - Dealing with limited resources
    - Dealing with the CO2 emission problem
- Developing integral design concepts including sustainability by smart (conceptual) design



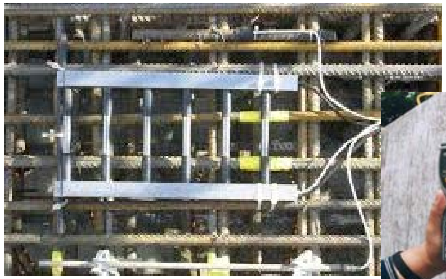
Looking back at *fib* MC2010

- Enabling use and benefiting from new technologies:
  - FEM: powerful computers and numerical programs
    - For structures with special characteristics (size, complications, advanced shapes, unknown boundary conditions) reliable numerical analyses can provide valuable information on structural behavior, safety and remaining service life



Bridge support NL-FEM analysis (DIANA)

  - SHM: monitoring with advanced sensor techniques
    - Structural Health Monitoring enables to assess reliable data on the real conditions and behavior of structures, and may enable reduction of uncertainties in the assessment and prediction of the performance of the structures



Corrosion measuring & monitoring systems

General principles of *fib* MC2010

- Design of concrete structures should be aimed at creating a **reliable** structure with required **durability**, i.e. structure that meets specified demands for **safety** and **serviceability** for a defined number of years in a **sustainable** way

  - Even though concrete structures have a limited service life, it can considerably be extended by:
    - good (conceptual) design
    - good (conceptual) re-design based on objective assessment
    - adequate threw-life management

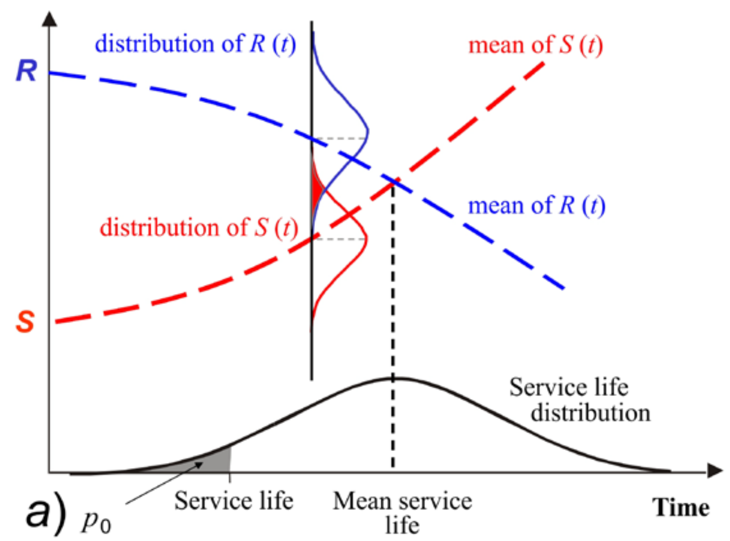
1930

Van Nelle factory
Rotterdam, The Netherlands

2016

General principles of *fib* MC2010

- Safety, serviceability, durability and sustaianbility as design criteria
  - **Performance-based approach**
    - In the context of the performance-based design of structures, reliability refers to the ability of a structure or structural member to fulfil the performance requirements during the service life for which it has been designed at a required probability level.
  - **Limit state concept**
    - To represent the transition between the desired state and the adverse state the concept of limit state is adopted:
      - reliability differentiation for new and existing structures
      - application of reliability concepts for/in service life design
      - introduction of reliability concepts for/in numerical analysis
  - **Reliability-based safety philosophy**
    - Reliability concepts shall cover at the same time **new and existing structures**

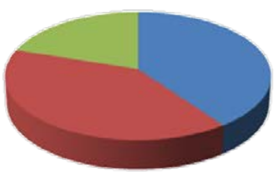


General principles of *fib* MC2010

- Reliability based safety philosophy

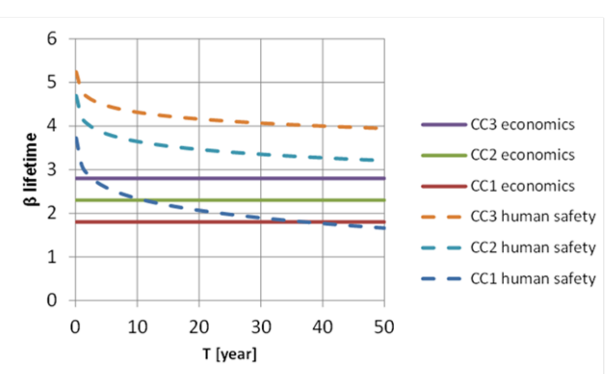
Fundamental parameters for definition of target reliability levels

Construction costs



Corresponding risk level

Failure probability **x** Failure costs
(Direct and Indirect)



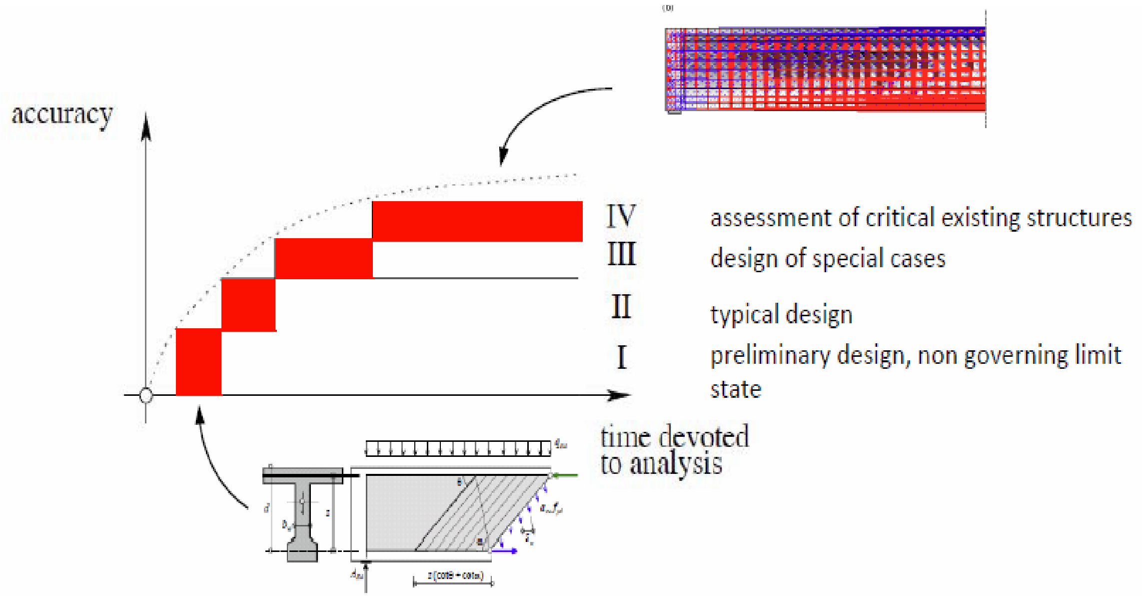
Buildings – floor area A > 500 m²

General principles of *fib* MC2010

- Performance-based design and assessment
  - **Verification**
    - Procedures should demonstrate that no violation of the limit state criteria takes place during the entire (residual) service life.
    - Verification shall be done by scientifically based **descriptional models**, which are provided at different **level of approximations,** and considering **leven of knowledge.**
  - **Descriptional models**
    - Models for describing the behavior and the structural resistance shall be derived on physical basis. Empiricism has been largely removed, which is in favor of a clear definition of the limits of applicability.
    - The models should, however, not only include the relevant parameters, but reflect the stochastic character of proceses described.
  - **Level of Approximation approach**
    - The level op Approximation approach is considered to be a major step ahead:
    - A basic principle of this approach is that the highest LoA models represent the performance with the highest accuracy and the lower LoA models are consequent simplifications of this representation.
  - **Level of Knowledge**
    - Due considertaion of the level of knowledge and consistent tratment of parameter and model associated is needed.

General principles of *fib* MC2010

- Level of Approximation approach



Levels of approximation

IV System assessment of critical existing structures & design of special cases e.g. by FEM
III In-depth elemental evaluation of existing structures & design of special cases
II Typical elemental design & assessment
I Preliminary design & assessment, non governing limit state

Safety formats in safety evaluation: Levels I to IV

I Partial factor methods
II Updating of info / reduction in uncertainty
III Full probabilistic approach
IV Full probabilistic with cost (& sustainability) optimisation

General principles of *fib* MC2010

- Approach to service life design used in fib Model Code

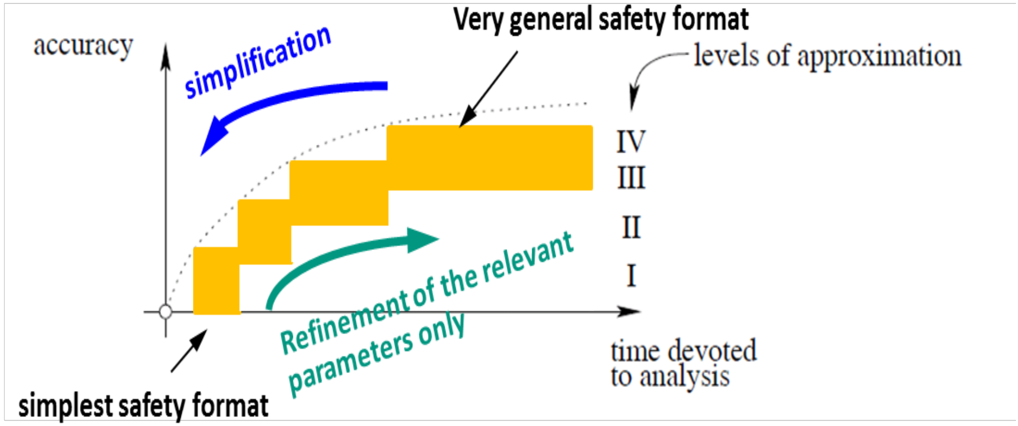
Safety formats in durability evaluation: Levels I to IV

I Avoidance of deterioration
Low likelihood of deterioration for particular limiting circumstances / concrete recipes, but not fool proof

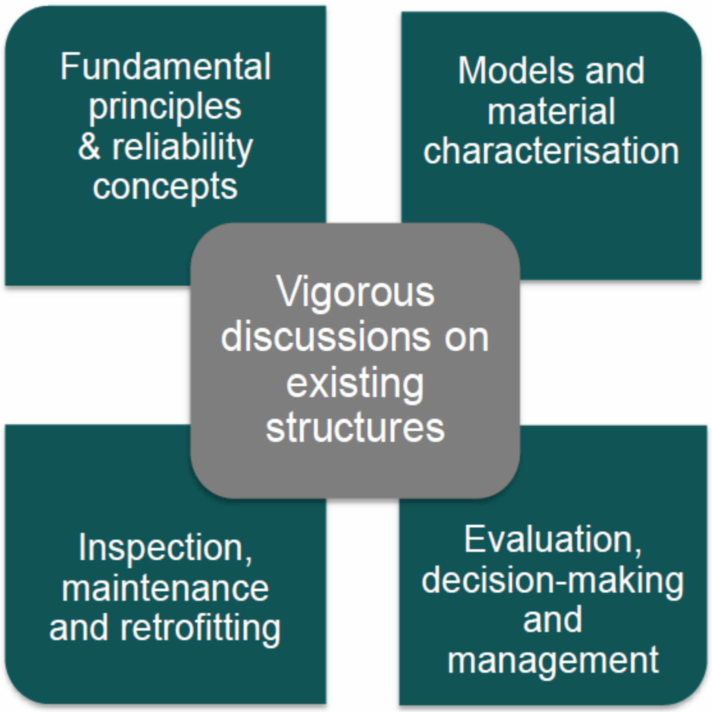
II Deemed-to-satisfy design
Tabulated approach most widely used at present

III Partial factor design
If calibrated, appropriate to standard structures

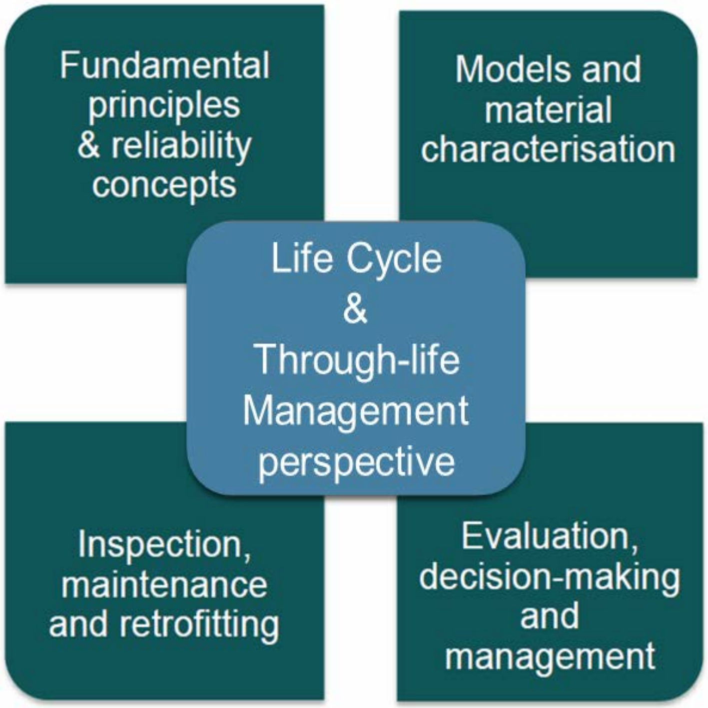
IV Full probabilistic design
Only realistic for special / monumental structures



Looking forward at *fib* MC2020

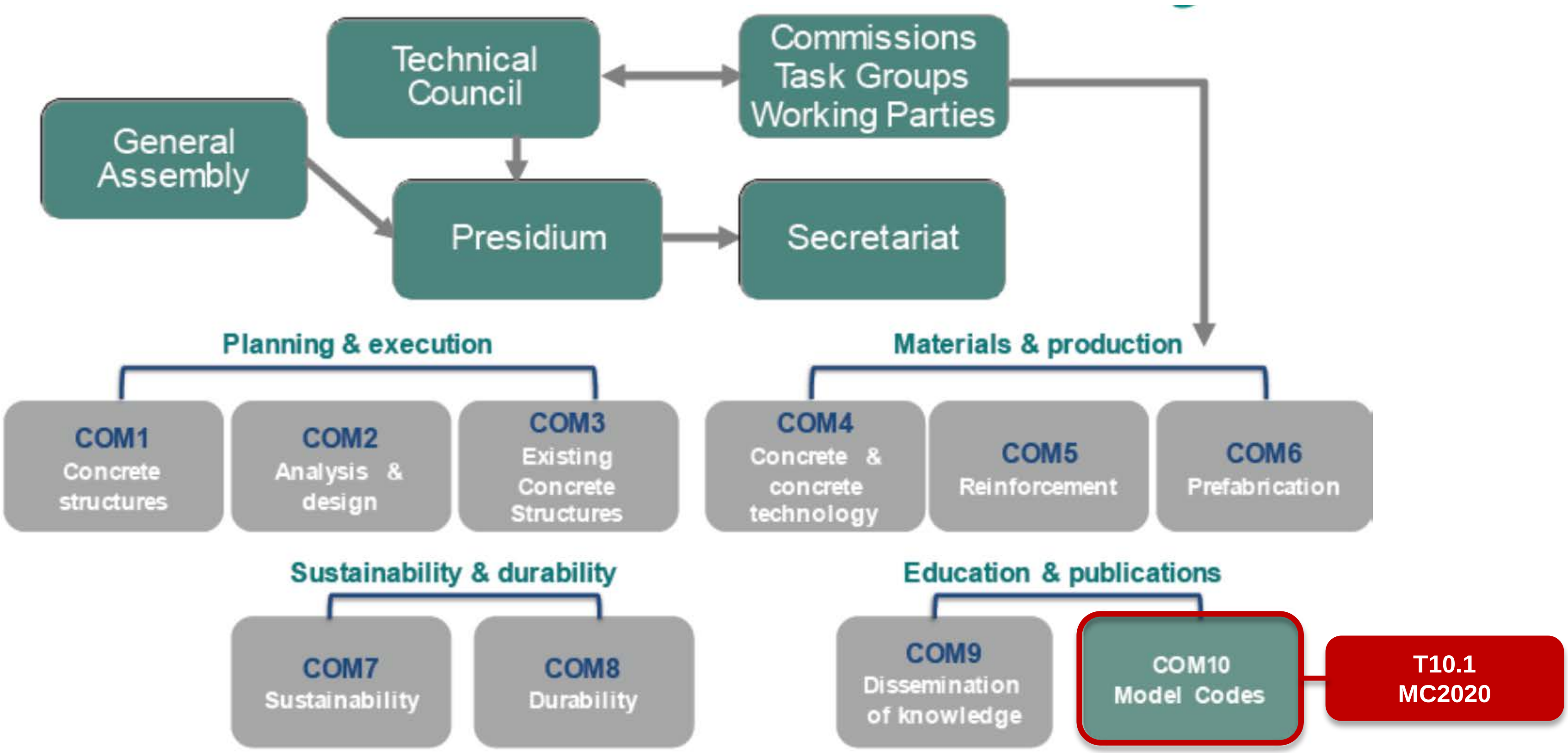


MC2010



MC2020

Looking forward at *fib* MC2020



General principles of *fib* MC2020

- **Single merged structural code for both new and existing concrete structures**
  - Integrated life cycle perspective with particular attention to through-life management,
  - Fundamental principles and a safety philosophy based on reliability concepts,
  - Implementation of performance based concept,
  - Holistic treatment of structural safety, serviceability, durability and sustainability,
  - Consistent approach to robustness and redundancy/resilance
  - Provisions based on generalized models and level of approximation approach,
  - Removing constraints for novel types of concrete and reinforcing materials and novel types of construction forms,
  - Considering material degradation and accounting for progressive damage
  - Full advantage of information acquired by testing and monitoring.

  - Strong international (world) perspective.

General principles of *fib* MC2020

- **Single merged structural code for both new and existing concrete structures**
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  - Full advantage of information acquired by testing and monitoring.

  - Strong international (world) perspective.

General principles of *fib* MC2020

- Safety philosophy for (existing) structures
  - Reliability concepts shall be extended and updated to cover at the same time new and existing structures considering that:
    - higher target reliability levels implies a larger cost increment in existing structures compared to the new ones
    - remaining service life is smaller for existing structures compared to design working life of new structures
    - updated information on actual resistance of an existing structure can be available

The target reliability values (β) may be reduced in existing structures compared to the new ones

General principles of *fib* MC2020

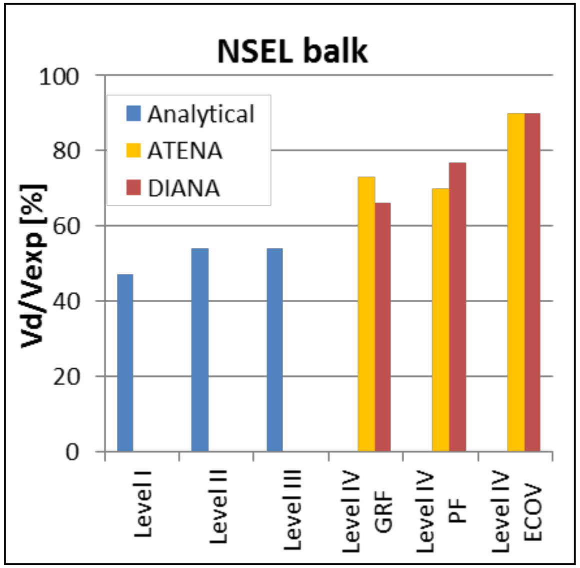
- Design strategies for robustness of (existing) structures

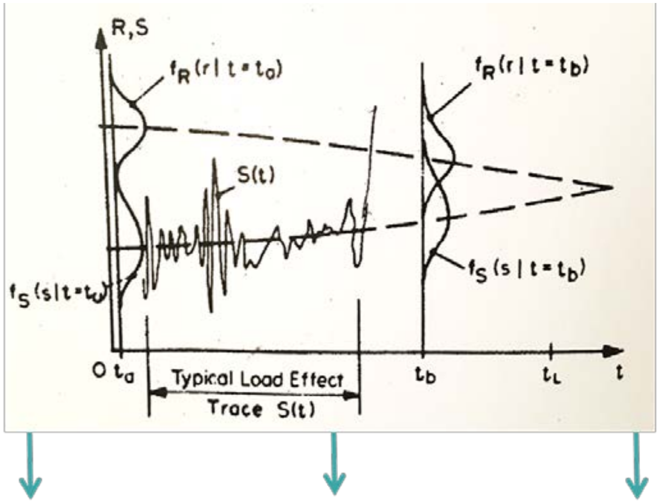
ACCIDENTAL DESIGN SITUATIONS

- STRATEGIES BASED ON IDENTIFIED ACCIDENTAL ACTIONS (e.g. explosions and impact)
  - DESIGN THE STRUCTURE TO HAVE SUFFICIENT MINIMUM ROBUSTNESS
  - PREVENTING OR REDUCING THE ACTION (e.g. protective measures)
  - DESIGN THE STRUCTURE TO SUSTAIN THE ACTION
- STRATEGIES BASED ON LIMITING THE EXTENT OF LOCALISED FAILURE
  - ENHANCED REDUNDANCY (e.g. alternative load paths)
  - PRESCRIPTIVE RULES (e.g. integrity and ductility)

General principles of *fib* MC2020

- Time–dependent reliability analysis by modern analysis tools (FEM)
  - Computation approaches shall enable accounting for progressive deterioration process
  - Computation approaches could be pursued for the reliability analysis of structures showing implicit limit state functions

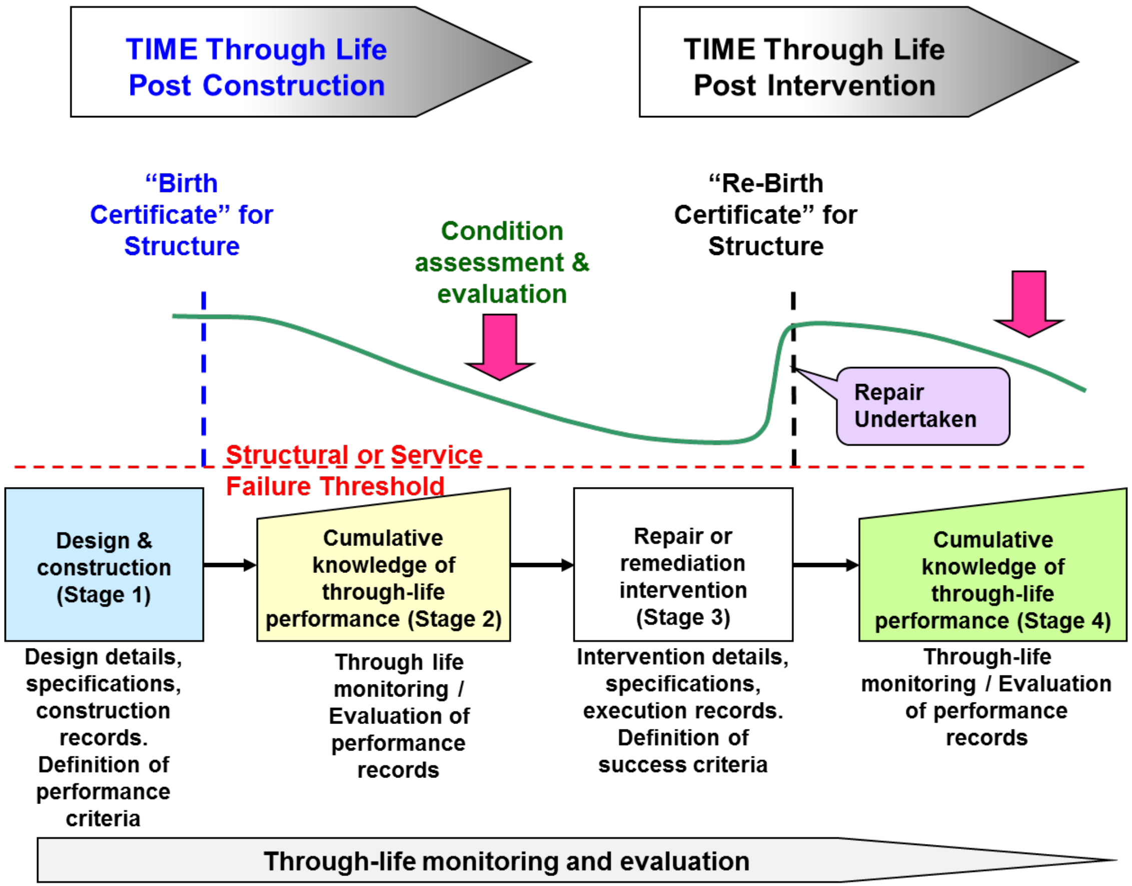




$R(t)$ $\qquad$ $R(t) and\ S(t)$ $\qquad$ $S(t)$

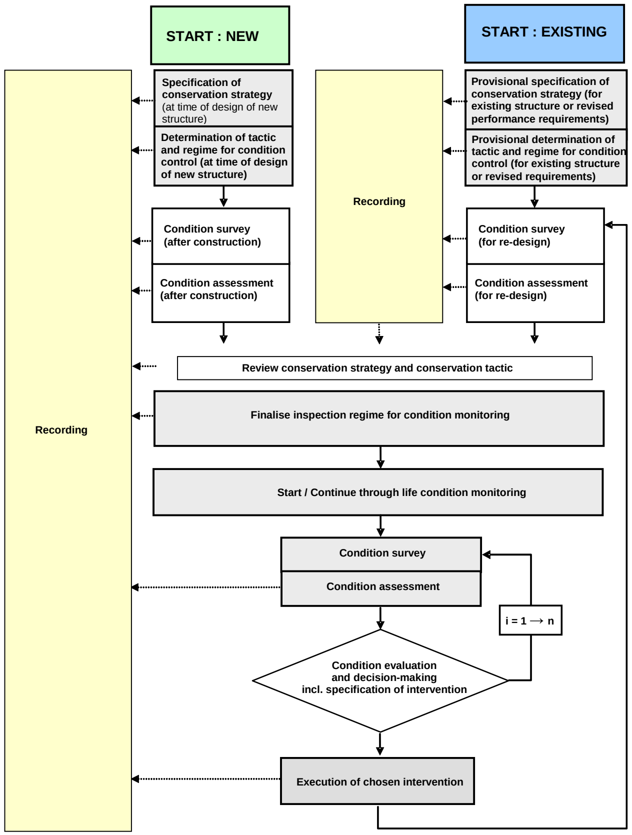
General principles of *fib* MC2020

- Through-life management of (existing) structures



General principles of *fib* MC2010

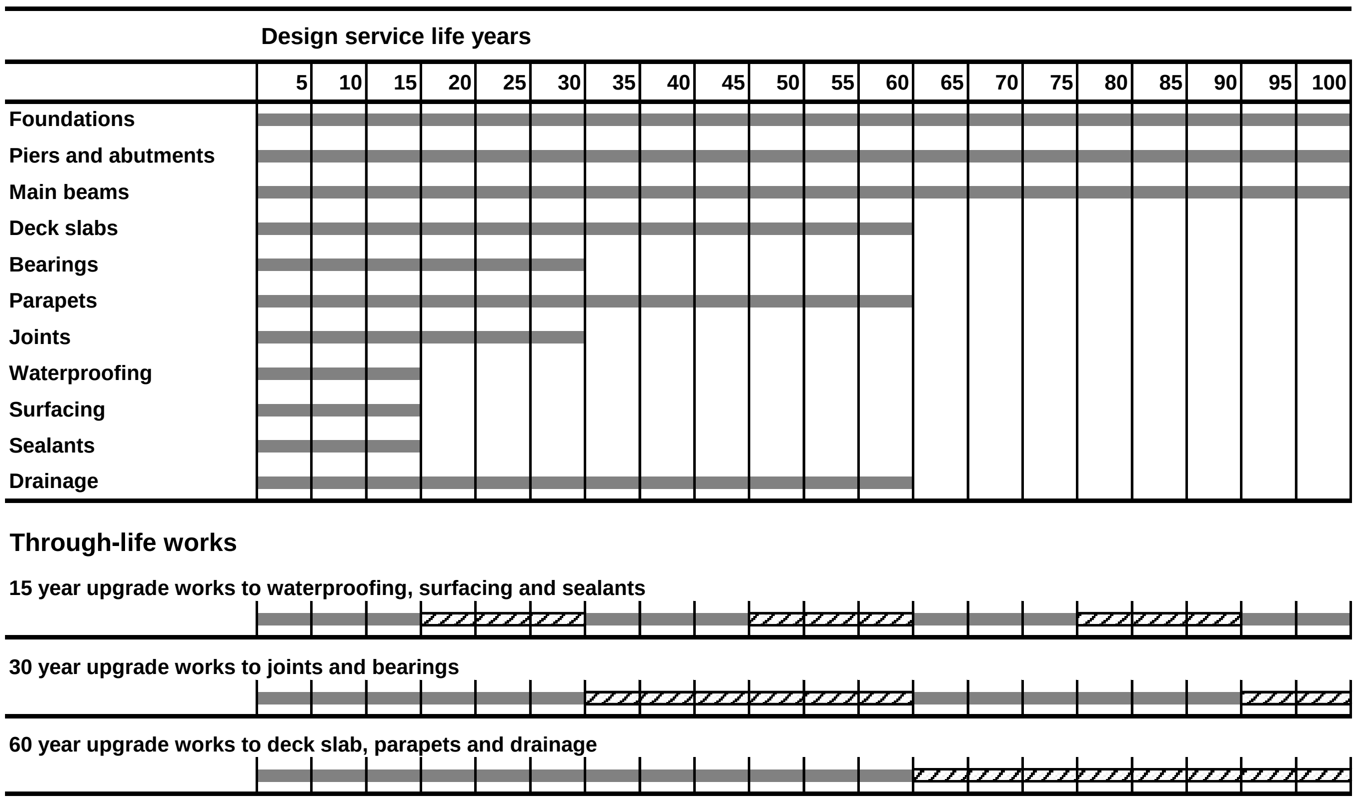
- Through-life management of (existing) structures
  - **Conservation strategies**
    - Strategy A: **Structures which are to be managed by planned condition control activities**.
      - suitable for structures where deterioration would be technically unacceptable or must not be seen e.g. monumental, important or sensitive buildings & structures.
    - Strategy B: **Structures or parts thereof which are managed by reactive activities.**
      - suitable for structures where remedial measures can be taken after deterioration becomes visible e.g. buildings and other common structures.
    - Strategy C: **Structures or parts thereof for which condition control is not practical.**
      - suitable for structures where it would be difficult economically and / or technically for preventative or remedial measures to be taken, such as foundations.



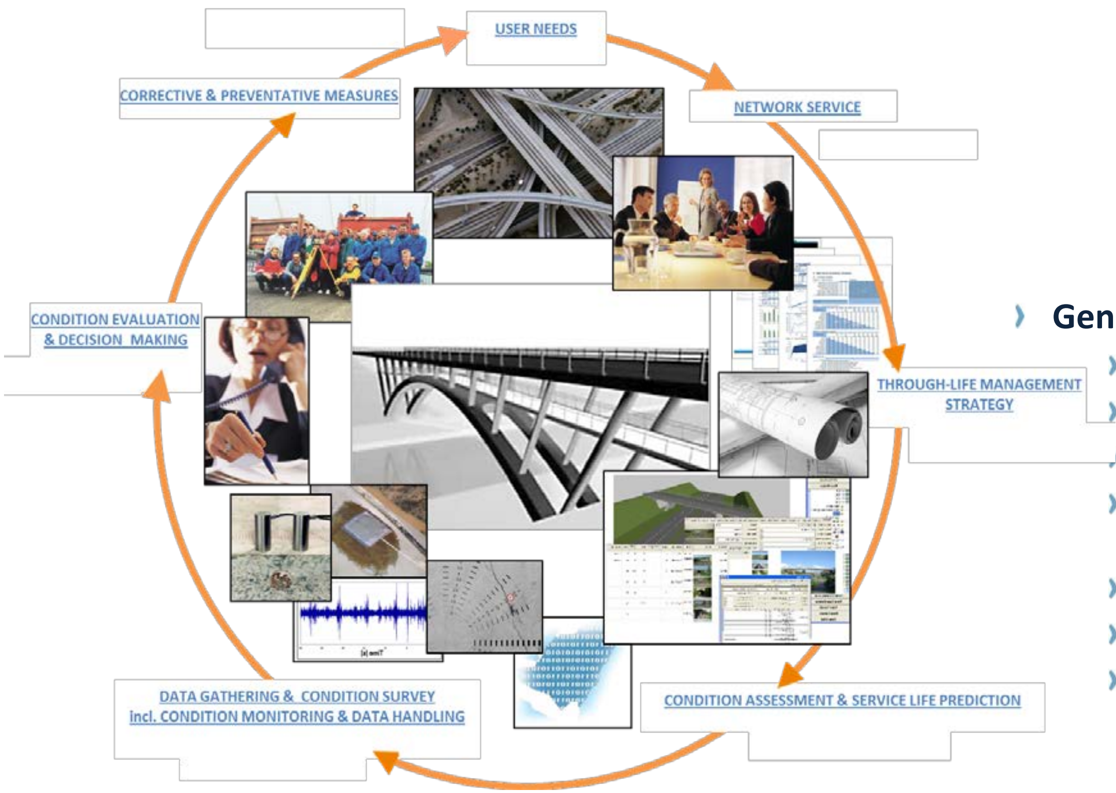
General principles of *fib* MC2020

- Through-life management of (existing) structures

A design service life performance plan for elements of a bridge

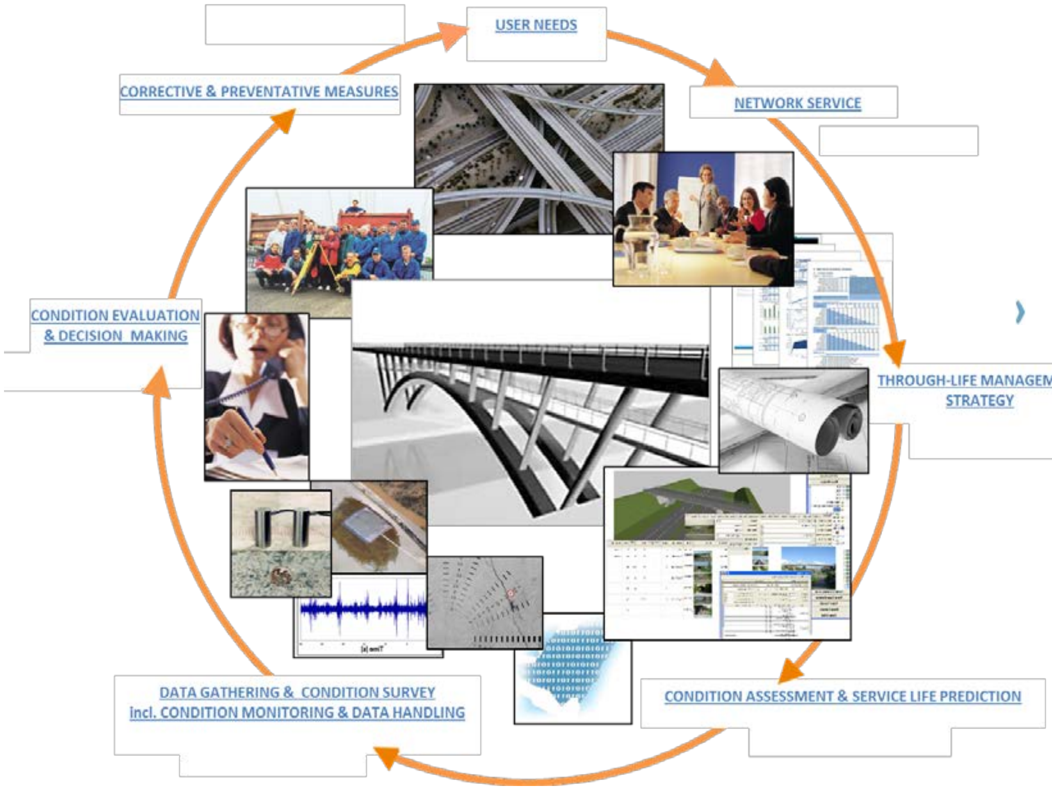


KPI and perfomance conctepts for AM



- **General flow from life cycle perspective**
  - **User needs recognition**
  - **Network service level definition**
  - **Through-life management strategy decisions**
  - **Condition assessment & service life prediction procedures**
  - **Data gathering & condition survey tools & techniques**
  - **Condition evaluation & decision-making methodology**
  - **Measures planning & execution**

KPI and perfomance conctepts for AM



- General flow from life cycle performance perspective
  - Network performance
  - Performance goals
  - Performance requirements
  - Performance criteria
  - Performance indicators
  - PI thresholds

MC2010 terminology

- **Performance:** Measure of service quality of a structure as seen by the customer, characterized by the behavior or by the appearance of a structure or structural element as a consequence of action, to which it is subject or which it generates, at present and in the future.
- **Performance goal:** Aspect of the required service of a structure, characterized by the behavior or by the appearance of a structure or its components for a specific action to which it is subject or which it generates.
- **Performance requirement:** Condition used to describe a required service quality with regard to specific performance goal, established by means of performance indicator(s) and associated performance criteria with constrains related to service life and reliability.
- **Performance criteria:** Quantitative limits (i.e. failure criteria*), defining the border between desired and adverse behavior .

  In context of Limit State Approach, performance criteria are the threshold values that describe for each limit state the conditions to be fulfilled i.e. failure criteria *

  * Failure is not synonymous with collapse

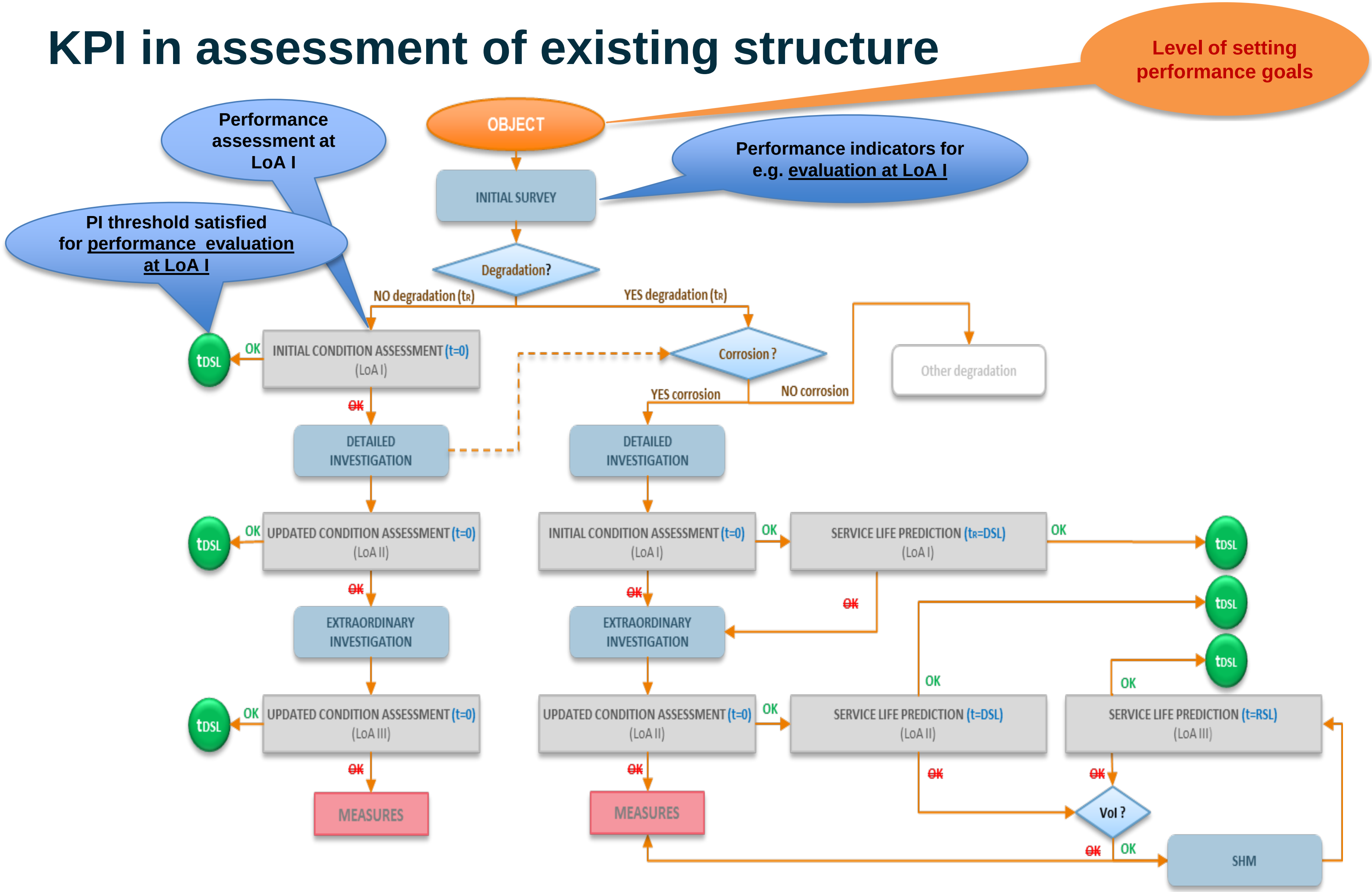
MC2010 terminology

- **Performance:** Measure of service quality
- **Performance goal:** Aspect of the required service
- **Performance requirement:** Condition used to describe a required service quality with regard to specific performance goal
- **Performance criteria:** Quantitative limits (i.e. failure criteria)
- **Performance indicators**: Assessable (i.e. measurable, testable or calculated) dimensional parameter of dimensionless index that quantitatively describes the behavior or the appearance of a structure or its components, relevant for a specific performance requirement at present and in the future.
  - performance indicator is associated with occurrence and development of defect(s), which affects ability od structure or its components to perform at the required performance level.
- **Thresholds for performance indicators**: A value that constitutes a boundary for performance indicator for a purpose of (a) monitoring, (b) assessing and/or (c) decision-making
  - thresholds are to be defined and determined by reliability-based approach considering relevant uncertainties

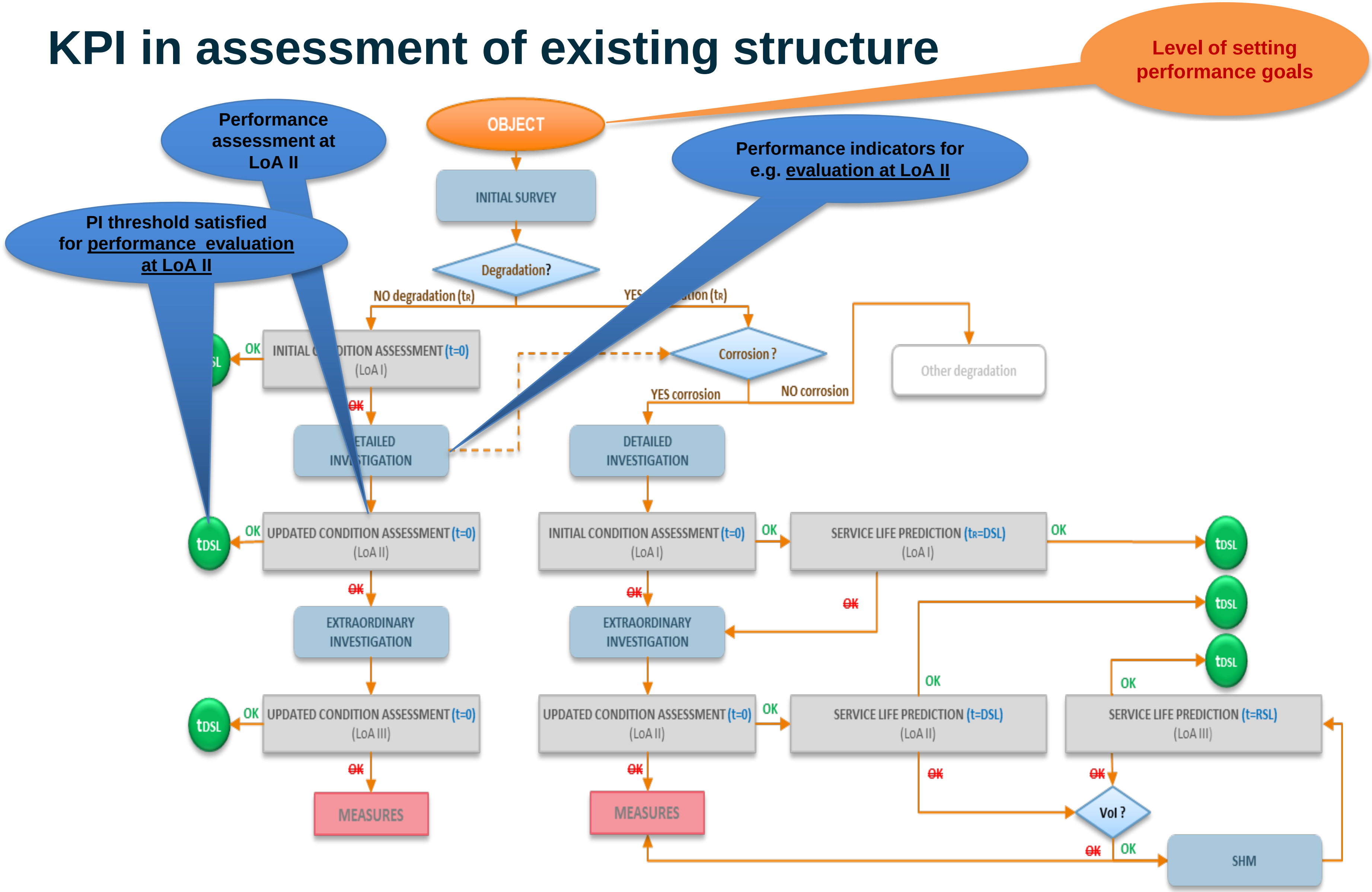
MC2010 terminology

- **Defect:** A specific imperfection or inadequacy in the structure or its components, which affects their ability to perform according to their intended functional required level, either now or at some future time
- **Defect as result of damage**: effect(s) of physical disruption or change in the condition of a structure or its components, caused by external actions, such that some aspects of either the current or future performance of the structure or its components will be impaired.
  - unfavorable change in integrity of the structural members
  - unfavorable change in mechanical properties of construction materials
  - unfavorable change in properties of structural system inc. changes in geometrical properties of the structural members, changes in member connections and changes in supports
  - unfavorable change related to equipment & protection
  - other unfavorable changes
- **Defect resulting from error(s)**: defect arising as a result of an error in design or construction
  - result of errors affecting (mechanical and durability) properties of construction materials
  - result of errors affecting integrity of structural members
  - result of errors affecting properties of structural system
  - result of errors affecting equipment & protection
  - other result of other errors in design of construction

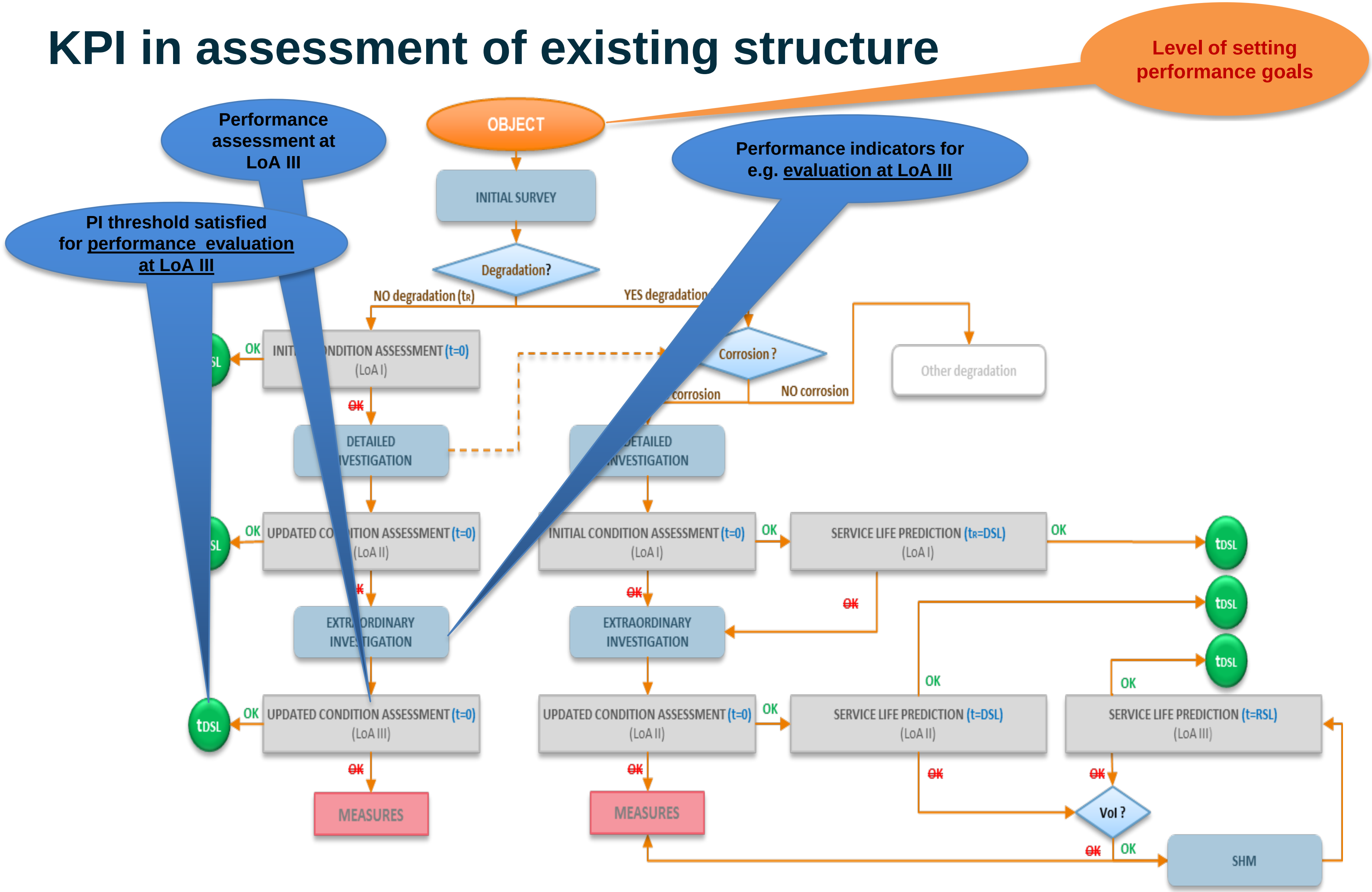
KPI in assessment of existing structure



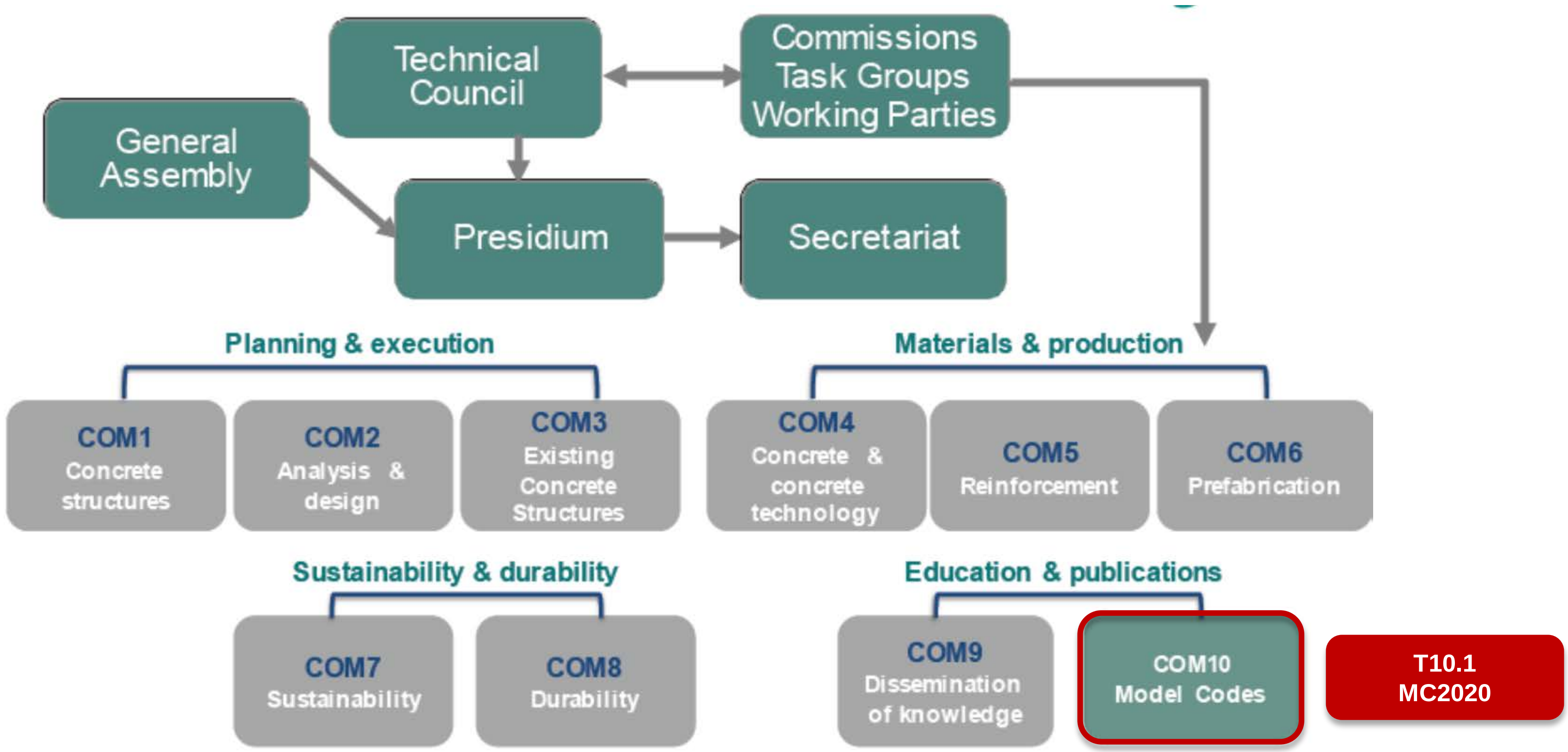
KPI in assessment of existing structure



KPI in assessment of existing structure



Organization framework for MC2020



Organization framework for MC2020

T10.1 : co-ordinating & drafting body for MC2020 (COM10: T10.1)

Core Group

Chair:	**Stuart Matthews (BRE)**
Co-Chair:	**Giuseppe Mancini (Uni Torino), Joost Walraven (TU Delft)**
Deputy-Chair:	**Agnieszka Bigaj-van Vliet (TNO)**
Technical Secretary:	**Gerrie Dieteren (TNO)**
Other core group members:	**Hugo Corres Peiretti, Frank Dehn, Aurelio Muttoni, Tamon Ueda**

Representatives of **all fib Commissions**

Regional contacts : **Africa, Asia, Europe, North America & South America**

Contacts for international organisations inc. bodies such as **CEN, ISO, JCI, ACI, PCI, JCSS & RILEM**

Other invited fib members

Timeline for MC2020

June 2015	*fib* workshop on Advancing the *fib* Model Code for Concrete Structures, The Hague, Netherlands: identifying needs and possibilities for provisions for existing concrete structures
Dec 2015	Core Group meeting to identify topics, the desired contents and roadmap for MC2020
Feb 2016	Submission of the roadmap for M2020 to the *fib* Presidium
Mar 2016	Decisions on the roadmap for M2020 by the *fib* Presidium
June 2016	Decision on establishing COM10 and T10.1 by the *fib* Technical Council
Sept 2016	Preparatory discussions on T10.1 JCI-*fib* Joint Workshop on Codes for Existing Structures and MC2020, Tokyo, Japan
Okt 2016	T10.1 MC2020 Kick-off meeting, Lausanne, Switzerland: establishing a shared vision on specific tasks, set draft contents for MC2020 & identify (main)authors / contributing groups, formulate final timetable proposal etc.

Timeline for MC2020

Nov 2016	MC2020 regional workshop, to stimulate participation of Africa - link to the *fib* Symposium in Cape Town, Republic of South Africa
Dec 2016	Dissemination of the vision on advancing *fib* MC2020 in *Structural Concrete*
2016	Start of delivery of *fib* Bulletins from the supporting work program of commissions
Mar 2017	MC2020 regional workshop: to stimulate participation of North America - link to ACI convention in Detroit
June 2017	Special session on Advancing fib Model Code for Concrete Structures on *fib* Symposium in Maastricht
Sept 2017	MC2020 regional workshop: to stimulate participation / link with RILEM in Chennai, India
2017	Delivery of *fib* Bulletins from the supporting work program of commissions

Timeline for MC2020

Oct 2018	Report of drafting of MC2020 presented at *fib* Congress in Melbourne
2018	Delivery of *fib* Bulletins from the supporting work program of commissions
May 2019	Report of drafting of MC2020 presented at *fib* Symposium in Krakow
2019/2022	Delivery of *fib* Bulletins from the supporting work program of commissions
2020	First compete draft of *fib* MC2020 for review and comment by *fib* Technical Council, *fib* Commissions, *fib* National Groups, *fib* members, etc.
2021	Final draft of MC2020 presented at *fib* Symposium / Congress & voting by *fib* General Assembly

TU1406

COST ACTION

TU1406
COST ACTION

WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

PERFORMANCE GOAL ASSESSMENT FOR EXISTING BRIDGES SUBJECT TO PIER LOCAL SCOUR

Mariano Angelo Zanini - University of Padova, Italy
Lorenzo Hofer - University of Padova, Italy
Flora Faleschini - University of Padova, Italy
Paolo Zampieri - University of Padova, Italy
Carlo Pellegrino - University of Padova, Italy

Università degli Studi di Padova

DIPARTIMENTO DI INGEGNERIA CIVILE, EDILE E AMBIENTALE - ICEA
DEPARTMENT OF CIVIL, ENVIRONMENTAL AND ARCHITECTURAL ENGINEERING

20th – 21st October 2016
Delft, Netherlands

INTRODUCTION

- In European roadway and railway networks, many multi-span unreinforced arch bridges were built in the last centuries.
- Piers are often characterized by shallow foundations placed in the river bed, which can be subject to hydrodynamic turbulences in case of river floods and can lead to localized scour phenomena.

INTRODUCTION

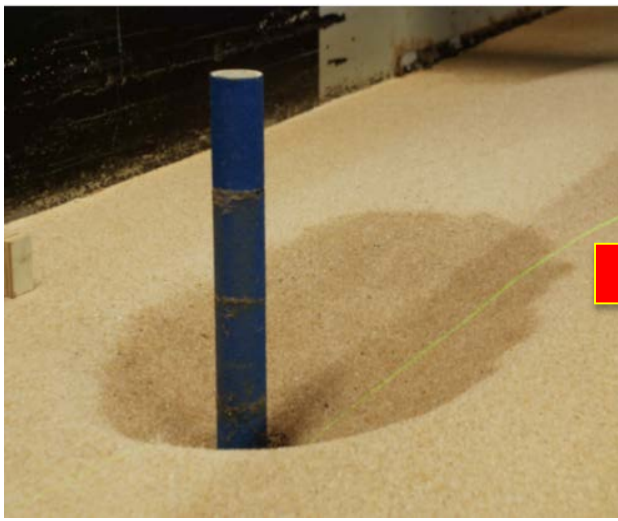
- The lack of adequate monitoring can cause over time scour-induced settlements able to involve bridge structural failure.
- The structural behaviour of existing masonry bridges through the failure analysis of a finite element model case study is investigated.

1901

2011 (retrieved online at www.lametino.it)

AIM OF THE WORK

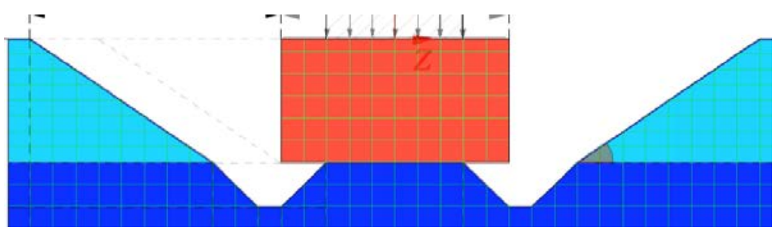
- Numerically simulating the evolution of the structural behaviour and finding a relationship with the local scour profile at the base of pier foundations.



P.I.?

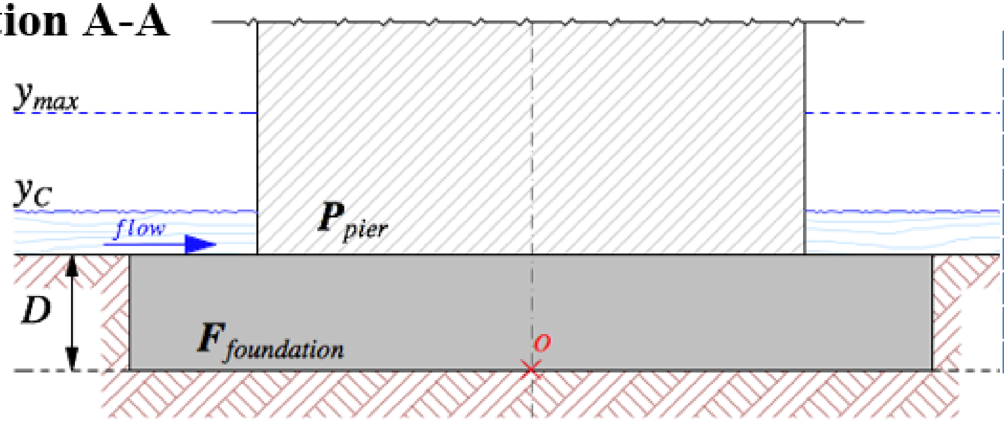
P.G.?

Il momento della caduta del ponte della foce(1834), avvenuta il 22 agosto 1965. Da archivio foto di Ballinari R., Verbania.



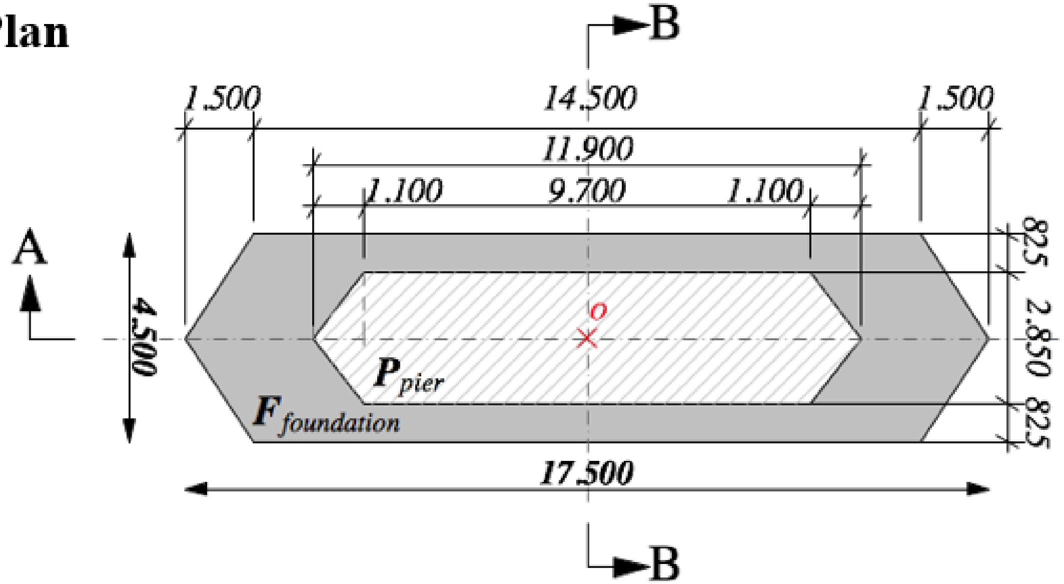
FOUNDATION GEOMETRIC CONFIGURATION

Section A-A

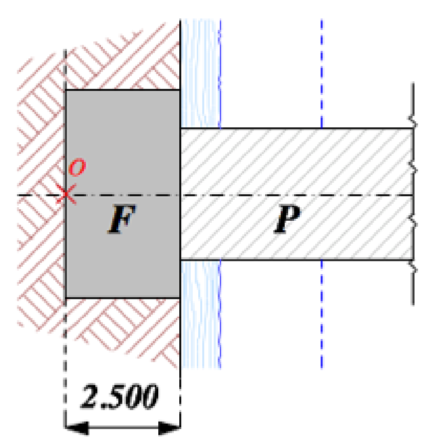


Parameter	Soil	Foundation
Specific weight	γ_S = 18 KN/m^3	γ_F = 20 KN/m^3
Saturated unit weight	γ'_S = 8 KN/m^3	-
Elastic modulus	E = 50 MPa	E = 20000 MPa
Poisson's coefficient	n = 0.25	n = 0.20
Finite element	Plate Quad8	Plate Quad8
Constitutive law	Elastic-Plastic	Elastic
Material model	Mohr-Coulomb	von Mises
Friction angle	φ' = 35°	-
Cohesion	c = 0.001 MPa	-

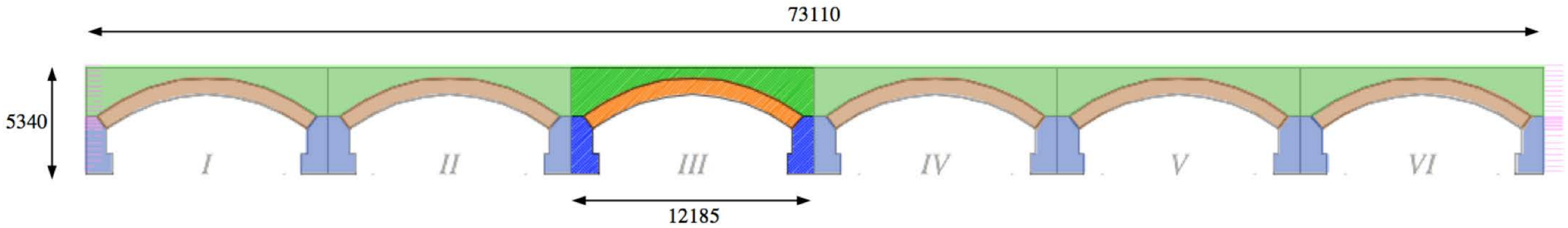
Plan



Section B-B

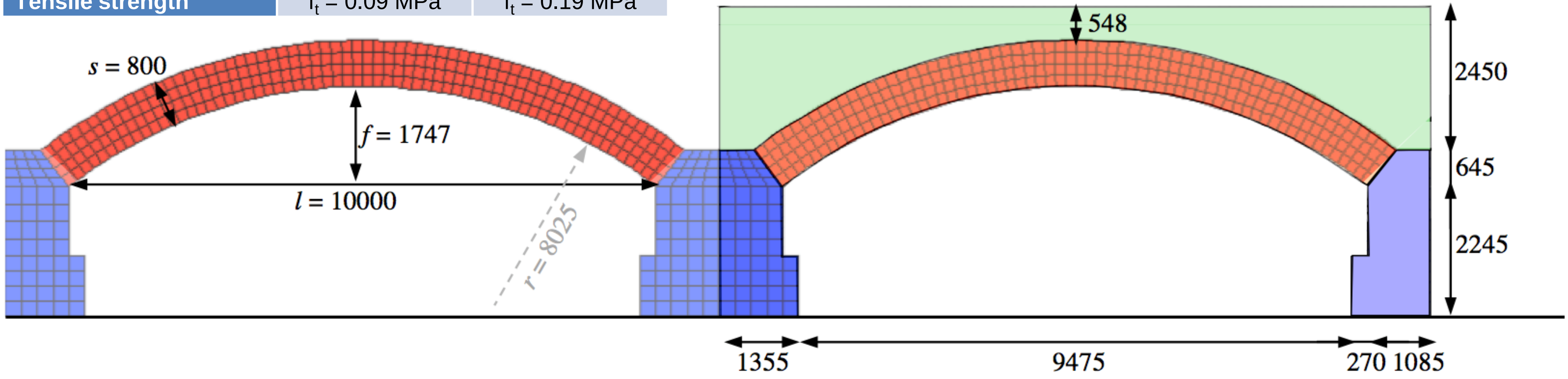


BRIDGE GEOMETRIC CONFIGURATION



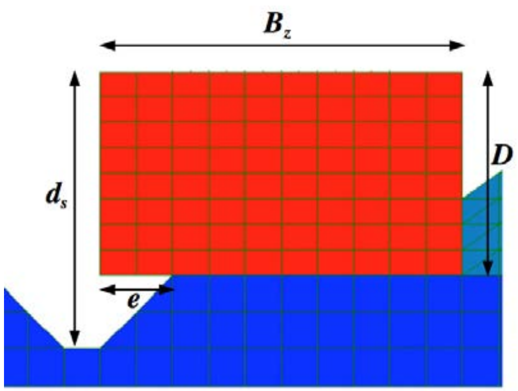
Parameter	Backfill	Arch / Piers
Specific weight	γ_S = 18 KN/m^3	γ_S = 18 KN/m^3
Elastic modulus	E =1400 MPa	E =4000 MPa
Poisson's coefficient	n = 0.20	n = 0.20
Constitutive law	Elastic-Plastic	Elastic-Plastic
Material model	Mohr-Coulomb	Drucker-Prager
Friction angle	φ' = 30°	φ' = 64.8°
Cohesion	c = 0.08 MPa	c = 0.43 MPa
Compression strength	f_c = 0.27 MPa	f_c = 3.85 MPa
Tensile strength	f_t = 0.09 MPa	f_t = 0.19 MPa

F.E. models were implemented with Strand7 v2.4 considering a 2-D plane strain.

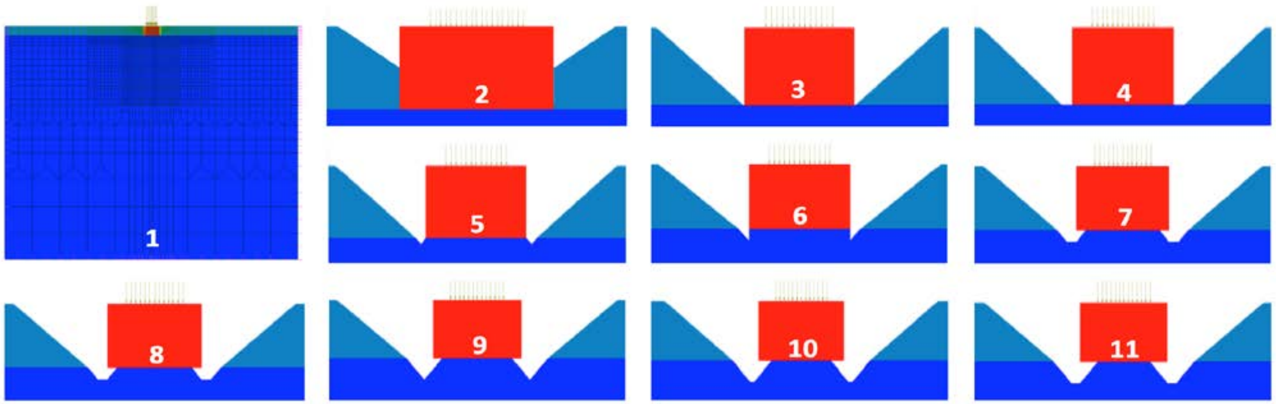


SCOUR LAYOUT

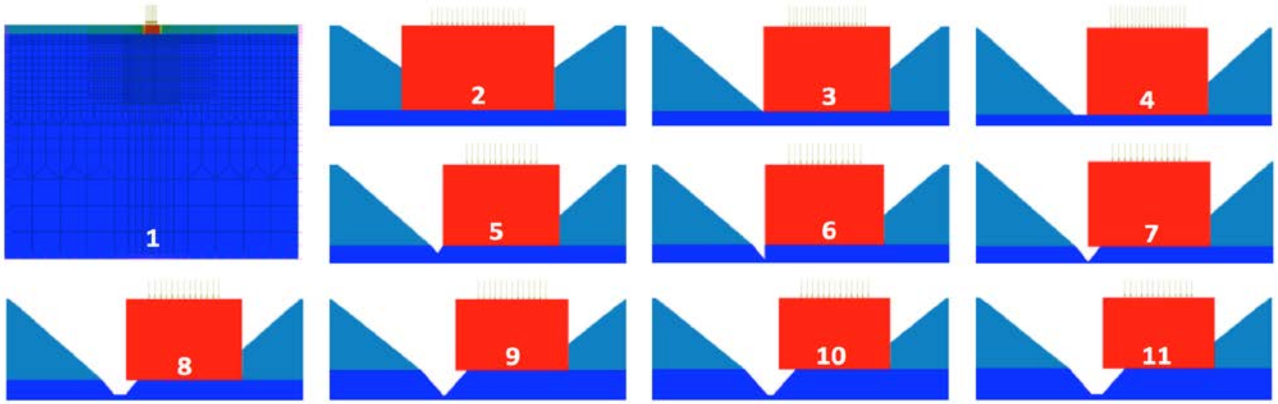
11 increasing scour layouts were considered in both cases:



SIMMETRICAL

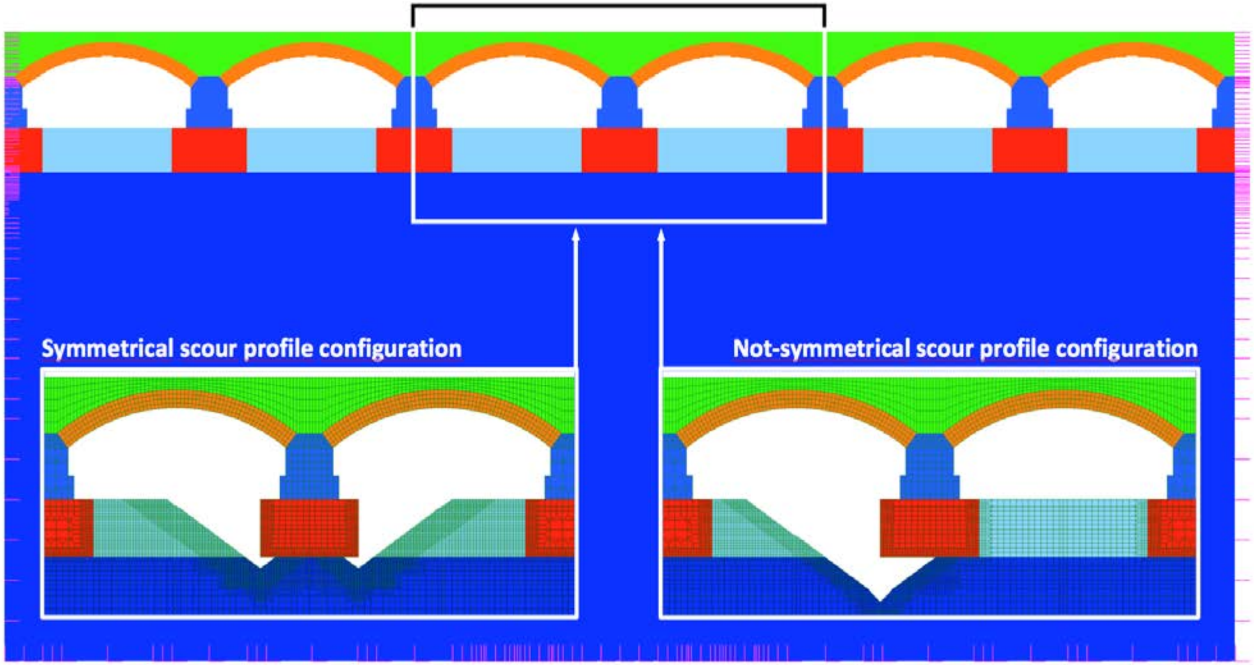


NOT-SIMMETRICAL

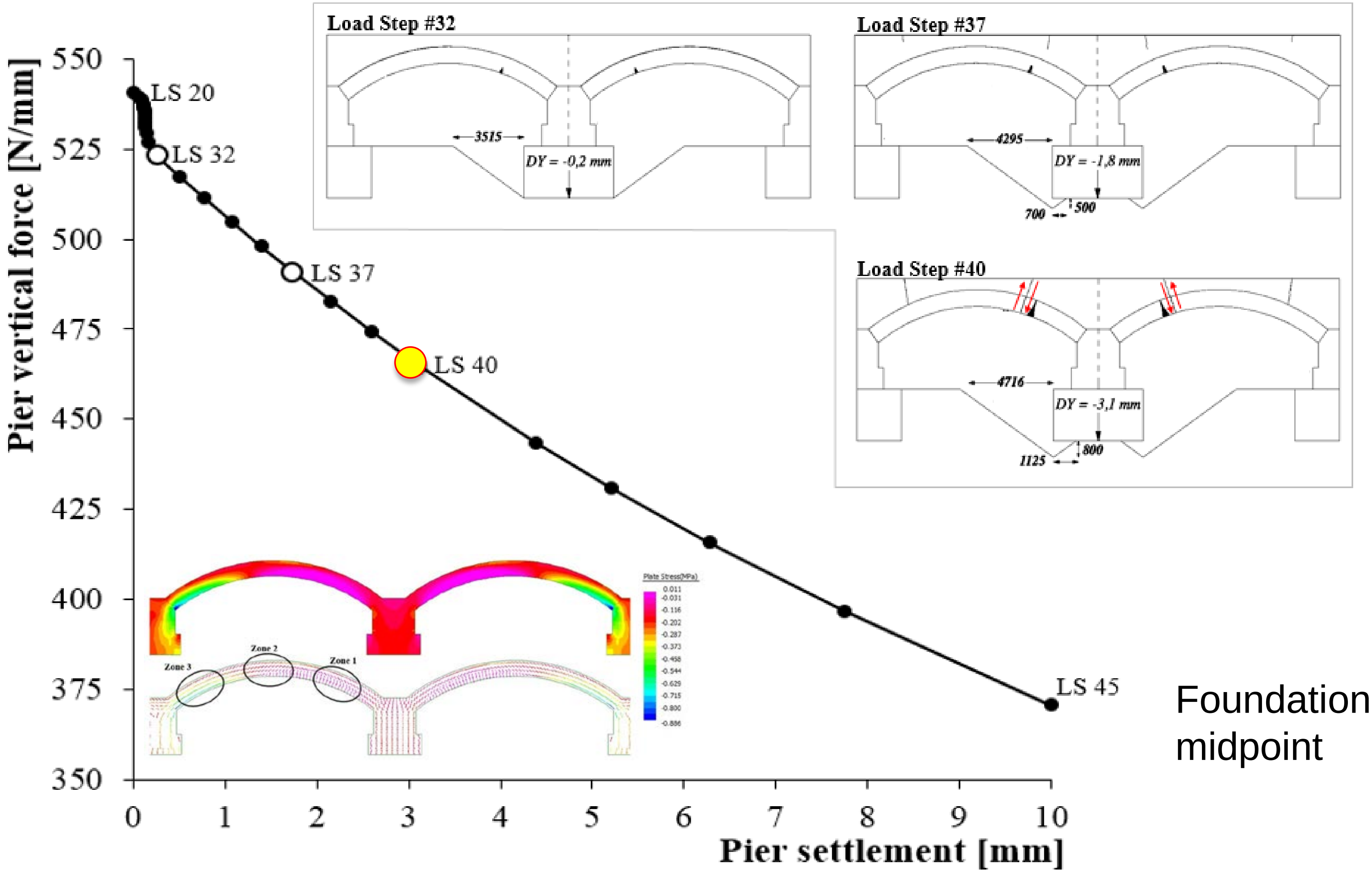


F.E. MODEL ANALYSIS

- Tensional stress evolution in the arches was monitored with the aim to identify potential kinematic collapse mechanisms, and the related scour configuration able to induce bridge structural failure.
- Non-linear construction stage analyses performed for increasing local scour profiles ($\varphi' = 35°$).



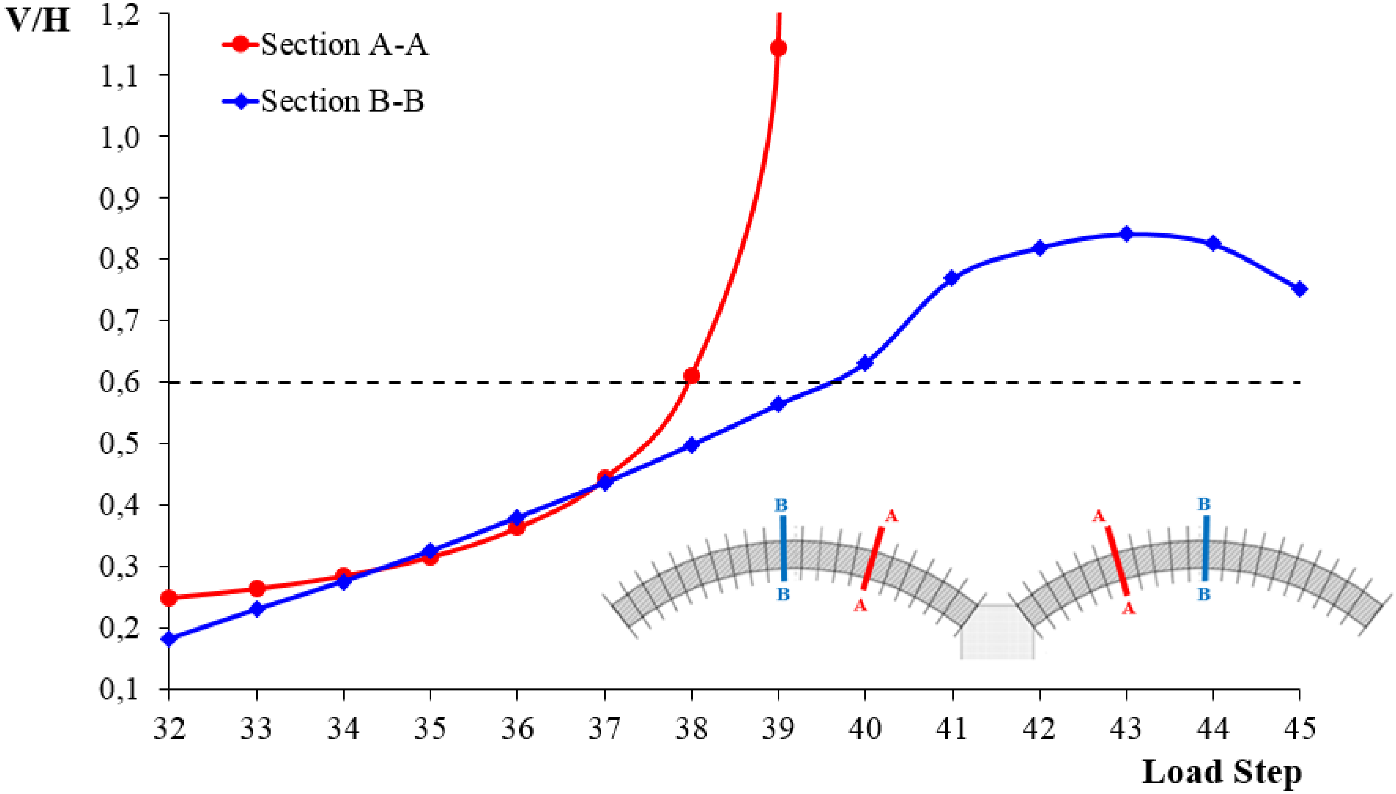
RESULTS: SYMMETRICAL CASE



Foundation midpoint

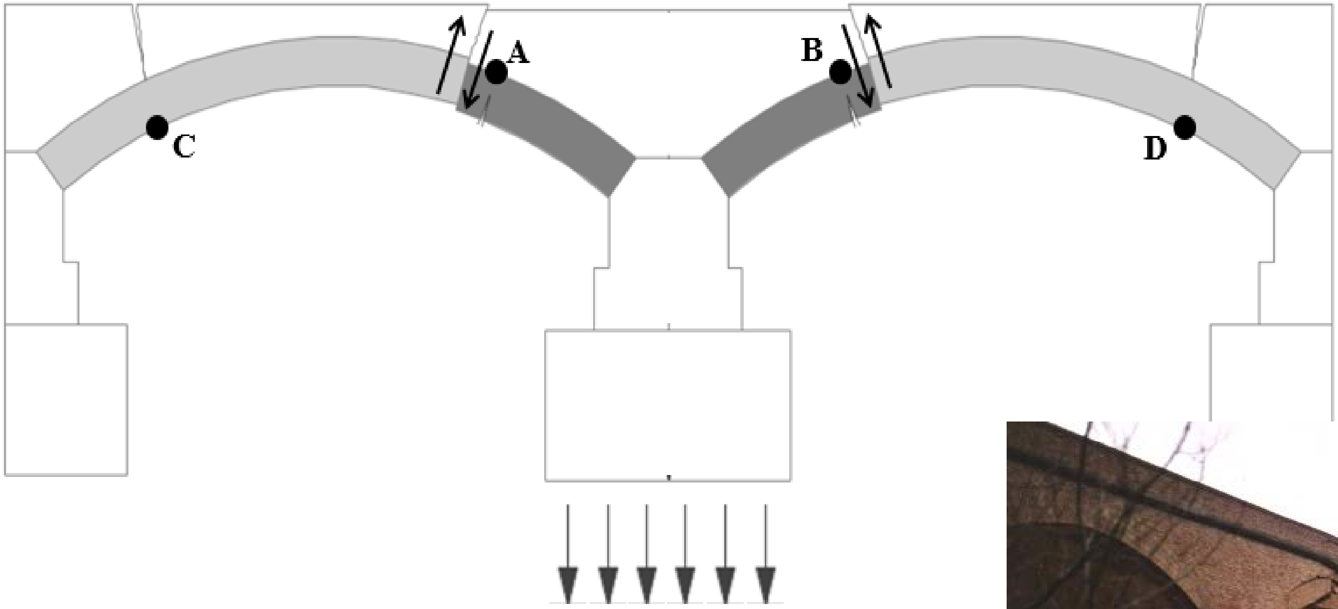
RESULTS: SYMMETRICAL CASE

- Drucker-Prager does not consider friction between arch blocks: however, it is indirectly derived. Evaluation of *V/H* ratio for each load step and comparison with threshold [Drosopoulos et al. 2006].



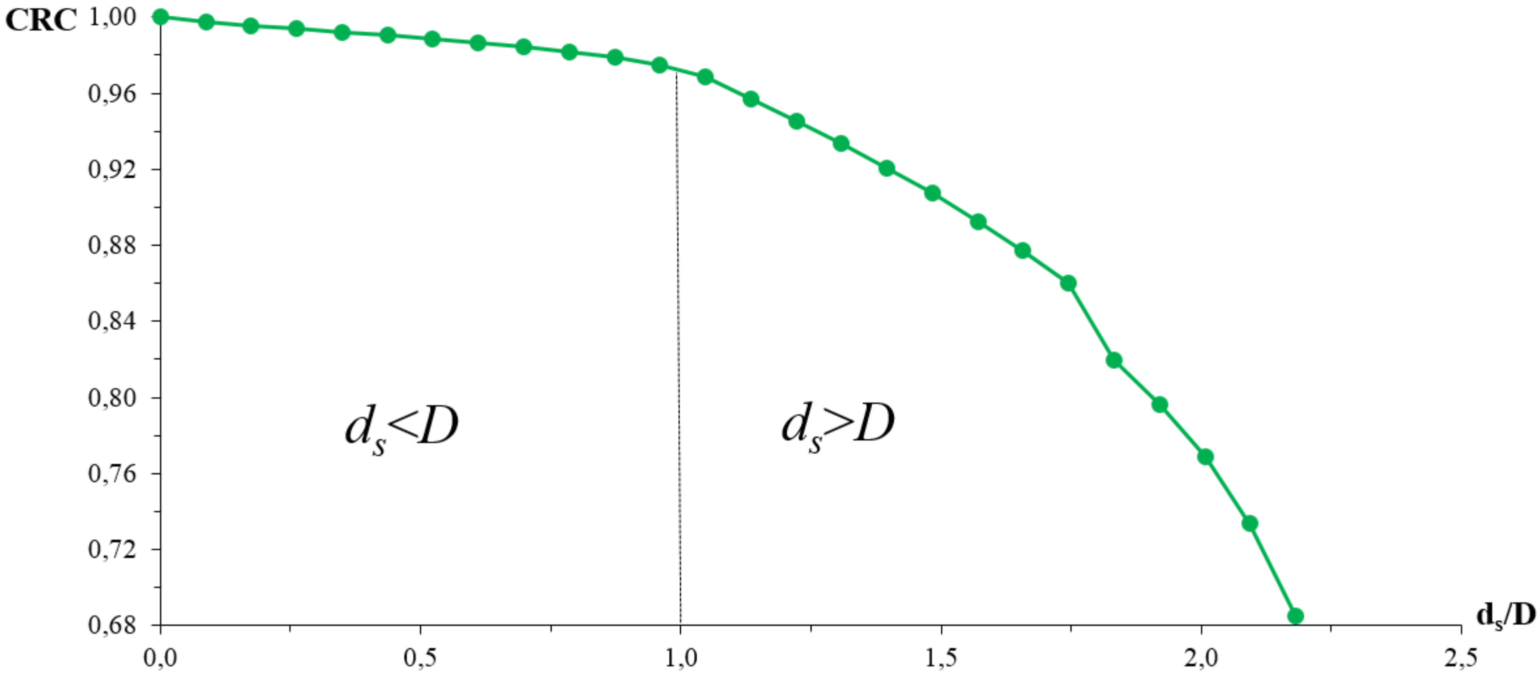
RESULTS: SYMMETRICAL CASE

- For higher displacement values, other additional hinges appear at the arches intrados close to piers adjacent to the settled central one.

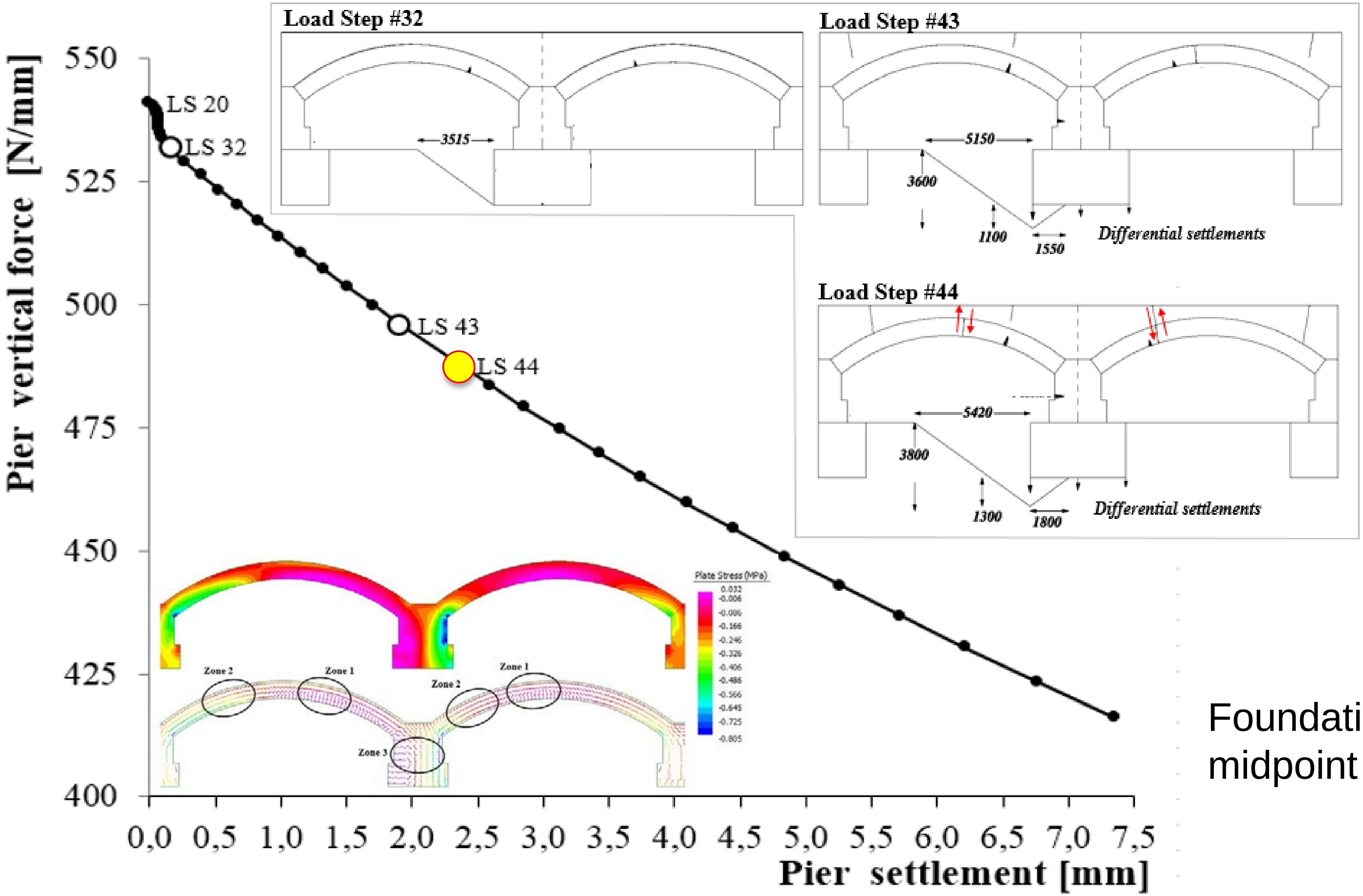


RESULTS: SYMMETRICAL CASE

- Relationship between a constrain reaction coefficient (*CRC*), (i.e. dimensionless coefficient representative of the reduction of the pier foundation vertical reaction) and dimensionless scour depth d_s/D:, seems able to quantitatively describe the phenomenon and defining threshold values.



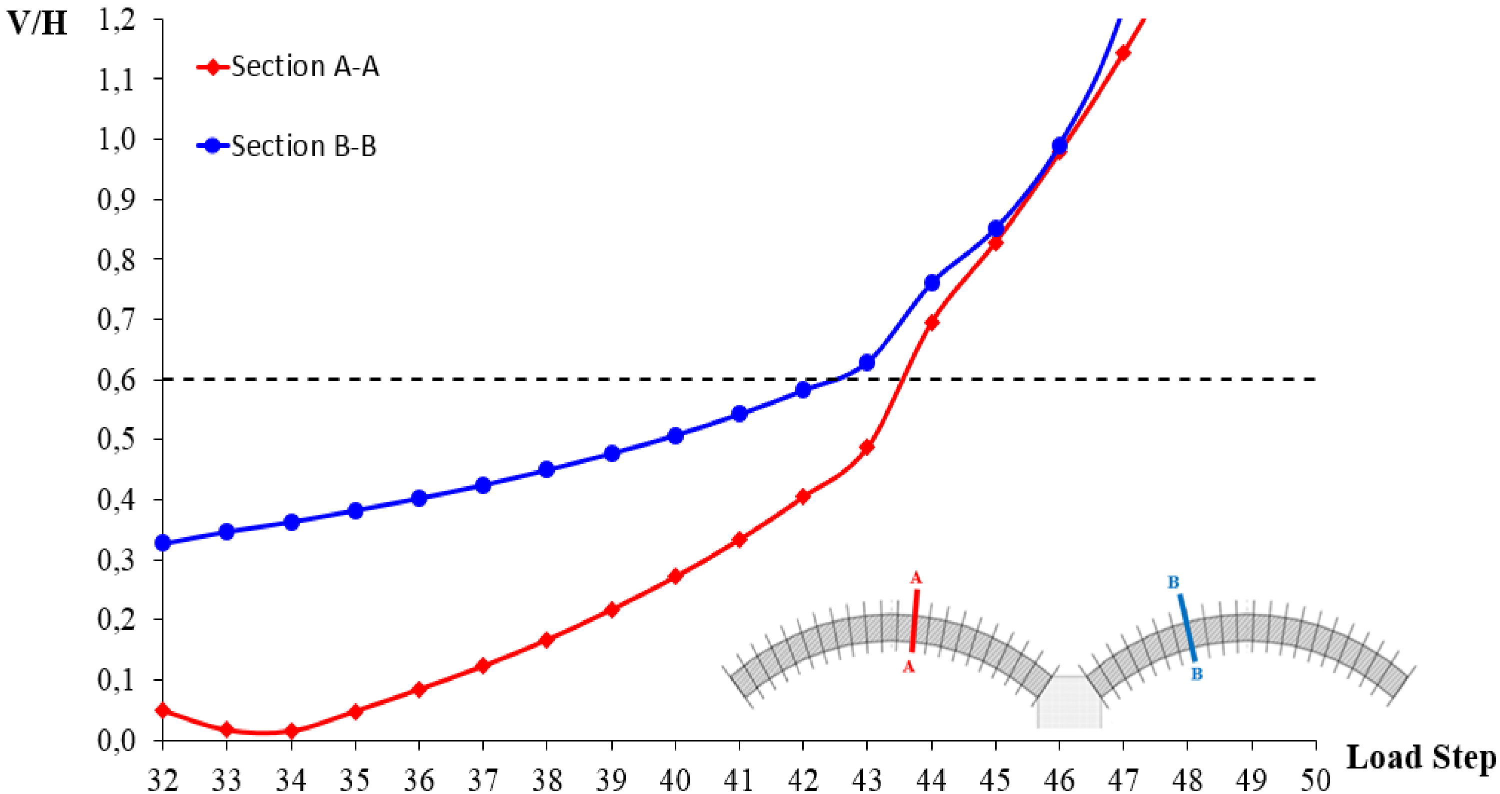
RESULTS: NOT SYMMETRICAL CASE



Foundation midpoint

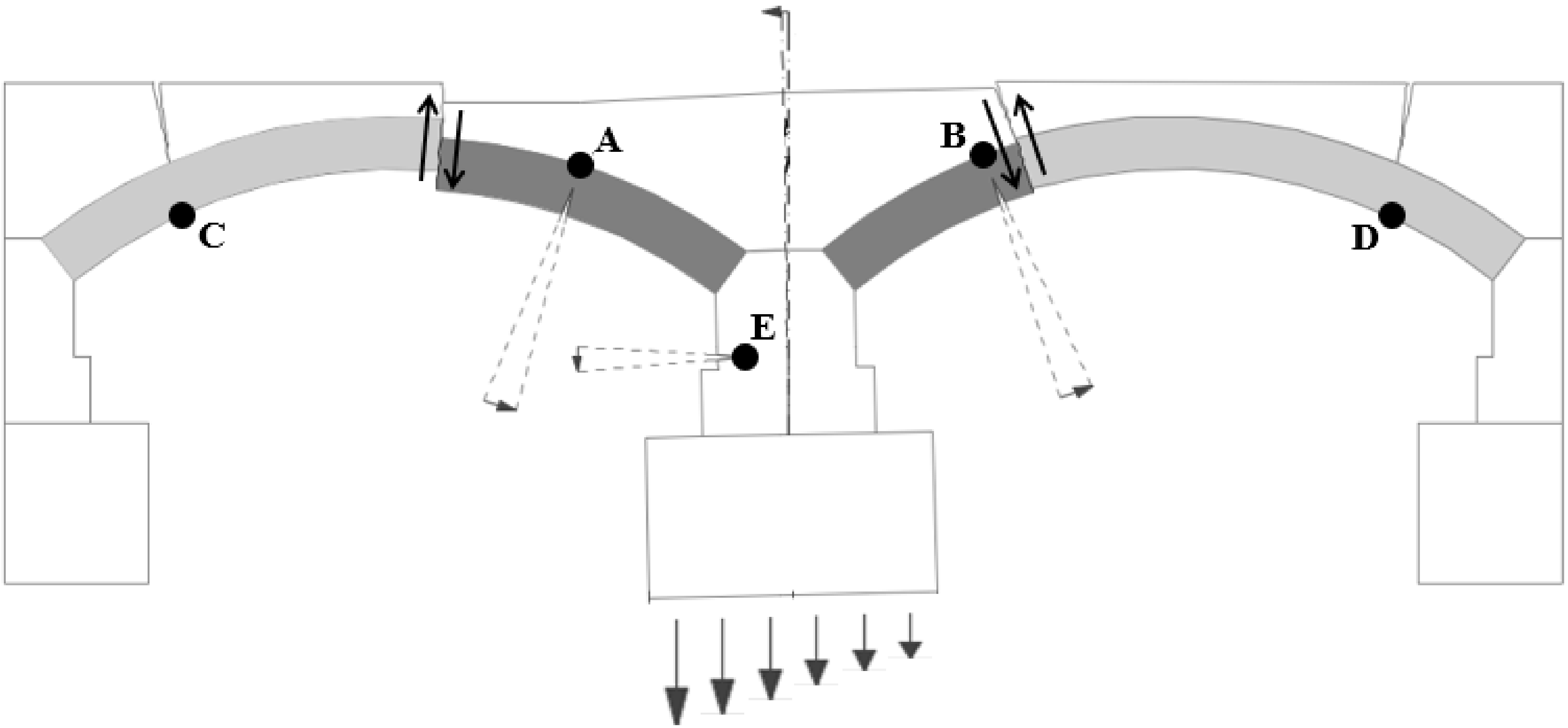
RESULTS: NOT SYMMETRICAL CASE

- Drucker-Prager does not consider friction between arch blocks: however, it is indirectly derived. Evaluation of *V/H* ratio for each load step and comparison with threshold [Drosopoulos et al. 2006].



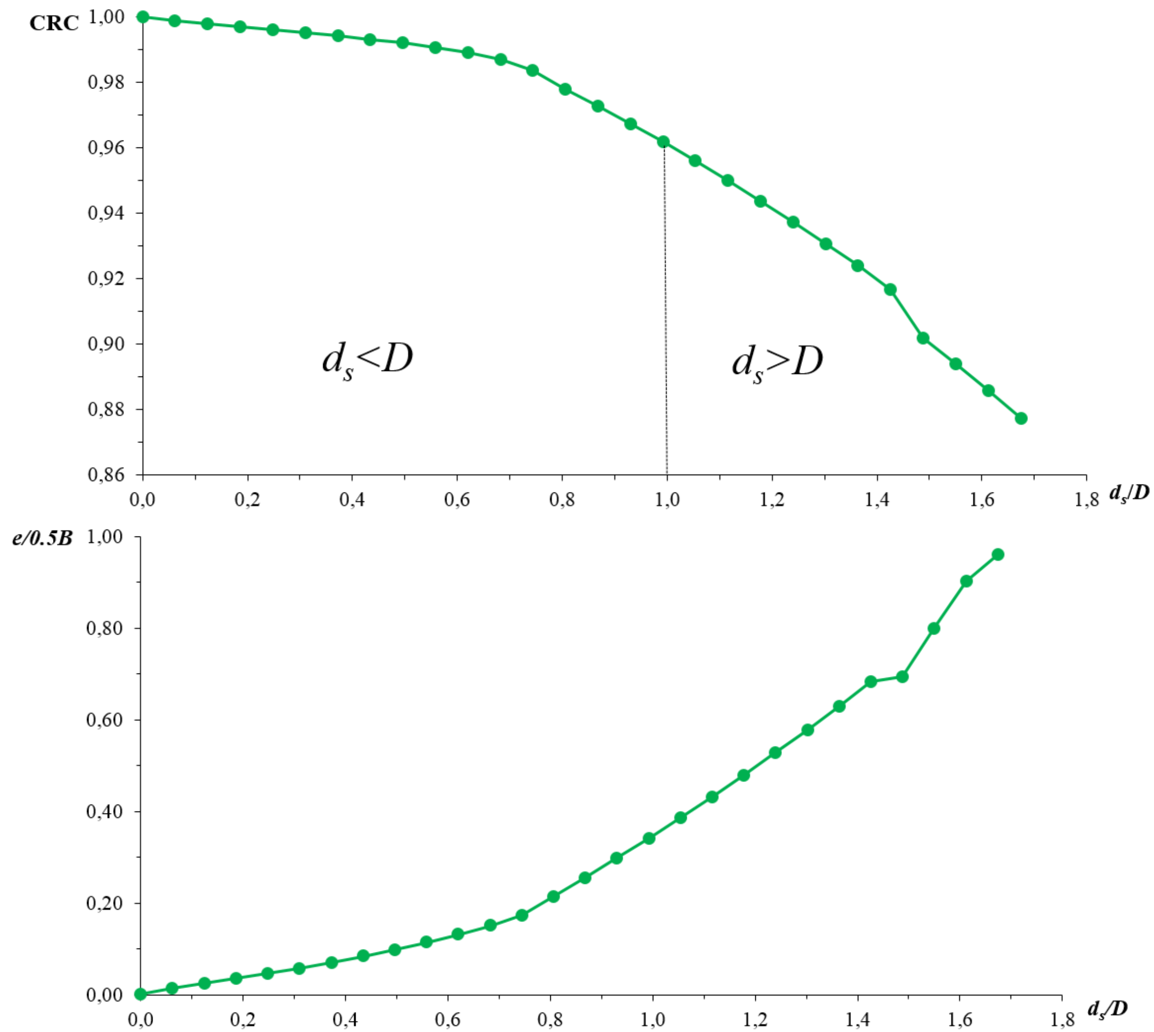
RESULTS: NOT SYMMETRICAL CASE

- Failure mechanism is characterized by two asymmetric sections with respect to the settled pier subject to rigid block sliding, two sections close to the adjacent piers in which hinges are localized at the arch intrados, cracking and a consequent hinge at the pier mid-height left.



RESULTS: NOT SYMMETRICAL CASE

- For d_s/D values lower than 0.8, *CRC* slightly decreases to 0.98, whereas for higher values, *CRC* reduction is remarkable.
- The dimensionless eccentricity of pier reaction *e/(0.5B)* is less than 0.2 until d_s/D<0.8, whereas increases up to the threshold value of 1 for a dimensionless scour depth d_s/D equal to 1.7.



CONCLUSIONS

- For d_s/D values lower than 1, scouring has a negligible influence on the structural response of masonry arch bridges, whereas when erosion starts to erode under the pier foundation base, settlements become evident, inducing cracking phenomena at the arch intrados.

- The failure of the structural system is reached when it turns into rigid blocks subject to relative sliding. This change of the structural scheme is mainly due to the settlement-induced increase of the V/H ratio up to 0.6 in some critical arch sections.

- Cracking phenomena appear at pier mid-height if differential settlements affect pier foundation and are indicators of the presence of eccentricity of the pier axial force.

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Detailed Evaluation of Deteriorated Highway Bridges in Greece

Dimitrios Nikolaidis- ConCentral Group, Larisa,Greece
Theodoros Rousakis- Democritus University of Thrace,Xanthi,Greece

Quality is not an act it is a habit
Aristotle

20th – 21st October 2016
Delft, Netherlands

Introduction.

The aim of the study is to develop a simplified evaluation tool regarding the structural redundancy of deteriorated existing bridges. Considering the difficulties arise from complex reliability models, based on time depended failure function of degradation models, further research has been performed towards an efficient assessment tool proposed for Highway Network in Greece.

The Idea of Structural Redundancy (r_{eff}).

Undamaged Sections.

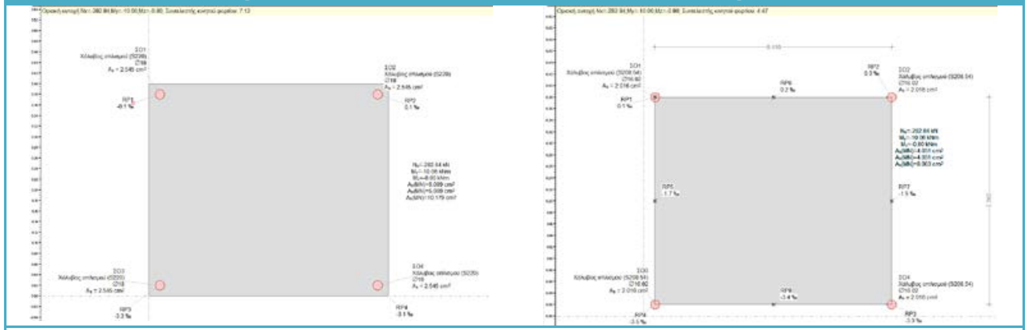
γ× S≤R and $\gamma = \frac{R}{S}$, where: R= Capacity & S= Actions.

Damaged Sections.

γ× S≤R and $\gamma' = \frac{R'}{S}$, where: R: Capacity &S: Actions.

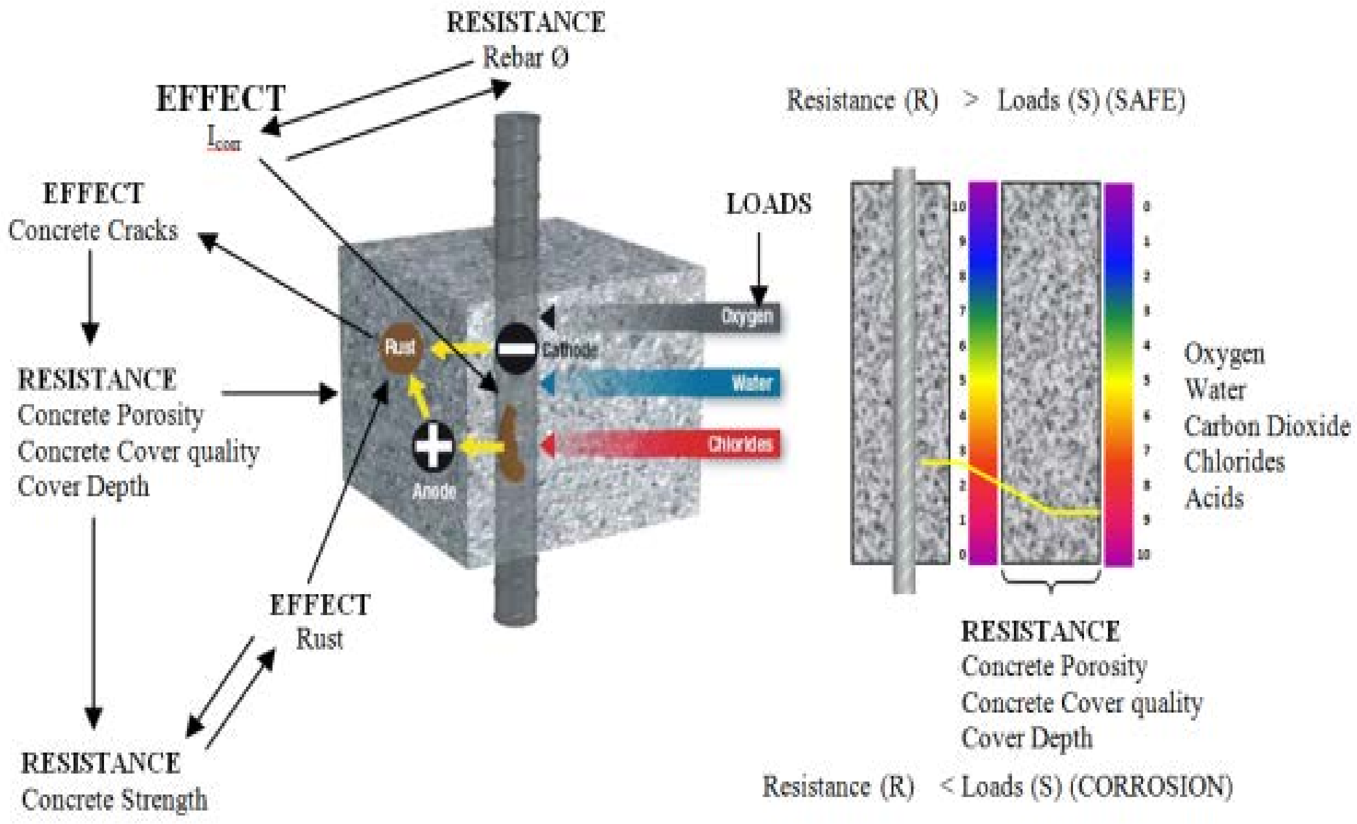
Structural Redundancy $= \frac{\gamma'}{\gamma}$ (r_{eff})

Undamaged Section	Damaged Section



γ =7.12 (R/S)	γ'=4.55 (R'/S)

Interdependence of multiple degradation factors causing “damage”.

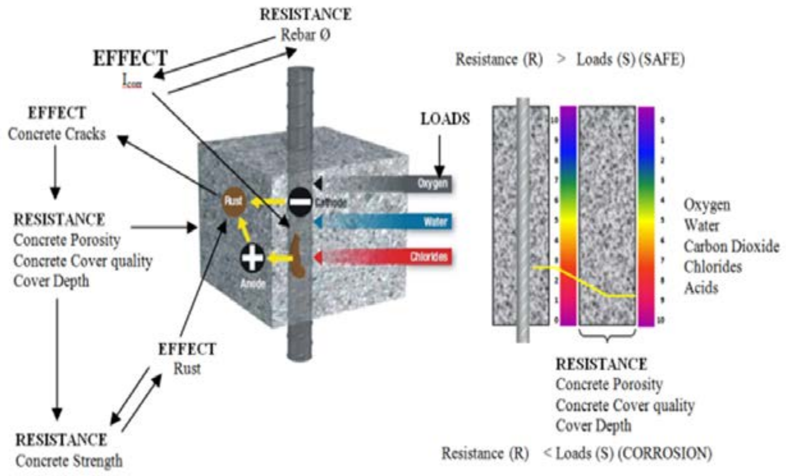


Structural Redundancy Factor Method (r_{eff}).

- The method is based on the implementation of predefined Non destructive testing (NDT) for assessing 11 main influencing “material damage (degradation) factors”. Their extent is expressed through the Damage Indices (DI) on a four Damage Index scale (1-4). The interdependence of multiple degradation factors can be expressed as the mean value of the summation of all existing DI. Therefore, the weight and the extent of damage due to bar corrosion related degradation is expressed through a final index called Corrosion Damage Index (DI), expressing the effect but not the cause.

Damage Index & Damage Weighing (RC).

No	Damage	Code	Damage Index			
			1	2	3	4
1	Carbonation Depth	CRB	0	<c	=c	>c
2	Chloride Front	CL	0	<c	=c	>c
3	% Cl w.t of concrete (at concrete cover)	CLC	<0.025	0.025<CLC< 0.03	0.025<CLC< 0.03	CLC>0.04
4	Cracks (RC Concrete)	CR	none	<0.3mm	>0.3	Spalling - cavities - large cracks
5	Concrete Resistivity (KΩcm)	R	>20	10< R<20	5<R<10	R<5
6	Half - Cell (mV)(Cu/$CuSO_4$)	HC	0-220	220-350	350-450	>450
7	Corrosion Rate (Icorr, LPR)	CORR	<0.1	0.1-0.5	0.5-1	>1
8	Ømm (Rebar Diameter)	D	>18mm	18mm-16mm	16mm-12mm	<12mm
9	% rebar mass loss	RD	0-2	2-5	5-10	>10
10	Concrete Moisture Content ,EMC (%)	MST	<2	2-4	4-6	>6
11	Concrete Strength	CS	≥C25/30	C20/25	C16/20	C12/15



$$DI = \frac{\sum_{i=1}^{n} Damage\,Weighing}{Damage\,No\,(n)}$$

Where n – Total number of damages.

Environmental Aggressivity.

Causes of deterioration can be expressed through Exposure Classes according to ELOT EN206-1 or KTS (concrete technology code) 2016. In the same way, each of the exposure classes is presented as a damage index due to environmental aggressivity (EA). Hence, the Structural Index considering the environmental impact (Lifecon) can be expressed as the mean .value of the summation of DI and EA

Class	X0	XC1	XC2	XC3	XC4	XD1	XD2	XD3	XS1	XS2	XS3
Weight (EA)	0	1	1	2	3	2	3	4	2	3	4

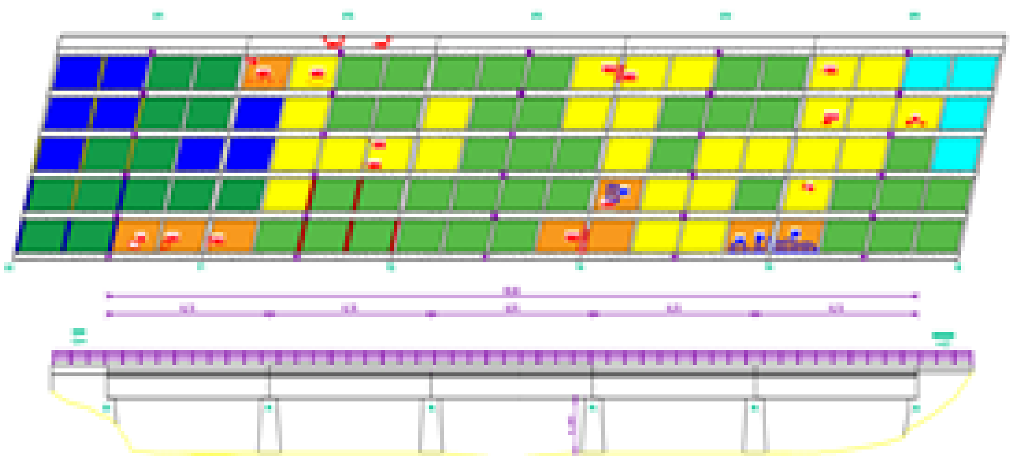
$$SDI = \frac{DI + EA}{2}$$

The final calculation of Structural Damage Index (SDI) is made, by averaging the weight of the exposure class with the actual Damage Index (DI) .Therefore, SDI provides sensitivity on damage evaluation by considering both cause and effect of a damage.

Evaluation of Damage Level.

	Damage Level	Damage Description	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

sensitivity factor FORM



Evaluation of r_{eff}.

$$r_{eff} = \mathrm{B}e^{-\alpha_R \beta V_R}$$

Where:

B –Reduction Factor (Related to geometrical uncertainties & material properties).

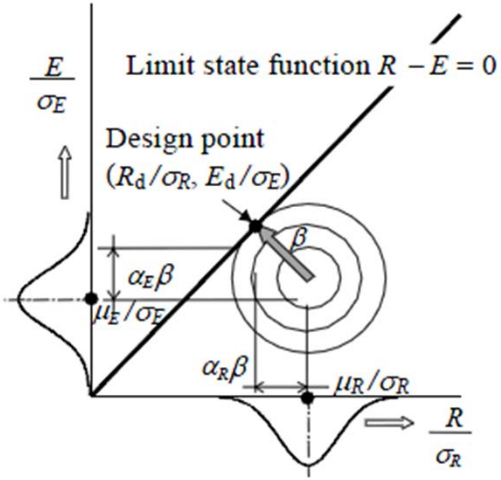
According to AASHTO B = 0.9 (Bending),B = 0.85 (Shear),B = 0.7 (Combination). These values can be used when the Knowledge Level is medium or low. For cases with extended in situ inspections ,B =1

a_R – sensitivity factor FORM

V_R – Coefficient of Variation.

β – Target Reliability Index, EN1990 for bridges (Design) β=4.5

Sensitivity Factor a



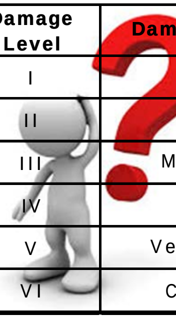
R < E

Reliability index β_t

simplified
Level II approach

Action reliability index	Resistance reliability index
$\beta_{E,t} = \alpha_E \beta_t$	$\beta_{R,t} = \alpha_R \beta_t$

Main action	Accompanying	Resistance
$\beta_{E,t} = -0{,}7\,\beta_t$	$\beta_{E,t} = -0{,}28\,\beta_t$	$\beta_{R,t} = 0{,}8\,\beta_t$



	Damage Level	Damage Description	SDI	a_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

$$\alpha_R = \sigma_R / \sqrt{\sigma_E^2 + \sigma_R^2} = 0{,}8$$

$$\alpha_E = -\sigma_E / \sqrt{\sigma_E^2 + \sigma_R^2} = -0{,}7$$

Target Reliabilities (β).

ISO 13882 – Target Reliabilities

Table F.1 — Illustrations of target reliability level

Limit states	Target reliability index, β	Reference period
Serviceability		
reversible	0,0	Intended remaining working life
irreversible	1,5	Intended remaining working life
Fatigue		
can be inspected	2,3	Intended remaining working life
cannot be inspected	3,1	Intended remaining working life
Ultimate		
very low consequences of failure	2,3	L_S years[a]
low consequences of failure	3,1	L_S years[a]
medium consequences of failure	3,8	L_S years[a]
high consequences of failure	4,3	L_S years[a]

[a] L_S is a minimum standard period for safety (e.g. 50 years).

JCSS– Target Reliabilities

Table 1: Tentative target reliability indices β (and associated target failure rates) related to one year reference period and ultimate limit states

1	2	3	4
Relative cost of safety measure	Minor consequences of failure	Moderate consequences of failure	Large consequences of failure
Large (A)	β=3.1 ($p_F \approx 10^{-3}$)	β=3.3 ($p_F \approx 5\ 10^{-4}$)	β=3.7 ($p_F \approx 10^{-4}$)
Normal (B)	β=3.7 ($p_F \approx 10^{-4}$)	β=4.2 ($p_F \approx 10^{-5}$)	β=4.4 ($p_F \approx 5\ 10^{-6}$)
Small (C)	β=4.2 ($p_F \approx 10^{-5}$)	β=4.4 ($p_F \approx 5\ 10^{-6}$)	β=4.7 ($p_F \approx 10^{-6}$)

Example NEN 8700 (Netherlands)

Minimum values for the reliability index β with a minimum reference period

Consequence class	Minimum reference period for existing building	β-NEW		β-EXISTING	
		wn	wd	wn	wd
0	1 year	3.3	2,3	1.8	0.8
1	15 years	**3.3**	2,3	1.8[a]	1.1[a]
2	15 years	**3.8**	2.8	2.5[a]	2.5[a]
3	15 years	**4.3**	3.3	3.3[a]	3.3[a]

Class 0: As class 1, but no human safety involved
wn = wind not dominant
wd = wind dominant
(a) = in this case is the minimum limit for personal safety normative

EN1900:2002– Target Reliabilities

Table 1. Reliability classification in accordance with CEN [5]

Reliability classes	Consequences for loss of human life, economical, social and environmental consequences	Reliability index β: β_a for T_a= 1 year	Reliability index β: β_d for T_d= 50 years	Examples of buildings and civil engineering works
3 – high	High	5,2	4,3	Bridges, public buildings
2 – normal	Medium	4,7	3,8	Residential and office buildings
1 – low	Low	4,2	3,3	Agricultural buildings, greenhouses

Damage Index & Damage Weighing (PS)

No	Damage	Code	Damage Index			
			1	2	3	4
1	Carbonation Depth	**CRB**	0	<c	=c	>c
2	Chloride Front	**CL**	0	<c	=c	>c
3	% Cl w.t of concrete (at concrete cover)	**CLC**	<0.025	0.025<CLC<0.03	0.025<CLC<0.03	CLC>0.04
4	Cracks (PS concrete-mm)	**CR**	<0.05mm	0.05-0.3mm	0.3-1mm	1-3mm
5	Concrete Resistivity (KΩcm)	**R**	>20	10< R<20	5<R<10	R<5
6	Half - Cell (mV)(Cu/$CuSO_4$)	**HC**	0-220	220-350	350-450	>450
7	Corrosion Rate (Icorr, LPR)	**CORR**	<0.1	0.1-0.5	0.5-1	>1
8	Ømm (Rebar Diameter)	**D**	>18mm	18mm-16mm	16mm-12mm	<12mm
9	% rebar mass loss	**RD**	0-2	2-5	5-10	>10
10	Concrete Moisture Content ,EMC (%)	**MST**	<2	2-4	4-6	>6
11	Concrete Strength	**CS**	≥C25/30	C20/25	C16/20	C12/15
12	Tendon Corrosion	**TC**	Low	Moderate	High	Possible Failure

Damage Index (Chlorides).

Potential of Corrosion can be identified through Half Cell readings close to tendon Duct (Metallic). Further investigation should be carried out regarding the grouting chloride content by sampling (Specified No of samples according to tendons No).Damage Index due to chloride presence $DI_{(Cl)}$ is defined for different chloride levels.

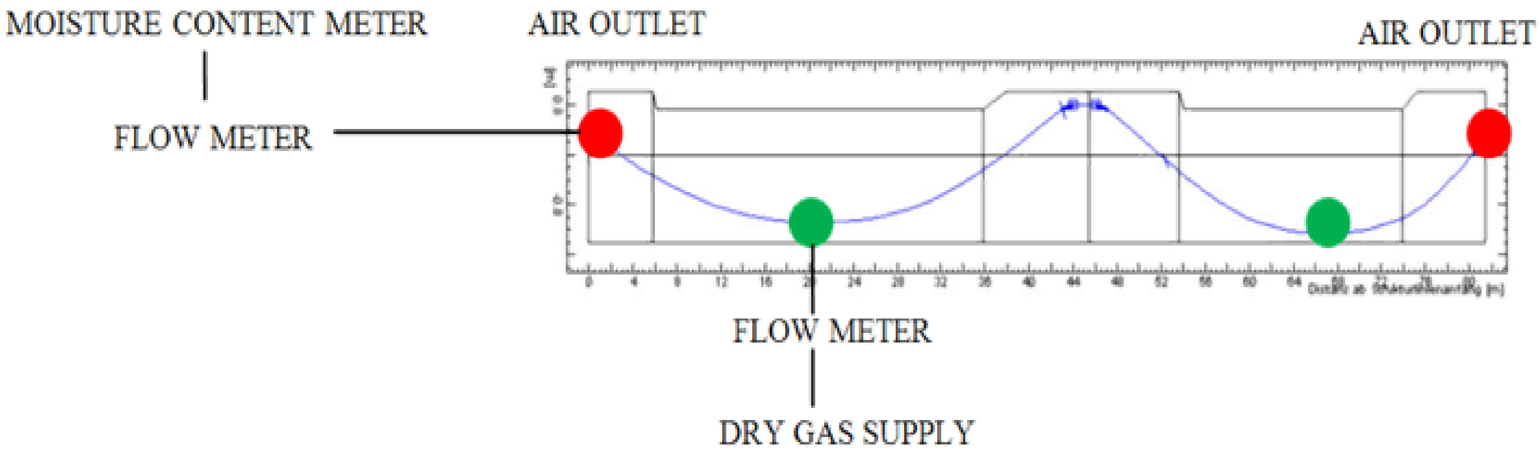
Chlorides (% w.t cement grout)	<0.08	0.08<Cl≤0.20	0.2<Cl≤0.40	Cl>0.40
Damage Index $DI_{(Cl)}$	1	2	3	4

Additional Tendon Inspection.

The second inspection of Tendons is related to grouting condition (voids, moisture), where a Damage Index defines the combination of possible voids and humidity (presence of water inside ducts). Hence, an inlet is placed on the lower point of tendon profile and two outlets at the upper points at equal distances. A Dry Nitrogen gas (0% RH) is supplied from a nozzle at the lower point, while a flow meter records the flow inside the ducts. Gas moves upwards towards outlets, where a flow and gas moisture content is recorded. The following figure presents the procedure should followed in order to assess internal tendon moisture and voids

Damage Index (Tendon Condition).

$DI_{(PT)}$ Evaluation Scale	Measure	Condition	Potential for Corrosion
1	No Air Flow	No Air Flow	Unknown
2	<0.3%	Dry	Low
3	0.3 -0.7 %	Moist	Moderate
4	>0.7%	Wet	High



Evaluation of Final Damage Index TC.

The average of Damage Index $DI_{(Cl)}$ and Damage Index $DI_{(PT)}$, gives the Damage Index of Tendon Corrosion $DI_{(TC)}$.

$$DI_{(TC)} = \frac{DI_{(Cl)} + DI_{(PT)}}{2}$$

Tendon Corrosion (TC)	Damage Index (TC)
Low	1
Moderate	2
High	3
Possible Failure	4

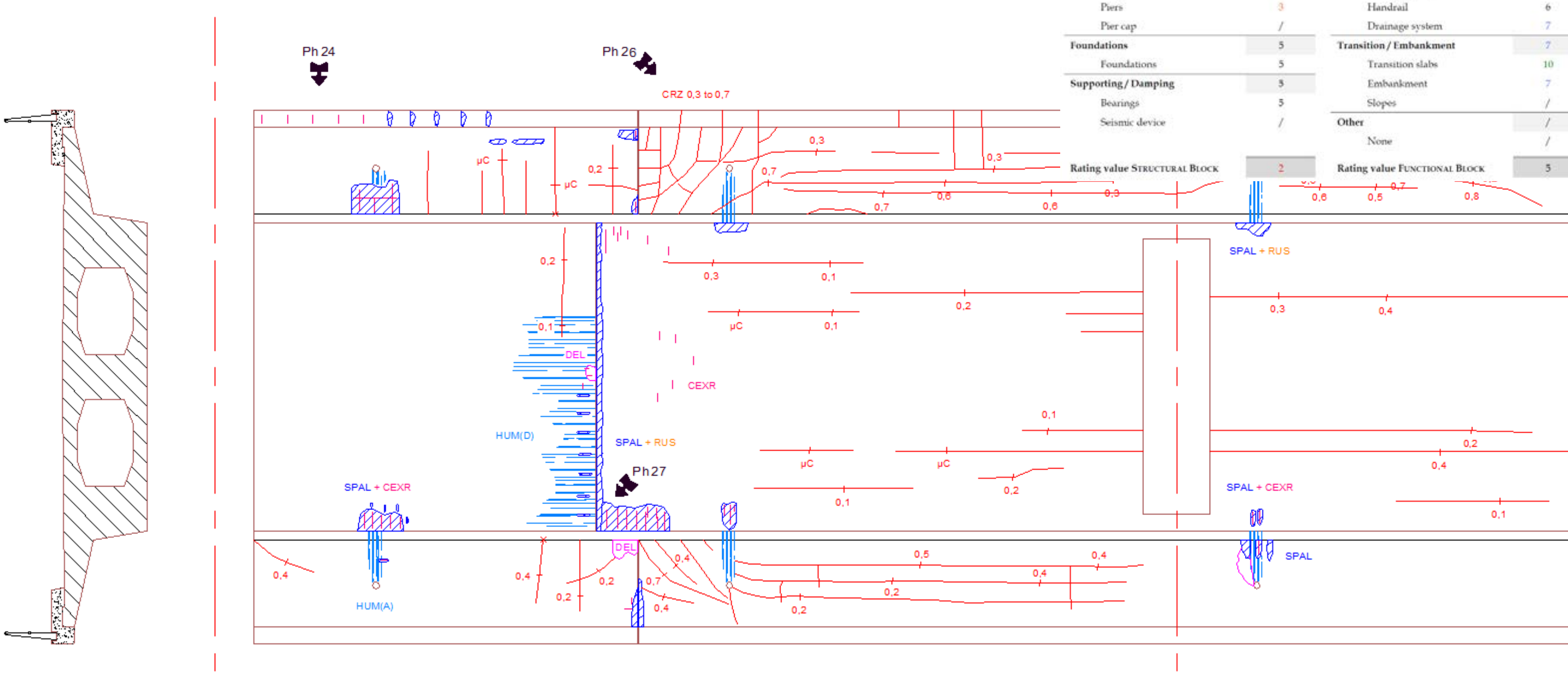
Condition Rating – 9 Scale CR.

Condition	#	Condition Description	EXPERT
As-built	10	*As-built condition*	
Excellent	9	*No problems noted or insignificance deterioration*	
Very Good	8	*Very good condition with light deteriorations*	
Good	7	*First signs of aging*	
Satisfactory	6	*Structure sound but with deficiencies*	
Fair	5	*Structure aging. Moderate deteriorations*	
Poor	4	*Advanced deterioration*	X
Poor	3	*Advanced deterioration. Eventual influence in structural condition*	X
Very poor	2	*Severe condition with important deteriorations*	X
Failed	1	*Failed*	X

A. Structural Block	
Components groups / components	*Rating value*
Superstructure	2
Box Girder	2
Substructure	3
Multicolumn abutment	5
Wingwalls	6
Piers	3
Pier cap	/
Foundations	5
Foundations	5
Supporting / Damping	5
Bearings	5
Seismic device	/
Rating value Structural Block	2

B. Functional Block	
Components groups / components	*Rating value*
Road surface	5
Pavement	7
Expansion joints	5
Sidewalk	10
Railing / parapets	6
Handrail	6
Drainage system	7
Transition / Embankment	7
Transition slabs	10
Embankment	7
Slopes	/
Other	/
None	/
Rating value Functional Block	5

Damage Mapping – Condition Rating.



A. Structural Block	
Components groups / components	*Rating value*
Superstructure	2
Box Girder	2
Substructure	3
Multicolumn abutment	5
Wingwalls	6
Piers	3
Pier cap	/
Foundations	5
Foundations	5
Supporting / Damping	5
Bearings	5
Seismic device	/
Rating value Structural Block	2

B. Functional Block	
Components groups / components	*Rating value*
Road surface	5
Pavement	7
Expansion joints	5
Sidewalk	10
Railing / parapets	6
Handrail	6
Drainage system	7
Transition / Embankment	7
Transition slabs	10
Embankment	7
Slopes	/
Other	/
None	/
Rating value Functional Block	5

Case Study 1.

Initial (SR): 2 (Assessment)
Very poor condition.
Further Assessment.

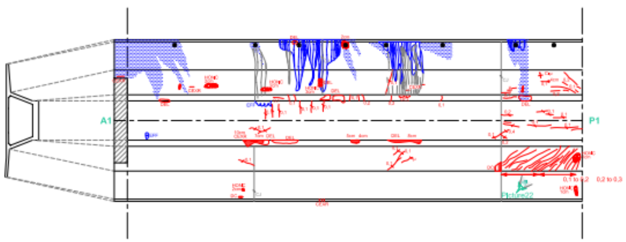
CDI	1.20
EA	1.00
CI	1.10
α R	0.40
B	1.00
β	4.5
VR	0.14
rreff	0.78

Calibrated (SR): 7
B = 1, r_{eff} = 0.78
Bridge is safe for operation

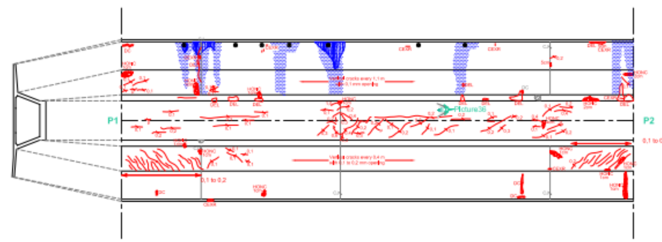
Total Damage Level– Low– SCI 1.15 - **α_R=0.4**

	Damage Level	Damage Description	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

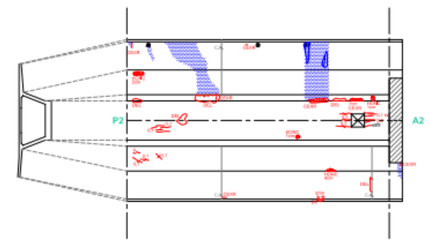
Case Study 1- Damage Mapping.



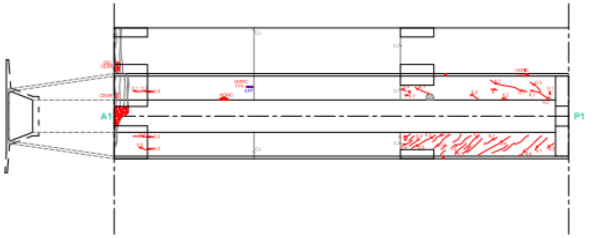
BOTTOM VIEW SP1



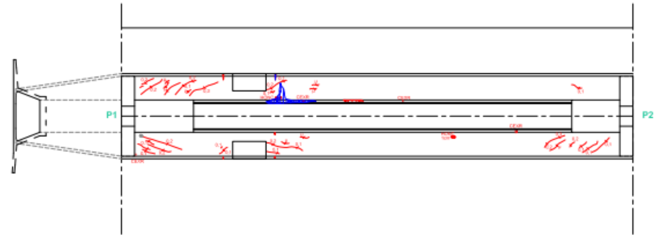
BOTTOM VIEW SP2



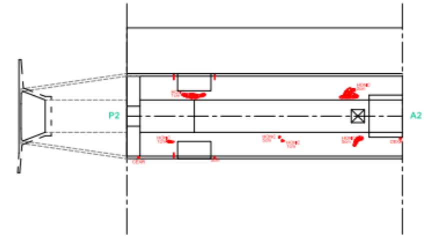
BOTTOM VIEW SP3



INSIDE VIEW SP1



INSIDE VIEW SP2



INSIDE VIEW SP3

Case Study 2.

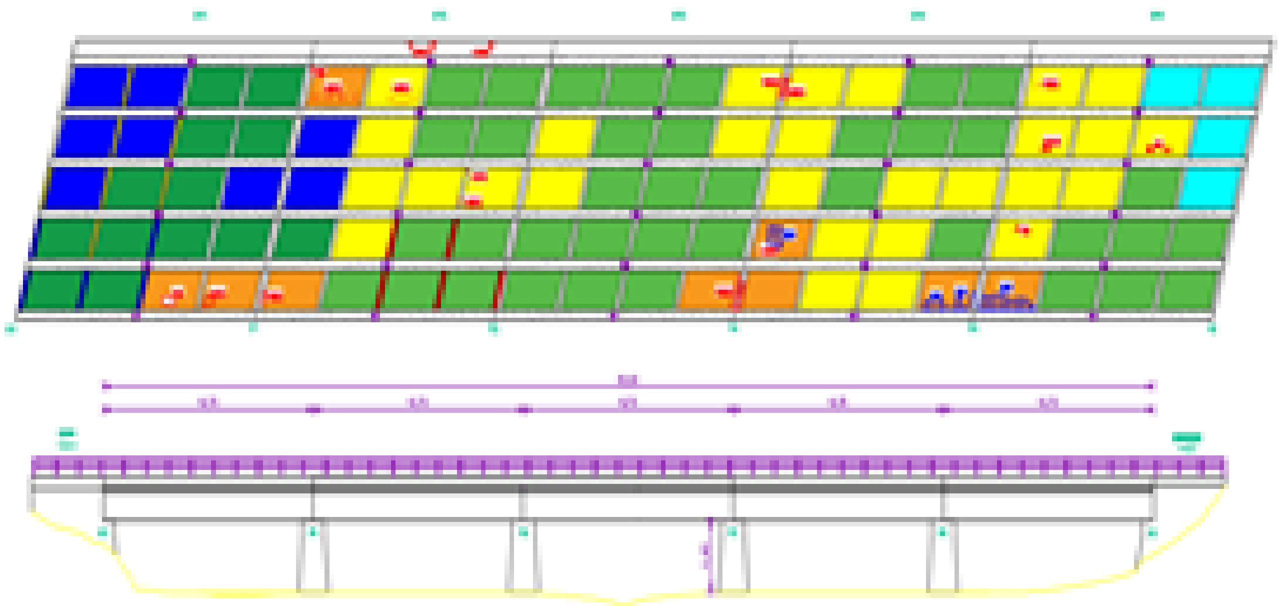
Initial (SR): 3 (Assessment)
Very poor condition.
Further Assessment.

Calibrated (SR): 5
B = 1.0, r_{eff} = 0.68
Retrofitting of specific members

Total Damage Level– Medium– SCI 1.65 - **α_R=0.5**

	Damage Level	Damage Description	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

Case Study 2 – Damage Mapping



	Damage Level	Damage Description	SDI	a_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

Case Study 3.

Initial (SR): 1 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

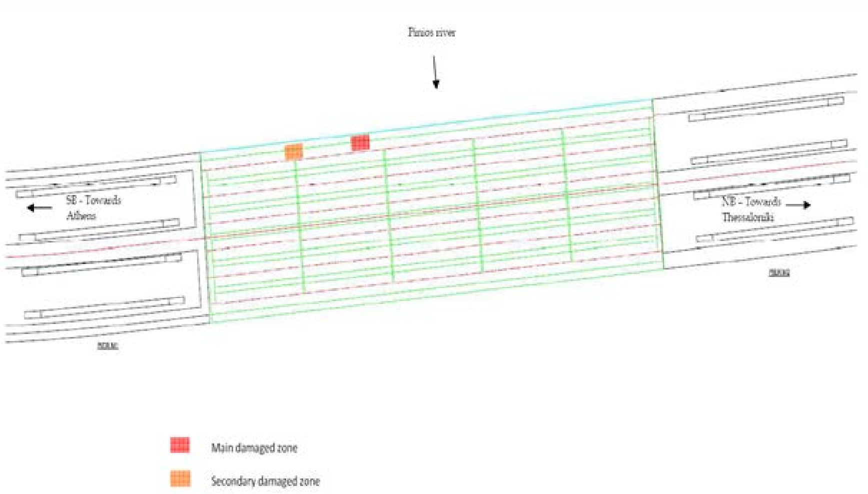
Condition	#
As-built	10
Excellent	9
Very Good	8
Good	7
Satisfactory	6
Fair	5
Poor	4
	3
Very poor	2
Failed	1

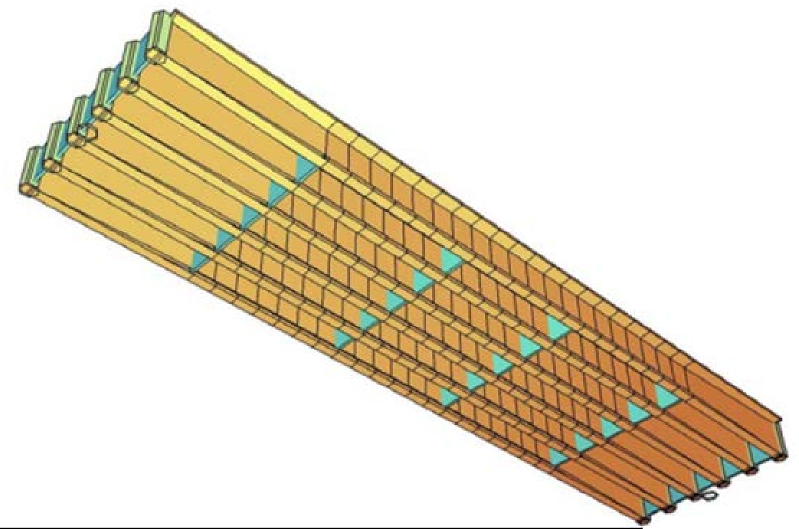
Total Damage Level– High– SCI 2.45 - **α_R=0.6**

Calibrated (SR): 5
B = 1.0, r_{eff} = 0.62
Bridge Safe for operation (traffic restrictions)

	Damage Level	Damage Description	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

Case Study 3 – Structural Evaluation.





Safety Factors according to EN 1990/EN 1992-2						
Capacity R	20006	18803	18803	18803	18803	16087
Required (DIN 1072 Loads)	19231	18563	17139	16517	15807	15301
UF (S/R)	0.96	0.99	0.91	0.88	0.84	0.95
reff	0.62	0.88	0.82	0.84	0.86	0.88
Remain Capacity R'	12403.72	16546.64	15418.46	15794.52	16170.58	14156.56
Updated UF	1.55	1.12	1.11	1.05	0.98	1.08
Required(Military Vehicles)	17240.00	16615.00	15448.00	15497.00	15430.00	15353.00
UF (S/R)	0.86	0.88	0.82	0.82	0.82	0.95
Updated Uf	1.39	1.00	1.00	0.98	0.95	1.08

Case Study 4.

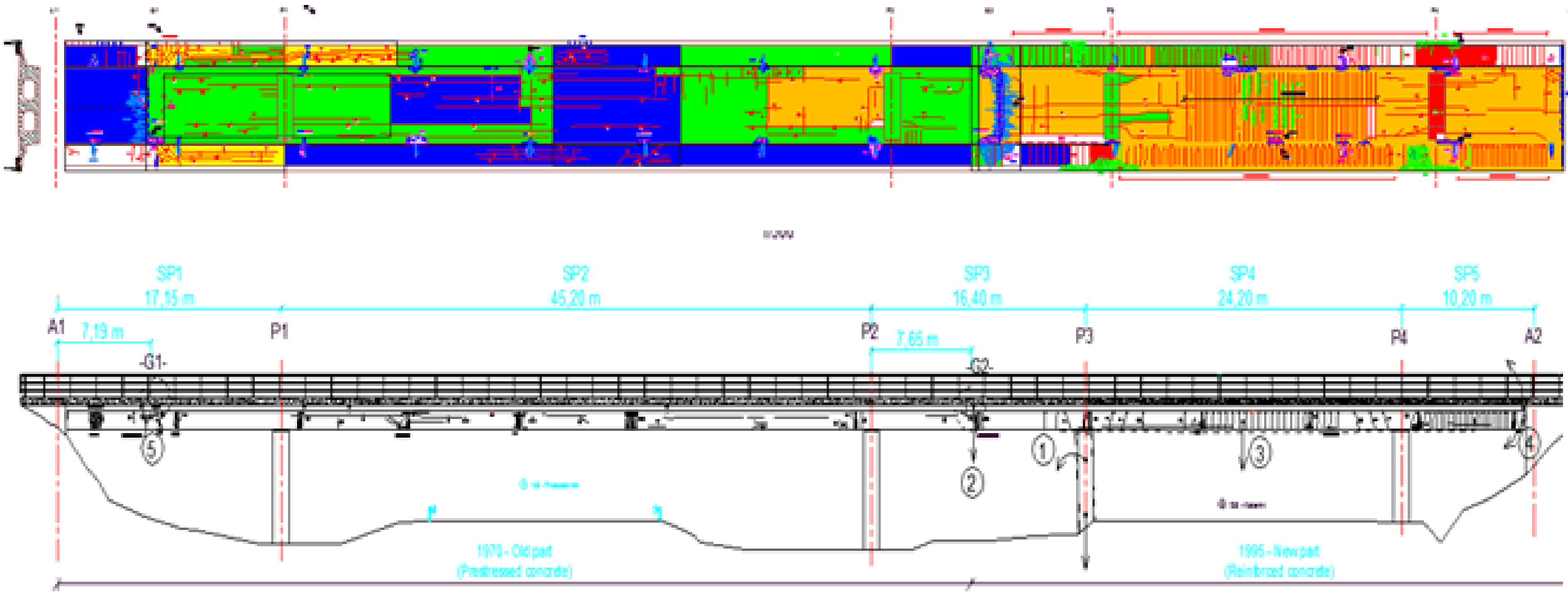
Initial (SR):2 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

Condition	#
As-built	10
Excellent	9
Very Good	8
Good	7
Satisfactory	6
Fair	5
Poor	4
	3
Very poor	2
Failed	1

Calibrated (SR): 4
$B = 1.0$, $r_{eff} = 0.65$
Bridge Safe for Class 30.
Retrofitting of Bridge

Total Damage Level– High– SCI 2.20 - **α_R=0.6**

Case Study 4 – Damage Mapping.



	Damage Level	Damage Description	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

Case Study 5.

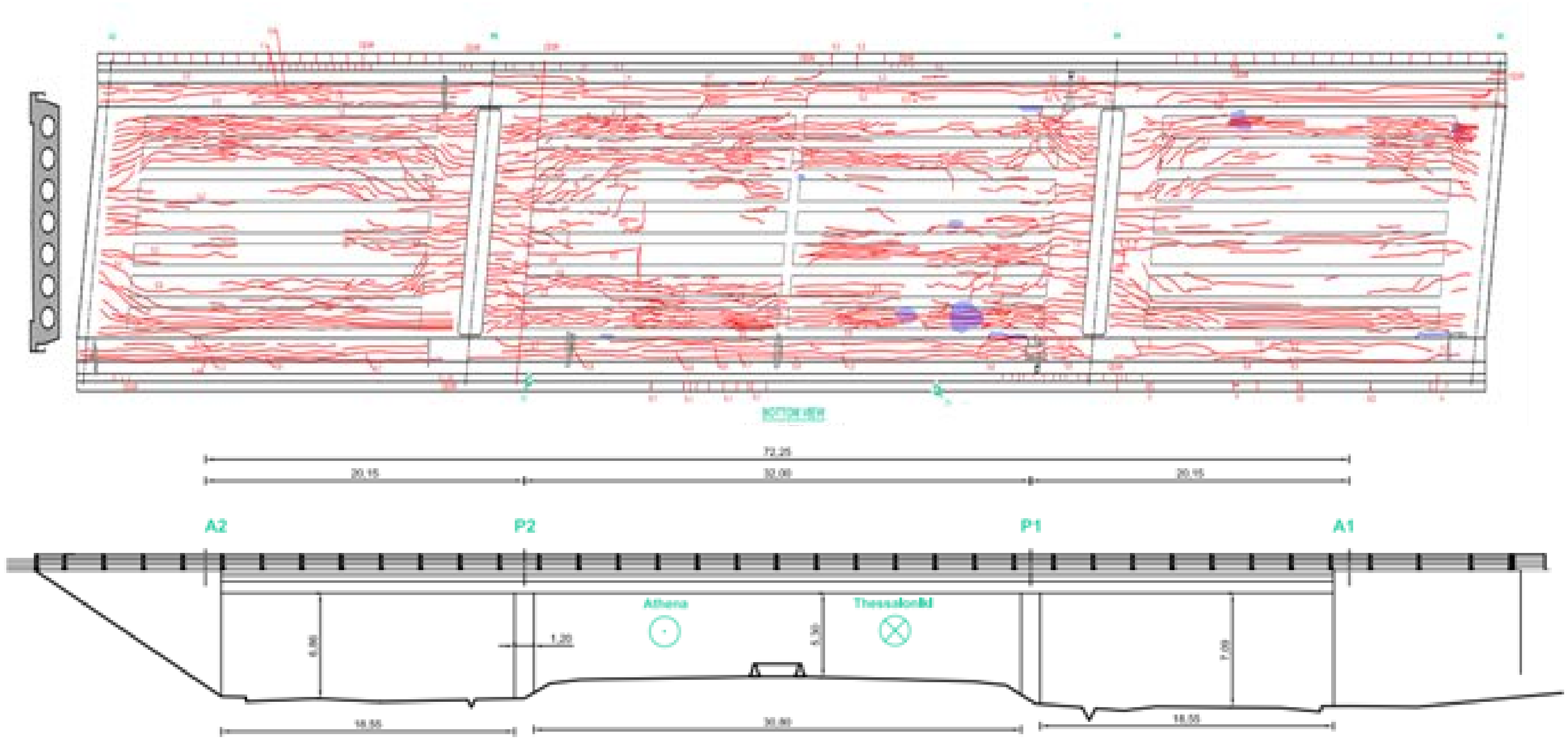
Initial (SR): 3 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

Condition	#
As-built	10
Excellent	9
Very Good	8
Good	7
Satisfactory	6
Fair	5
Poor	4
	3
Very poor	2
Failed	1

Calibrated (SR): 6
B = 1.0, r_{eff} = 0.70
Bridge Safe for operation
Repair AAR cracks

Total Damage Level– Medium– SCI 1.70 - $\boldsymbol{\alpha_R}$**=0.5**

Case Study 5 – Damage Mapping.



	Damage Level	Damage Description	SDI	a_R
	I	No Damage	0-0.65	0.3
	II	Low Damage	0.65-1.20	0.4
	III	Medium Damage	1.20-1.90	0.5
	IV	High Damage	1.90-2.55	0.6
	V	Very High Damage	2.55-3.5	0.7
	VI	Critical Damage	>3.5	0.8

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Integrating multivariate techniques in bridge management systems for life-cycle prediction

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UCC University College Cork, Ireland Coláiste na hOllscoile Corcaigh

UCD DUBLIN

20th – 21st October 2016

Delft, Netherlands

OVERVIEW

- BMS retains large quantities of data and metadata
  - Data – condition states through visual inspection
  - Metadata – crossing type, structural form, age (sometimes), etc.
- Possible to use multivariate data analysis techniques to analyse contents of bridge management systems (BMS)
  - "Big data analysis"
- Principal component analysis (PCA) is an effective data-reduction technique
  - Can define latent variables/underlying data structure
- Integration to existing BMS for improved asset base description

INTRODUCTION

- Aging, deteriorating bridge stock
  - Post-WW2 construction
  - Popular practice of “deferred maintenance”
- Increasing maintenance demands, less available funding
- Modern BMSs are a popular method to track and plan maintenance activities
  - Inventory (metadata)
  - Inspection (data)
  - Assessment
  - Maintenance
  - Financial

INTRODUCTION

- Large amounts of visual inspection data
  - Condition rating of individual elements
  - Overall based on worst condition primary structural element
- Simple to compare small number of bridges
  - Problem for larger data-sets
- “BIG DATA analysis”
  - Pattern extraction
- Data reduction techniques for latent variable structure
  - Principal component analysis
  - Latent variable – unobservable
    - Classic example: conflict prediction (turmoil, revolution, subversion)

MULTIVARIATE ANALYSIS OF BMS: Description of Dataset

- *Infraestruturas de Portugal*
  - Condition rating data for **3,036** bridges
  - 1,667 single span reinforced concrete
  - 713 single span masonry arch

Condition Rating	Description
0	No damage
1	Minor damage
2	Some damage
3	Significant damage
4	Critical damage
5	Ultimate damage

- Primary elements
  - Structural – deck, abutments, retaining/wing walls
  - Non-structural – surface, barriers, embankment

MULTIVARIATE ANALYSIS OF BMS: Assessment Methods

- Principal component analysis (PCA) is a multivariate analysis technique, the primary purpose of which is to reduce the dimensionality of a set of data

- Redefine the input variables as principal components (PC) – a linear combination of the original variables, but having a magnitude less than the original data set, but while preserving most of the information

- Highlighting the variables that demonstrate the most variance in the data set

MULTIVARIATE ANALYSIS OF BMS: Assessment Methods

- The first principal component Y_1 is defined as:

$$Y_1 = \boldsymbol{\alpha}'_1 \mathbf{x} = \alpha_{11}x_1 + \alpha_{12}x_2 + \ldots + \alpha_{1p}x_p = \sum_{j=1}^{p} \alpha_{1j}x_j$$

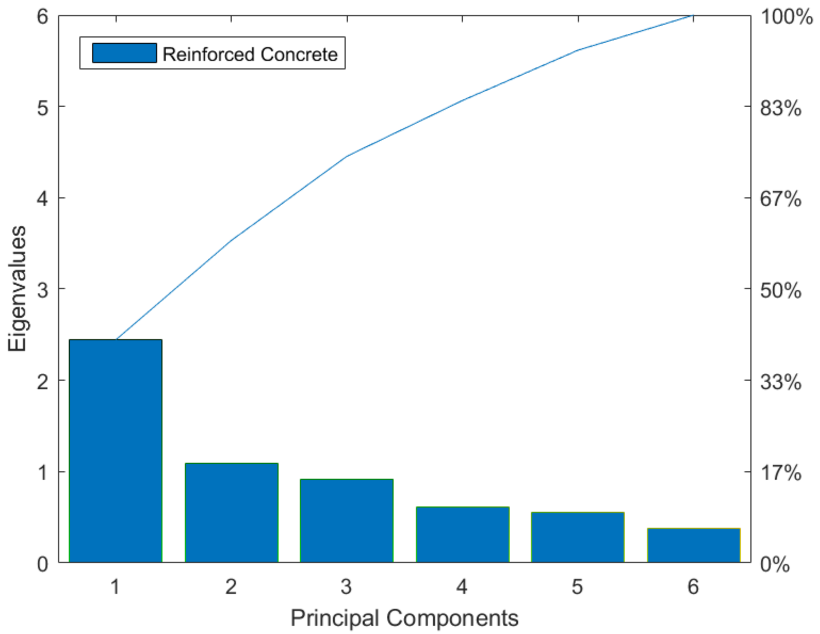
- The sum of the square of the coefficients α_i is equal to unity, and is a better indicator of the influence the coefficient has than the raw value:

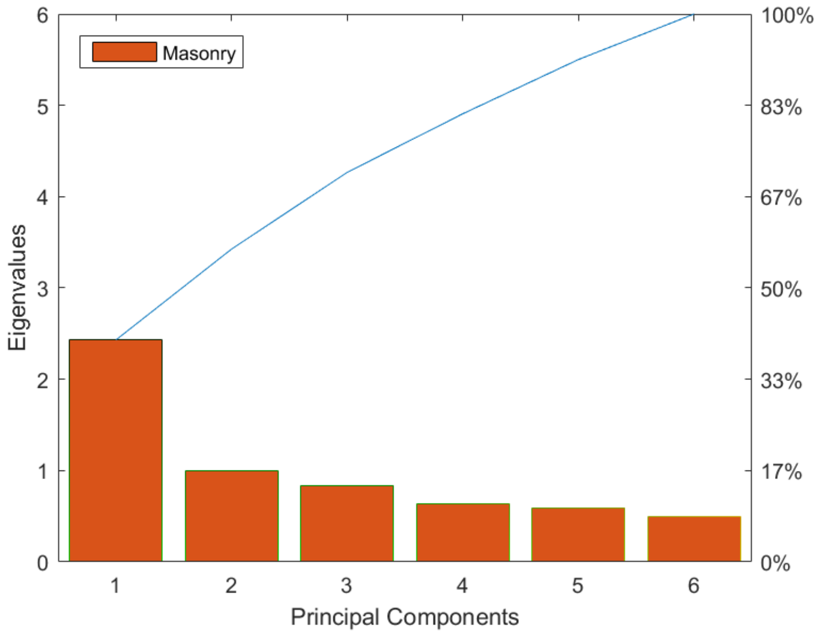
$$\sum_{i=1}^{p} \alpha_i^2 = \alpha'\alpha = 1$$

- The first principal component is the direction along which the data-set shows the largest variation
- The second PC is determined under the constraint of being orthogonal to the first PC and to have the largest variance

RESULTS: Number of PCs to Retain

- Determine the number of PCs to retain with scree plot of the eigenvalues
- Eliminate PCs with eigenvalues less than unity

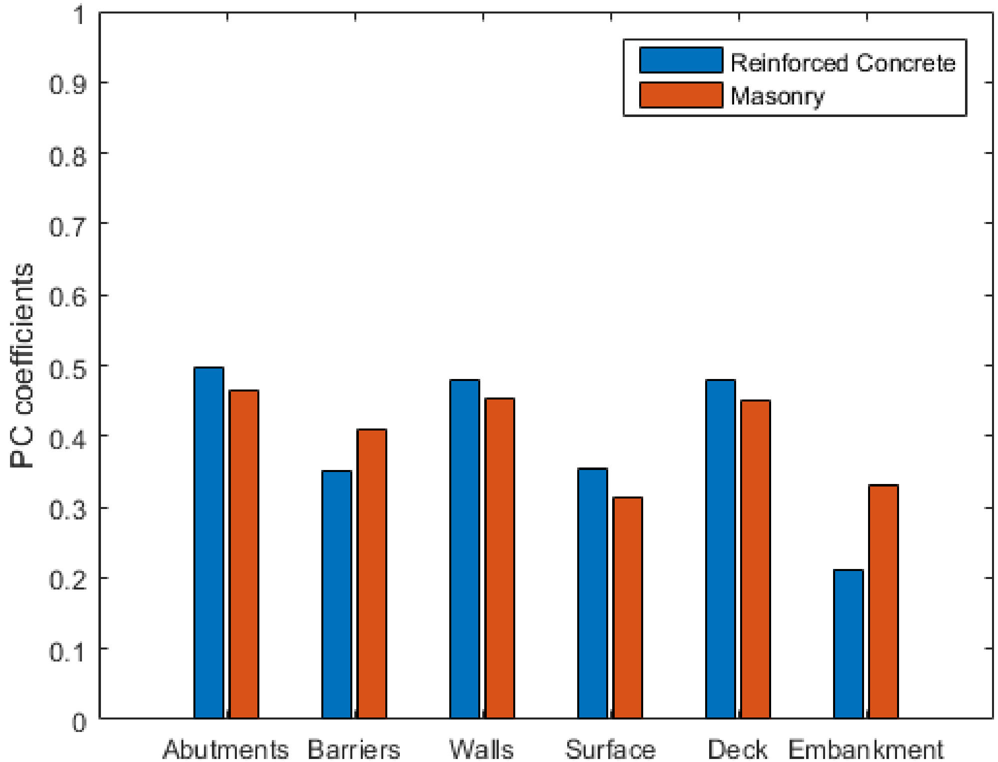




- Retaining: 1 PC (~41%), 2 PCs (~58%), 3 PCs (~72%)

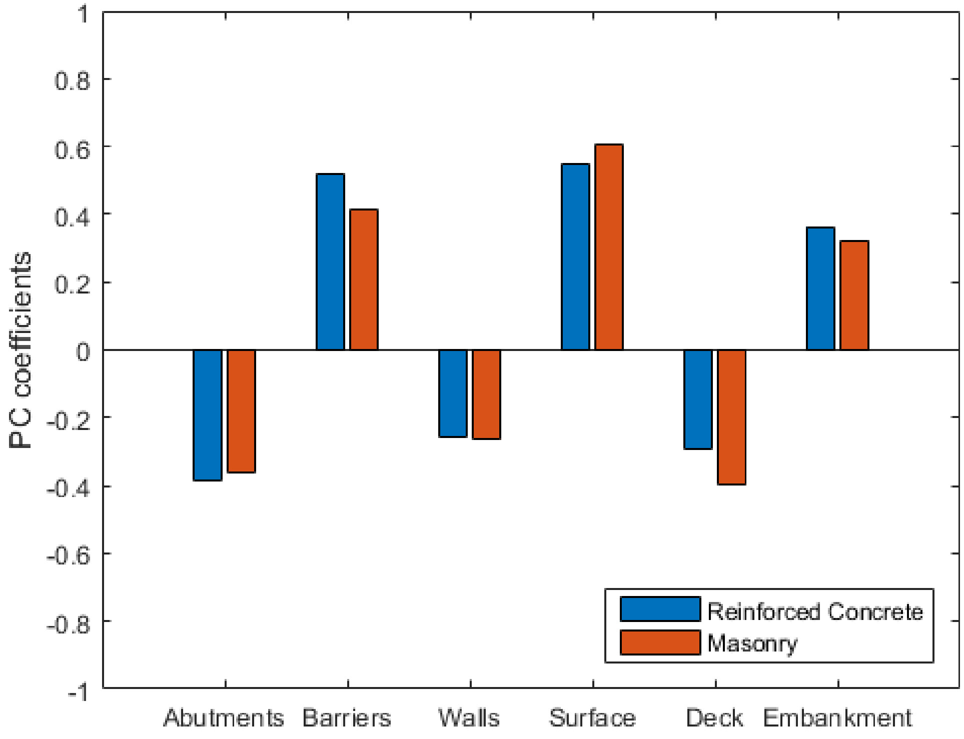
RESULTS: First Principal Component Coefficient, α_{1i}

- Indicate the relationship between original variables and new latent variables
- For this BMS, positive coefficients indicate unfavourable state, negati[ve] indicate favourable state
- 1st PC has all positive coefficients, indicating unfavourable state in each variable

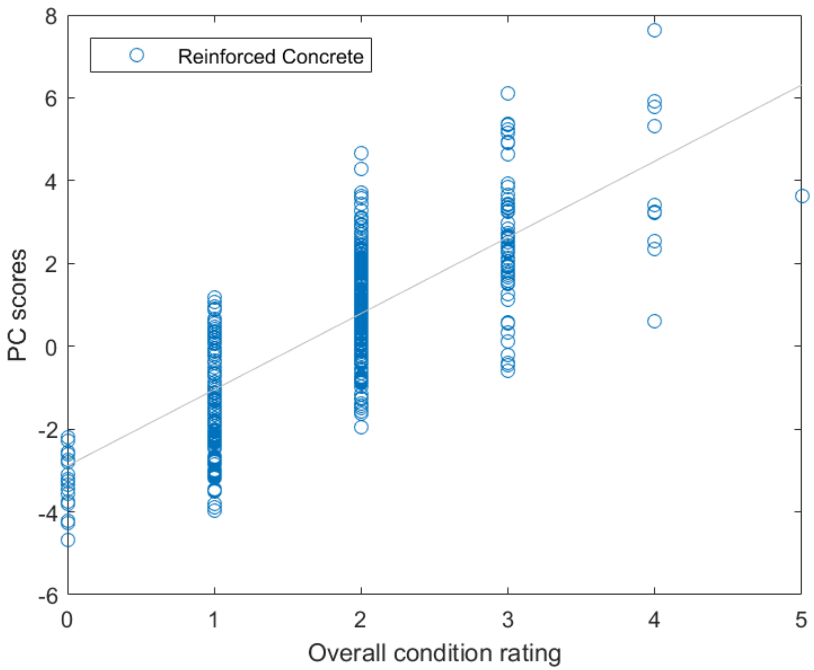


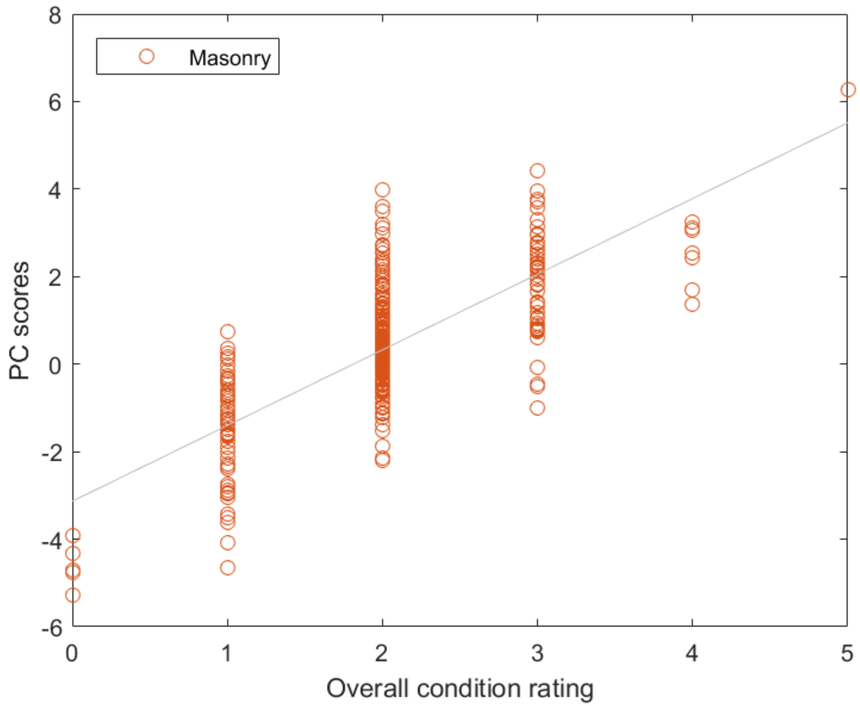
RESULTS: First Principal Component Coefficient, α_{2i}

- 2^{nd} PC has mix of positive and negative coefficients, indicating disconnect between states for groups of variables
- Describes bridges where the structural and non-structural elements are in opposing states



RESULTS: First Principal Component Score, Y_1





BMS INTEGRATION: Weighted Condition Ratings

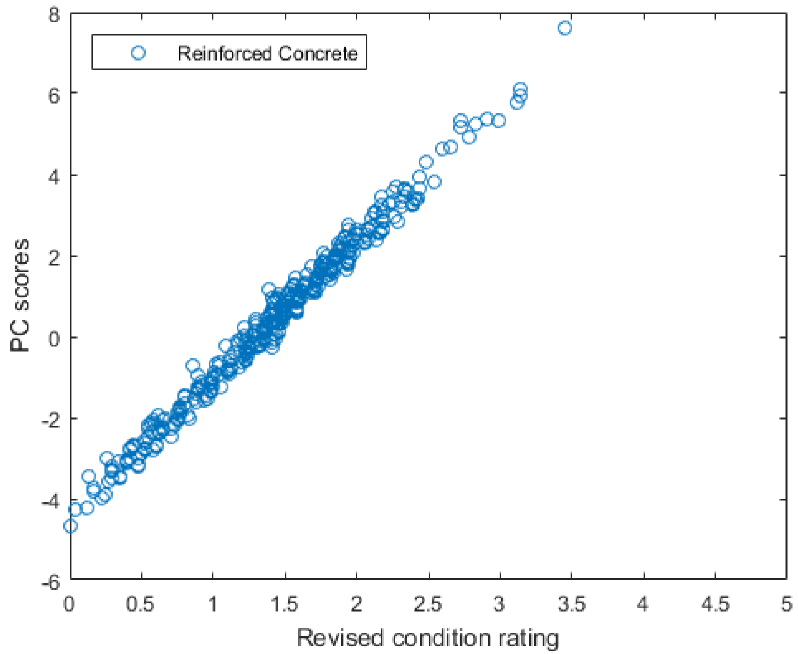
- Scope for improved overall condition rating
  - Weighted linear combination
- PC coefficients as first point guides

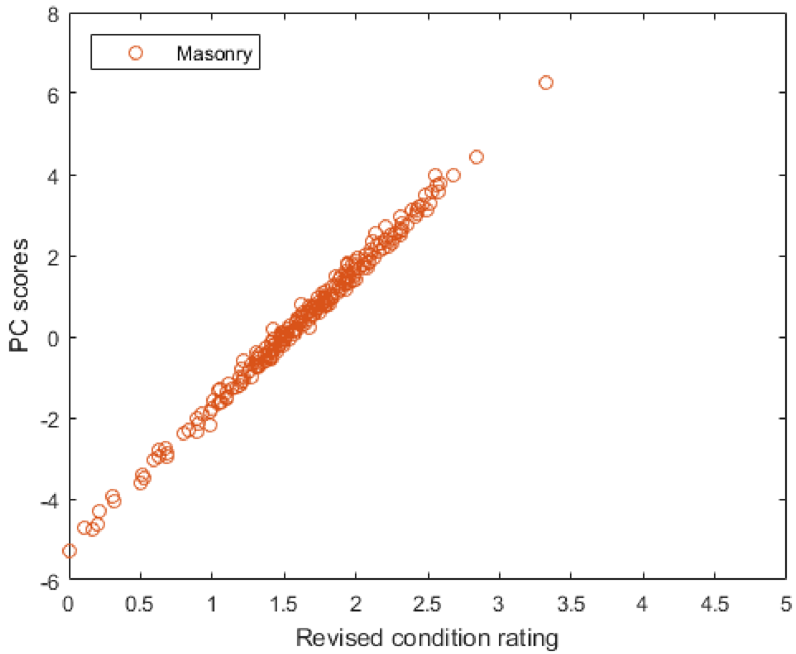
$$\zeta = \sum_{j=1}^{p} \lambda_j x_j, \quad \lambda_j = \alpha_{1,j}^2$$

- Where ζ is a linear combination of the new weighting factors λ_j and the original condition ratings x_j for the individual elements

Element	Reinforced concrete	Masonry
Abutments	0.2466	0.2166
Barriers	0.1238	0.1669
Walls	0.2290	0.2043
Surface	0.1255	0.0989
Deck	0.2300	0.2033
Embankment	0.0451	0.1099
Σ	1.0000	1.0000

BMS INTEGRATION: Weighted Condition Ratings





CONCLUSIONS

- PCA demonstrated commonality in latent structure between reinforced concrete and masonry arch bridges
  - Variances enough to caution against untargeted application
- Y1 is a good indicator of overall bridge condition state
  - Better descriptor than existing model
- Data extracted weighting factors good first point for BMS integration
  - Refinements based on current experience and knowledge base
- Further studies needed on larger, more diverse asset bases needed for reliable model
  - Region specific factors
  - Structure specific factors

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Reliability of existing bridges determined with chloride induced corrosion

Ivan Zambon, Anja Vidović and Alfred Strauss
University of Natural Resources and Life Sciences, Vienna

20th – 21st October 2016
Delft, Netherlands

CONTENT

1. Introduction
2. Chloride ingress in concrete structures
3. Material parameters identification
    1. Cement types
    2. Concrete classes
    3. Concrete cover
4. Environmental parameters identification
    1. Chloride content
    2. Temperature
5. Benchmarking of deemed-to-satisfy provisions in Austria
    1. Development of reliability indices
    2. Target reliabilities
    3. Ages of inspection
    4. Sensitivity study
6. Conclusions

INTRODUCTION

Condition assessment

A process of reviewing information gathered about the current condition of a structure or its components, its service environment and general circumstances, allowing a prognosis to be made of current and future performance

Prediction models

- mathematical (statistical)
- empirical
- physical models

<u>Chloride ingress</u>

Carbonation

Corrosion

Freeze thaw

Alkali-silica reaction …

Levels of information:

Monitoring

Testing level

Engineering level

CHLORIDE INGRESS

$$g(c,t) = C_{crit} - C(c,t)$$

$$C_x(x,t) = C_0 + \left(C_{S,\Delta x} - C_0\right) \cdot \left[1 - erf \frac{x - \Delta x}{2 \cdot \sqrt{k_e \cdot D_{RCM,0} \cdot \left(\frac{t_0}{t}\right)^{\alpha} \cdot t}}\right]$$

$$k_e = exp\left[b_e\left(\frac{1}{T_{ref}} - \frac{1}{T_{real}}\right)\right]$$

where:

C_0	initial chloride content;
$C_{S,\Delta x}$	chloride content at depth Δx;
x	depth with a corresponding content of chlorides ($x = c_{nom}$ – depth of reinforcement);
Δx	depth of convection zone;
k_e	environmental variable (considers temperature);
$D_{RCM,0}$	apparent diffusion coefficient (rapid chloride migration test);
t_0	reference point of time;
α	ageing exponent;
b_e	temperature coefficient (proportional to activation energy) [K]
T_{ref}	reference temperature [K]
T_{real}	temperature of the structural element or the ambient air [K]

MATERIAL PARAMETERS – CEMENT TYPES

		- 1993	1994	1994 - 1980	1980 - 1963	$D_{RCM,0}$ [ND (μ,σ=0.2*μ)]						α [BD (μ, σ, a=0, b=1)]
						0,35	0,4	0,45	0,5	0,55	0,6	
CEM I	Portland cement	CEM I	PZ	PZ	-	-	8,9	10,0	15,8	19,7	25,0	0,30 / 0,12
CEM II	Portland-slag cement	CEM II/A-S	PZ (H)	PZ (H)	PZ (H)	-	7,0	8,0	-	-	-	0,35 / 0,16
		CEM II/B-S	EPZ	EPZ	EPZ	-	5,0	7,7	8,3	-	-	-
	Portland-fly ash cement	CEM II/A-V	PZ (F)	PZ (F)	PZ (F)	5,6	6,9	9,0	10,9	14,9	-	0,60 / 0,15
		CEMII/A-W	PZ (F)	PZ (F)	PZ (F)	-	-	-	-	-	-	-
	Portland-limestone cement	CEM II/A-L	PZ (K)	PZ (K)	PZ (T)	-	-	-	-	-	-	-
		CEM II/A-LL	PZ (K)	PZ (K)	PZ (T)	-	9,4	12,8	15,1	-	-	0,30 / 0,12
	Portland-composite cement	CEM II/A-M	PZ (C)	PZ (C)	PZ (C)	-	-	-	-	-	-	-
CEM III	Blastfurnace cement	CEM III/B	HOZ	HOZ	HOZ	-	1,4	1,9	2,8	3,0	3,4	0,45 / 0,20

- 1993	1994 - 1963
Class	Class
-	Z 275
32,5 L (CEM III)	Z 375
32,5 N	Z 375
32,5 R	Z 375
42,5 L (CEM III)	Z 475
42,5 N	Z 475
42,5 R	Z 475
52,5 L (CEM III)	-
52,5 N	-
52,5 R	-

	w/c ratio per cement type					
Concrete class	Z 275		Z 375		Z 475	
B 160	0,74	0,92	0,87	1,03	-	-
B 225	0,61	0,75	0,71	0,83	0,77	0,88
B 300	0,51	0,64	0,61	0,71	0,66	0,74
B 400	0,32	0,54	0,51	0,61	0,57	0,64
B 500	-	-	0,39	0,52	0,48	0,56
B 600	-	-	0,32	0,43	0,37	0,48

MATERIAL PARAMETERS – CONCRETE CLASSES

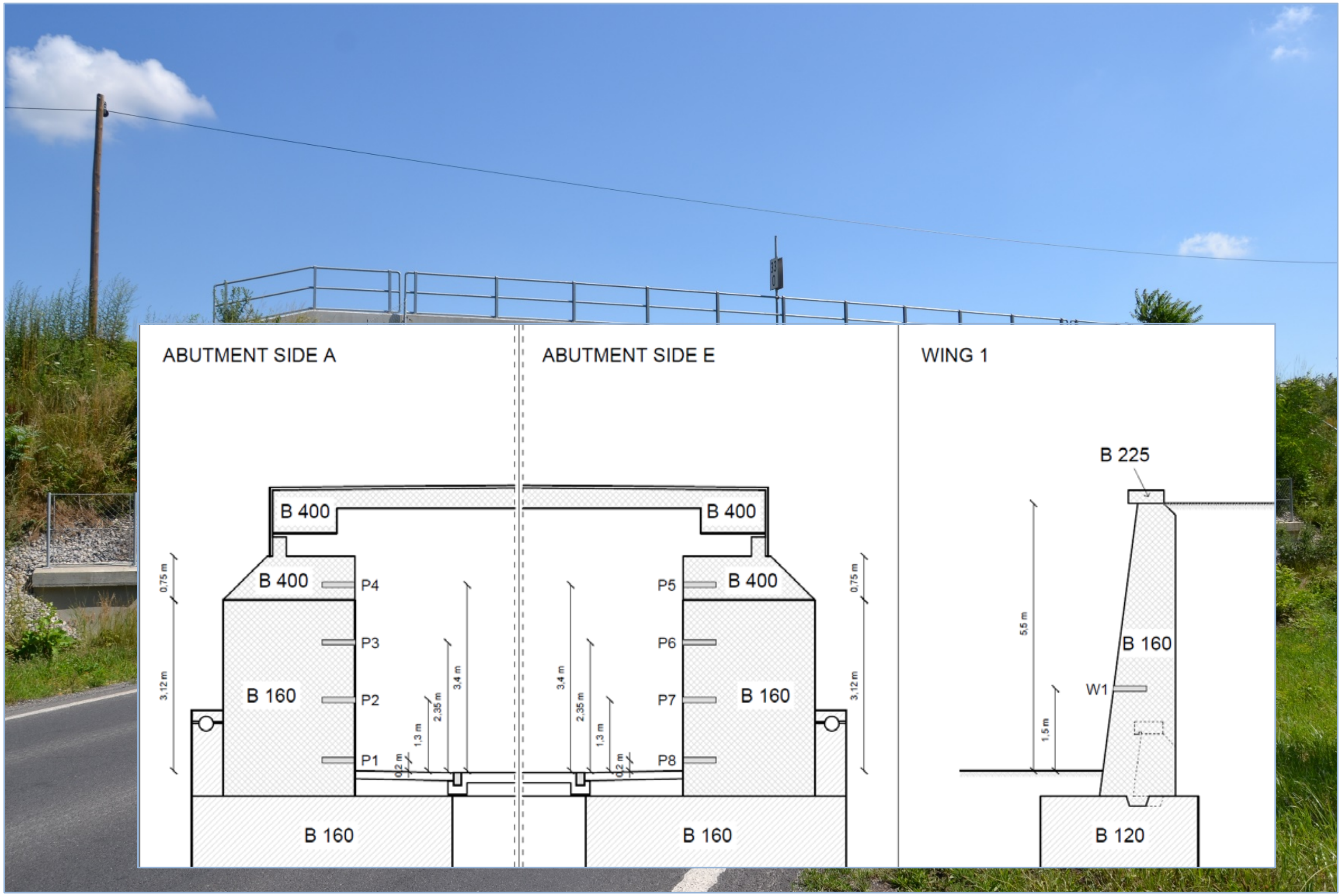
Determining the w/c ratio through connection of concrete class with cement strength classes

- 2002	2002 - 1954
-	B 0
-	B 50
-	B 80
C8/10	B 120
C12/15	B 160
C16/20	B 225
C20/25	B 300
C25/30	-
C30/37	B 400
C35/45	-
C40/50	B 500
C45/55	-
C50/60	B 600
C55/67	-
C60/75	-
C70/85	-
C80/95	-
C90/105	-
C100/115	-

- Historically used cement strength classes Z275, Z375 and Z475
- Historically used concrete classes B160, B225, B300 … B600
- Chosen w/c ratios 0.4, 0.45, 0.5, 0.6, 0.7 and 0.8

Usage and demands	Concrete class
Blinding concrete for reinforced concrete foundations, strength verification is not required	B 0
Unreinforced foundations	B 225
Basement walls of reinforced concrete in accordance with ÖNORM B4200 - Part 3	B 160
Bridges piers, standing on the roadside, with the influence of thaw	B 300
Bridge structures, exposed concrete, structures where resistant to frost is needed, …	B 400
Bridge cornices, visible concrete, concrete with freeze-thaw effect	B 300
Precast ceiling of residential building	B 225
Reinforced concrete ceiling, consisting of slabs, beams and joists, and columns	B 300
Water tanks	B 225
Retaining walls, unreinforced (static required)	B 160
Reinforced concrete foundations	B 225
…	…

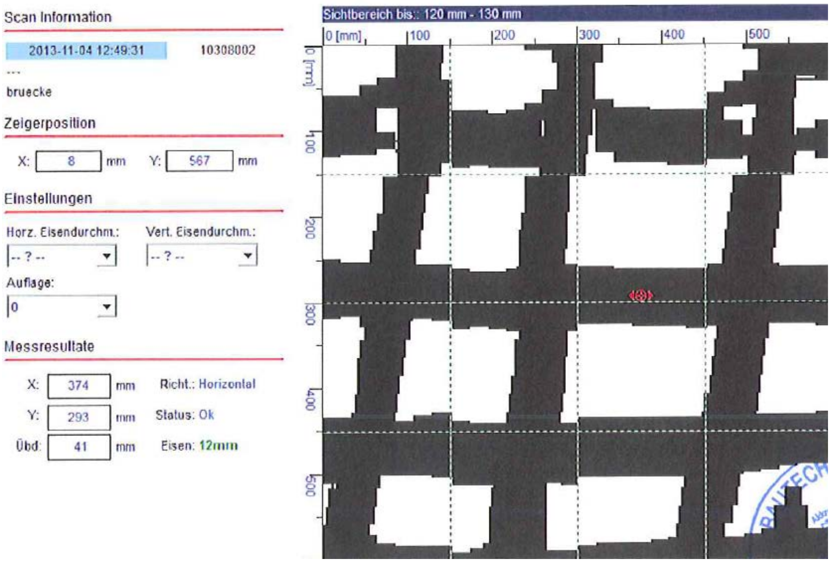
MATERIAL PARAMETERS – CONCRETE CLASSES



MATERIAL PARAMETERS – CONCRETE CLASSES

- mean was variated
- μ=(30, 35, 40, 45, 50)
- σ used σ=6 mm

Normal distribution
Beta-distribution
Weibull (min)-distribution
Lognormal distribution
Neville distribution

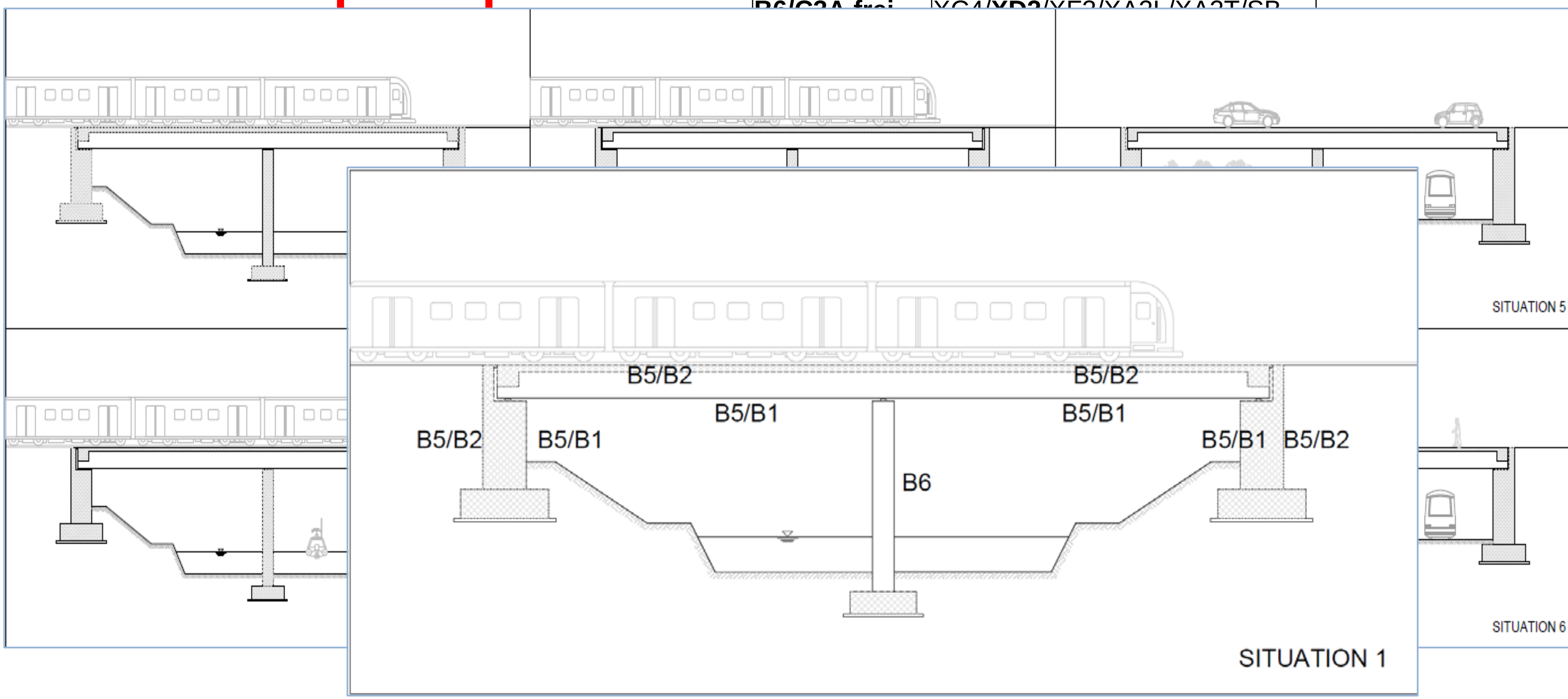


Component or demands	Concrete cover [mm]
Plates and ribbed ceilings inside buildings	15 mm
Traffic structures from in situ concrete	25 mm
In all other cases	20 mm
In freeze-thaw attack, **chloride attack**, strong and very strong chemical attack	**35 mm**
In underwater concrete	50 mm
In underwater concrete, visible surfaces	60 mm
Bulky components	50 mm

ENVIRONMENTAL PARAMETERS - EXPOSURE

Exposure class	$C_{S,0}$ / $C_{S,\Delta x}$ [wt.-/b] LND		
XD1	0.5 ≤ μ ≤ 1.5	**μ = 1.5**	CoV = 0.75
XD2	2.0 ≤ μ ≤ 4.0	**μ = 3.0**	CoV = 0.75
XD3	2.0 ≤ μ ≤ 4.0	**μ = 3.0**	CoV = 0.75

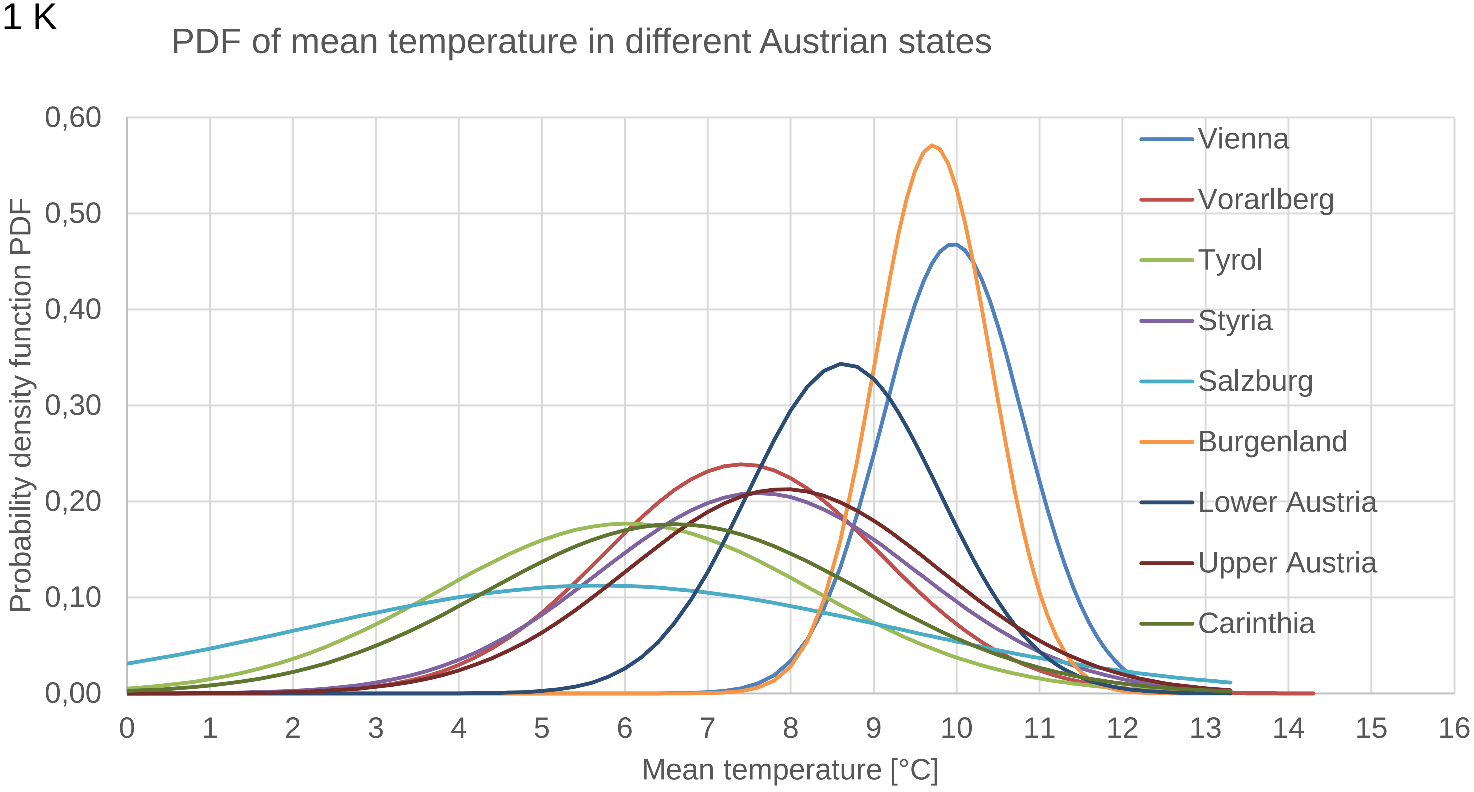
Concrete	Environmental class
B1	XC3
B2	XC3/**XD2**/XF1/XA1L/SB
B3	XC3/**XD2**/XF3/XA1L/SB
B4	XC4/**XD2**/XF1/XA1L/SB
B5	XC4/**XD2**/XF2/XA1L/SB
B6/C3A-frei	XC4/**XD3**/XF2/XA2L/XA2T/SB



ENVIRONMENTAL PARAMETERS - TEMPERATURE

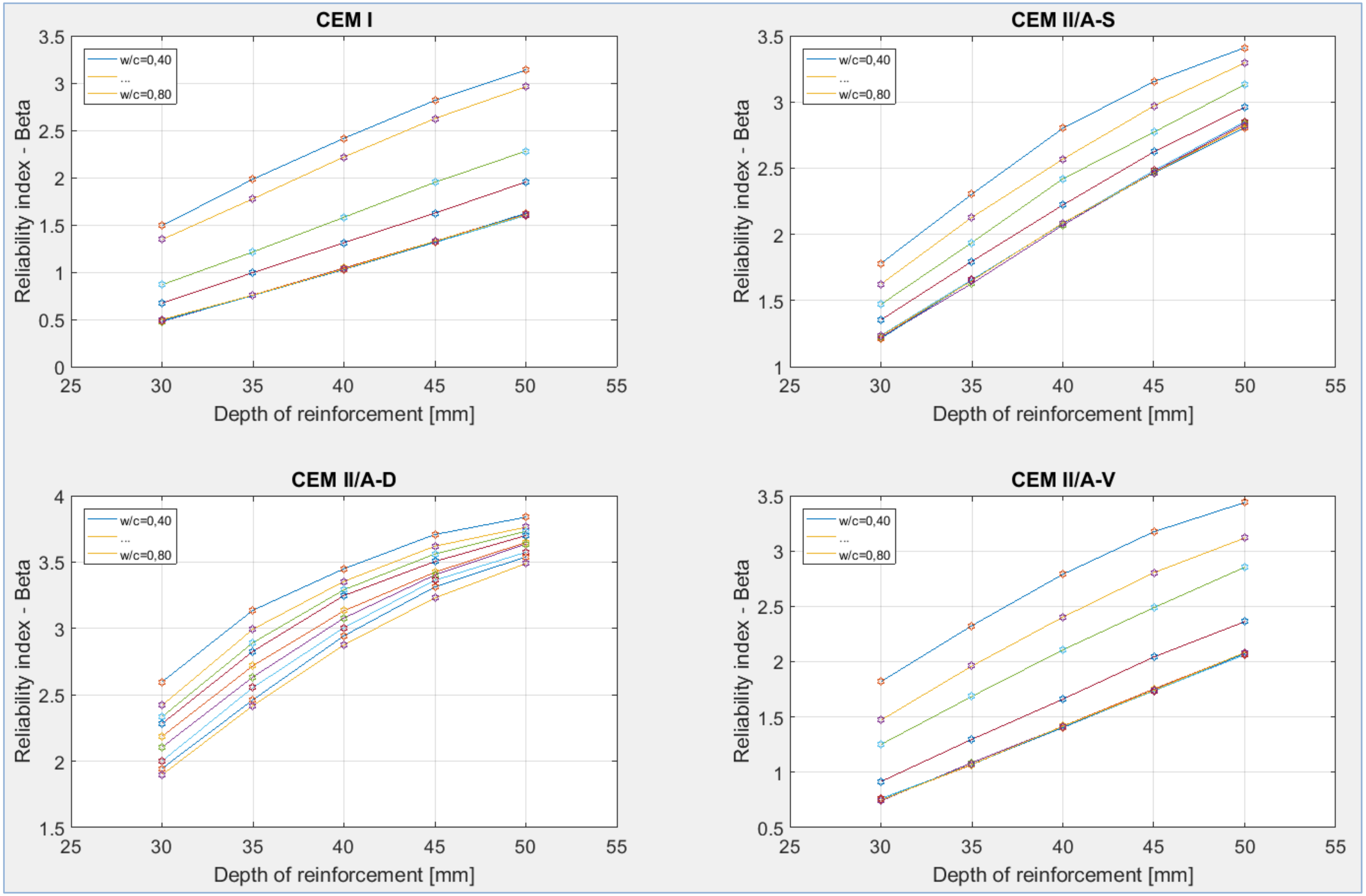
Study of 170 weather stations
Some of them from 1900
Used values:

μ=281 K
σ=3



RELIABILITY INDICES

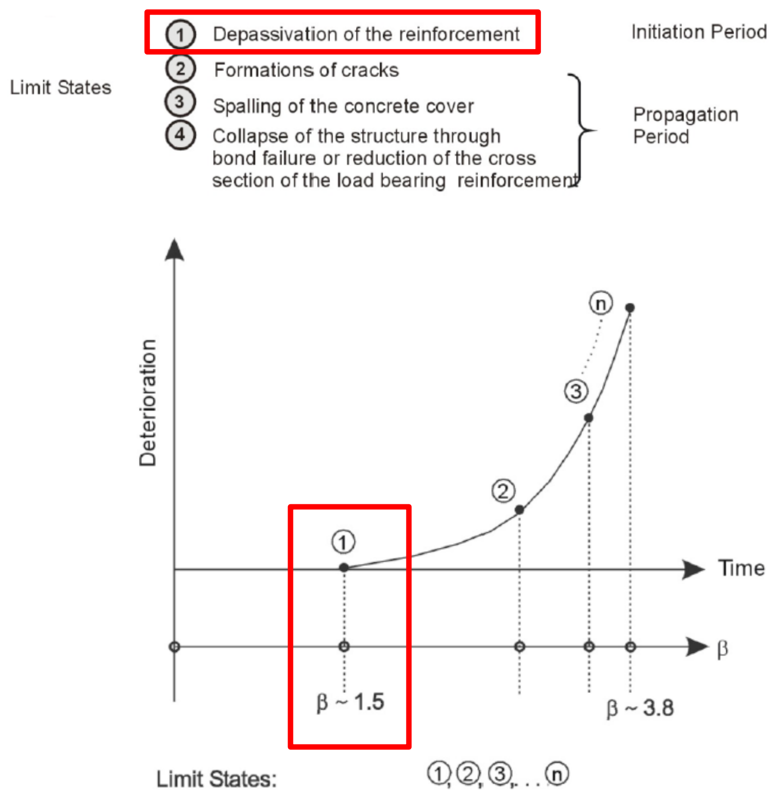
For exposure class XD2



TARGET RELIABILITIES

The following values of target reliability β at the age of 50 years are proposed:

- β ≥ 1,5 for bridge elements exposed to exposure class XD3
- β ≥ 0,5 for bridge elements exposed to exposure class XD1 and XD2



Lower target reliability levels for existing structures may be used if they can be justified on the basis of socio-economic criteria

Table F.1 — Illustrations of target reliability level

Limit states	Target reliability index β	Reference period
Serviceability		
Reversible	0,0	Intended remaining working life
Irreversible	1,5	Intended remaining working life
Fatigue		
Inspectable	2,3	Intended remaining working life
Not inspectable	3,1	Intended remaining working life
Ultimate		
Very low consequences of failure	2,3	L_S years[a]
Low consequence of failure	3,1	L_S years[a]
Medium consequence of failure	3,8	L_S years[a]
High consequence of failure	4,3	L_S years[a]

[a] L_S is a minimum standard period for safety (e.g. 50 years).

INDICATIVE VALUES FOR DEEPER TESTINGS

Made for CEM III/B

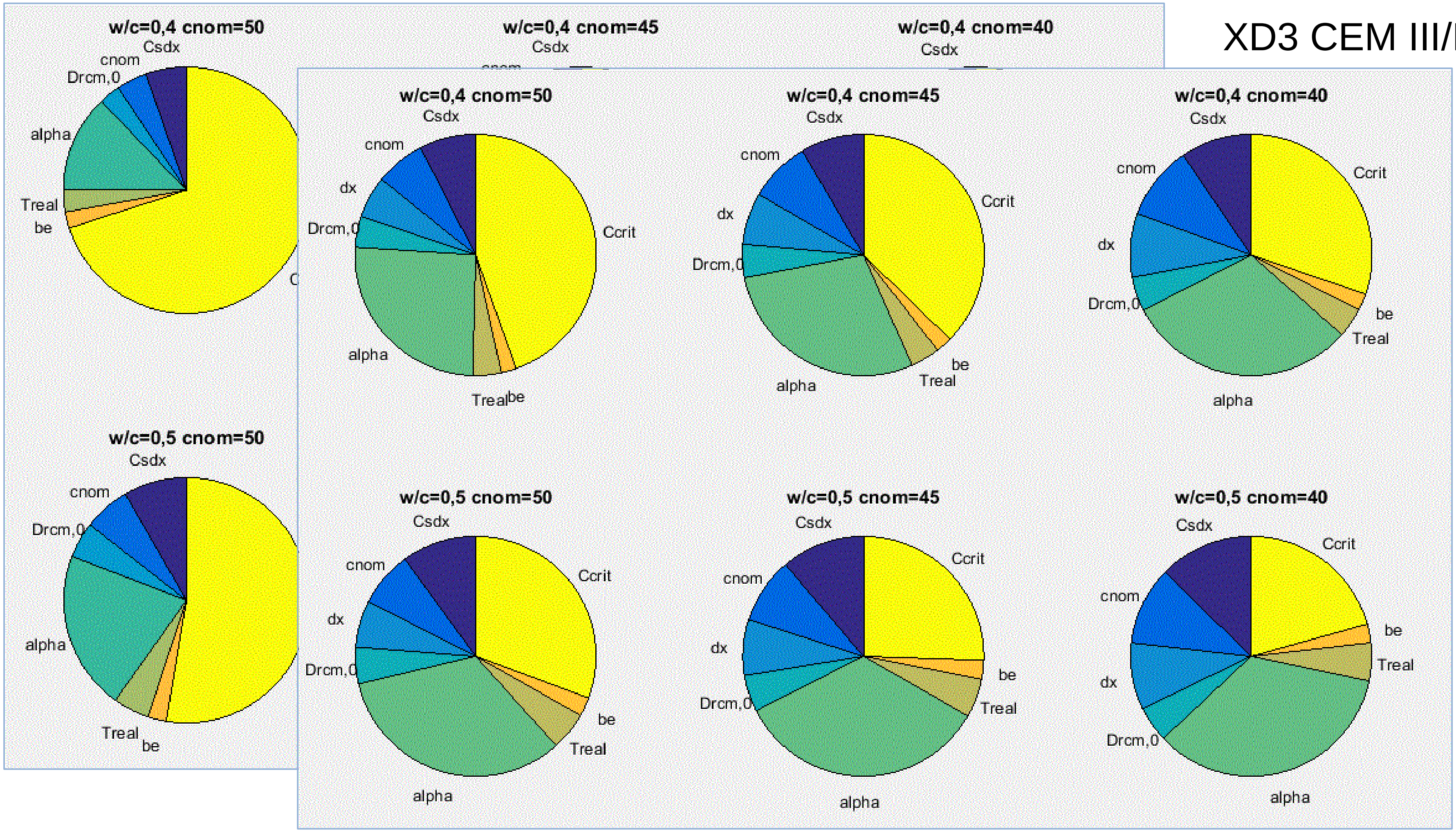
$\beta=0.5$ ($p_f \approx 0.3$)

		Concrete type					
		B 160	B 225	B 300	B 400	B 500	B 600
XD1	min	> 100	> 100	> 100	> 100	> 100	> 100
	mean	> 100	> 100	> 100	> 100	> 100	> 100
	max	> 100	> 100	> 100	> 100	> 100	> 100
XD2	min	> 100	> 100	> 100	> 100	> 100	> 100
	mean	45	70	> 100	> 100	> 100	> 100
	max	27	40	65	85	> 100	> 100
XD3	min	37	58	> 100	> 100	> 100	> 100
	mean	13	20	33	46	81	> 100
	max	8	12	18	25	44	68

SENSITIVITY STUDY

XD1 CEM I

XD3 CEM III/B



CONCLUSIONS

- **There is a gap between research and implementation that needs to get more intention!**
- **The model inputs and material characteristics should be studied in more detail!**
- **The target reliabilities should be defined by codes!**

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COST ACTION

THANK YOU FOR YOUR ATTENTION!

WG2 and WG3 WORKSHOP
Bridge performance goals and quality control plans

QUALITY CONTROL OF MASONRY BRIDGES BASED ON EMPIRICAL INFLUENCE LINES OF DISPLACEMENTS

Tomasz Kamiński – Wrocław University of Science and Technology, Poland
Czesław Machelski – Wrocław University of Science and Technology, Poland

20th – 21st October 2016
Delft, Netherlands

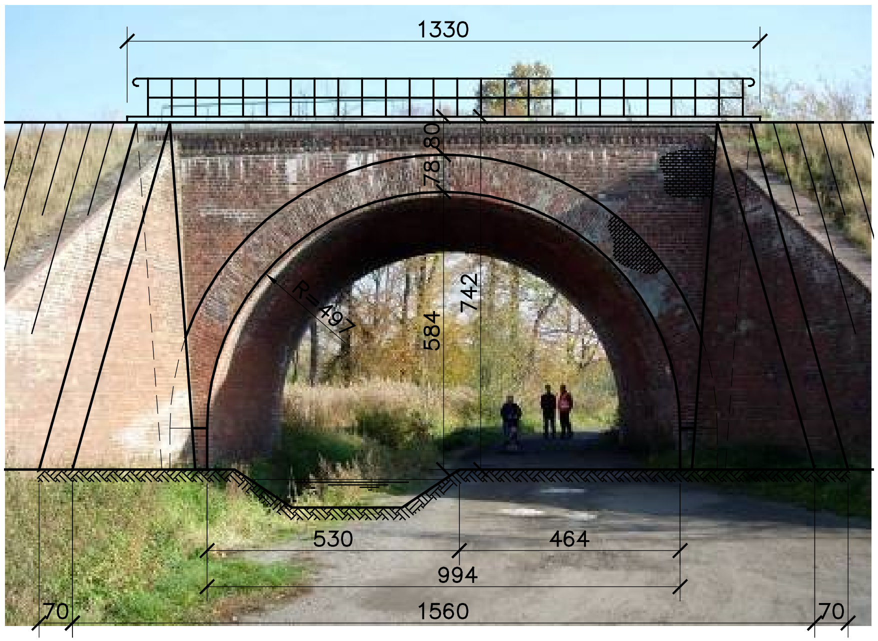
CONTENTS

INTRODUCTION

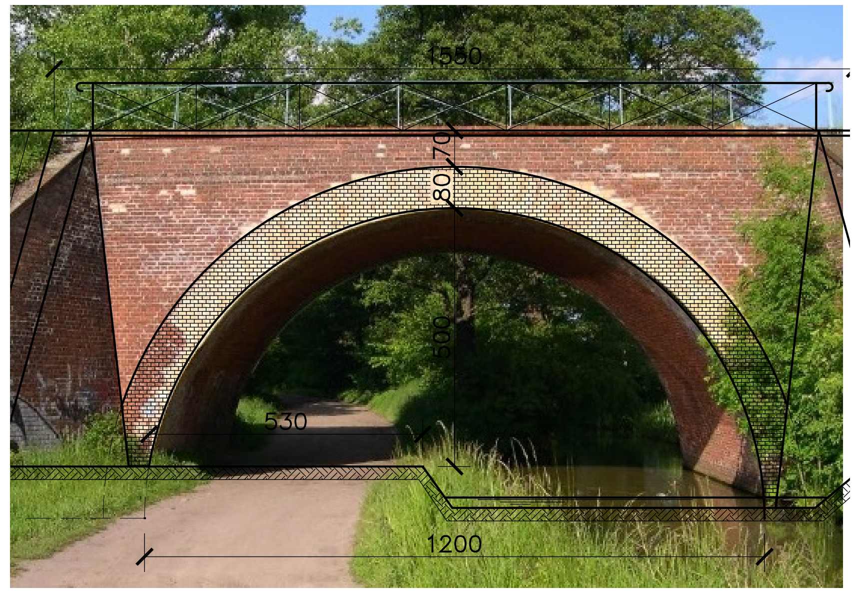
- Masonry arch bridges are complex structures characterised by many difficult to determine parameters including:
  - ✓ material properties (requiring laboratory tests)
  - ✓ geometrical characteristics (including hidden and inaccessible ones and effective width for 2D models)
- A simpler solution may be a careful study on the structure's deformation under live loads (especially in case of railway bridges)
- Based on measurements on site carried out during regular exploitation
- Even when limited to a single point of the arch, the obtained results can provide information on structural response to many independent loading cases (global calibration of a bridge model)
- The analysis is presented in two case studies of railway arch bridges with a similar structure under various railway vehicle types running with different speed

ANALYSED STRUCTURES

- single-span railway bridges with circular brickwork arches from 1875

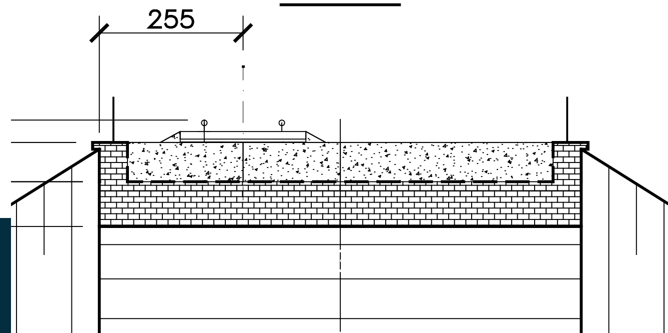


Oleśnica



Milicz

A - A



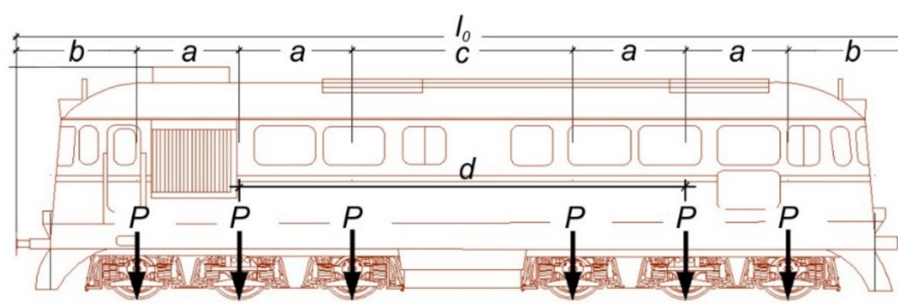
EXPERIMENTAL TESTS

APPLIED LIVE LOADS

- Analysis concentrated on effects of a locomotive itself crossing a bridge
- Four types of locomotives used

Bridge	Locomotive type	Axle load *P* [kN]	Bogie type	Space between bogies *d* [m]	Spaces between axles [m]		
					a	***b***	***c***
Oleśnica	ET22	200	Co'Co'	10,30	1,75	2,720	6,80
Milicz	ET22	200	Co'Co'	10,30	1,75	2,720	6,80
	EU07	196,2	Bo'Bo'	8,55*	3,05*	2,317	5,50
	E31	203	Co'Co'	10,95	2,40	1,525	6,15
	Dragon	202,2	Co'Co'	10,50	1,95	2,965	6,60

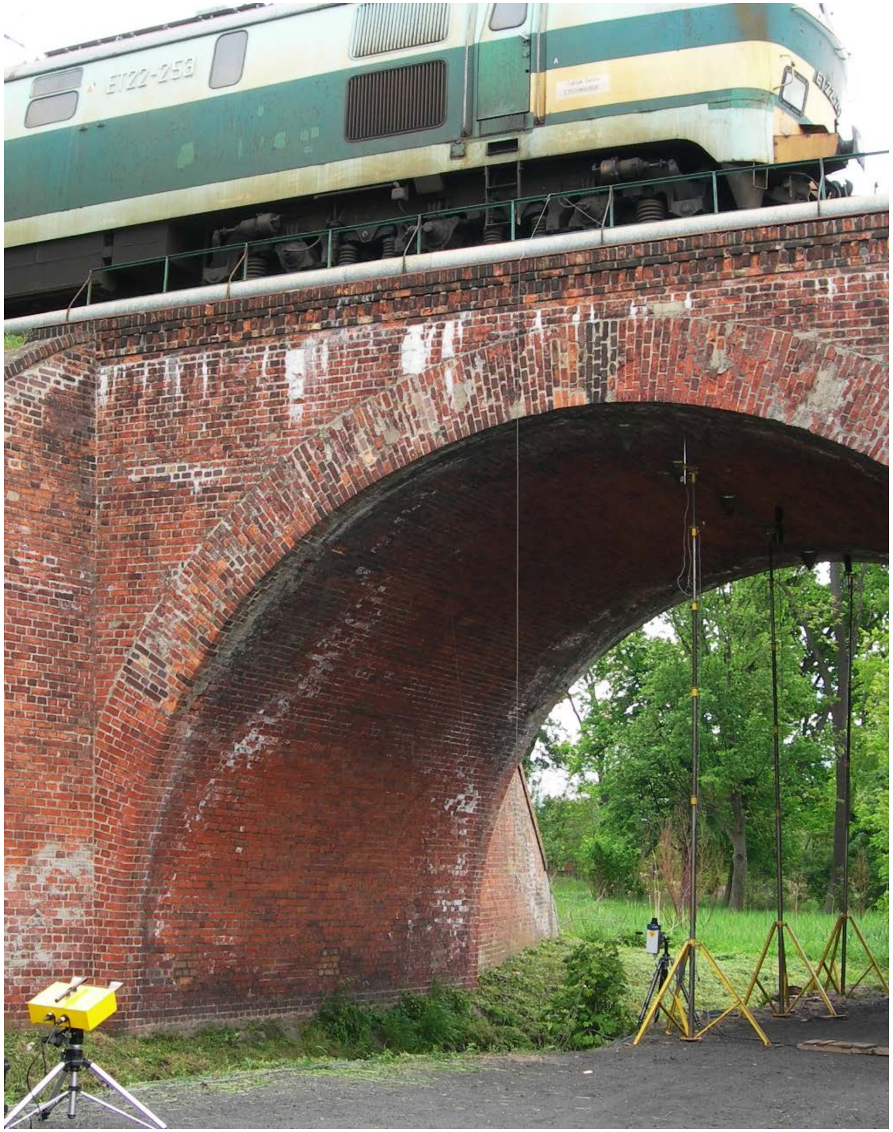
* 4-axle locomotive



EXPERIMENTAL TESTS
TESTING PROCEDURE

- Testing procedure based on measurements of the arch barrel deflection in the midspan (crown cross-section)

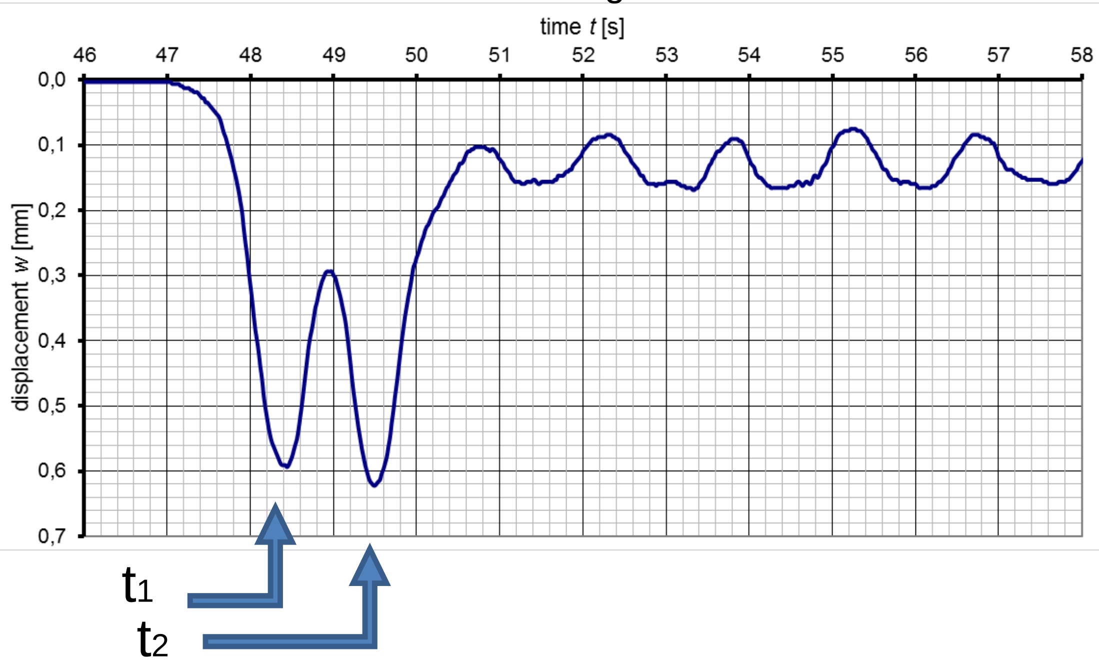
- Measurements carried out by means of various types of gauges (LVDT gauges, laser vibrometer sensor and microradar equipment)



EXPERIMENTAL TESTS
TESTING PROCEDURE

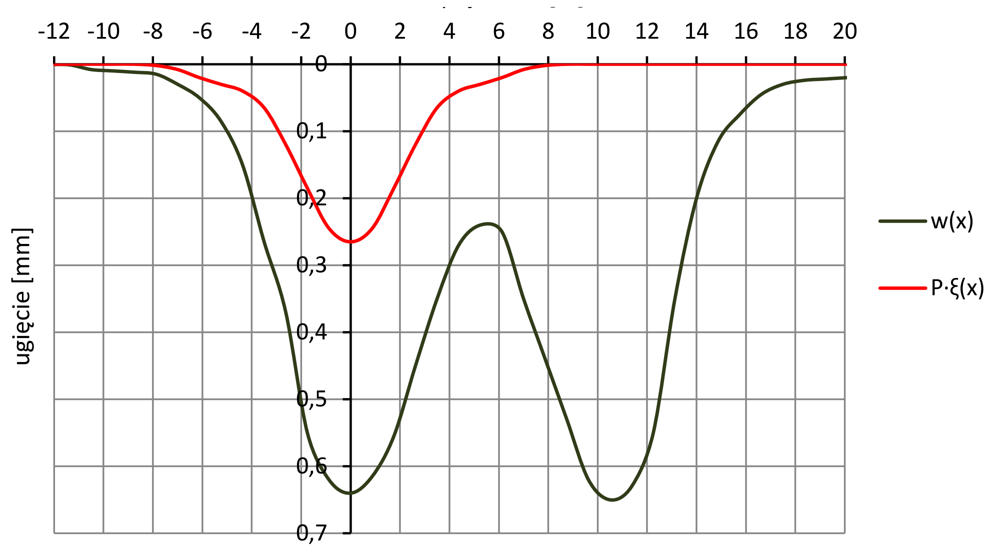
- Moments of the front and the rear locomotive bogie axes crossing the arch crown section are identified
- On the basis of the time difference $t_2 - t_1$ and the distance between bogies the average speed of the locomotive crossing a bridge can be evaluated
- The diagram $w(t)$ is transformed into the diagram $w(x)$

passage of a train (with ET22 locomotive) across the bridge in Milicz



Bridge	Passage	Locomotive type	$t_2 - t_1$ [s]	v [m/s]
Oleśnica	V11	ET22	0,95	10,8
Milicz	V10	ET22	1,03	10,0
	V13	EU07	0,66	12,9
	V16	E31	0,70	15,6
	V20	Dragon	0,53	19,8

EMPIRICAL INFLUENCE FUNCTIONS OF DISPLACEMENTS



- Empirical influence function of deflection $\xi(x)$ is developed from $w(x)$
- General relationship between the arch crown deflection $w(x)$ and $\xi(x)$ is:

$$w(x) = P\sum_{i=1}^{n}\xi(x + x_i)$$

- A progressive calculation procedure is applied starting from the point $x = x_0$, for which the measured deflection is equal to $w(x_0)$:

$$w(x_o) = P \cdot \xi(x_0 + a))$$

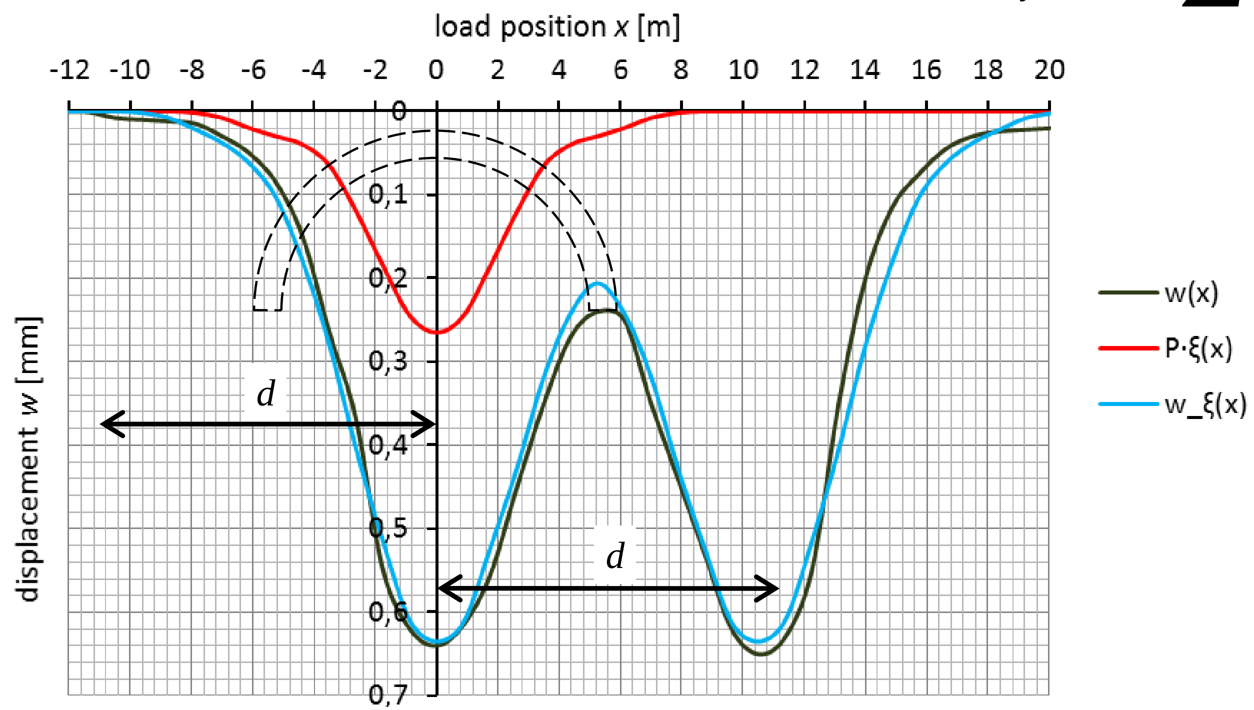
EMPIRICAL INFLUENCE FUNCTIONS OF DISPLACEMENTS

- Further procedure carried out with subsequent positions of the locomotive allows to find the next ordinates of $\xi(x)$

$$w(x_o + a) = P\left[\xi(x_o + a) + \xi(x_o + 2a)\right]$$

$$w(x_o + 2a) = P\left[\xi(x_o + a) + \xi(x_o + 2a) + \xi(x_o + 3a)\right]$$

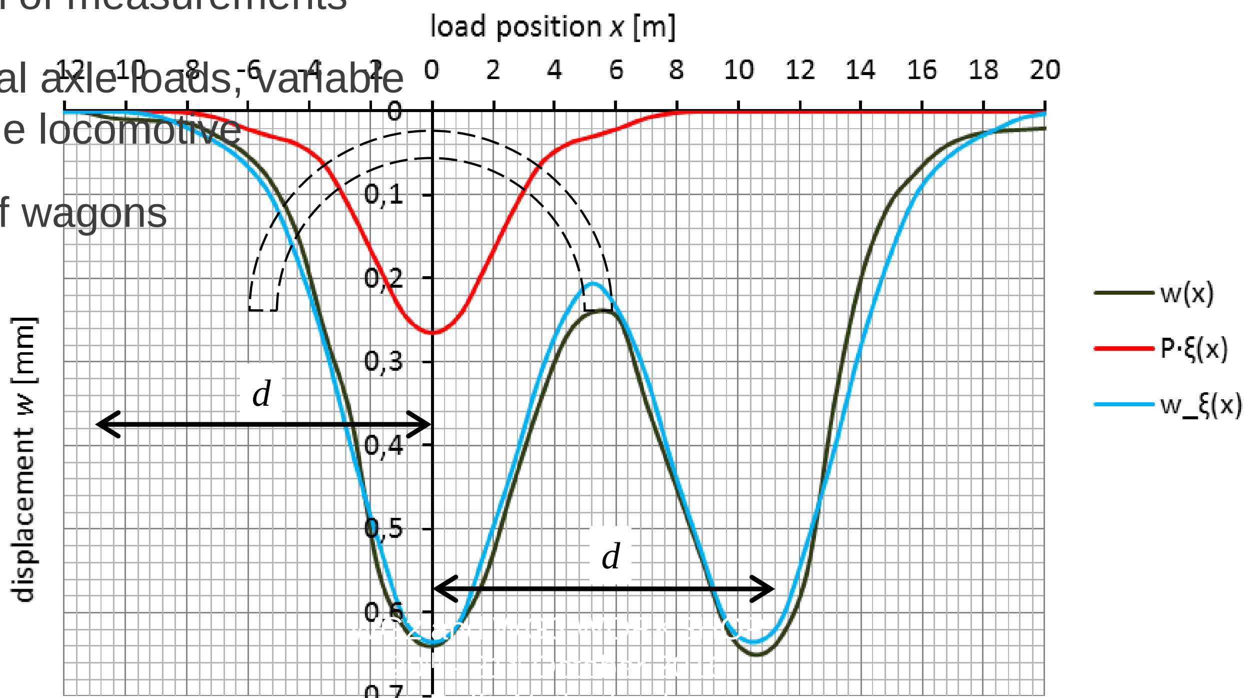
- Correctness control (for a single locomotive): $w(x) \cong w_\xi(x) = P\sum^{n} \xi(x + x_i)$



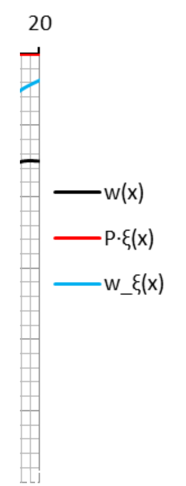
EMPIRICAL INFLUENCE FUNCTIONS OF DISPLACEMENTS

Oleśnica

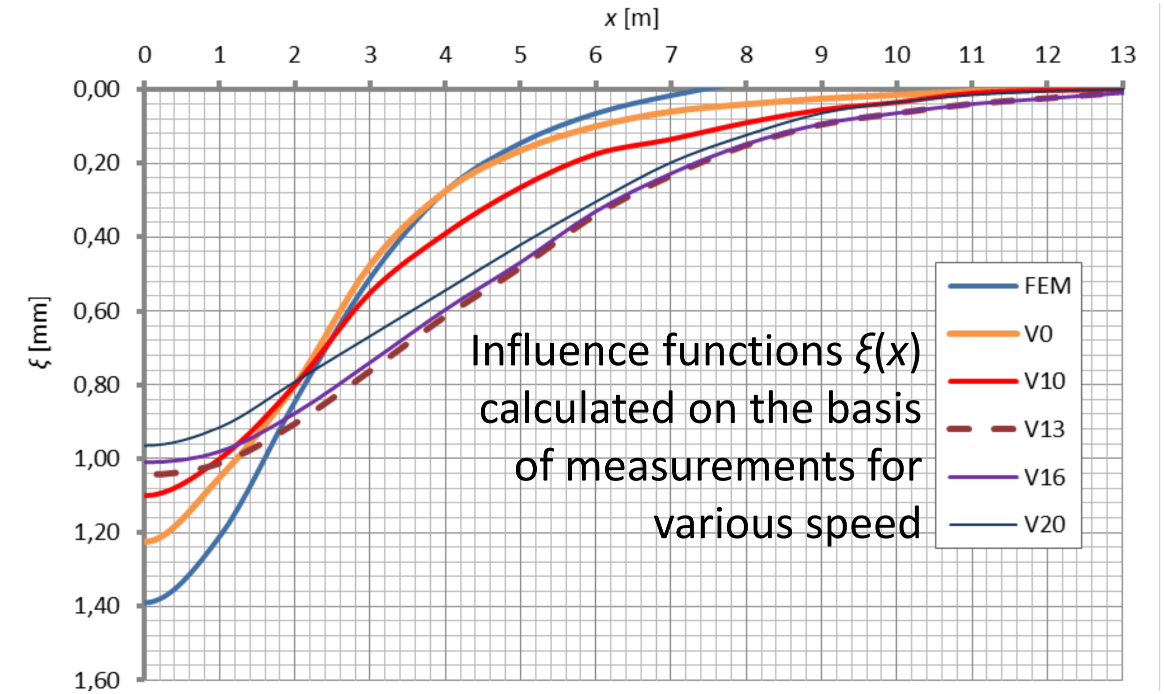
- Analogical diagrams is formed for the bridge in Milicz
- The lack of a prefect agreement between *w*(*x*) and $w_\xi(x)$ can arise from:
  - imprecision of measurements
  - different real axle loads, variable speed of the locomotive
  - influence of wagons



licz



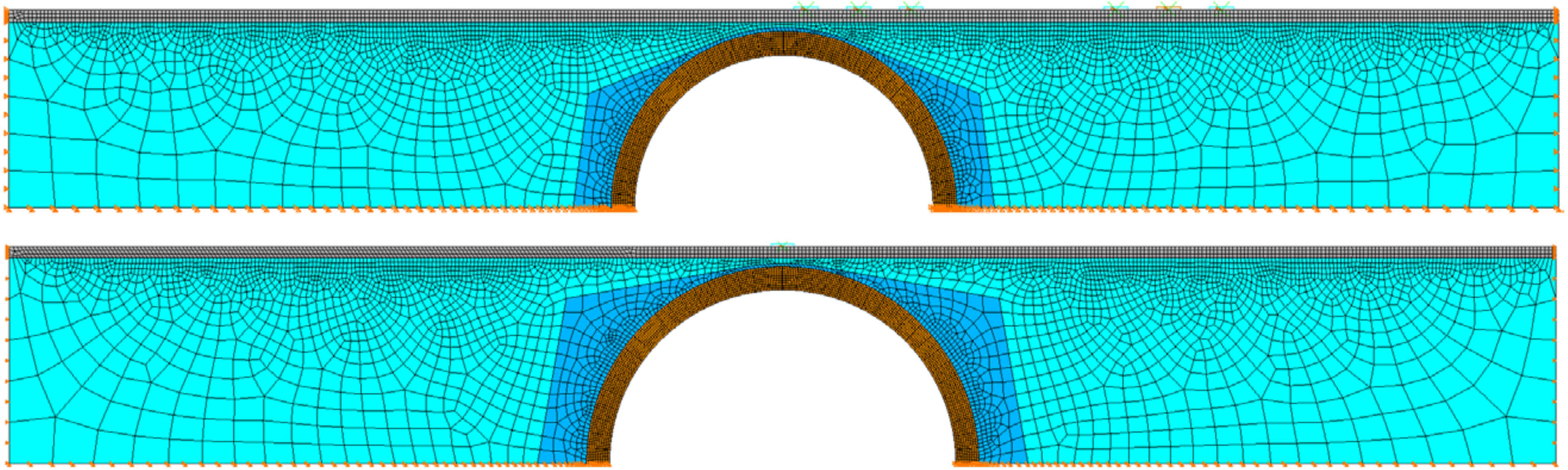
INFLUENCE LINES OF THE ARCH CROWN DISPLACEMENTS



- Different shapes are related to the speed of the locomotives
- $\xi_{max} = \xi\,(x{=}0)$ is getting higher with the decrease of the speed v
- $\xi(x)$ should be equal to the influence line of deflection $\eta(x)$ when $v \approx 0$
- extrapolation of $\xi(x)$ to speed $v = 0$ eliminates all unknown, speed dependent effects and give the experimental **influence line $\eta(x)$**

FINITE ELEMENT ANALYSIS

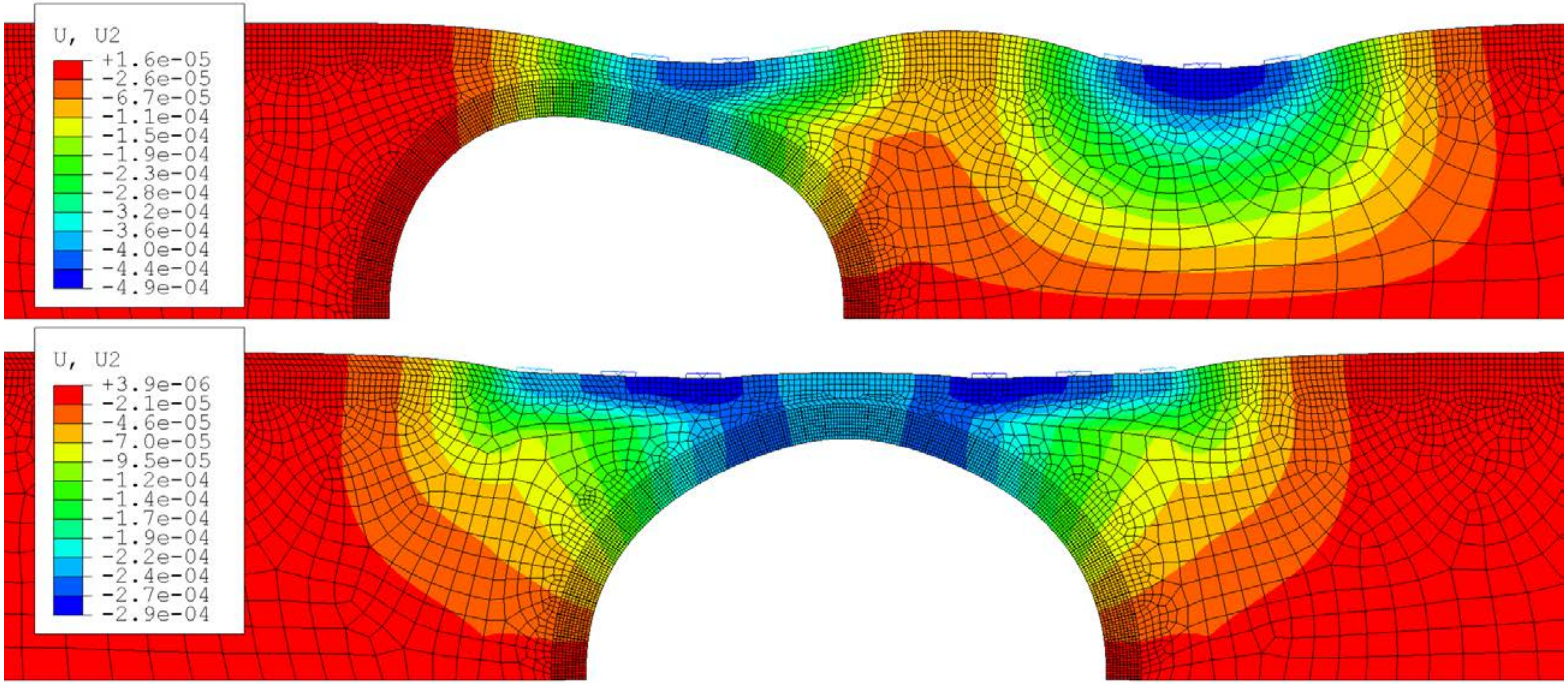
- Two-dimensional FE models used (arch barrel, masonry backing, soil backfill and pavement layer)



- The extent of the models reaches about 20 m away from the arch
- Mezomodelling technique applied to the masonry arch
- The live loads represent the action of locomotive axles applied to the top of the pavement layer as a pressure a width equal to 80 cm

FINITE ELEMENT ANALYSIS
RESULTS

- Two types of numerical results are collected and compared with the measured values:
  - the influence lines of deflection in the arch midspan,
  - deformation of the whole structure (with selected positions of the locomotives)



$\overline{\omega}$

FINITE ELEMENT ANALYSIS

Comparison of the measured and calculated deflections of the bridge in Milicz

Passage	Loco-motive position	Directly measured deflection w_v [mm]	Extrapolated measured deflection w_0 [mm]	Calculated deflection w_c [mm]	Discrepancy $\omega = (w_c - w_0)/w_0$ [%]
V10	A	0,508	0,595	0,562	-5,5
	S	0,245	0,236	0,233	-1,3
V13	A	0,375	0,385	0,369	-4,2
	S	0,256	0,274	0,261	-4,7
V16	A	0,504	0,522	0,487	-6,7
	S	0,453	0,248	0,235	-5,2
V20	A	0,509	0,573	0,536	-6,5
	S	0,410	0,240	0,233	-2,9

$\overline{\omega}$ = -4,6%

- Only the comparison of the calculated (w_c) values with the extrapolated (to v=0) measured (w_0) deflections shows good compatibility
- The average discrepancy between w_0 and w_c considering all loading cases is very low being equal to $\overline{\omega}$ = -4.6 %.

CONCLUSIONS

- The presented procedure may be an **effective method of a comprehensive calibration** of numerical models as well as **of the quality control procedures (**by monitoring of structural behaviour changes in time)
- It provides sufficient **results** from measurements carried out during regular exploitation of a bridge **without any disturbance to the traffic**
- It can be based as well on other mechanical effects (including both vertical and horizontal displacements or strains) in any point
- The difference between $\xi(x)$ corresponding to various speeds is not caused by the dynamic vibrations but most probably is related to **large inertia of masonry bridges** responding with some delay to the loads
- Calibration process revealed specific mechanical features of the bridges:
  - **large area of the soil** in the approaching zones of the bridge (reaching at least *L* outside the arch springing) needs to be included in the model
  - Essential meaning of the **backfill properties** (found to be defined by E > 100 MPa) as well as the shape of the **masonry backing** is discovered
  - Evident participation of the **pavement** in the bridge stiffness is visible

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Overview and preliminary results of the Long Term Bridge Performance Program

Bruce Johnson – Oregon Department of Transportation, United States of America

Hamid Ghasemi – FHWA Turner Fairbanks Highway Research Center, Fairfax, Virginia, United States of America

20th – 21st October 2016
Delft, Netherlands

Org Chart

LTBP Program

Office of Infrastructure R&D
Infrastructure Management Team

Dr. Hamid Ghasemi
Team Leader

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LTBP - Program Coordinator

NDE Center
Dr. Hoda Azari
NDE Program Manager

Susan Lane
LTBP - Field Operations Engineer

Frank Jalinoos
LTBP - Research Engineer

Fred Faridazar
LTBP - Research Engineer

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LTBP - Outreach & Development Engineer

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Sarah Skeen
Structural/Geotechnical Engineer
California Division

Tom Saad
Bridge Engineer
Resource Center

Maria Drozd
Bridge Engineer
Maine Division

Kareem Naji – NH Division
Kenny Young – OH Divsion

Outline

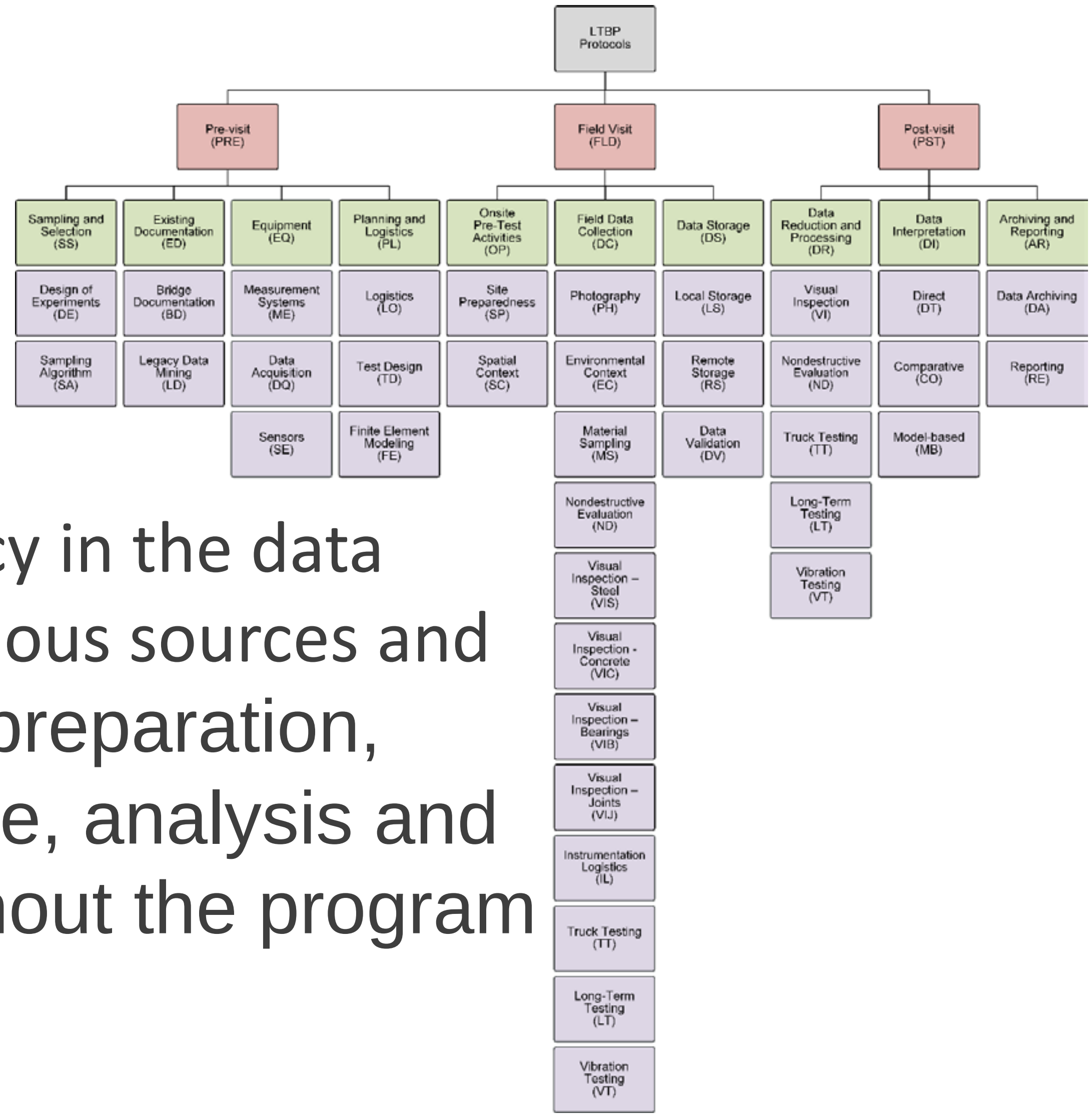
- Products
- LTBP Data Collection
- LTBP Status Update
- LTBP Strategic Plan
- Other Items

LTBP Program

LTBP Products

- Protocols
- Bridge Portal (A Data-Driven Decision Tool)
- Deterioration and Forecasting Model
- Bridge Performance Index
- Bridge Practice Timelines (Concrete, Steel, Reinforcing Bars)
- Automated Data Collection
- NDE Data Analysis - 2D Condition Maps
- Summaries of State Practices

LTBP Protocols



- Objective
  - Ensures consistency in the data collected from various sources and parties including preparation, collection, storage, analysis and reporting throughout the program

LTBP Products

LTBP Program

LTBP Protocols

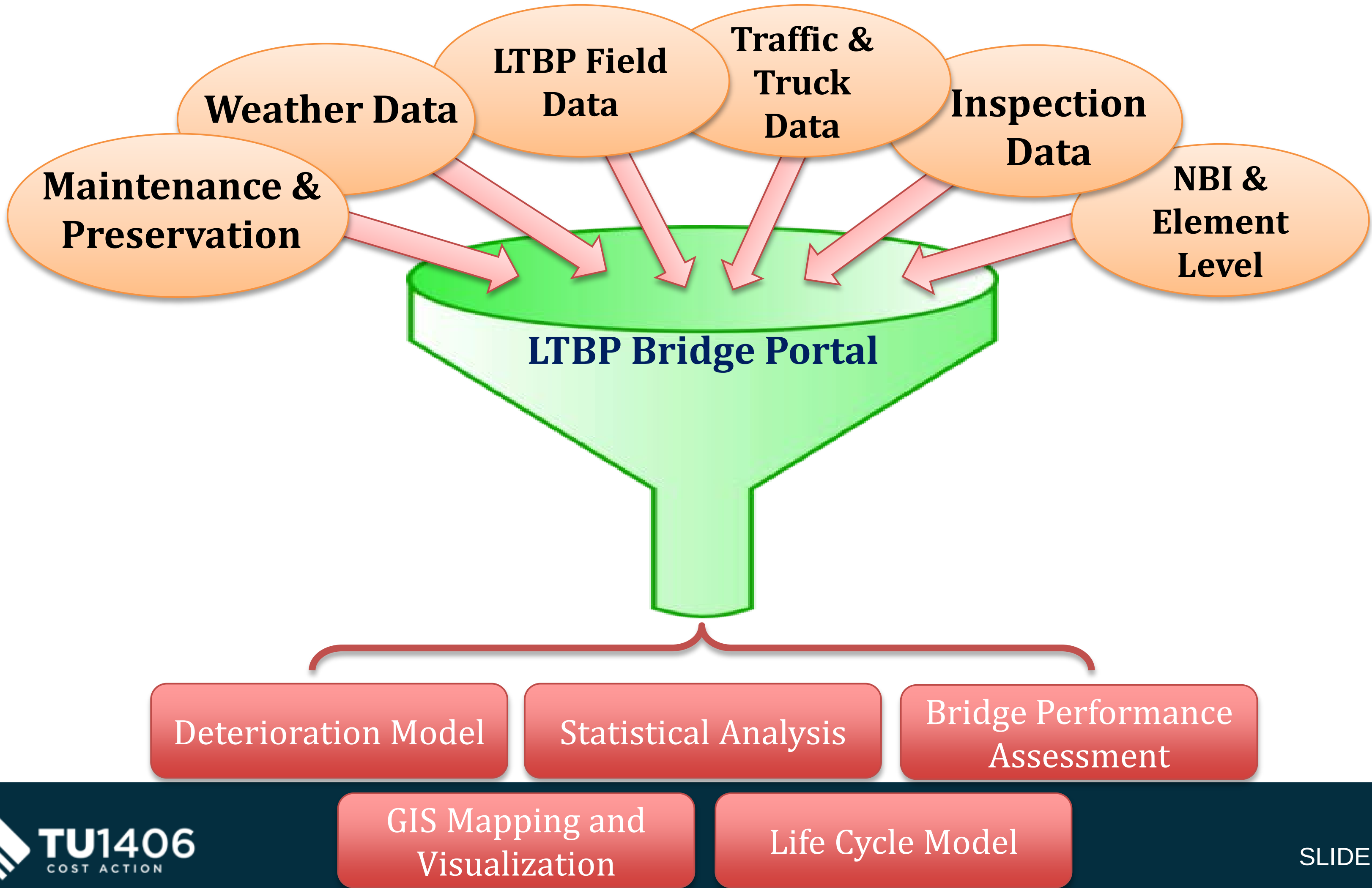
Status:

- First edition of the LTBP protocols were published in January 2016 (FHWA-HRT-16-007)
  - **Protocols successfully employed by Penn State University, Larson Transportation Institute and PenDot**
- A framework for tracking comments and implementing revisions to these protocols is in place
- Future protocols (135) are drafted and undergoing review for publication.
- An addition 32 protocols are identified for development

Bridge Portal

Data-Driven Tool

A centralized, national-level repository for efficiently and quickly accessing and querying bridge performance-related data and information



LTBP Products

LTBP Program

Bridge Portal

Data Sources

NBI	**Element Level Data**	Traffic and WIM Data	Environment Data
Maintenance	Bridge Documents	Visual Inspection	NDT/NDE

Tools

Advanced Security	**Advanced Search**	**Reporting**	**Map Visualization**
Multi dimensional Charts	**Map charts**	Bridge Historical data	**Deterioration Model**

Future Development

More Statistical features	Performance Comparison	Bridge Timeline	Advanced Deterioration Model
Android App	iPad App	Custom Groups	

Features available in V 1.0 Highlighted

LTBP Products

LTBP Program

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Bridge Portal

- Version 1 of the LTBP Bridge Portal was integrated with UPACS system and is available through FHWA network
- Establishing small working group to guide future development
- Rutgers delivered version 1.1 which has new features and capabilities
- Version 1.1 presented at 2016 TRB/LTBP workshop

Deterioration and Forecasting Model

Current Status:

- Theoretical Model has been developed
- The model needs to be validated with field data as they become available
- BCOM is very supportive of the methodology

Remaining Work:

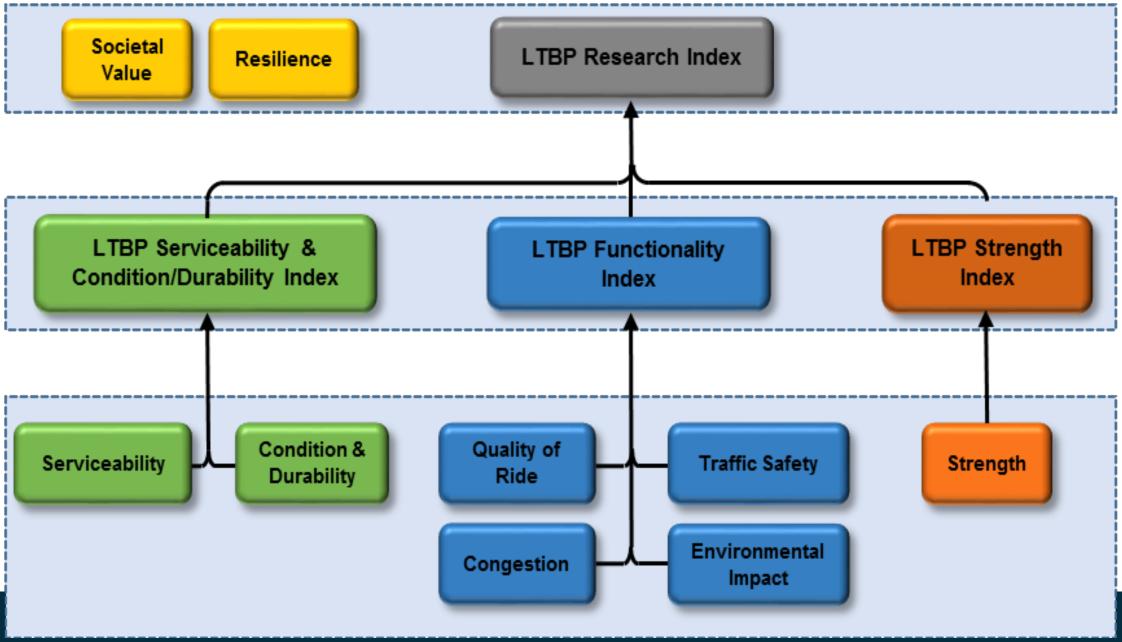
- Model validation (NJ, NY, NH)
- Model implementation
- Model integration with Bridge Portal

LTBP Products

LTBP Program

LTBP Research Performance Index

- Objective
  - Develop a data-driven bridge research index
    - Will help to understand the role of various aspects of bridge performance <u>on health</u> and condition of bridges



Bridge Practice Timelines (Concrete, Steel, Reinforcing Bars)

Creating timelines of changes in bridge practices from 1960 to the present to provide context and assistance for analyzing results obtained from field evaluations of bridges

- LTBP Tech Brief Published: National Changes in Bridge Practices for Reinforcing Bars

LTBP Products

LTBP Program

Bridge Practice Timelines (Concrete, Steel, Reinforcing Bars)

- **NATIONAL TIMELINES OF CHANGES IN BRIDGE PRACTICES**
  - **TELL US HOW & WHEN MATERIALS CHANGED**
  - **TELL US HOW & WHEN TECHNOLOGIES CHANGED**

- **STATE TIMELINES OF CHANGES IN BRIDGE PRACTICES**
  - **TELL US WHEN EACH STATE ADOPTED THE CHANGED MATERIAL OR CHANGED TECHNOLOGY**

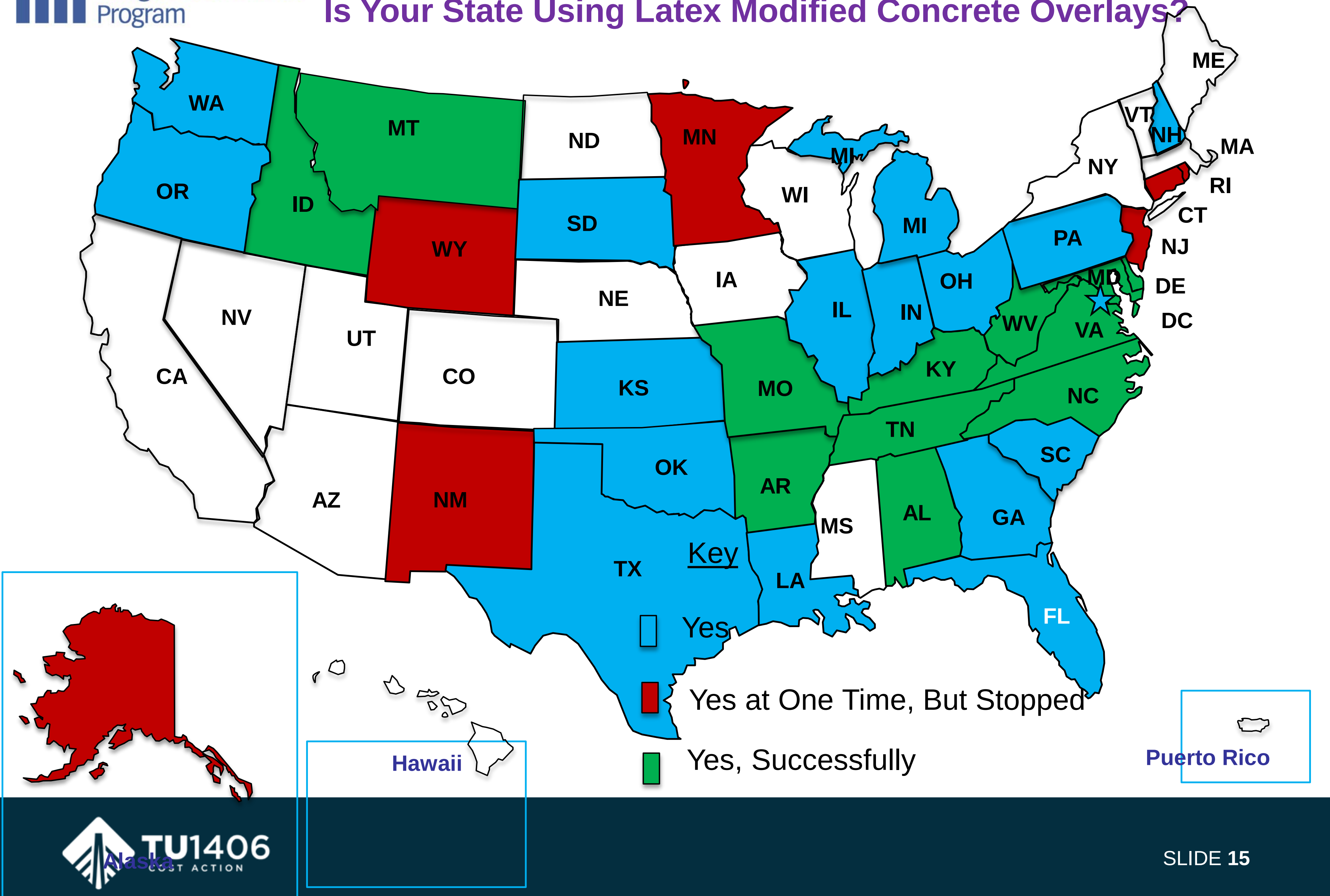
Bridge Practice Timelines (Changes in materials and technologies)

- **SUMMARIES OF CURRENT STATE PRACTICES FROM STATE COORDINATORS' MEETINGS**
  - **TELL US WHICH STATES HAD SUCCESS**
  - **TELL US WHICH STATES STOPPED USING**

- **EXAMPLE: BRIDGE DECK OVERLAYS**
  - **LATEX MODIFIED CONCRETE (Old Technology)**
    - **Why Success in Some States and Stopped Using in Others?**
  - **POLYESTER POLYMER CONCRETE (New Technology)**
    - **Why Success in Western U.S. But Not Many States Tried in Eastern U.S.?**

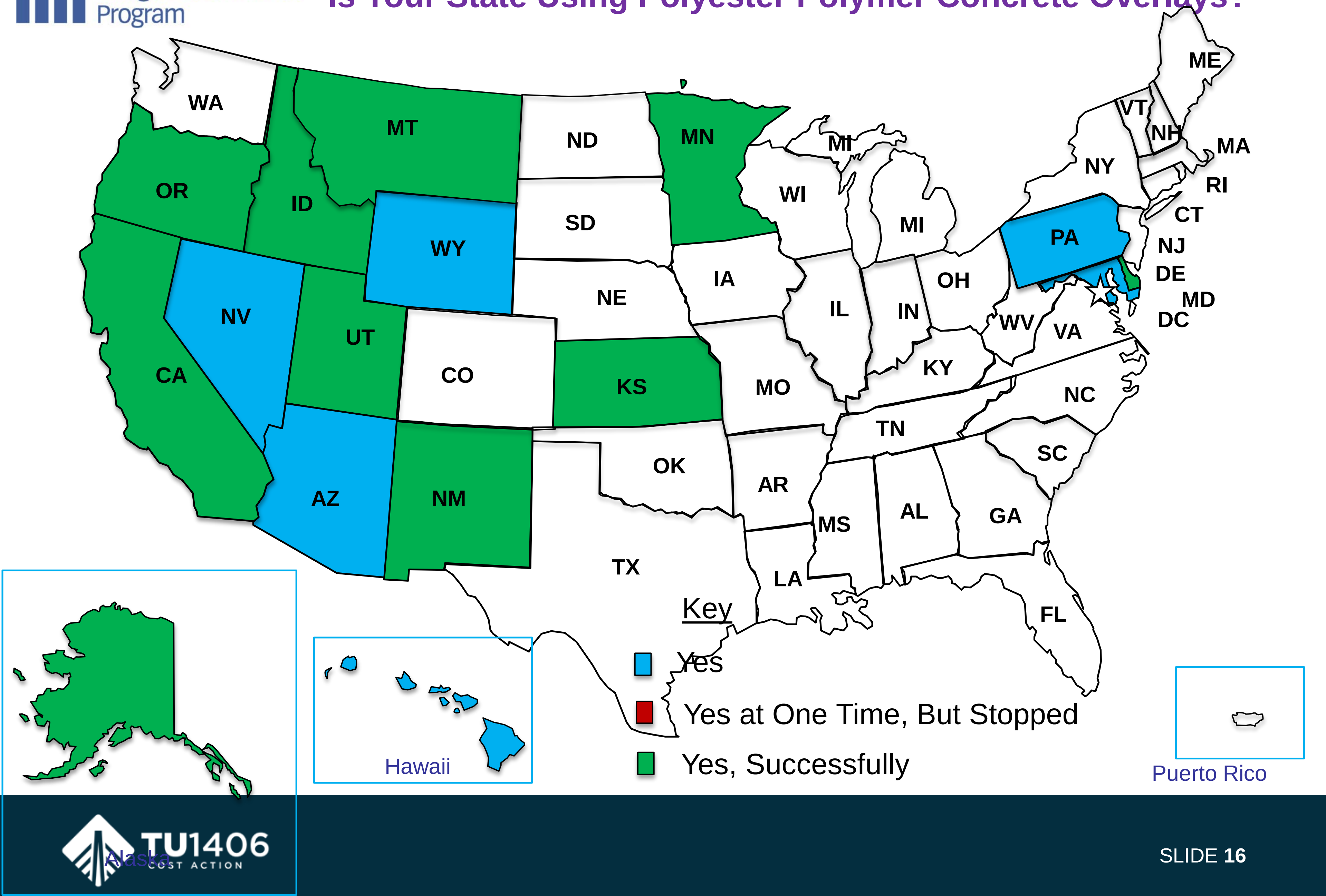
Long-Term Bridge Performance Program

Is Your State Using Latex Modified Concrete Overlays?



Long-Term Bridge Performance Program

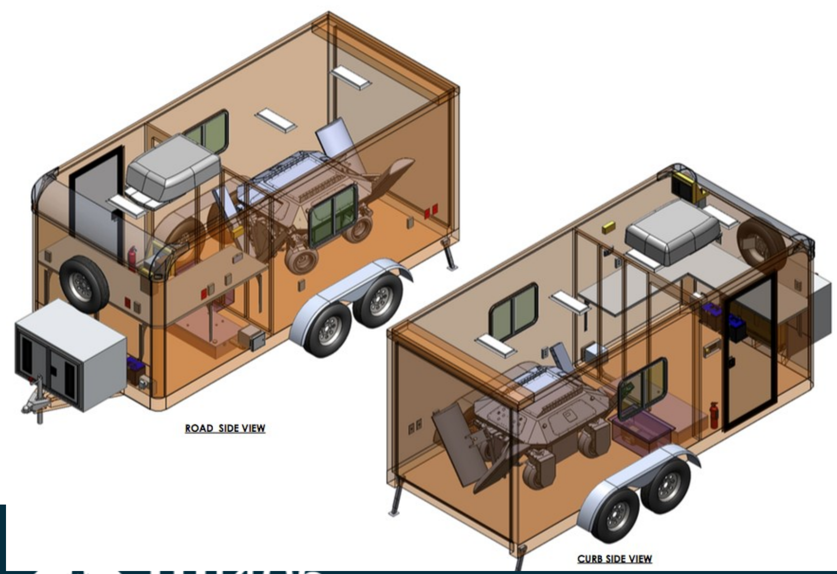
Is Your State Using Polyester Polymer Concrete Overlays?

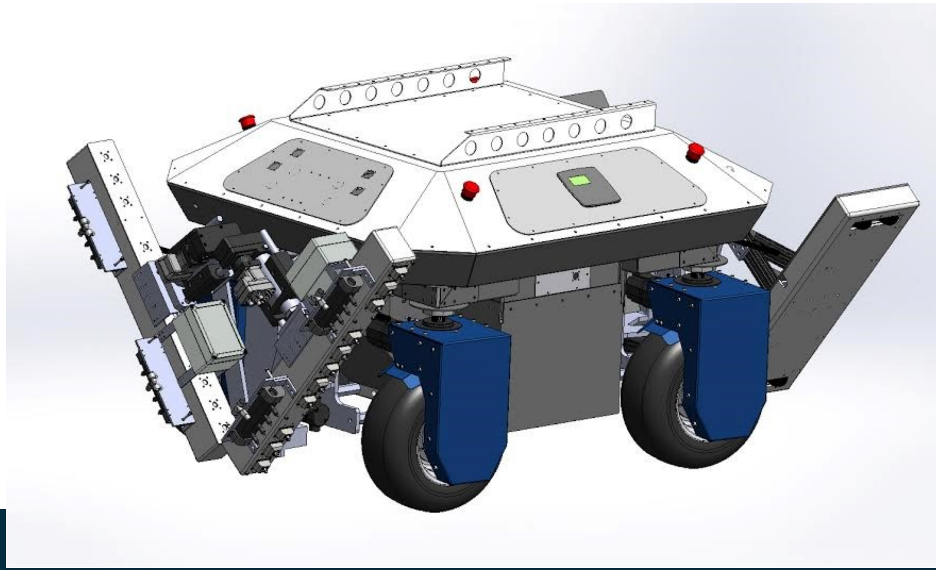


RABIT Bridge Deck Assessment Tool

Objective:

- Procurement of four autonomous robotic bridge deck inspection tools inclusive of training to LTBP contractors for the proper deployment of the technology in field data collection activities



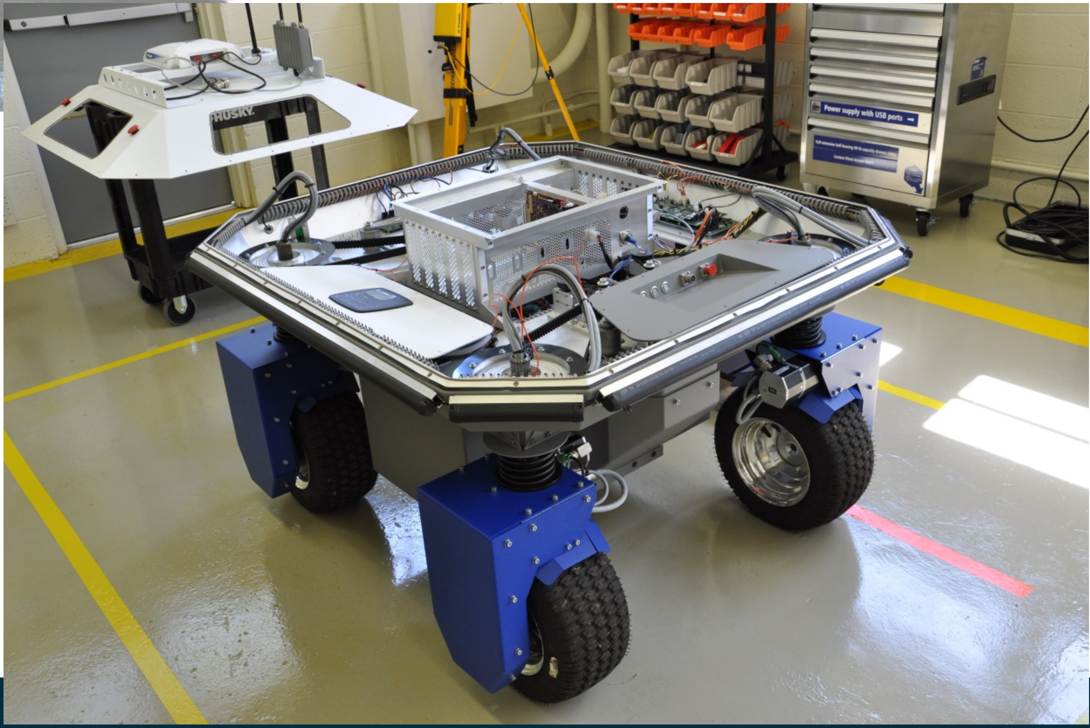


RABIT Bridge Deck Assessment Tool

Status:

- Development of the software is on-going
- Validation plan for each RABIT is in place
- First robotic platform for RABIT #1 is being modified
  - ✓ New acoustic array is under construction
  - ✓ New GPR array is being constructed
  - ✓ Other NDE technologies are in stock and are being mounted

Visit to Infratek



RABIT Bridge Deck Assessment Tool

Future Work:

- Completion of RABIT #1 in late 2016, with others to follow at 4 month intervals
- Validation of each system prior to handoff to FHWA
- Training of contractors and other personnel to use the RABIT
- Continual refinement and improvement for future generations of the RABIT system

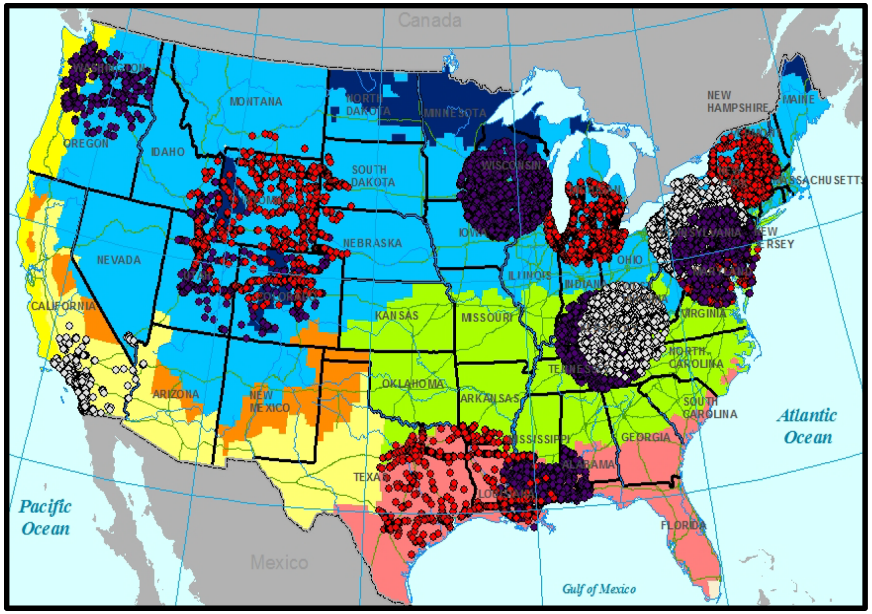
Modal Testing

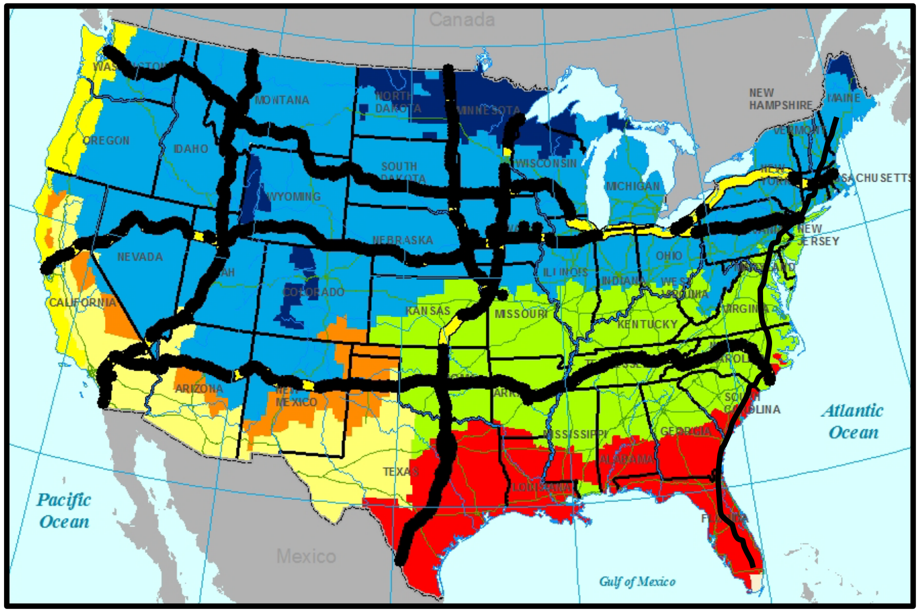
Objectives:

- Demonstrate the rapid, repeatable nature of the Modal Testing system as a technology for field data collection **on reference bridges**

- Collect dynamic signatures on bridge superstructure and produce reliable FE models based on field collected data

Data Collection –





14 Clusters
4 Bridge Types

Corridors

Data Collection Strategy –

- Legacy Data Mining
- Detailed Visual Inspection
- Hand-Held NDE Tools
- RABIT Deployment
- Automated Modal Testing

Data Collection – Legacy Data Mining

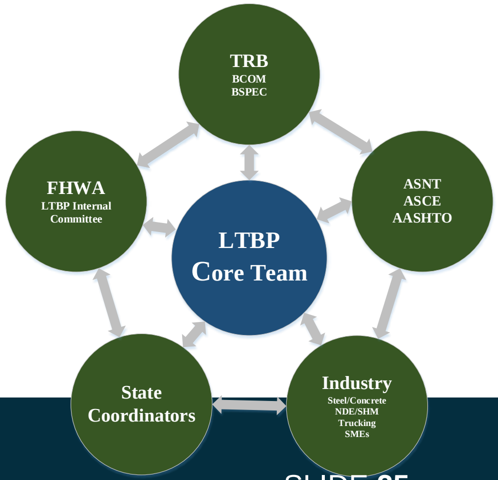
Objective – Employing readily available bridge documentation to develop an understanding of the:

- Distribution of bridge characteristics and condition among and between bridge clusters;
- Correlation of bridge condition parameters between bridge cluster data sets; and;
- Comparison of multiple parameters contributing to bridge performance

Data Collection – Legacy Data Mining

Benefits

- LTBP Program
  - “Desktop” understanding of bridge performance
  - Framework for field data collection and analysis
- Bridge Owners
  - Preliminary understanding of performance differences by bridge types
  - Preliminary comparison of bridge performance by type and by geography
- Stakeholders/LTBP Community
  - Preliminary identification of critical bridge condition trends and “hot topic” issues



Data Collection – Legacy Data Mining Status

Status

- Data collected and extracted on 445 of 965 bridges
  - This covers bridges from 19 of 40 states
- Basic statistical analysis completed providing histograms and data distribution studies
- Correlation coefficient analyses and multi-variate analyses are on-going

Future Work

- Completion of data collection and extraction on remaining 520 bridges
- Continued statistical analyses

Data Collection –

Data Collection: Untreated Bridge Decks, Joints, and Bearings

Contractor	Cluster	States
Rutgers	Mid-Atlantic Steel; Mid-Atlantic Prestressed Concrete (Visual/NDE/RABIT + LDM)	DE, NJ, MD, PA, VA, WV
Michael Baker	Gulf Steel; Gulf Prestressed Concrete (Visual)	AL, AR, FL, LA, MS, TX
PSI	Mid-Atlantic Steel, Mid-Atlantic Prestressed Concrete; Mid-Atlantic Concrete Box; NE Steel (LDM)	CT, DC, DE, MA, MD, ME, NH, NJ, NY, OH, PA, RI, VA, VT, WV
PB	NW Prestressed Concrete; SW Concrete Box (Visual)	AZ, CA, NV, OR, WA
Pennoni	NA	WIM Study, NDE/RABIT™ Construction/Validation

Field Work

Data Collection

Data Collection – Projects at a Glance

Rutgers

- Data-Driven Modeling
- Bridge Portal
- Field Data Collection
  - Mid-Atlantic Region
- Legacy Data Mining
- Other Projects

PB

- NW and SW visual inspection

Michael Baker

- Gulf Visual inspection, material sampling

PSI

- Legacy Data Mining

Pennoni

- WIM
- LTBP Performance Index
- Develop an accelerated testing bridge DB
- Website & Newsletter
- Protocol Publication
- RABIT Acquisition

Rutgers – Tasks Description

1. LTBP Protocols Training
2. LTBP Bridge Portal Development
3. LTBP Field Test – Mid Atlantic Cluster
   – 24 Bridges, NDT, Visual Inspection & Modal Testing
4. Legacy Data Mining
   – 14 Clusters and 10 Corridors (1100 Bridges)
5. Corrosion Studies
6. Quarterly Progress Reports
7. Other Special Projects
8. LTBP Advanced Deterioration and Cost Analysis Models

Status Update

LTBP Program

PSI – Task Description

- Legacy Data Mining for Mid Atlantic and North East Cluster
- 415 bridges

Status Update

LTBP Program

Michael Baker – Task Description

- Field Data Collection of Gulf Cluster bridges
- 24 bridges
- Visual Inspection of all 24 bridges
- Material Sampling of some of the bridges

Status Update

LTBP Program

PB – Task Description

- Field Data Collection of South West and North West bridges
- 24 bridges
- Visual Inspection of all 24 bridges

Pennoni – Tasks Description

1. LTBP Protocols
2. Technical Assistance
3. LTBP Newsletter & Website
4. Onsite Staff
5. Validate NDT Collected Data
6. Develop Bridge Traffic DB
7. Develop an Accelerated Bridge Testing DB
8. Quarterly Progress Report
9. LTBP Research Index

Long-Term Bridge Performance Program

- LTBP Data Collection
- LTBP Status Update
- LTBP Strategic Plan
- Summary

Long-Term Bridge Performance Program

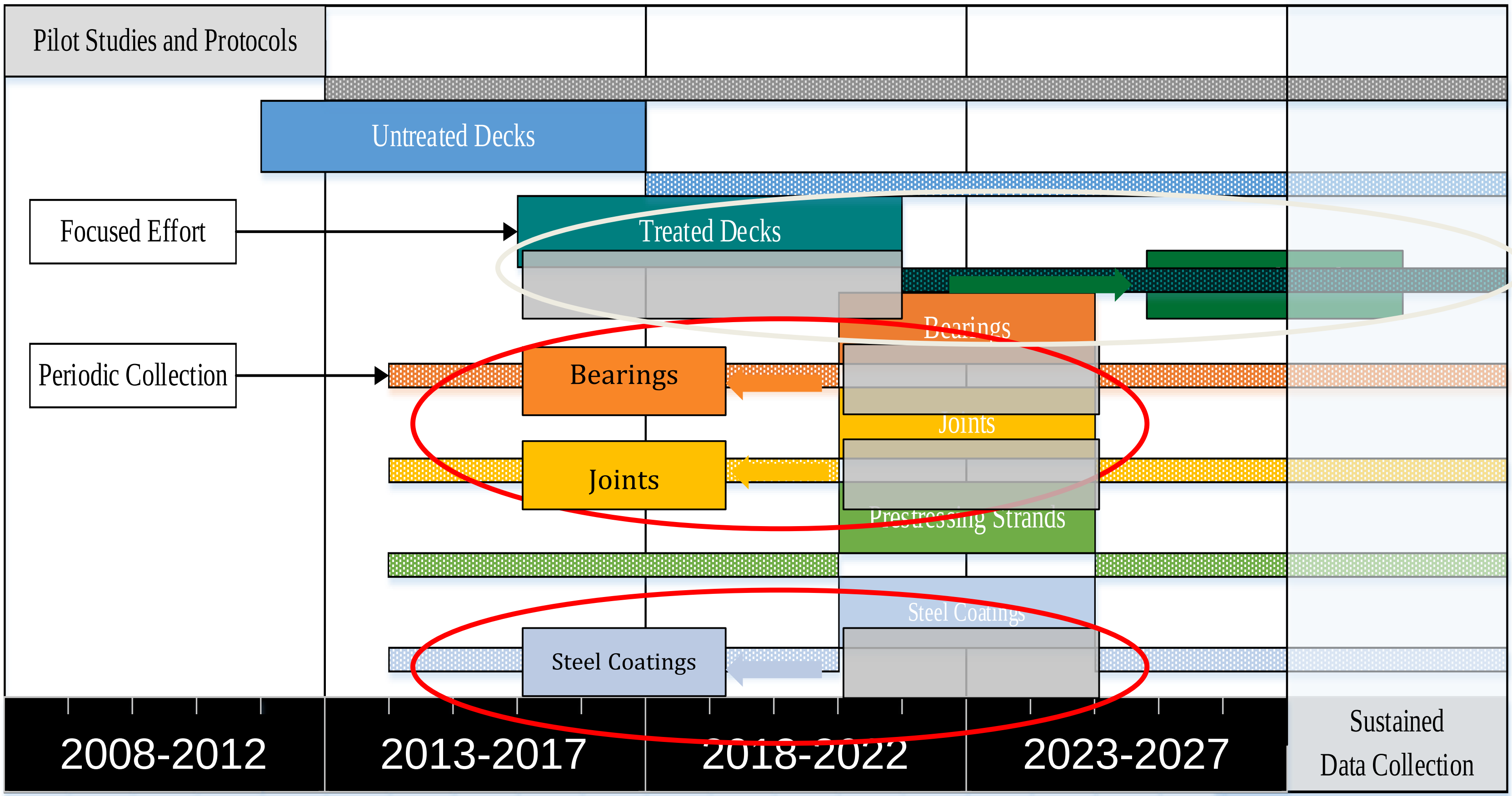
LTBP Bridge Breakdown

Untreated Decks				Treated Decks			
Steel Multigirder		Concrete		Steel Multigirder		Concrete	
		Prestressed Concrete Multigirder	Prestressed Concrete Box			Prestressed Concrete Multigirder	Prestressed Concrete Box
Steel Coatings		Prestressing Strands		Steel Coatings		Prestressing Strands	
Bearings	Joints	Bearings	Joints	Bearings	Joints	Bearings	Joints

Refocus Field Efforts

Long-Term Bridge Performance Program

Schematic Timeline for LTBP Long-Term Data Collection



Refocus of Efforts – Untreated Decks, Steel Multi-Girder Superstructures, Steel Coatings, Joints, and Bearings

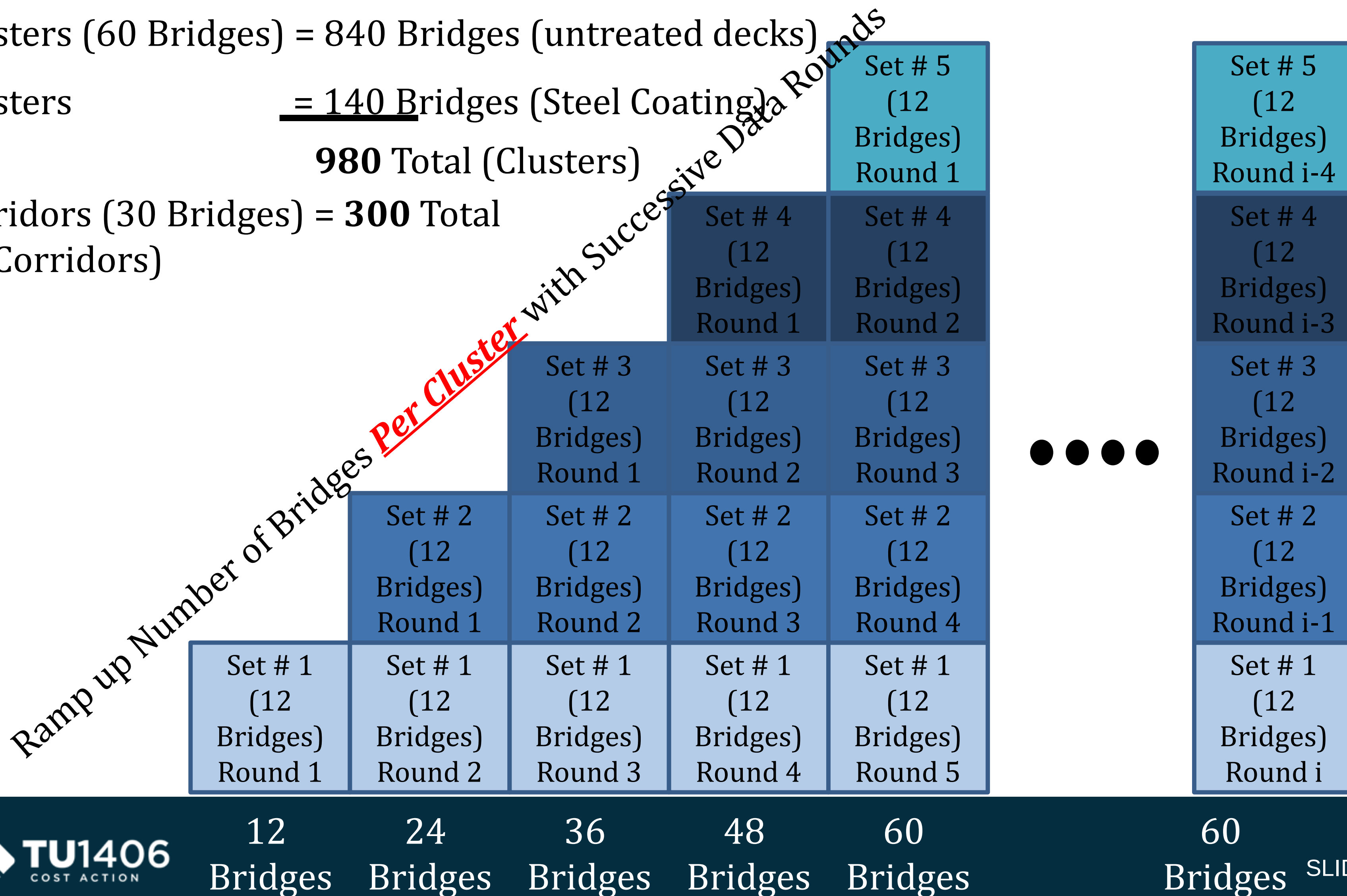
Long-Term Bridge Performance Program

14 clusters (60 Bridges) = 840 Bridges (untreated decks)

14 clusters = 140 Bridges (Steel Coating)

980 Total (Clusters)

10 Corridors (30 Bridges) = **300** Total (Corridors)



Other Items

- Draft LTBP Business Plan (Developed and Submitted to TRB/BCOM)
- Strategic Roadmap (Developed and Submitted to TRB/BCOM)
- Pooled- Fund Study (impact of live load on bridge performance)
- VA Pilot Bridge Deck Sections will be at TFHRC in June 2016 for additional testing and autopsy
- Subcommittee on Bridges and Structures Technical Committee for Bridge Preservation (T-9) and LTBP

Summary

- **The LTBP is a significant effort that will define bridge performance using repeatable, data-driven NDE information.**
- **The Program aims to show the value of NDE and provide a large data set to evaluate the value of current inspection and evaluation methods.**
- **The Protocols establish a consistent basis for collecting data by the Program and others that can add to the large database on information to produce reliable predictions of performance and extension of service life after remedial or preservation actions.**
- **The Portal is available for use by US agencies and will be available in he future to other researchers to improve deterioration modeling and evaluate the benefits of high performance materials and preservation actions.**

TU1406

COST ACTION

SLIDE **40**

Pooled Fund Project TPF-5(283) MN, OR, NC, GA, WI, PN, and IA: The Influence of Vehicular Live Loads on Bridge Performance

- Goals
  - Develop a National Bridge Traffic Database
  - Determine and quantify influence of trucks on bridge component performance
  - Develop protocols for collecting research quality bridge traffic data
  - Develop tools and products that bridge owners can use to manage and operate loading conditions on existing network of highway bridges

Two Fundamental Questions

- **What are the current truck loads on our nation's bridges?**
  - There have been changes in truck geometry, axle configurations, suspension, and tire characteristics.
  - Common loaded truck weights are stipulated to have increased in some cases from 72 kips (design truck) to more than 110 kips.
- **What are the impacts of increased truck loads on the durability of the nation's bridges?**
  - Which bridge elements are especially being affected by truck loads?
  - The freight industry has been requesting increases in allowable truck loads on bridges.
  - Bridge owners need to address the effects of live loads on bridge component durability by using better tools and strategies to manage and operate

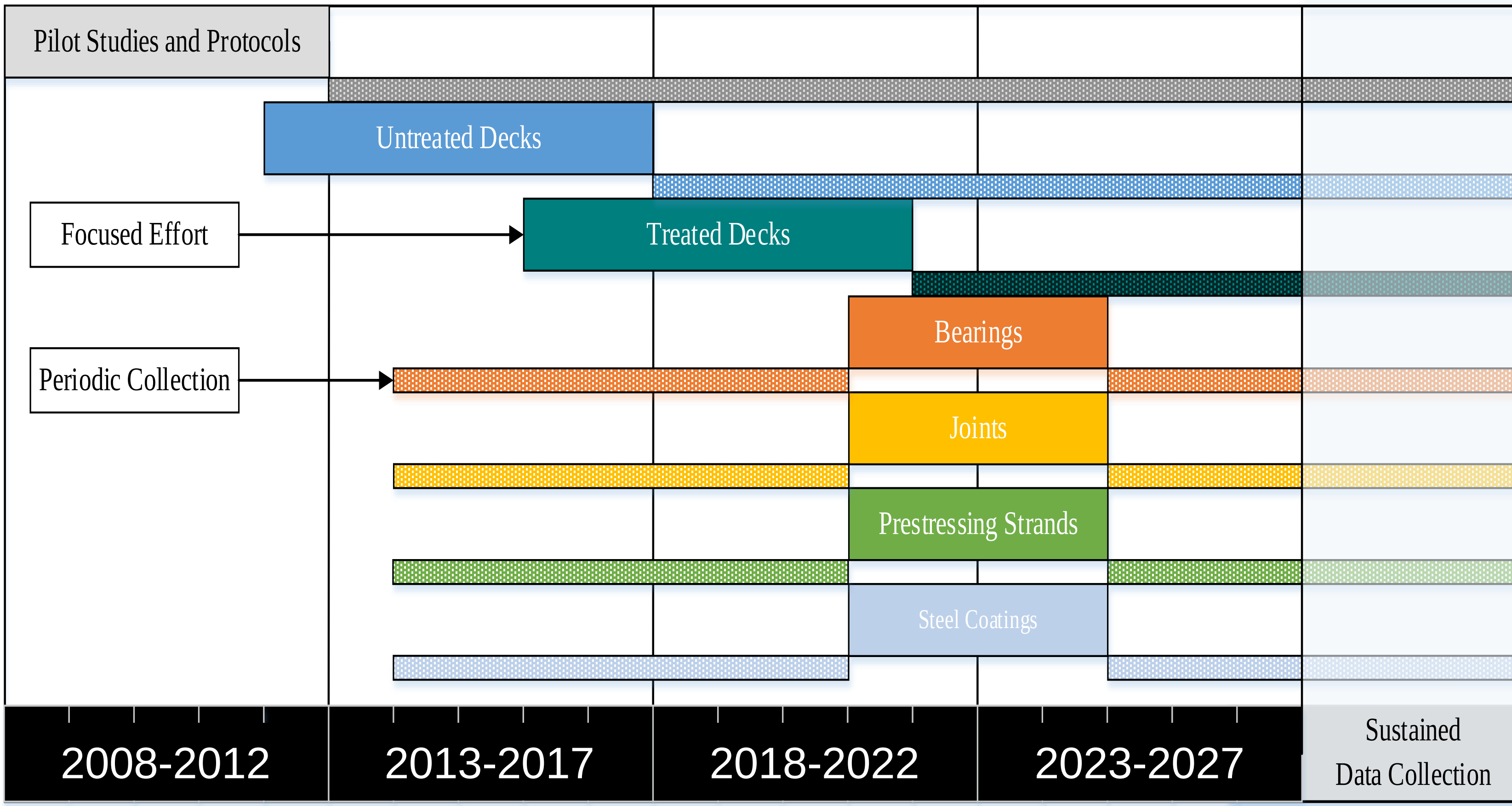
Long-Term Bridge Performance Program

<table>
<tr><th colspan="8">LTBP Bridge Breakdown</th></tr>
<tr><td colspan="4">Untreated Decks</td><td colspan="4">Treated Decks</td></tr>
<tr><td colspan="2" rowspan="2">Steel Multigirder</td><td colspan="2">Concrete</td><td colspan="2" rowspan="2">Steel Multigirder</td><td colspan="2">Concrete</td></tr>
<tr><td>Prestressed Concrete Multigirder</td><td>Prestressed Concrete Box</td><td>Prestressed Concrete Multigirder</td><td>Prestressed Concrete Box</td></tr>
<tr><td colspan="2">Steel Coatings</td><td colspan="2">Prestressing Strands</td><td colspan="2">Steel Coatings</td><td colspan="2">Prestressing Strands</td></tr>
<tr><td>Bearings</td><td>Joints</td><td>Bearings</td><td>Joints</td><td>Bearings</td><td>Joints</td><td>Bearings</td><td>Joints</td></tr>
</table>

Current Focus – Field Efforts

Long-Term Bridge Performance Program

Schematic Timeline for LTBP Long-Term Data Collection

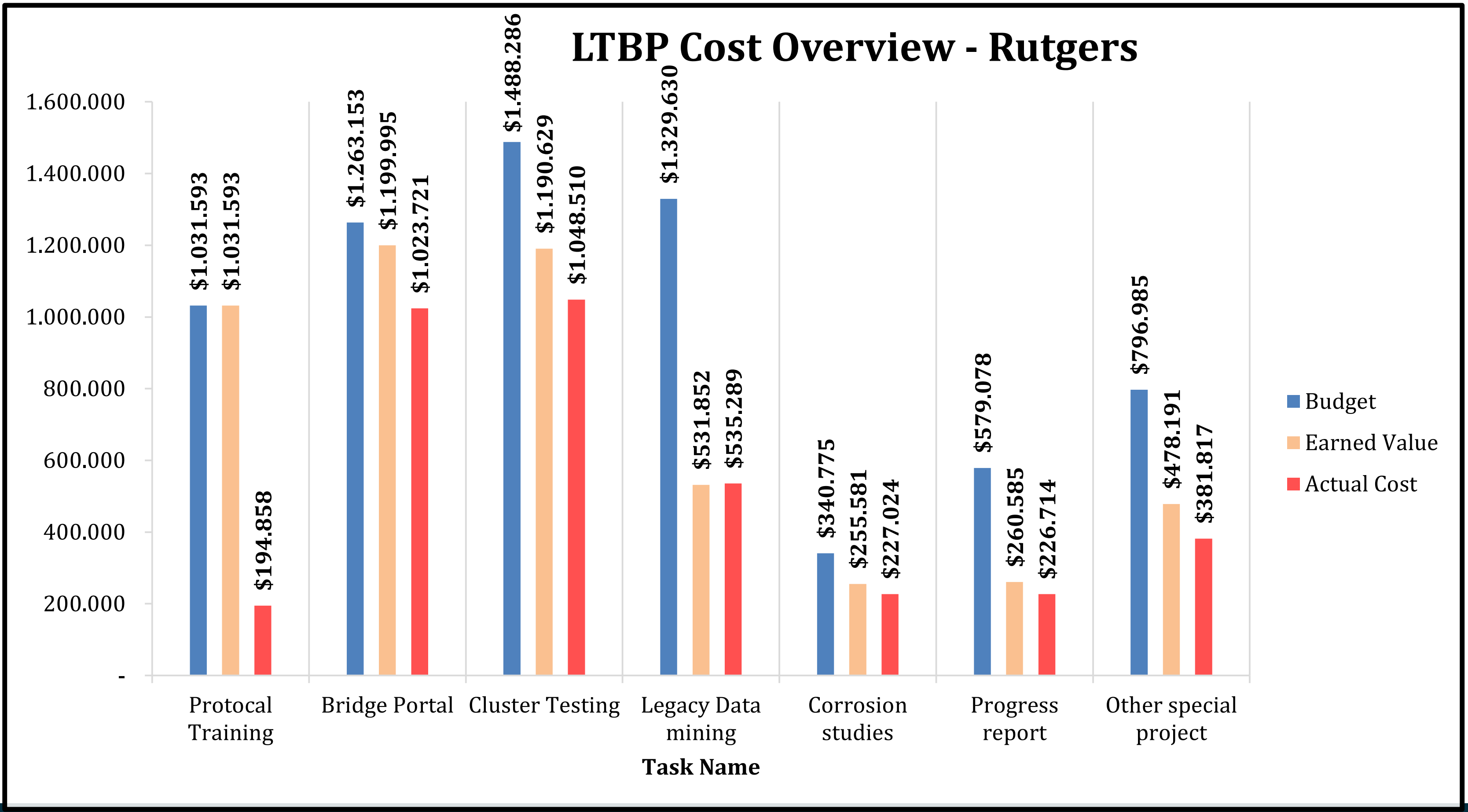


Timeline to be adjusted based on available resources

Data Collection –

Project Tracking system

Rutgers- Financial Status



Status Update

LTBP Program

TU1406 COST ACTION

Rutgers – Project Status

Data Collection –
Project Tracking system

Rutgers - Protocols Training

TUE 4/1/14 - FRI 10/31/14

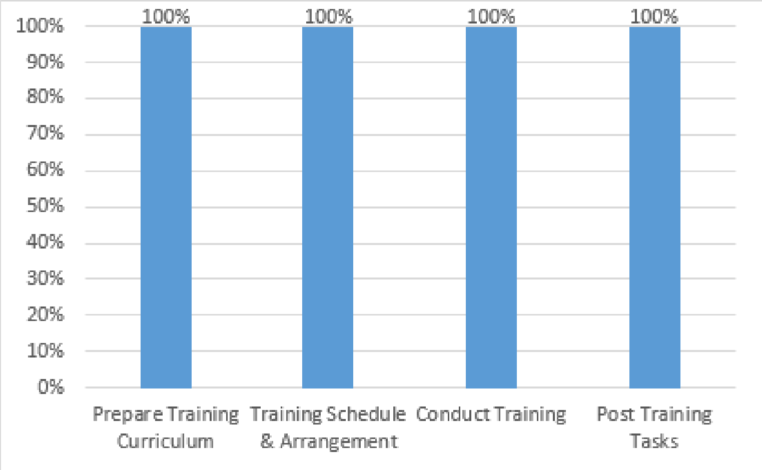
% COMPLETE

100%

MILESTONES

Name	Finish
Develop Training Curriculum	Mon 8/25/14
Conduct Training	Fri 10/10/14

TOP LEVEL TASKS - % COMPLETE



LATE TASKS

Name	Start	Finish	Duration	% Complete

LTBP Program Status Update

Rutgers – Project Status

Data Collection –

Project Tracking system

Rutgers - Legacy Data Mining

THU 5/1/14 - FRI 7/15/16

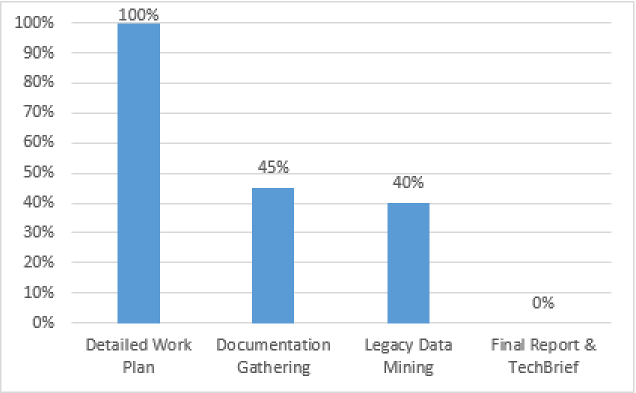
% COMPLETE

42%

MILESTONES

Name	Finish
Summary Finding Report Submittal	Fri 3/25/16
Final Report & Techbrief submittal	Fri 6/10/16

TOP LEVEL TASKS - % COMPLETE



LATE TASKS

Name	Start	Finish	Duration	% Complete
Receive Documents from States	Tue 8/5/14	Mon 12/8/14	90 days	31%
Conduct Paper Studies	Thu 5/1/14	Wed 4/15/15	250 days	48%
Draft Summary of Findings For Comparison	Thu 4/16/15	Wed 5/13/15	20 days	0%
Review of Summary of Findings	Thu 5/14/15	Wed 6/3/15	15 days	0%
Revised Summary of Findings	Thu 6/4/15	Wed 6/24/15	15 days	0%
Summary Finding Report Submittal	Fri 3/25/16	Fri 3/25/16	0 days	0%
Draft Final Reports	Mon 3/28/16	Fri 4/29/16	25 days	0%

LTBP Program Status Update

Rutgers – Project Status

Data Collection –

Project Tracking system

Status Update

LTBP Program

Rutgers - Bridge Portal

TUE 4/1/14 - WED 3/23/16

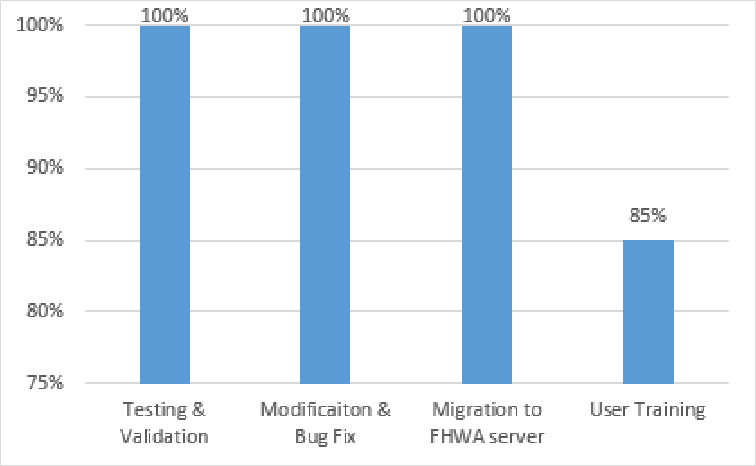
% COMPLETE

96%

MILESTONES

Name	Finish
Test & Validation Plan generated	Mon 6/23/14
Test Results Report Generated	Mon 9/15/14
Modifications Implemented	Mon 3/23/15
Deployment of System in FHWA servers	Fri 7/31/15

TOP LEVEL TASKS - % COMPLETE



LATE TASKS

Name	Start	Finish	Duration	% Complete
Create Adobe captivate	Thu 12/24/15	Wed 2/24/16	45 days	96%
Conduct webinars and workshops	Thu 2/25/16	Wed 3/23/16	20 days	0%

Rutgers – Project Status

Data Collection –

Project Tracking system

Rutgers - Field Data Collection

THU 5/1/14 - MON 7/4/16

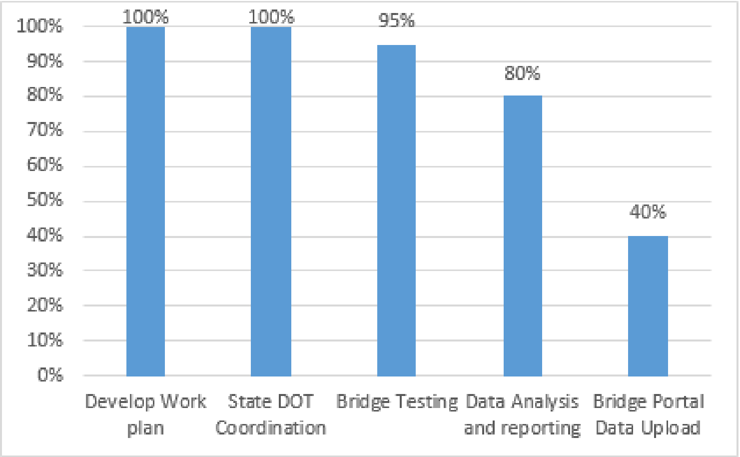
% COMPLETE

86%

MILESTONES

Name	Finish

TOP LEVEL TASKS - % COMPLETE



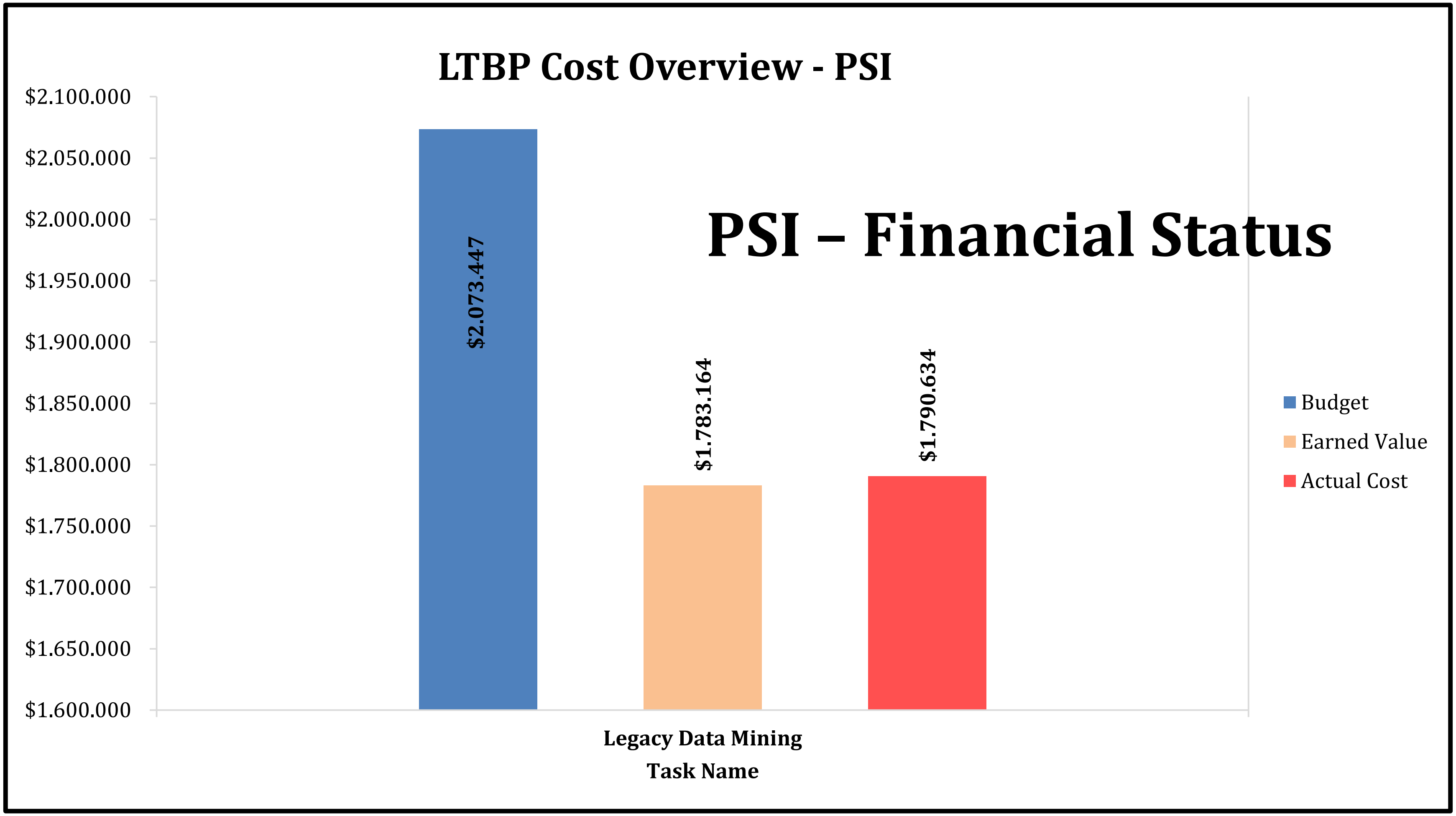
LATE TASKS

Name	Start	Finish	Duration	% Complete

LTBP Program

Status Update

Data Collection – Project Tracking system



Task Name	Budget	% done	Earned Value	Actual Cost	Variance	Variance %
Legacy Data Minin	$ 2,073,447.00	86%	$ 1,783,164.42	$ 1,790,634.05	$ (7,469.63)	0%

LTBP Program

Status Update

TU1406 COST ACTION

PSI – Project Status

Data Collection –

Project Tracking system

PSI - Legacy Data Mining

THU 6/12/14 - THU 7/21/16

% COMPLETE

92%

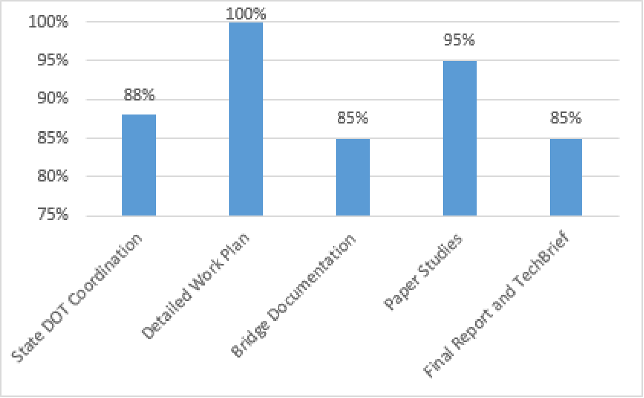
MILESTONES DUE

Milestones that are coming soon.

Name	Finish

% COMPLETE

Status for all top-level tasks. To see the status for subtasks, click on the chart and update the outline level in the Field List.



LATE TASKS

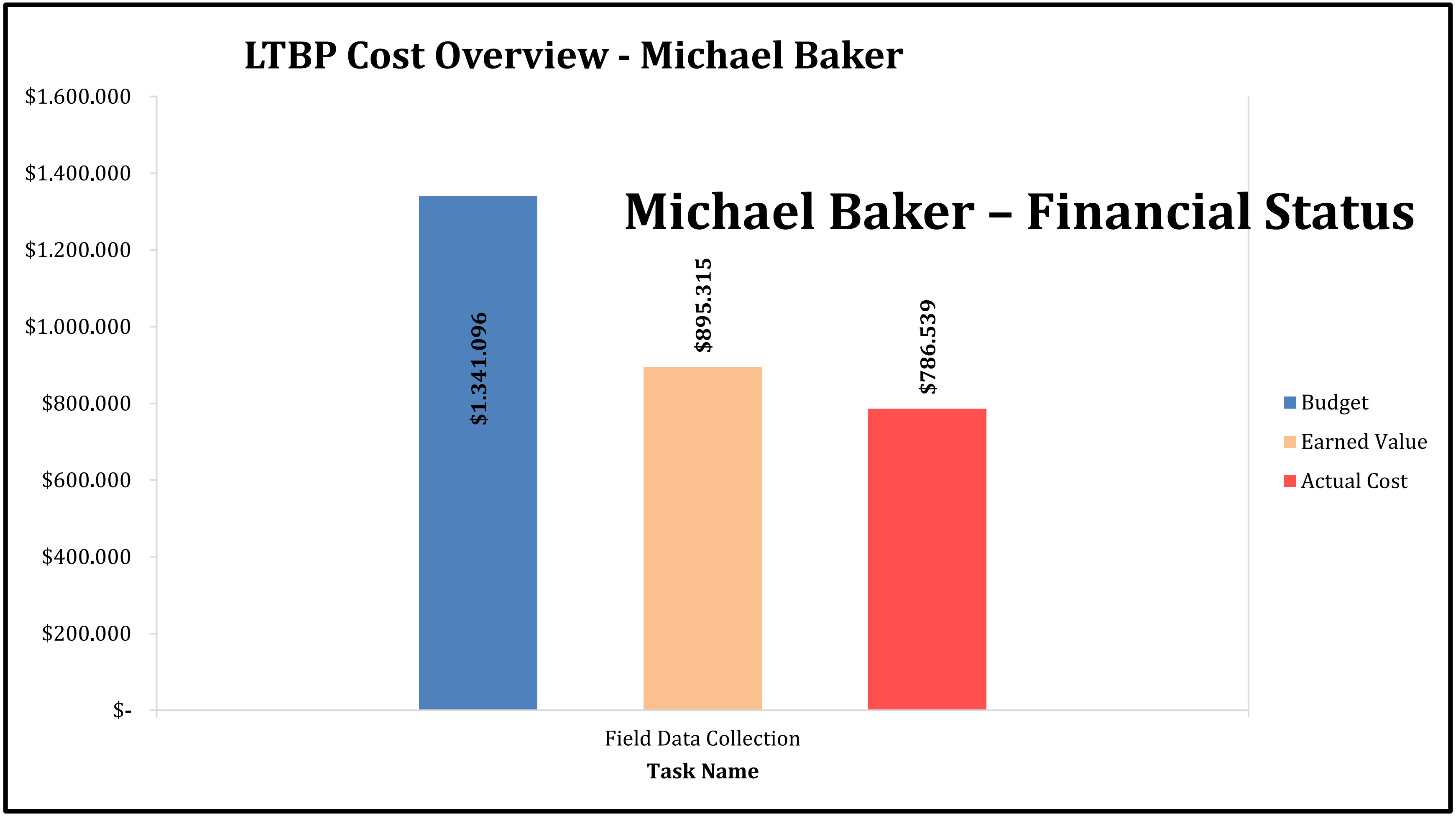
Tasks that are past due.

Name	Start	Finish	Duration	% Complete
State DOT Semi-annual Meetings	Mon 2/22/16	Tue 3/22/16	22 days	50%
Collecting Bridge Documentation Data	Mon 9/15/14	Fri 7/1/16	470 days	85%
Conducting Paper Studies	Mon 5/18/15	Thu 12/31/15	164 days	90%
Revised Summary of Findings	Thu 2/11/16	Fri 3/11/16	22 days	80%
Final Report	Fri 2/12/16	Sun 3/13/16	1 emon	60%

Status Update

LTBP Program

Data Collection – Project Tracking system



Task Name	Budget	% done	Earned Value	Actual Cost	Variance	Variance %
Field Data Collecti	$ 1,341,095.69	67%	$ 895,315.48	$ 786,539.00	$ 108,776.48	8%

Status Update

LTBP Program

Data Collection – Michael Baker - Project Status

Project Tracking system

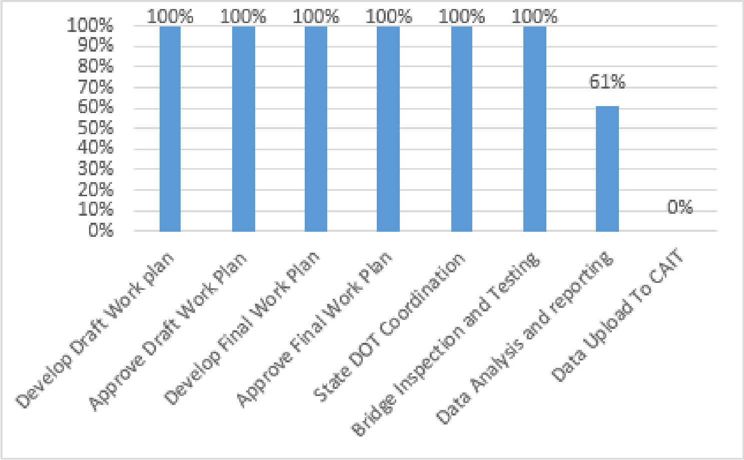
Michael Baker- Field Data Collection

TUE 4/21/15 - MON 5/9/16

% COMPLETE

78%

TOP LEVEL TASKS - % COMPLETE



MILESTONES

Name	Finish
Data Upload To CAIT	Mon 5/9/16

LATE TASKS

Name	Start	Finish	Duration	% Complete
Generate Report	Sun 11/22/15	Fri 5/6/16	121 days	25%

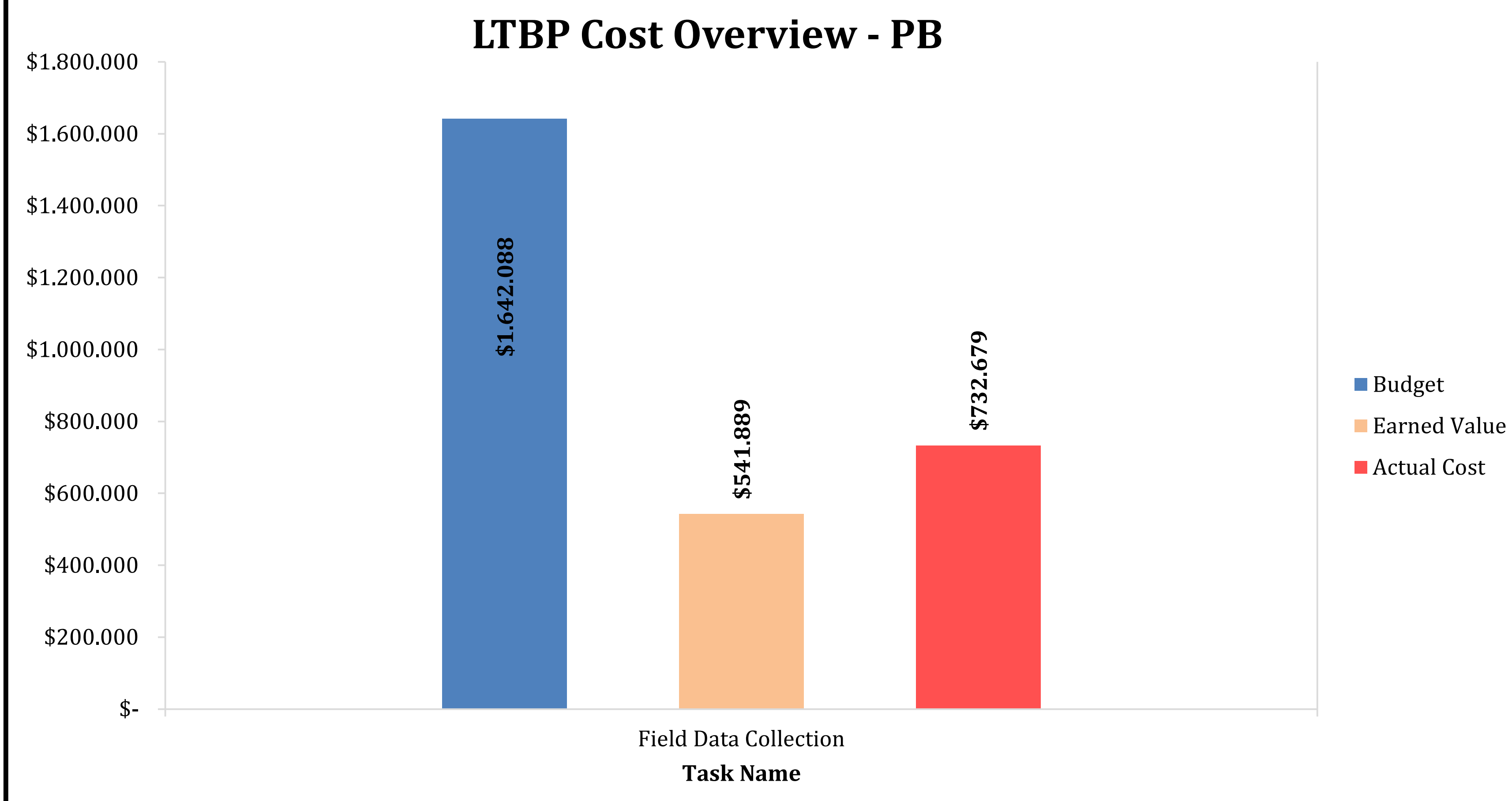
Status Update

LTBP Program

TU1406 COST ACTION

PB - Financial Status

Data Collection – Project Tracking system



Status Update

LTBP Program

TU1406 COST ACTION

Task Name	Budget	% done	Earned Value	Actual Cost	Variance	Variance %
Field Data Collecti	$ 1,642,088.00	33%	$ 541,889.04	$ 732,679.00	$ (190,789.96)	-12%

Data Collection – PB - Project Status

Project Tracking system

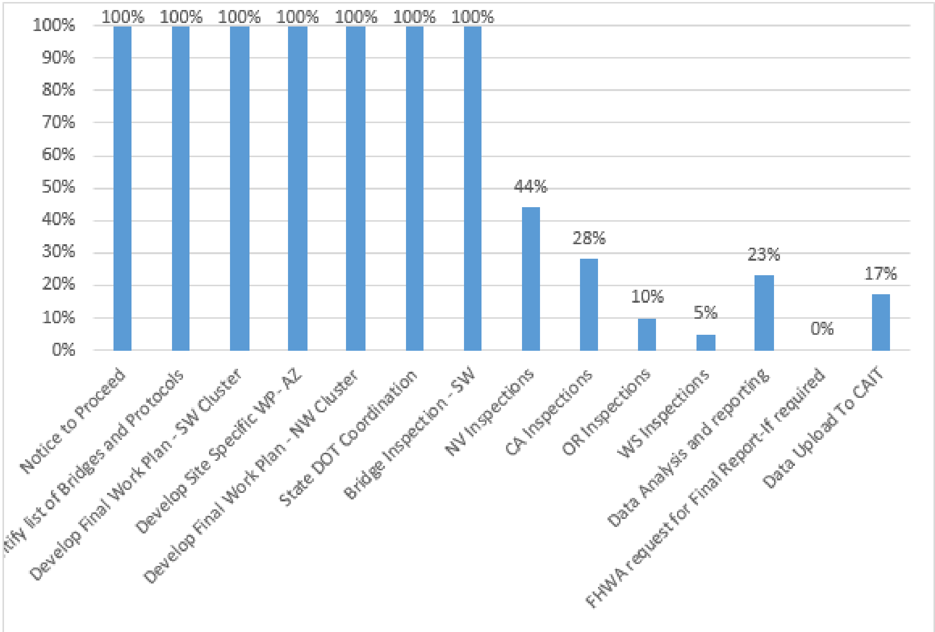
PB - Field Data Collection

TUE 9/9/14 - FRI 12/23/16

% COMPLETE

41%

TOP LEVEL TASKS - % COMPLETE



MILESTONES

Name	Finish
Data Upload To CAIT	Thu 8/25/16

LATE TASKS

Name	Start	Finish	Duration	% Complete
Data Analysis	Mon 8/3/15	Thu 8/25/16	245 days	23%

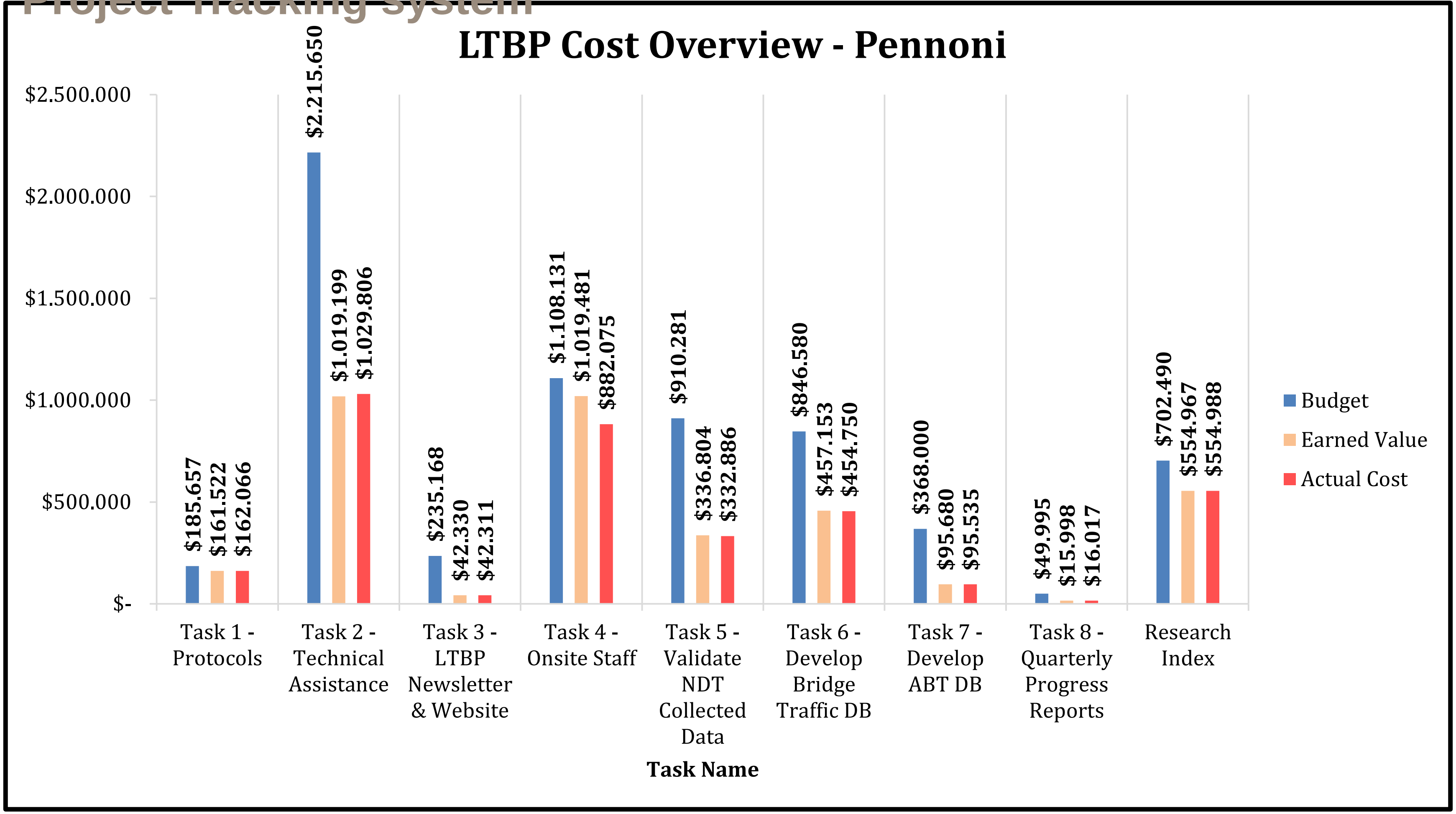
Status Update

LTBP Program

Data Collection –

Project Tracking system

Pennoni - Financial Status



Status Update

LTBP Program

Current and Planned Contracts

Critical items (requiring continued funding or initiating):

- Portal
- Deterioration and Cost Analysis Modeling (continue)
- Protocols development
- Data collection steel coatings (Initiate)
- Data Collection untreated decks – Initiate untreated deck, steel girder, joints, and bearings data collection (Initiate) – Contingency Plan

- Technical Support Services Contract
- Data Collection Contract

Contingency Plan

Potential Options For Reduced Funding:

Option 1 – Reduce the Number of Bridge Clusters - $5M

Option 2 – Reduce the Number of Bridges - $5M

Option 3 – Reduce Number of Bridge Types - $5M

Option 4 – Hybrid Approach (combination of full-scale accelerated testing and field calibration of data-driven models) $5M

Each potential option raises concerns both with research methodology as well as stakeholder involvement

TU1406
COST ACTION

WG2 and WG3 WORKSHOP

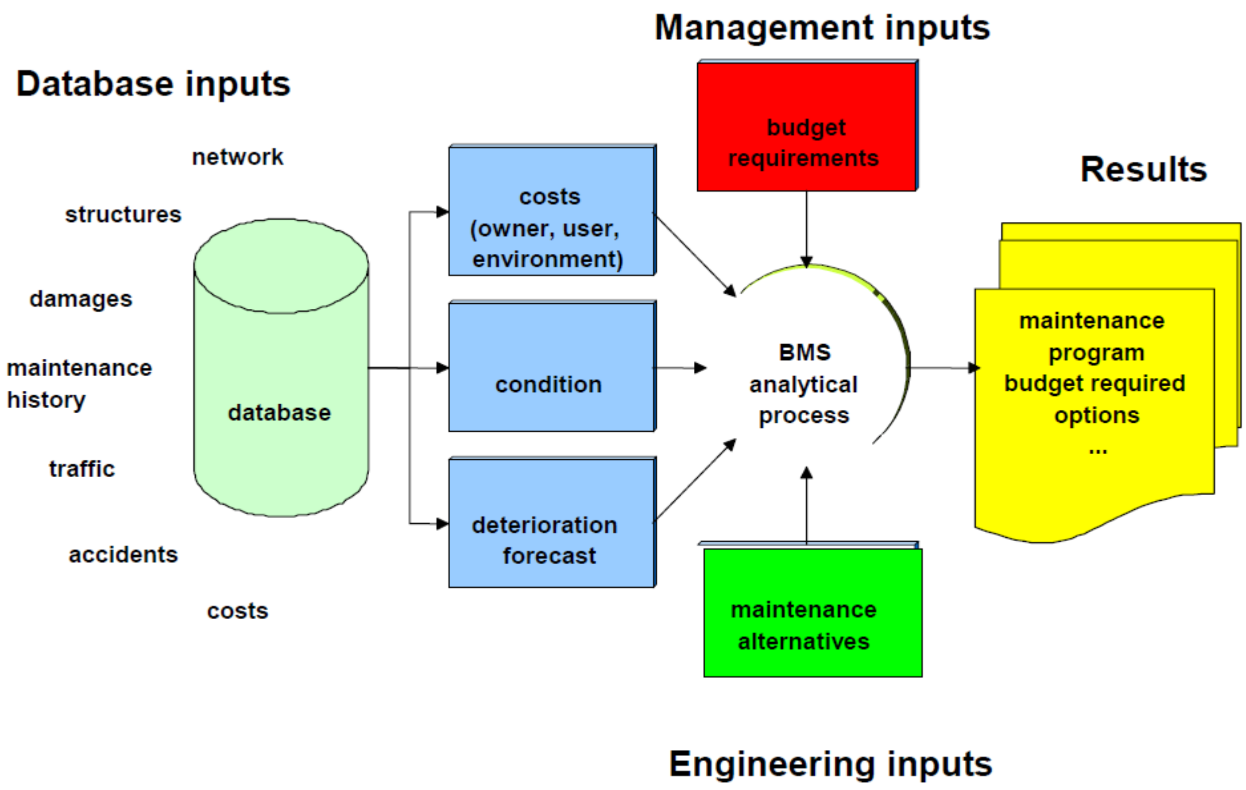
Bridge performance goals and quality control plans

Reflections on Quality Control Plans for Girder and Frame Bridges

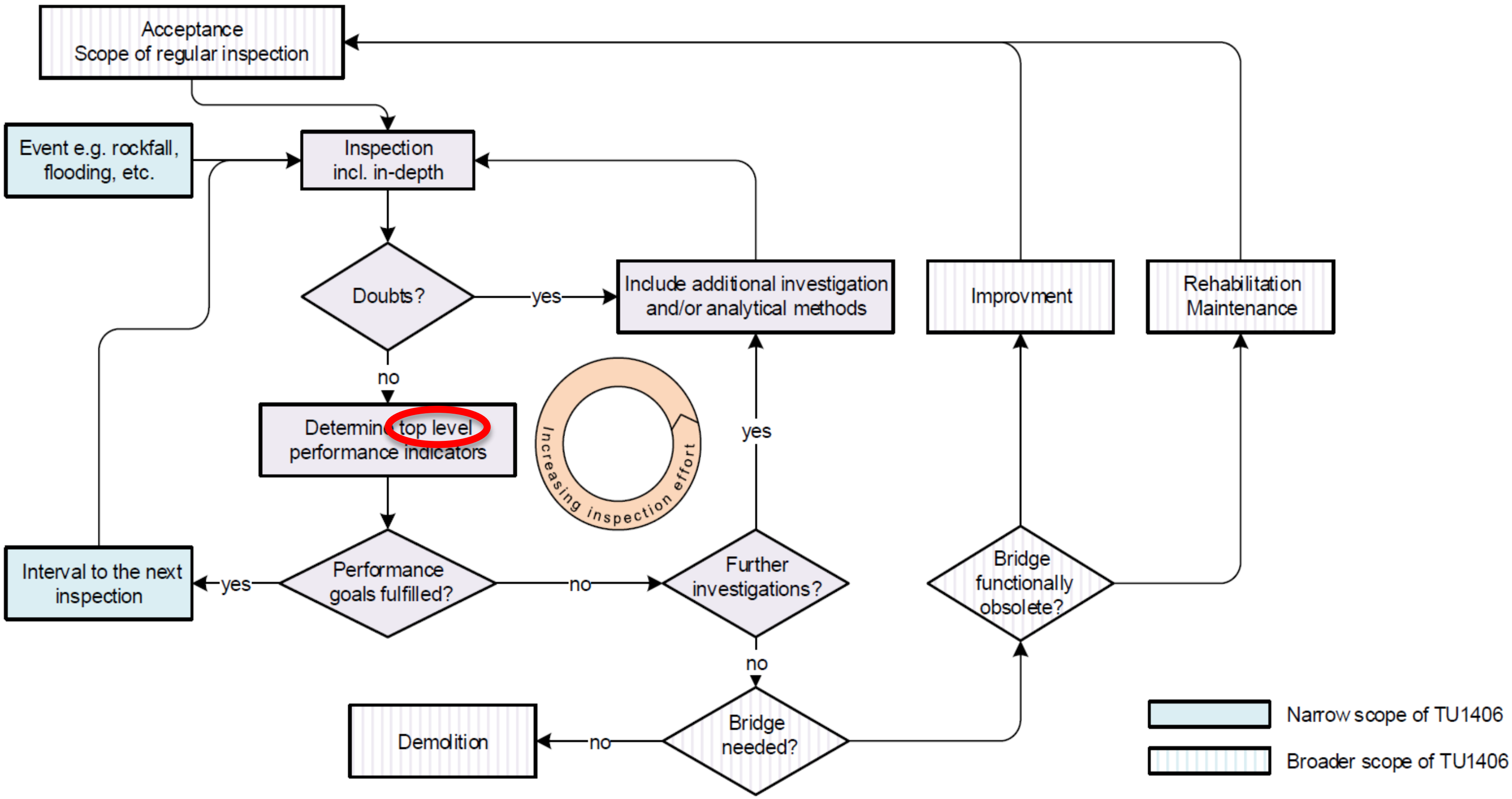
Poul Linneberg – COWI A/S, Denmark

20th – 21st October 2016
Delft, Netherlands

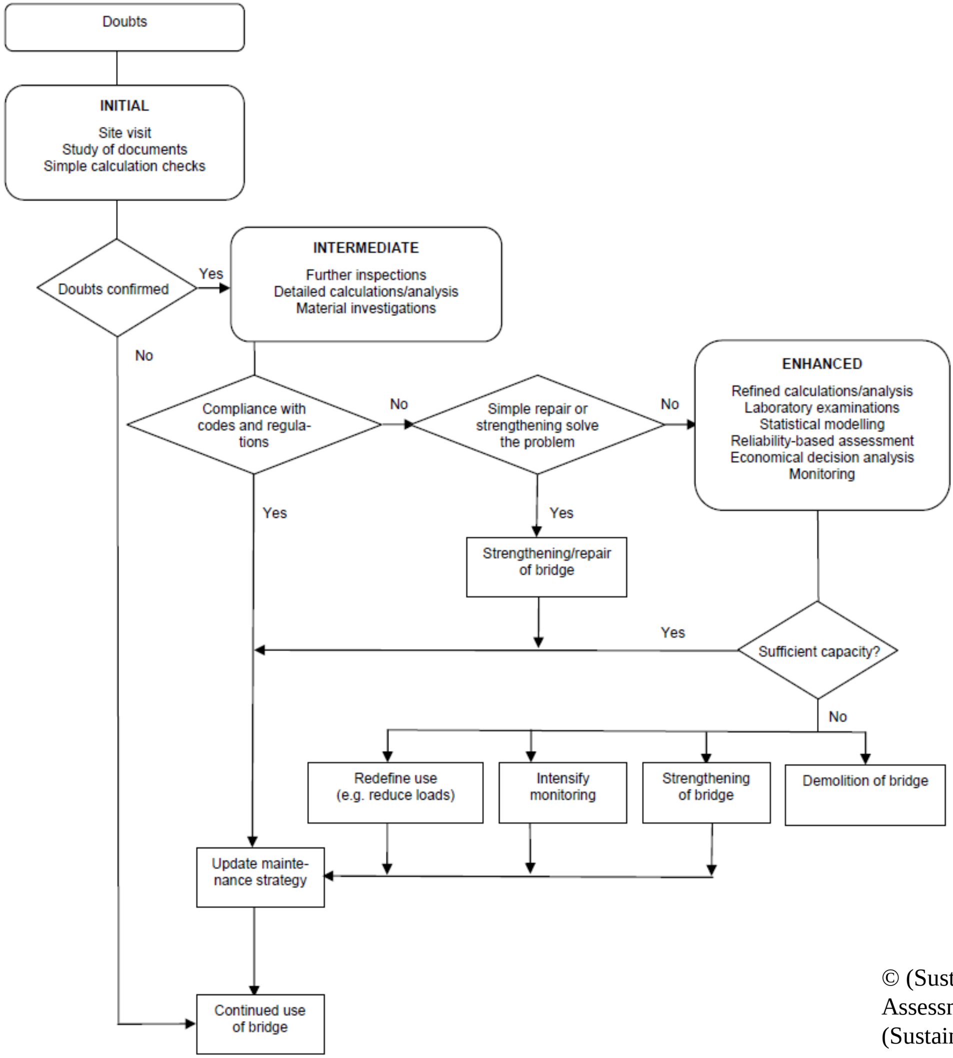
Bridge Management System (BMS), OECD 1992



Quality Control Plans



QC Plans based on a multilevel procedure



© (Sustainable Bridges, 2007).
Assessment methods for each level recommended by (Sustainable Bridges, 2007) or (COST 345, 2002)

Performance Indicators (PI)

In relation to establishment of QC plans, a breakdown into low-level and top-level PI's is appropriate:

Low-level PI's:

- Findings: Cracks, spalling, colour change, leakage, efflorescence etc.
- In-situ measurements: Crack widths, half-cell measurements, accelerometer measurements, measurement of electrical resistance, geo radar map etc.
- Lab testing: Chemical analysis, mechanical testing on specimens etc.

Top-level PI's (ref. Figure) are based on low level PI's:

- Qualitative: Condition rating
- Quantitative: Probabilities of undesired scenarios

PI's are evaluated at the time of commissioning, inspections and interventions (maintenance, repair and replacement).

Performance goals (PG)

Performance goals are understood as quality requirements.

Performance goals could be satisfying a threshold or extremising (no threshold, i.e. maximising or minimising).

Scope of Work

Based on results from COST TU1406 WG1 and WG2 as well as existing approaches WG3 shall provide a methodology with detailed step-by-step explanations for establishment of QC plans.

Network issues (downstream consequences) as well as landmark bridges are not treated by WG3.

WG3 work in general comprise:

- Guideline on inspection intervals, methods and instruments
- Criteria for triggering more detailed investigations, safety and serviceability checks or maintenance interventions

QC plans will address the dynamics and uncertainty of the processes that may significantly comprise the bridge performance.

Methodology for handling uncertain information and multi-criteria analysis etc. for prioritization is not included in this presentation, refer to e.g. (Belgrade, 2016). Refer e.g. Saydam (2013) on simplified risk-based life-cycle management framework.

This presentation only deals with Girder and Frame bridges. A vast majority of these bridges are made of concrete, i.e. metallic bridges are excluded from this presentation.

Triggering Criteria related to Degradation Processes

The condition of a highway bridge can be detrimental affected by various factors. These may act singly or in combination to generate functional, load-carrying and long-term durability problems.

The defects that may appear on a bridge are the result of active degradation processes. The analysis of degradation processes and their relations with defects allows identification of defect causes. The information on degradation processes is very important for planning of preventive maintenance as well as for planning rehabilitation of a structure. By means of reliable information on the degradation processes, a maintenance strategy can be optimized in order to reduce the number of interventions (i.e. reduce Life Cycle Costs and traffic disturbance).

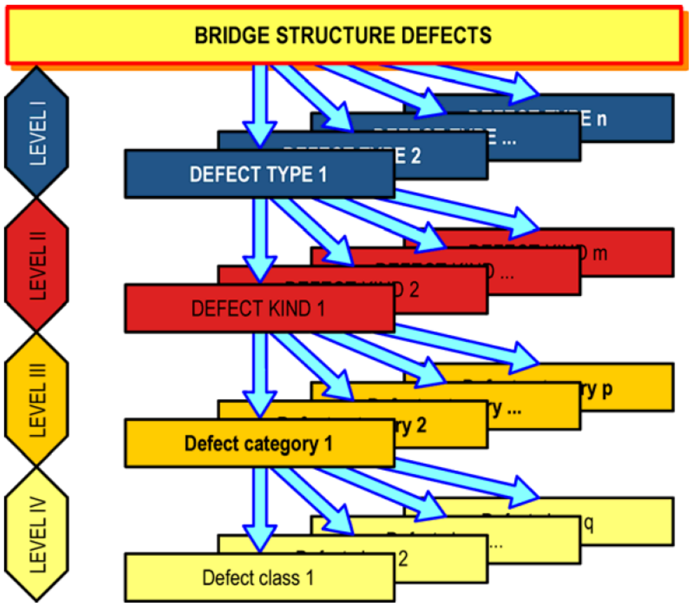
Defects result from:

- Design
- Materials
- Construction
- Loading
- Environmental conditions

Degradation mechanisms / Defect type	Chemical				Physical							Biological					
	Alkali – Aggregate Reaction (AAR)	Carbonation	Corrosion	Crystallization	Leaching	Oil and fat influence	Salt and acid actions	Creeping	Fatigue	Freeze-thaw action	Influence of high temperature	Modification of foundation conditions	Overloading	Shrinkage	Water penetration	Accumulation of dirt or rubbish	Living organisms activity
Contamination	■		■		■	■	■				■				■	■	■
Deformation				■				■			■	■	■	■			
Deterioration	■	■	■	■	■	■	■		■	■	■				■		■
Discontinuity			■	■					■	■	■	■	■	■			
Displacement												■	■			■	
Loss of material			■	■						■	■		■		■		■

In order to assess degradation processes, defects should be graded with respect to their nature, intensity and extent. Gradation should be done in a manner that fits the type of damage, the cause of damage, and the material forming the structural element. Degradation mechanisms in relation to defects for concrete bridges has been summarized by (Sustainable Bridges, 2007) and (COST 345, 2002).

Triggering Criteria related to Degradation Processes



© (Sustainable Bridges, 2007).

CLASSIFICATION OF CONCRETE BRIDGE DEFECTS			
LEVEL I	LEVEL II	LEVEL III	LEVEL IV
Contamination	Concrete	Inorganic	Aggressive
			Neutral
		Organic	Penetrating
			Superficial
	Protection	Inorganic	Aggressive
			Neutral
		Organic	Penetrating
			Superficial
Deformation	Concrete	Deflection	
	Protection	Deflection	
Deterioration	Concrete	Modification of chemical features	Calcium hydroxide reduction
			pH factor reduction
			Salt concentration increase
		Modification of physical features	Absorbability increase
			Elastic modulus change
			Embrittlement increase
			Frost-resistance reduction
			Permeability increase
			Porosity increase
			Strength reduction
	Protection	Modification of chemical features	Calcium hydoxide reduction
			pH factor reduction
			Salt concentration increase
		Modification of physical features	Adhesion reduction
			Embrittlement increase
			Fading
			Frost-resistance reduction
			Permeability increase
			Porosity increase
	Reinforcement and prestressing system	Modification of physical features	Bond reduction
			Strength reduction
Discontinuity	Concrete	Crack	Irregular
			Longitudinal
			Skew
			Transverse
		Delamination	
		Fracture	
	Protection	Crack	
		Delamination	
		Fracture	
	Reinforcement and prestressing system	Crack	
		Fracture	
Displacement	Excessive	Rotation	
		Translation	
	Limited	Rotation	
		Translation	
Loss of material	Concrete		
	Protection		
	Reinforcement and prestressing system		

Triggering Criteria related to Degradation Processes

Inspections are undertaken to:

- check the design assumptions underlying the quantification of some actions
- detect changes in use that could affect the safety or serviceability of a structure
- detect damage due, for example, to vehicle impact, ground movement and vandalism
- detect signs of structural distress due to overloading
- identify areas of material degradation (cause, extent and rate)
- provide a basis for determining structure-specific loads
- provide information of the cost-effectiveness of various remedial measures

As analyzed by (Sustainable Bridges, 2007) various inspection regimes have been implemented within Europe. The Sustainable bridges project suggested a common procedure, if no other procedure is required by national recommendations. Recommendations are also provided in Fib bulletin 22 (2003).

Furthermore, as summarized by A. Kedar at Cost TU 1406 meeting (Belgrade, 2016) inspection interval and type is influenced by:

- Inspection quality
- Bridge condition
- Cost
- Bridge type
- Bridge location

Triggering Criteria related to Degradation Processes

Proposal by SB-project	*Name of inspection*	*Inspection intervals*	*Education of bridge inspector*	*Inspection technique/ Result*
Regularly inspection	Surveillance	continuously	Track staff	Visual from ground
	Routine inspection	1 year	Inspector	Visual
	Principal inspection	3 years	Inspector + Structural engineer (if needed)	Visual + simple tests - if needed to guarantee the bridge safety level (e.g. fatigue, scour)
	General inspection	6 years	Structural engineer + specialist (in case of doubts)	Visual inspection in touching distance
	Special investigation	In case of doubts (accident, earthquake, flooding)	Specialised laboratories	NDT, static, dynamic load tests, monitoring, depending on the cause of doubt High tech tests,
Long-term evaluation	Monitoring	In case of doubts	Specialised laboratories	If needed: Strain, load, temperature, inclination
Condition Assessment (CA)	Digital implementation of inspection results into the CA	Continuous data update using results from inspection and/or monitoring	Railway administrator with Bridge-inspection- and IT-specialisation	Data processing of information in a database

© (Sustainable Bridges, 2007).

With due reference to Figure 2 (multilevel assessment procedure) a phase wise division of inspection methods may be formulated, see Figure 3.

Triggering Criteria related to Degradation Processes

With due reference to multilevel assessment procedure a phase wise division of inspection methods may be formulated

Phase I Initial Visual inspection, NDT with hand hold tools	Phase II Intermediate NDT with hand holdtools	Phase III Enhanced Extensive NDT image processing automised NDT
Visual inspection Simple NDT, e.g.: - Cover meter - Passive thermography - Rebound hammer	Update of drawings, Detection of In-service defects - NDT hand hold tools - Radar, Ultrasonic echo - Impact echo - Active thermography	Update of models (e.g.: Reinforcement) - Data fusion of different NDT-Data - SAFT-reconstruction - Frequency or phase evaluation of thermo-data

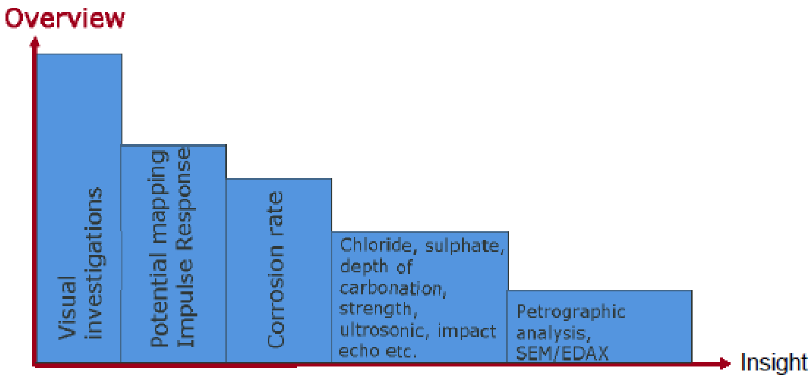
Ultrasionic echo: steel plates

Ultrasonic-echo: RC-Bridge

Automised NDT-scanner

Triggering Criteria related to Degradation Processes

Some test methods provide good overview others a good insight. The challenging in any investigation is finding the optimum combination.



For a summary of NDT and semi-destructive testing capabilities for concrete structures, ref. (COST 345, 2002).
For a more elaborate guidance on inspection and condition assessment of exiting bridges, ref. (Sustainable Bridges, 2007) and (Fib Bulletin 22, 2003).
Guidance on load tests and monitoring, ref. (COST 345, 2002) and (Fib Bulletin 22, 2003).

Triggering Criteria related to Degradation Processes

Table 5-4: Limitation and interference of NDT-methods with railway operating infrastructure, time consumption only for pure measurement (without equipment installation)

NDT-Method	***Investigated details***	***Limitation in use*** *Accuracy of the method incl. characteristics of the material*	***Railway specific issues / Traffic interruption***	***Roughly estimated time expenditure*** *for 1 point/m or 1 m or 1 m²*	***Remarks***
Reinforced concrete structures, prestressed concrete structures					
Visual	Contamination, material loss, deterioration, displacements, cracks	Cracks <0,1 mm, only surface observation, loose cover	No, except upper side of slabs below ballast	Main inspection MI (every 6th year); M an days: l< 10m 0,5 days l< 30m 2 days l< 50m 3 days l< 100m 6 days l> 100m 20 days Simple inspection (once a year); < 100m 0,5 MI > 100m 0,7 MI	Inspection time depends on span; Time according to highway bridge inspection in Germany
Radar-echo 500 MHz, 900 MHz, 1,5 GHz (Ground penetrating radar, Georadar, impulse radar)	Girder web thickness, slab thickness embankments/ retaining wall reinforcement, tendon ducts, inhomogeneity, humidity, metal inclusions (anchors)	Appropriate for depth max. 2 m Defect size: in homogeneous material: ~ 5% of the depth, in heterogeneous material: ~ 10 % of the depth, Dense near surface reinforcement prevents deep penetration	No influence 500 GHz >> 50Hz	No Hand driven tool, line scan: < 1 min/m Automated scanner: < 0,1 m/s ~ 80 s/m	Access from one side, Use in assessment phase 3
Radar transmission 500 MHz, 900 MHz, 1,5 GHz	Girder web thickness, reinforcement tendon ducts inhomogeneity	Method in development, better imaging of inhomogeneity expected	No: 1500 GHz >> 50Hz	High time consumption for data acquisition and processing	Research level, Access from both sides
Ultrasonic echo (US) (US-array without coupling agent): transversal: 50 kHz longitudinal: 100 kHz	Reinforcement, tendons location, grouting of tendons, local inhomogeneity, thickness, metal inclusions	Depth: 5-40 cm Defect size: 10-30 mm Train traffic noise, building activities, as drilling, anchor dysfunction or other noise can influence the acoustic signal acquisition	Low influence from electromagnetic signalling: >> 50 Hz Interrupt measurement during train traffic	Point by point (hand hold tool) < 6 s/point Line scan 1 m (stepper), grid á 5 cm: 100 s/m Area scan 1 m² (automated scanner US+IE): c< 80 min/m²	Access from one side: Depending on the task to solve: Frequency 50-300 kHz. Assessment phase 3

Table 5-4: Limitation and interference of NDT-methods with railway operating infrastructure, time consumption only for pure measurement (without equipment installation)

NDT-Method	***Investigated details***	***Limitation in use*** *Accuracy of the method incl. characteristics of the material*	***Railway specific issues / Traffic interruption***	***Roughly estimated time expenditure*** *for 1 point/m or 1 m or 1 m²*	***Remarks***
Reinforced concrete structures, prestressed concrete structures					
Acoustic emission	Determination and localisation of propagating (active) cracks	10 % of the sensor distance in the array, Influenced by low temperature, defects, deformation rate	No influence of the signalling Check data plausibility during train traffic	Only global assessment with monitoring	Method feasible, research level (Experts use only)
Air coupling Ultrasonic	Voids	Reflected surface waves influence the emitted waves	No investigations, comparable to US		Research level
Impact-echo	Thickness, delamination between two concrete layers, location of voids, quantification of cracks	Train traffic noise, building activities, as drilling, anchor dysfunction or other noise may influence the acoustic signal acquisition	Signalling not affected, influence from passing train	< 3s/ point Area scan 1 m² (automated scanner US+IE): < 80 min/m²	Surface waves may influence the result, solution: IE in transmission
Active thermography	Near subsurface voids, moisture, plaster delamintion, control of strengthening measures	Safe , no radiation, Accuracy depends on depth of the void, camera, distance and further limits, e.g.: 1 m²: 65000 pxl.: ± 1 cm²	No, if applied from underneath the track No el.-magn. waves	Depending on the required surface and depth resolution	No moving / scanning technique
Radiography (cobalt, γ-ray)	Metal inclusions, cables, wires, tubes	Defect size ~20 x 1-2 mm, Restriction due to radiation, no traffic during test	Safety requirements for radiation required, Signalling does not influence	Radiation exposure depending on activity of the source, thickness of the concrete	Last phase in reassessment, Phase 3

Triggering Criteria related to Degradation Processes

Table 6-2: Use of non-destructive for detection of defects in concrete structures (coloured NDT methods are enhanced or developed within the project Sustainable Bridges)

	Design compliance testing							***Defects from deterioration (durability, environmental effects, excess loading)***											
Defect/ void/ inhomogeneity/ defect from deterioration	Thickness measurement	Concrete cover	Honeycombs	Reinforcement or tendon location (diameter estimation)	Insufficient grouting	Concrete properties (strength, elastic modulus etc.)	Hidden degradation, insufficient backfill	Surface (concrete) condition	Delamination/ spalling	Moisture content	Crack location	Physical degradation (freeze-thaw-cycling, abration)	Surface crack depth	Active cracks	Cracked strands	Remaining thickness	Remaining diameter	Corrosion of reinforcement	Chemical attack (sulphate, chlorine, AKR)
Visual and simple NDT																			
Visual inspection								■	■		(■)	■		(■)		(■)	(■)	(■)	(■)
Tapping (Hammer)								■	■		■	■							
Rebound hammer						■		(■)	(■)										
Covermeter		■		■															
Geodetic measurement	■																		
Acoustic NDT-methods																			
Ultrasonic echo	■				■	■			■		■	(■)	(■)			■			
Ultrasonic transmission			■	■	■				■							■			
Ultrasonic wave velocity																			
Acoustic emission 4)									■					■	(■)			(■)	
Impact echo	■		■	(■)	(■)				■				■						
Electromagnetical methods																			
Impulse radar (GPR)	■	■	■	■						(■)	(■)	■ 3)							
Radar tomography			■ 3)	■ 1)						■						■			
Electromagnetic induction															■				
Radiography																			
Radiography (High safety requirements)		■	■	■	■									■					
Radio-isotope moisture-meter										■									
Thermography (in WP6)																			
Active thermography		■ 3)	■						■	■ 2)									
Electrochemical and spectroscopic methods																			
LIBS																			■
Potential field																		■	■
Measurements and minor destructive test																			
Static load test														■					
Dynamic load test														■					
Sampling (Coring)	■	■				■	■			■	■	■	■						
Microscopy																		■	■

1) only tendon ducts, not simple reinforcement
2) only moisture near surface
3) not reliably applied
4) in combination with static or dynamic load test

Triggering Criteria related to Degradation Processes

A triggering criteria related to a degradation process defines if more detailed investigation/assessment or an intervention (maintenance, repair or replacement) is needed.

The condition of a highway bridge can be detrimental affected by various factors. These may act singly or in combination to generate functional, load-carrying and long-term durability problems. Because of this, there is no 1:1 relation between a vast amount of damage processes and a possible triggering criterion.

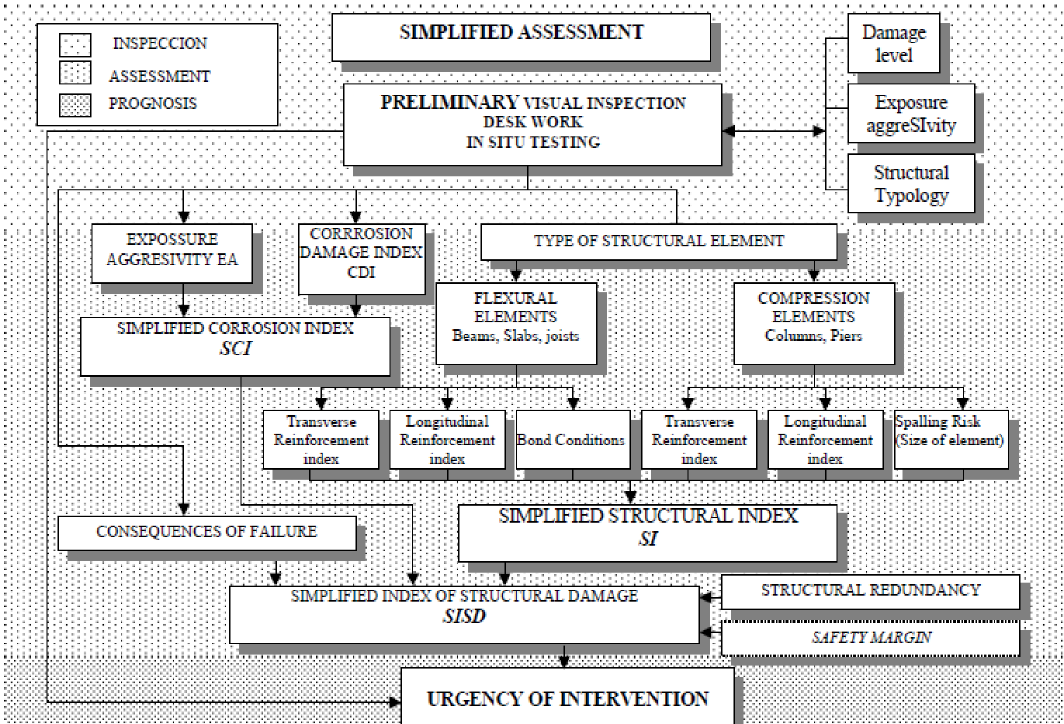
In many countries triggering criteria's are related to a top level performance indicator that may be qualitative (condition rating) or quantitative (probabilities of undesired scenarios). As a remark, condition rating in some countries does not account for in-service loads acting on the structure, i.e. it does not provide a direct measure of the level of safety and therefore of the priority of interventions.

ContectVet (ContectVet, 2001) has proposed a simplified and a detailed assessment procedure. These procedures are formulated for corrosion-affected concrete structures. Similar guidance is provided by ContectVet for frost and ASR affected concrete structures. For the two assessment procedures, ContectVet has formulated associated triggering criteria's (urgency of intervention). They may serve as inspiration for COST TU1406.

A detailed assessment ends in a management strategy. The management strategy should be technical and economic optimal given the actual constraints (budgets, politics etc.).

Fib bulletin 22 (Fib bulletin 22, 2003) also summarize reliability states, attributes and maintenance actions. The project REHABCON (REHABCON, 2004) has also summarized the decision process.

Triggering Criteria related to Degradation Processes



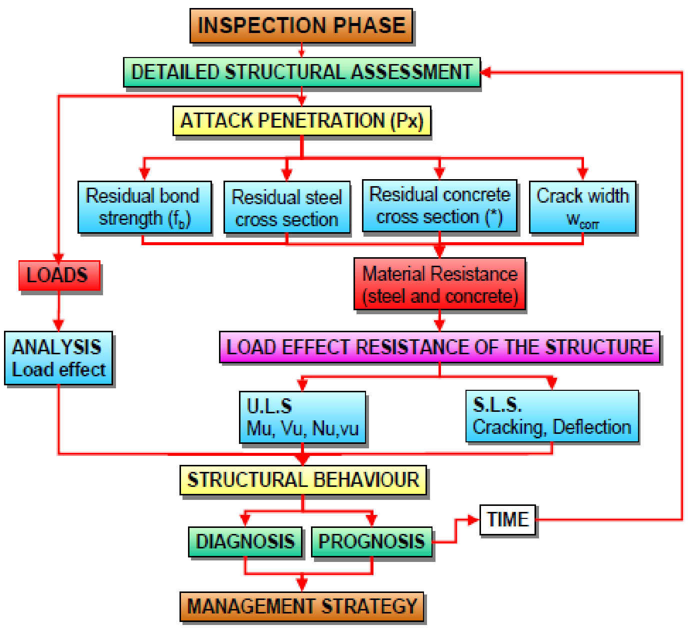
SISD value	**Urgency of intervention**	**Action needed**
Negligible	> 10	Periodic inspections
Medium	5 – 10	Reassess structure during this time
Severe	2 – 5	Structural assessment within this time
Very Severe	0 – 2	Repair or detail structural assessment within this time

The type of intervention will differ depending on the case under study:

- For structures whose Urgency of Intervention, determined by means of Table 3.14, is higher than 5 years, recommendation is given to reassess the structure after that time, preferably after having monitored the actual corrosion rates in the structure.
- For structures which Urgency of Intervention is situated between 2 and 5 years, is recommended that a specialist consultant, in no more than that time carry out a detailed assessment.
- For structures whose Urgency of Intervention is lower than 2 years, most probably a repair action should be needed. The development of such repair project should previously require in most cases an immediate detailed assessment.

Triggering Criteria related to Degradation Processes

The general process of the detailed assessment is exposed in the following diagram:



Five main steps can be distinguished:

1. Inspection phase, to collect all relevant data regarding the structure and its environment.
2. Determination of corrosion effects on concrete and steel, and specifically, on how bond properties, the steel cross section, the geometry of concrete section and cracking are modified by corrosion.
3. Load evaluation and analysis, taking into account the modified sections of concrete and steel.
4. Determination of the load effect resistance of the structure considering the new material properties.
5. Verification of the structural behaviour in both the present state (diagnosis) and in the future (prognosis) by means of the ULS and SLS Theories.

		Action to be taken
ULS	R vs. S	
	R > S	No actions are required until the estimated time when the effects of the actions become higher than the structure resistance is reached
	R<S<1.1R	Reassessment within 1 year
	S > 1.1R	urgent repair work
SLS		Agreement with owner's requirements

Triggering Criteria related to Demand Processes

Demand	Inspection interval	Primary inspection method
› **Operational** › Traffic volume (max) › Traffic loading › Gross Vehicle Weight (max) › Axle loads (max) › Width (max) › Height (max) › Maintainability (max) › Life Cycle Costs (min) › Visual appearance (max)	› **Operational** › Ad hoc › Traffic loading › Ad hoc › Ad hoc	› **Operational** › Survey › Traffic loading › Inspection and assessment › Inspection and assessment
› **User** › Reliability (max) › Availability (max) › Safety (min. fatalities/injuries) › Affordable travel (min)		
› **General (regulation by law or other measures)** › Human health (max) › Environ. protection › Climate change (min) › Noise (min) › Waste (min) › Etc.	› **General (regulation by law or other measures)** › Ad hoc › Environ. protection › Ad hoc › Ad hoc › Ad hoc › -	› **General (regulation by law or other measures)** › Survey › Environ. protection › Desk study › Survey › Survey › -

As shown above the concept of RAMS is included in the above list of demands. Also the Dutch concept of RAMSHEЄP is included in the above list of demands (Klatter at al, 2009).

The above demands may be contradictionary and are resolved in a political process and set by agencies in collaboration with professional organizations.

Triggering Criteria related to Demand Processes

Demand	Triggering criteria	Investigation/assessment or intervention
› **Operational**	› **Operational**	› **Operational**
› Traffic volume (max)	› User delay	› Capacity calc. / widening of bridge
› Traffic loading	› Traffic loading	› Traffic loading
› Gross Vehicle Weight (max)	› Bridge Class	› Reassessment or strengthening
› Axle loads (max)	› Axle load	› Reassessment or strengthening
› Width (max)	› Width	› Widening of bridge
› Height (max)	› Height	› Change of roadway profile (if possible)
› Maintainability (max)	› Budget constraints	› Refurbishment, upgrading
› Life Cycle Costs (min)	› Budget constraints	› Reassessment of operation and maintenance regime
› Visual appearance (max)	› Public perception	› Maintenance
› **User**		
› Reliability (max)		
› Availability (max)		
› Safety (min. fatalities/injuries)	› Fatality/injuries	› Reassessment of operation, refurbishment
› Affordable travel (min)	› Cost	› Reassessment of operation
› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**
› Human health (max)	› …	› Survey
› Environ. protection	› Environ. protection	› Environ. protection
› Climate change (min)	› Global Warming Potential etc.	› Reassessment of operation and maintenance regime
› Noise (min)	› dBA	› Reassessment of operation and maintenance regime
› Waste (min)	› Tonnes	› Reassessment of operation and maintenance regime
› etc.	› -	› -

Triggering criteria's may be technical or economic, but even technical triggering criteria's are balanced by budget constraints.

TU1406
COST ACTION

Thank you for your attention

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

PROJECT PERFORMANCE APPRAISAL FRAMEWORKS AS BLUEPRINTS FOR BRIDGE QUALITY CONTROL

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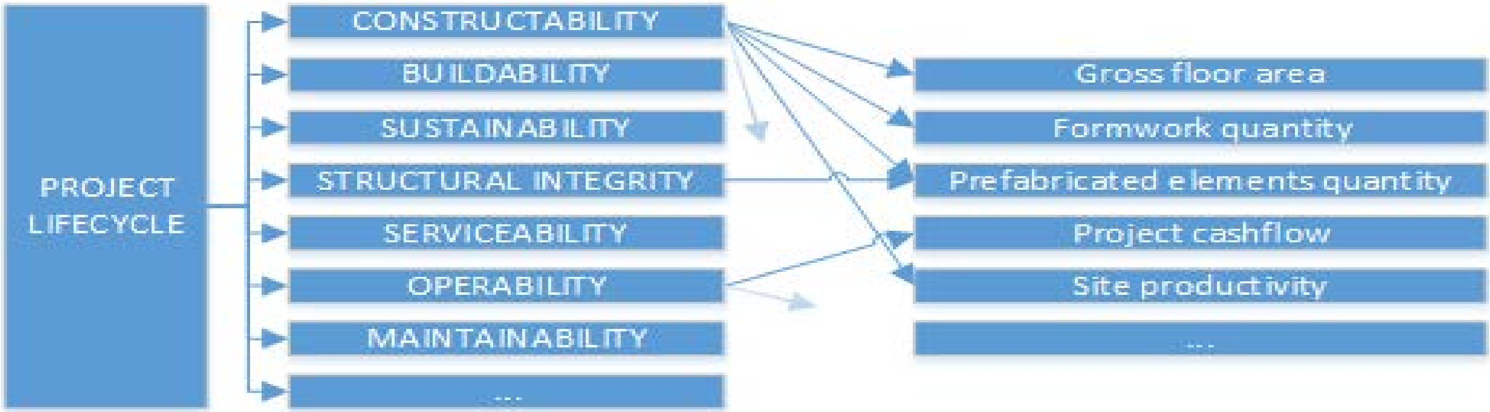
20th – 21st October 2016
Delft, Netherlands

PROJECT PERFORMANCE APPRAISAL FRAMEWORKS (PPAFs) – GENERAL REMARKS (1/2)

- Products of research efforts and the development of cognitive, mathematical and software tools
- Developed and established in practice by agents of the AEC industry and/or national institutions, authorities and regulations
- Utilized in all kinds of projects (buildings, infrastructure and special project cases)

PROJECT PERFORMANCE APPRAISAL FRAMEWORKS (PPAFs) – GENERAL REMARKS (2/2)

- Implemented, simultaneously or separately, in:
  - The monitoring of the full project lifecycle
  - The evaluation of distinct project lifecycle notions (e.g. constructability, buildability, sustainability, structural integrity, serviceability, operability, maintainability etc.)
  - The computation of constituents of the project lifecycle notions (e.g. gross floor area, formwork quantity, prefabricated elements quantity, project cash flow, site productivity etc.)



PRESENTED PPAFs

- CONQUAS (CONstruction QUality Assessment System) – Singapore
- PASS (Performance Assessment Scoring System) – Hong Kong
- BDAS (Buildable Design Appraisal System) – Singapore
- BAM (Buildability Assessment Model) – Hong Kong
- SBTool – Portugal, Spain & Italy

CONstruction Quality Assessment System (CONQUAS) (1/2)

Table I. Items assessed regarding structural frame

Item	Points
Formwork	5
Reinforcement	10
Finished concrete	15
Concrete quality	8
Reinforcement quality	2

Table II. Items assessed regarding architectural works

Item	Points
Floors	8
Internal walls	8
Ceiling	4
Doors and windows	4
Rainwater down-pipes, plumbing, sanitary fittings	3
Installation of mechanical and electrical services	3
Components (permanent fixtures)	3
Roof	5
External walls	6
Material and functional tests	6

Table III. Items assessed regarding facilities of external works

Item	Points
Aprons and drains	2
Roadworks and carparks	2
Footpaths and turfing	2
Fencing and gates	2
Other areas, specific to the project	2

Checklist of the CONQUAS scoring system

- Developed by the Construction Industry Development Board (CIDB) of Singapore and compulsorily in effect since 1989
- Appraises the quality of public sector *buildings* in terms of (i) the **structural frame**, (ii) the assorted **architectural works** and (iii) the **external works**
- A scoring system with a checklist related to aspects (i)-(iii) is utilized by state evaluators to produce the CONQUAS score for the whole building and/or certain elements of it

CONstruction Quality Assessment System (CONQUAS) (2/2)

- Showcases validated positive correlations with site productivity
- Incorporates scoring thresholds that grant tendering advantages to contractors achieving or surpassing them

Bonus/Discount Threshold Scores (1/4/2016 to 31/3/2017)

Building Category	Bonus Threshold Score for FY16	Discount Threshold Score for FY16
Residential	91.3	85.3
Commercial	92.9	86.9
Institution	89.2	83.2
Industrial/Others	85.9	79.9

- Appraises **finished projects** and focuses on the **classification of contractors**
- Followed by the establishment of CE CONQUAS for various, and not only building, public sector projects (e.g. sewage networks, marine structures etc.)

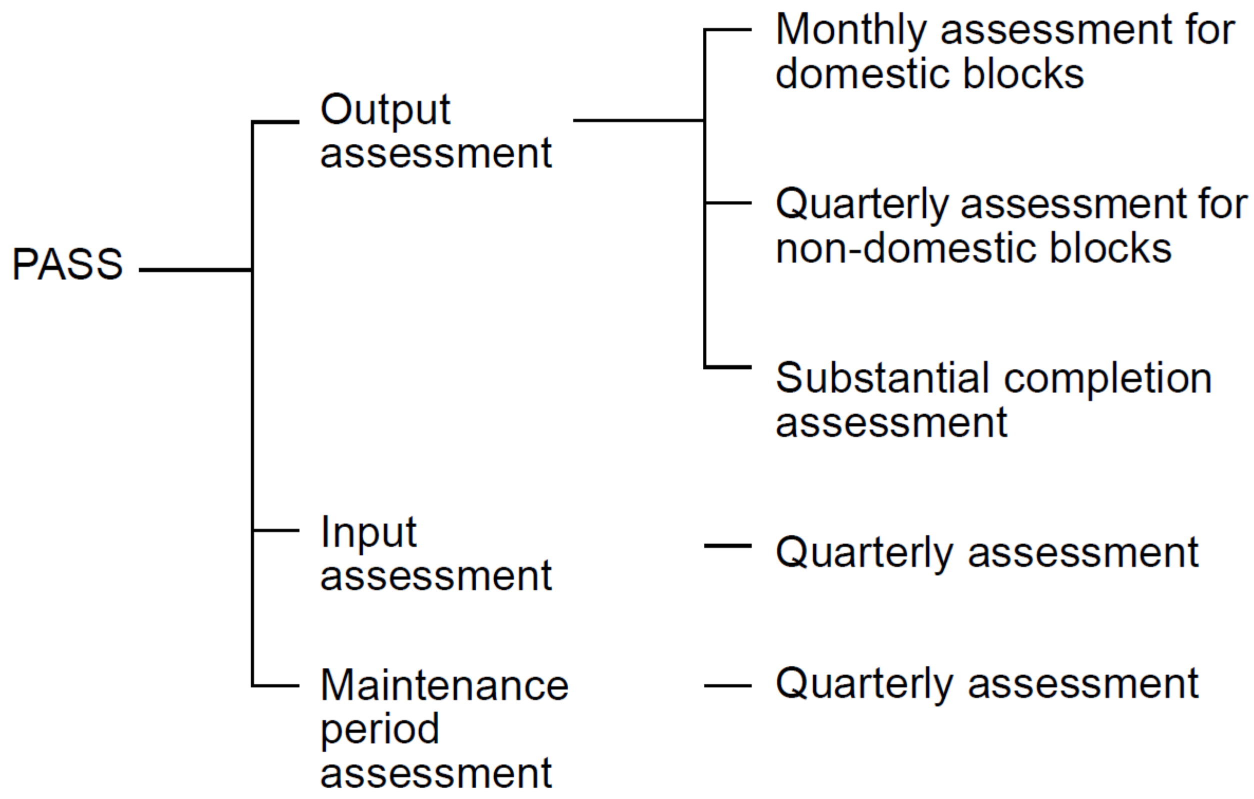
Performance Assessment Scoring System (PASS) (1/2)

- Adapted from CONQUAS for Hong Kong and in effect since 1990
- Utilizes a similar to CONQUAS scoring system (with similar categories (i)-(iii), but also an additional category (iv) **other obligations**)

- Apart from completed buildings, also monitors **projects currently under construction**, *taking into account*
  - the contractor's managerial performance
  - the contractor's productivity
  - the contractor's conformance to the specified quality thresholds

  and allotting points for
  - the management, organization, coordination and control of works
  - the resources flow
  - the real-time schedule progress
  - the project documentation

Performance Assessment Scoring System (PASS) (2/2)



PASS schema hierarchy

The depicted output assessment is related to the score of construction itself, and the input assessment to the productivity and managerial notions

Buildable Design Appraisal System (BDAS) (1/2)

- Developed by the Building and Construction Authority (BCA) of Singapore
- Put in effect complimentarily to CONQUAS since the mid-’90s. The two form a composite project quality and performance assessment framework, primarily targeted to *high-rise buildings*

- Appraising the conformance of building designs to the notion of buildability as “the extend to which the design of a building facilitates ease of construction, subject to the overall requirements for the completed building”, for
  - better practical integration of design and construction
  - better deliverables
  - fewer discrepancies between the as-designed and as-built project states
  - more thorough satisfaction of the defined project objectives

Buildable Design Appraisal System (BDAS) (2/2)

Buildability Score of Building	=	**Buildability Score of Structural System (including Roof System)**
		+ Buildability Score of Wall System
		+ Buildability Score of Other Buildable Design Features
		+ Bonus Points
BScore (In mathematical terms)	=	**$50[\sum(As \times Ss)] + 40[\sum(Lw \times Sw)] + N +$ Bonus points**
where As	=	Asa / Ast
Lw	=	Lwa / Lwt
Aw	=	Awa / Awt
As	=	Percentage of total floor area using a particular structural design
Ast	=	Total floor area which includes roof (projected area) and basement areas
Asa	=	Floor area using the particular structural design
Lw	=	Percentage of total external & internal wall length using particular wall system
Lwt	=	Total wall length excluding external basement wall for earth retaining purpose
Lwa	=	External & internal wall length using particular wall system
Aw	=	Percentage of total external & internal wall areas using particular wall design
Awt	=	Total wall area, excluding perimeter wall of the basement. All internal walls in the basement are to be considered.
Awa	=	External & internal wall areas using particular wall design
Ss	=	Labour saving index for structural design
Sw	=	Labour saving index for external & internal wall design
N	=	Buildability Score for other buildable design features
Bonus points	=	Bonus points for single integrated components

The BDAS scoring system

- The scoring system measures, classifies and awards points to the buildability attributes of construction designs
- The 3S principle is promoted:
  - Standardization (e.g. repetition of grids, component sizes and connection details)
  - Simplicity (utilization of construction systems and connection details of low complexity)
  - Single integrated elements (combination of multiple components to form composite elements)
- There are validated positive correlations between high BDAS and high CONQUAS scores

Buildability Assessment Model (BAM) (1/2)

- Adapted from BDAS for Hong Kong in the early ’00s
- PASS and BAM form a composite project quality and performance assessment framework, primarily targeted to *buildings*

- Extends the 3S principle of BDAS into nine buildability factors (BFs):
  - BF1: economic use of the contractors’ resources
  - BF2: easy visualization and coordination of design requirements by the site staff
  - BF3: development and adoption of alternative construction details
  - BF4: overcoming of restrictive site conditions
  - BF5: standardization and repetition
  - BF6: freedom of choice between prefabricated and on-site works
  - BF7: simplification of construction details in case of non-repetitive elements
  - BF8: mitigation of adverse weather impact by enabling flexible construction schedules
  - BF9: consideration of site work sequencing in the designs

Buildability Assessment Model (BAM) (2/2)

Construction Systems

Structural frames	$23\ \Sigma(V_s \times BI_s)$
Slabs	$14\ \Sigma(A_s \times BI_s)$
Envelopes	$19\ \Sigma(A_e \times BI_e)$
Roof	$19\ \Sigma(A_e \times BI_e)$
Internal walls	$3\ \Sigma(A_{iw} \times BI_{iw})$

Proportionate area using a particular finishing system in a particular location i.e. ceilings, internal walls, internal floors, external walls or roof coverings

Buildability Index for particular finishing system in a particular location i.e. internal ceilings, internal walls, internal floors, external walls or roof coverings

Finishing systems	$2\ \Sigma(A_f \times BI_f)$
Building features	$5\ \Sigma(BI_{bf} \times cov_{bf}) \div$ Sum of BI_{bf}
Building services aspects	$2\ \Sigma(BI_{bs} \times cov_{bs}) \div$ Sum of BI_{bs}
Site specific factors	$12\ \Sigma\ BI_{ss} \div$ Sum of applicable BI_{ss}
Innovative ideas of improving buildability	**10** (max.)

Total buildability score = 100 (max.)

BI_{bf}: Buildability Index for a particular building feature

BI_{bs}: Buildability Index for a particular building services aspect

Bi_{ss}: Buildability Index for a particular site specific factor

Cov_{bs}: Coverage of the building feature (in %)

The BAM framework

SBTool (1/3)

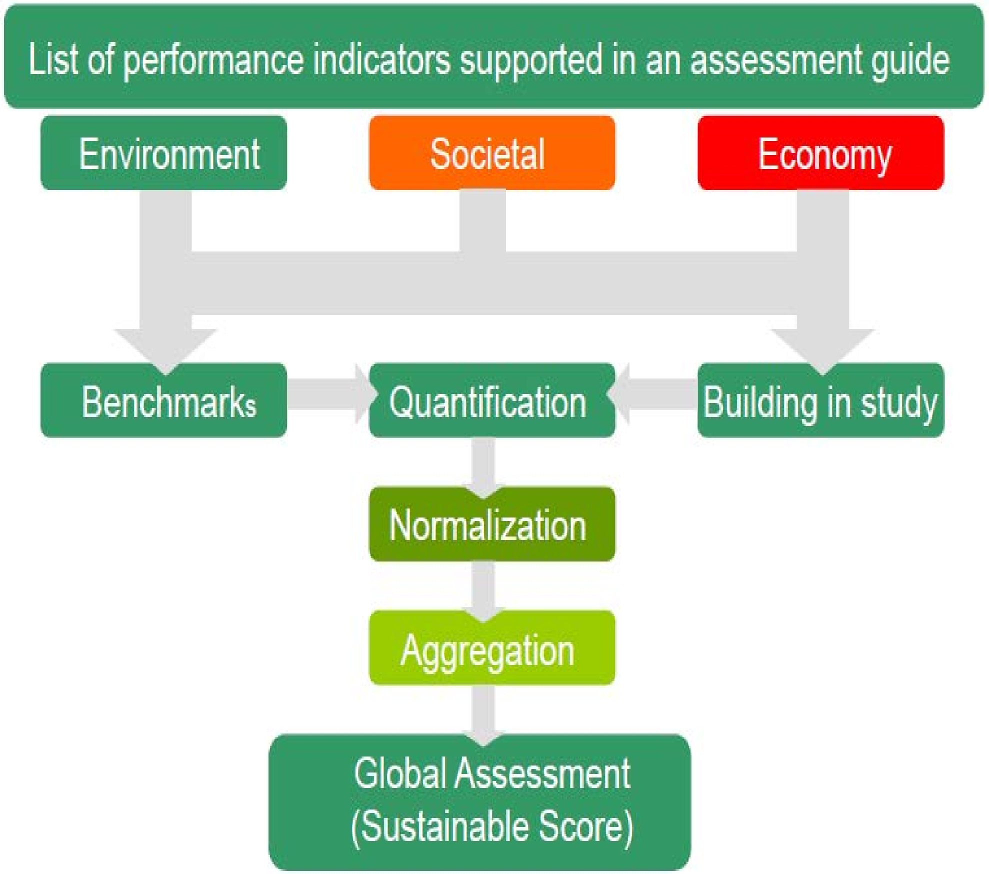
- Developed by iiSBE in 2007, overhauling the previous tool GBTool
- Customized and adapted for use in Portugal, Spain and Italy for the sustainability performance assessment both of *sites* and *building projects*

- Used primarily by:
  - authorized organizations (e.g. municipalities, non-governmental organizations (NGOs) etc.) for the establishment of rating systems suiting specific regions and building types
  - owners and managers of large building portfolios to specify their performance requirements to their staff and consultants
  - educators of graduate engineering students

- Takes into account sustainability performance indicators (SPIs), **discretized** by:
  - the social sustainability dimension
  - the environmental sustainability dimension
  - the economic sustainability dimension

  and **benchmarked** through the principles of:
  - conventional practice
  - best practice

SBTool (2/3)



The SBTool methodology

Features a top-down layout:

- a core framework encompassing established, regional and generic sustainability standards, requirements, thresholds and specifications
- separate and targeted computational sheets producing the sustainability score of specific projects

SBTool (3/3)

$$\overline{P}_i = \frac{P_i - P_{*i}}{P_i^{*} - P_{*i}} \forall_i$$

with,
P_i – Value of *i*th parameter;
P_{*i} – Conventional practice of *i*th parameter;
P^{*}_i – Best practice of the *i*th parameter.

	A+	$\overline{P} > 1,00$
Best practice	A	$0,70 < \overline{P} \leq 1,00$
	B	$0,40 < \overline{P} \leq 0,70$
	C	$0,10 < \overline{P} \leq 0,40$
Conventional practice	D	$0,00 \leq \overline{P} \leq 0,10$
	E	$0,00 < \overline{P}$

The indicator value normalization equation and the graded scale of the SBTool

- In the resulted sustainability score, the performance values obtained for each parameter and indicator are normalized on a scale between 0 (reference/conventional value) and 1 (best performance)
- The quantified values are converted in a graded scale, from A+ to E (sustainability grade of the project)

INTEGRATION OF KBPIs AND PPAFs (1/2)

- WG1 of TU1406 discretized Key Bridge Performance Indicators (KBPIs) utilizing five homogenized categories:
  - Defects corresponding to the KBPIs
  - Relations of the KBPIs to certain parameters (material properties, equipment and protection, geometry changes, bearing capacity, structural integrity and joints, original construction sequence and design, dynamic behavior, environmental exposure)
  - Rating of the KBPIs
  - Cost and importance of the KBPIs
  - Loads corresponding to the KBPIs

- All the presented PPAFs utilize indicators discretized in categories, databases in checklist format and inclusive computational methodologies

INTEGRATION OF KBPIs AND PPAFs (2/2)

- The presented PPAFs, integrated with the KBPIs, could serve as blueprints and practical examples of appraising frameworks **for a possible validation of the methodology developed by WG2 of TU1406**

- Possible modifications for any of the presented PPAFs to be used as validation drafts for WG2:
  - Swapping the overhead system categories with the five homogenized KBPI ones
  - Substituting the corresponding indicators with the KBPIs
  - Adapting of the weight/point allocation scheme
  - Adapting of the computational, normalized and interface-related elements

CONCLUSIONS (1/2)

- PPAFs already used in practice can provide valuable data concerning best practices and lessons-learned for the appraisal of project performance and quality

- Case studies and applicational examples of such frameworks, especially those easily adaptable for infrastructure projects and lifecycle performance (including sustainability and quality), should generally be collected, scrutinized and serve as validation blueprints for:
  - The establishment of the performance goals
  - The computational schema of a QC plan for bridges
  - The reclaiming of past experience
  - The more efficient dealing with problematic or bottlenecking aspects that may arise during the conceptualization and construction of a QC plan for bridges

CONCLUSIONS (2/2)

- Of the presented PPAFs, SBTool seems the most suitable for the validation purposes, since
  - it is the only sustainability-oriented PPAF, thus offering a head start for the sustainability considerations related to the KBPIs
  - it is the only adapted and validated in practice in Europe
  - its mathematical schema ensures that as many KBPIs as desired can be used, because all elements are in the end normalized into a single scale – no substitution is required, and all KBPIs can be taken into account in addition to the already existent SBTool indicators (if such a thing is deemed necessary)
  - its versatility ensures an easier adaptation to infrastructure projects
  - it is more robust, because it relies not only on expert input, but also in: (i) specific mathematical methodologies like multivariate and linear regression and (ii) machine learning schemes like artificial neural networks

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Quality Control Plan for Earth Retaining Walls

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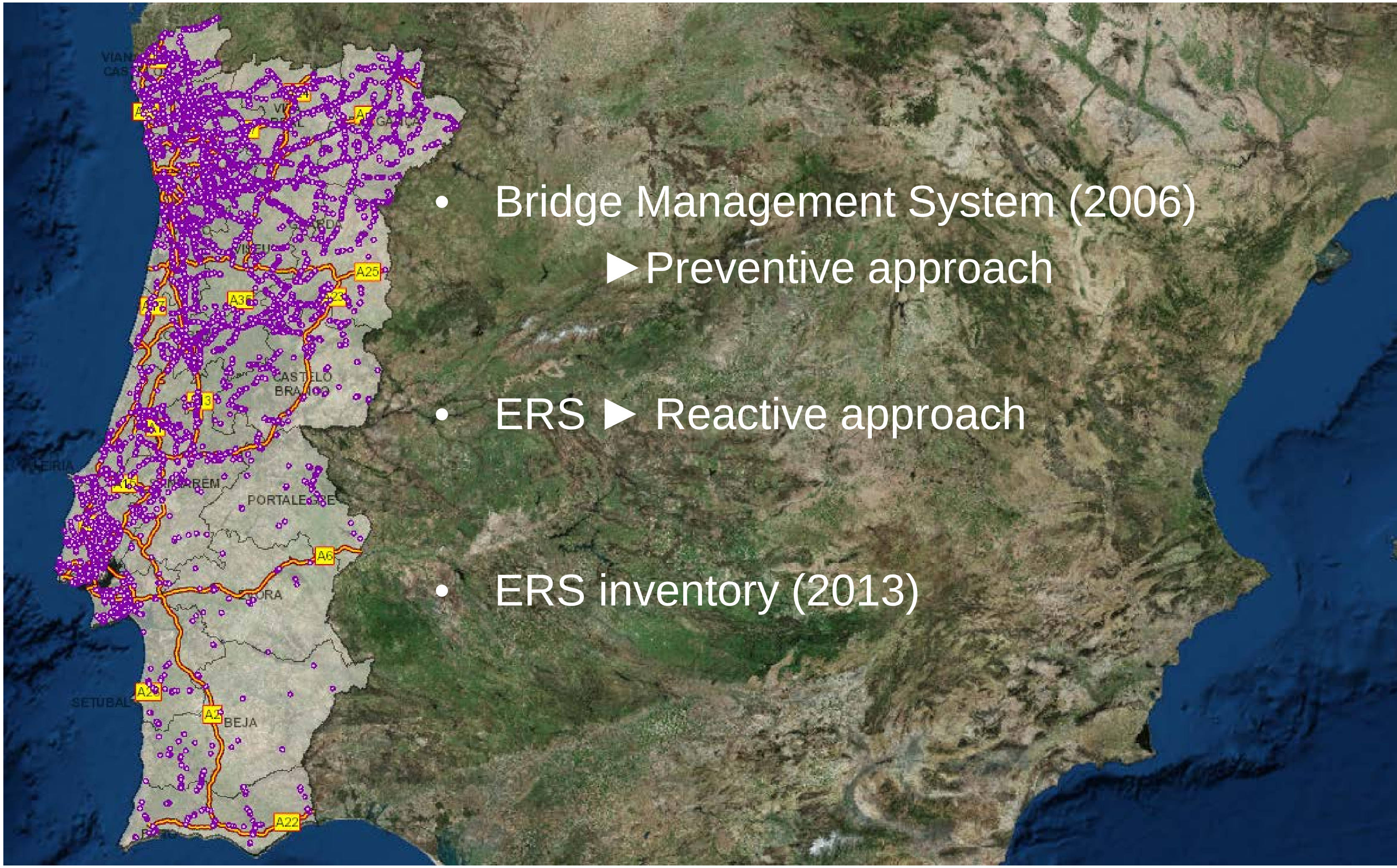
20th – 21st October 2016
Delft, Netherlands

CONTENTS

- Introduction
- Inventory Campaigns
- Assets to include in the QCP
- Activities considered in the QCP
- The QCP Process
- Conclusions

INTRODUCTION



- Bridge Management System (2006)
  ►Preventive approach
- ERS ► Reactive approach
- ERS inventory (2013)

INVENTORY CAMPAIGNS

Criterion:

- *Functional*: Geotechnical structures with the aim of soil retaining;
- *Location*: Included between lanes, roadsides, slopes, earthworks and expropriated land".

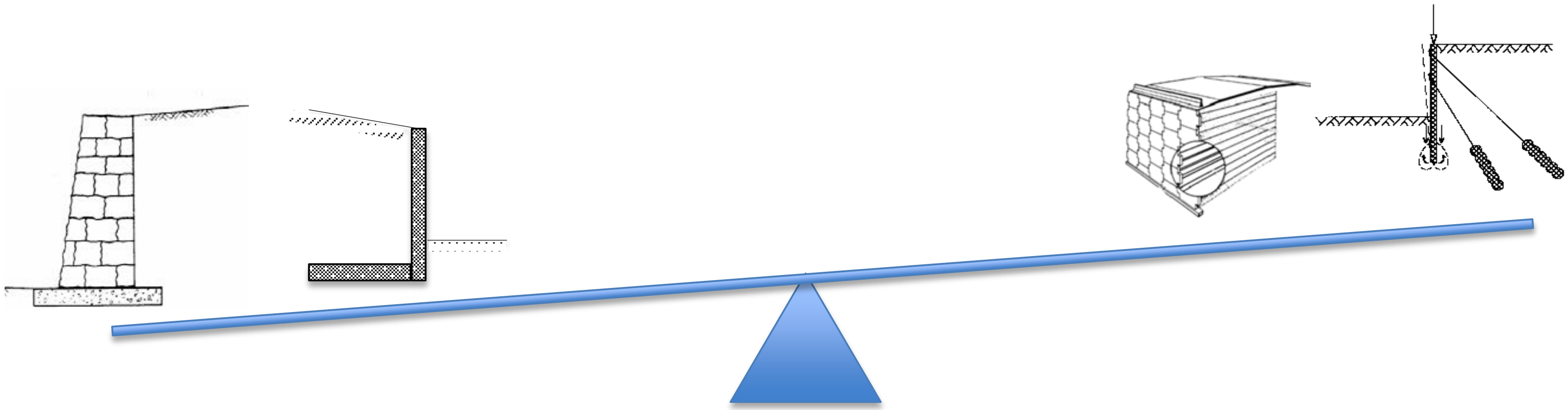
Types of Structures:

- Gravity walls, cantilevered walls, piling walls, anchored walls, soil-strengthened and soil nailing.

Results:

- 98% of the assets classified as gravity or cantilevered walls.

INVENTORY CAMPAIGNS



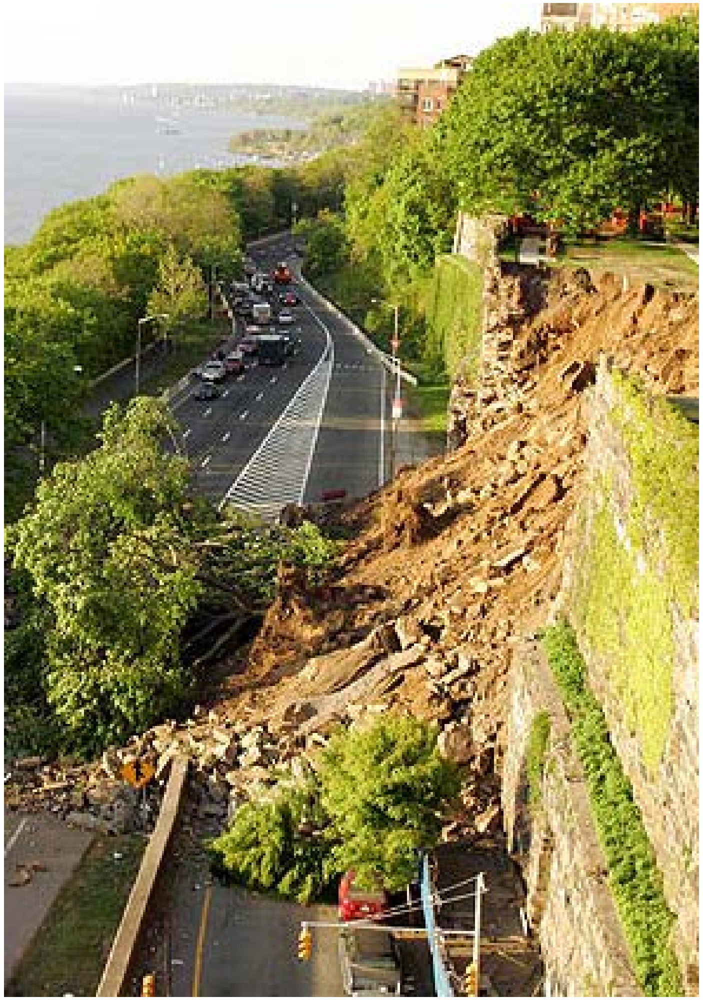
Simple Structures

- Visible condition is representative
- Visual inspection by technicians specifically trained to the task

Complex Structures

- Buried/ hidden elements
- In-depth inspection performed by structural/ geotechnical engineers

WHAT TO INCLUDE IN THE QCP?



J. Estrin, The New York Times

- All structures under the company jurisdiction with 2 meters high or more;
- All structures defined as “complex structures” independently from jurisdiction and height;
- All “simple structures” falling out of the company jurisdiction, with height ≥ 2m, if the minimum distance to the road is less than the maximum structure height.

ACTIVITIES CONSIDERED IN THE QCP

Starting point:

- *Feasible → Suitable to the previewed number of ERS;*
- *Aligned → Having in mind the company reality and BMS practices;*
- *Smart → Based on visual inspections.*

Periodic Activities:

- Periodic visual inspection as the core of the process:
  - Routine Inspections
  - Principal Inspections

Occasional Activities:

- Internal or external to the process, performed only when their necessity is detected.

ACTIVITIES CONSIDERED IN THE QCP

PERIODIC ACTIVITIES:

- **Routine Inspections:**
  - Technicians trained on the method;
  - Visual observations supported by a defect catalog;
  - Each 2 yrs (simple and complex structures)

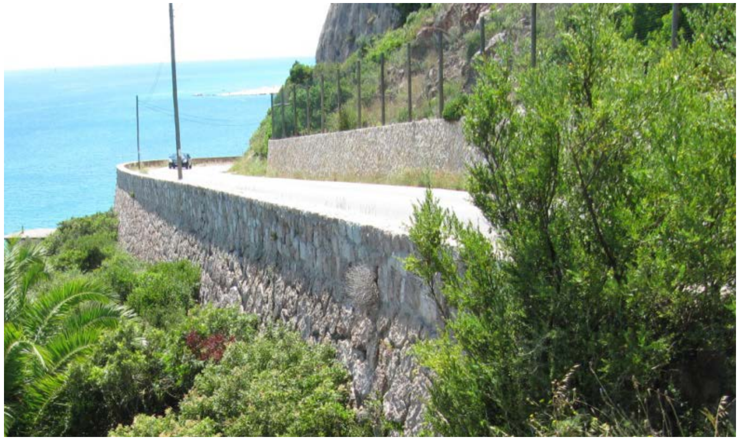
- **Principal Inspections:**
  - Specialized technicians (engineers);
  - Condition rating assignment derived from visual observations;
  - Each 6 yrs (complex structures) or when proposed by Routine.

ACTIVITIES CONSIDERED IN THE QCP

PERIODIC ACTIVITIES:

Complex Structurs

- Routine inps. each 2yrs
- Principal insp. each 6 yrs
- Further investigations if needed



Simple Structures

- Routine insp. each 2yrs
- Principal insp. if needed

ACTIVITIES CONSIDERED IN THE QCP

OCCASIONAL ACTIVITIES:

- **Monitoring:**
  - Considered since the design/ construction or assigned by Principal/ Detailed inspection.

- **Follow-up Visits:**
  - Annual visit to the structures with most critical condition.

- **Inventory update and Detailed inspections** (may include tests).

THE QCP PROCESS

What we have so far?

Set of interrelated or interacting activities that use inputs to deliver an intended result -> **Process**

ISO 9000:2015 (Quality management systems)

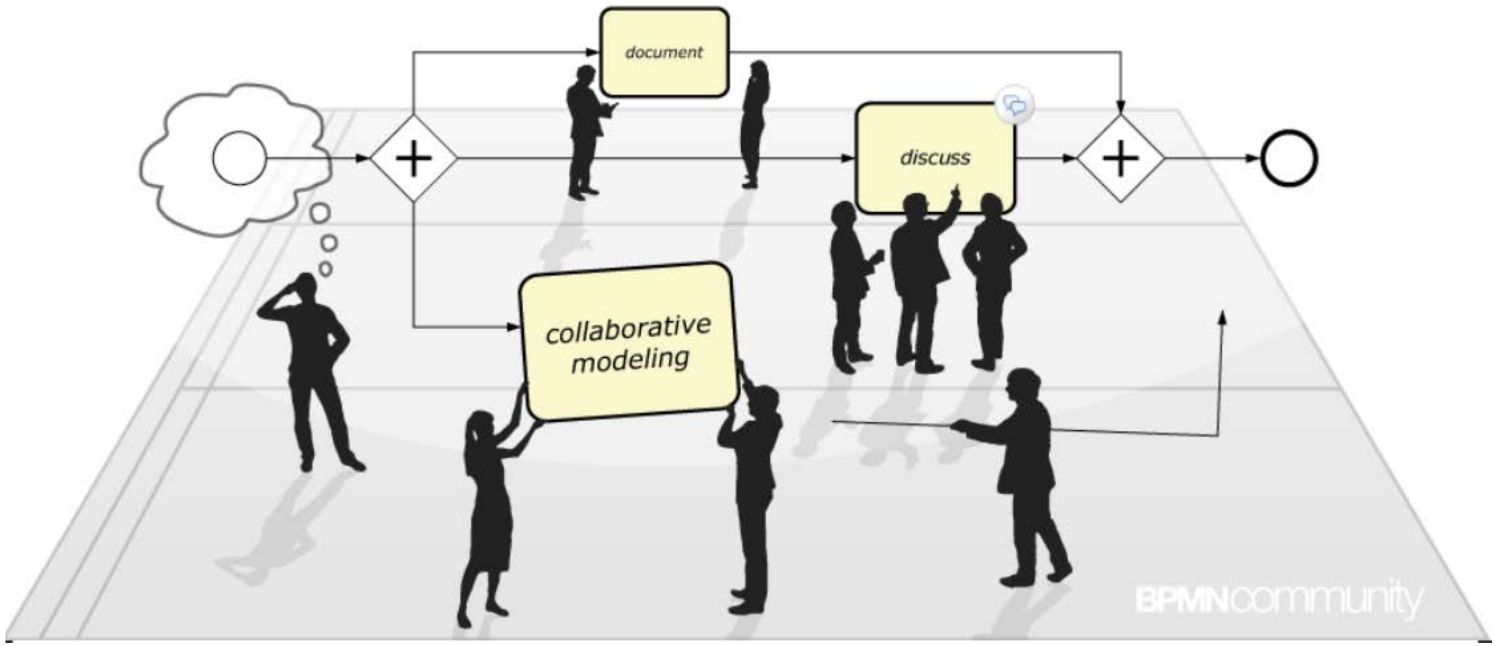
What we need?

Systematic approach for capturing, designing, executing, documenting and improvement of processes, seeking alignment with company strategies, better results and resources optimization.

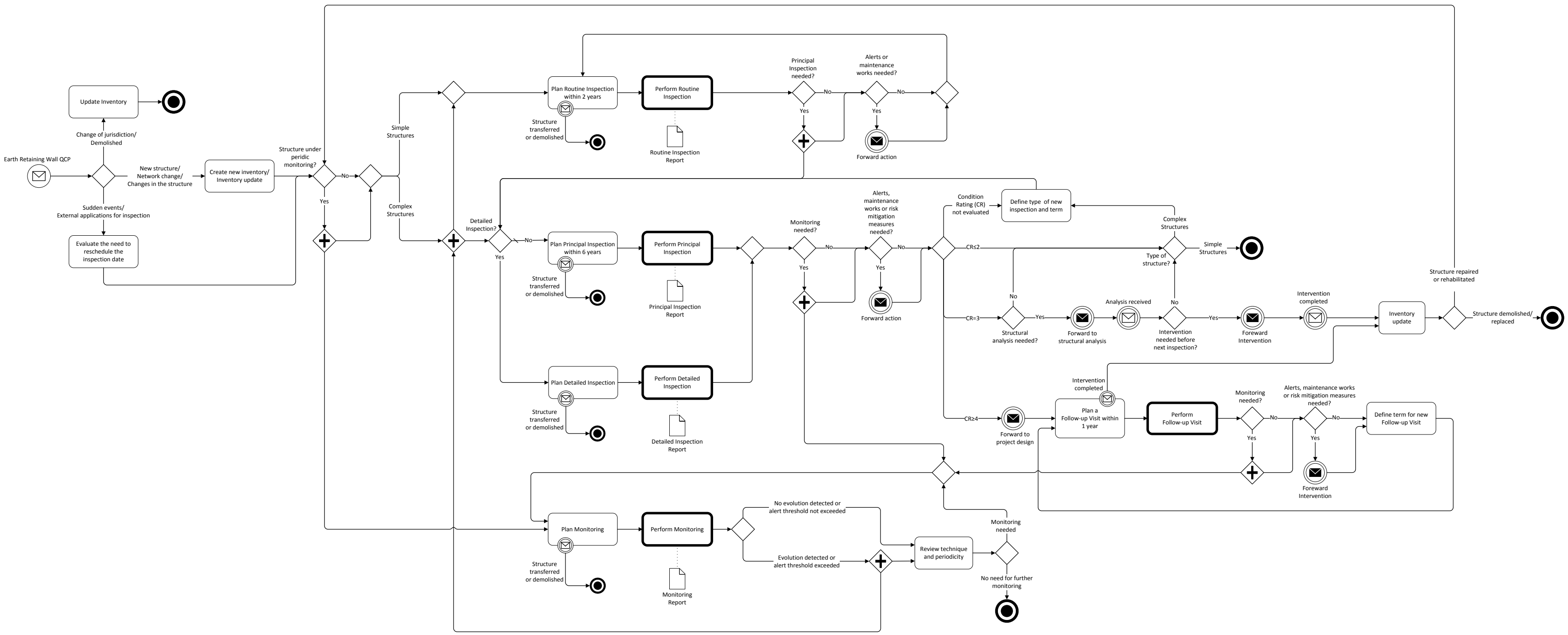
ISO/IEC 19510:2013 (Business Process Model Notation)

THE QCP PROCESS

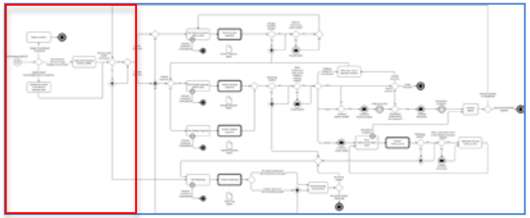
- Quality Control Plan as a BPMN process with specific intervenient, interrelated activities, both internal and external to the process, specific inputs and standard outputs:

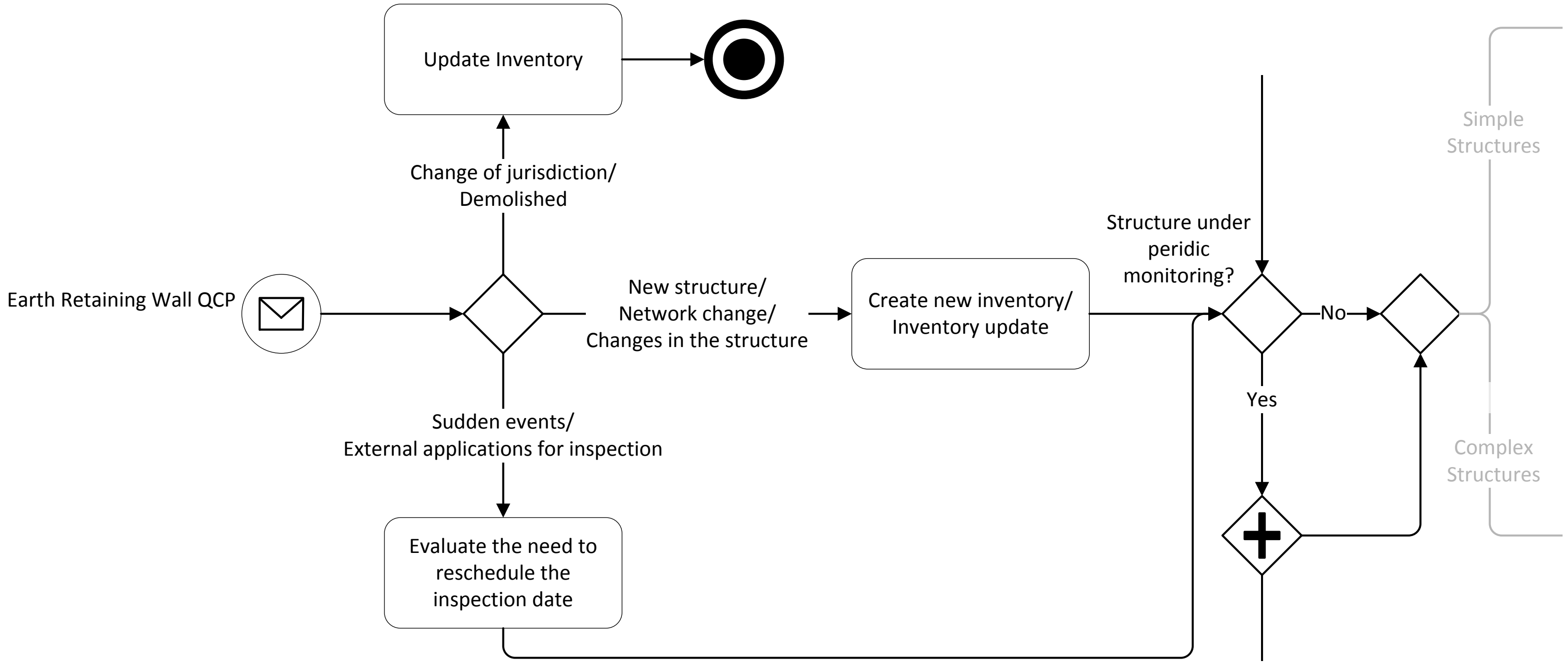


THE QCP PROCESS

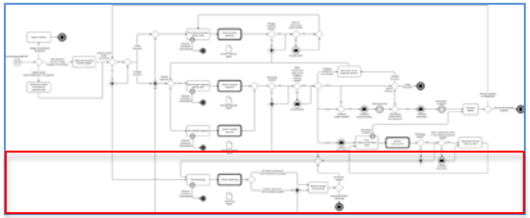


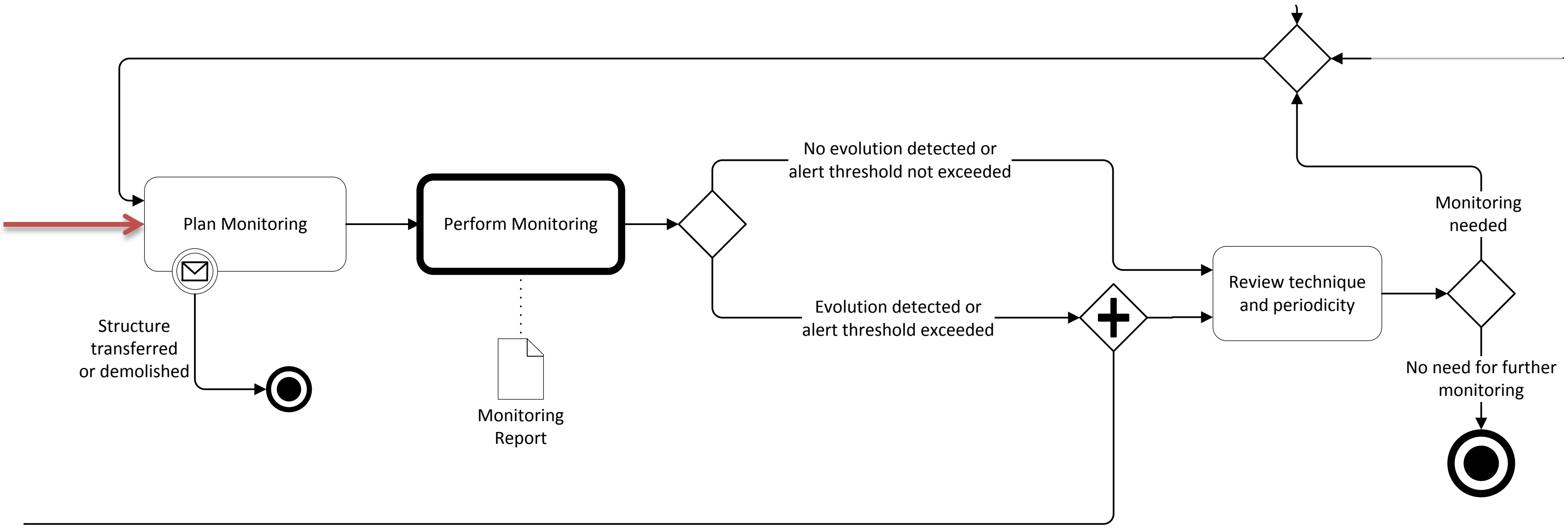
THE QCP PROCESS



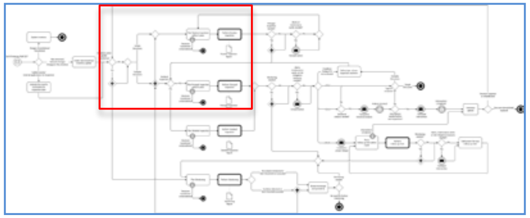


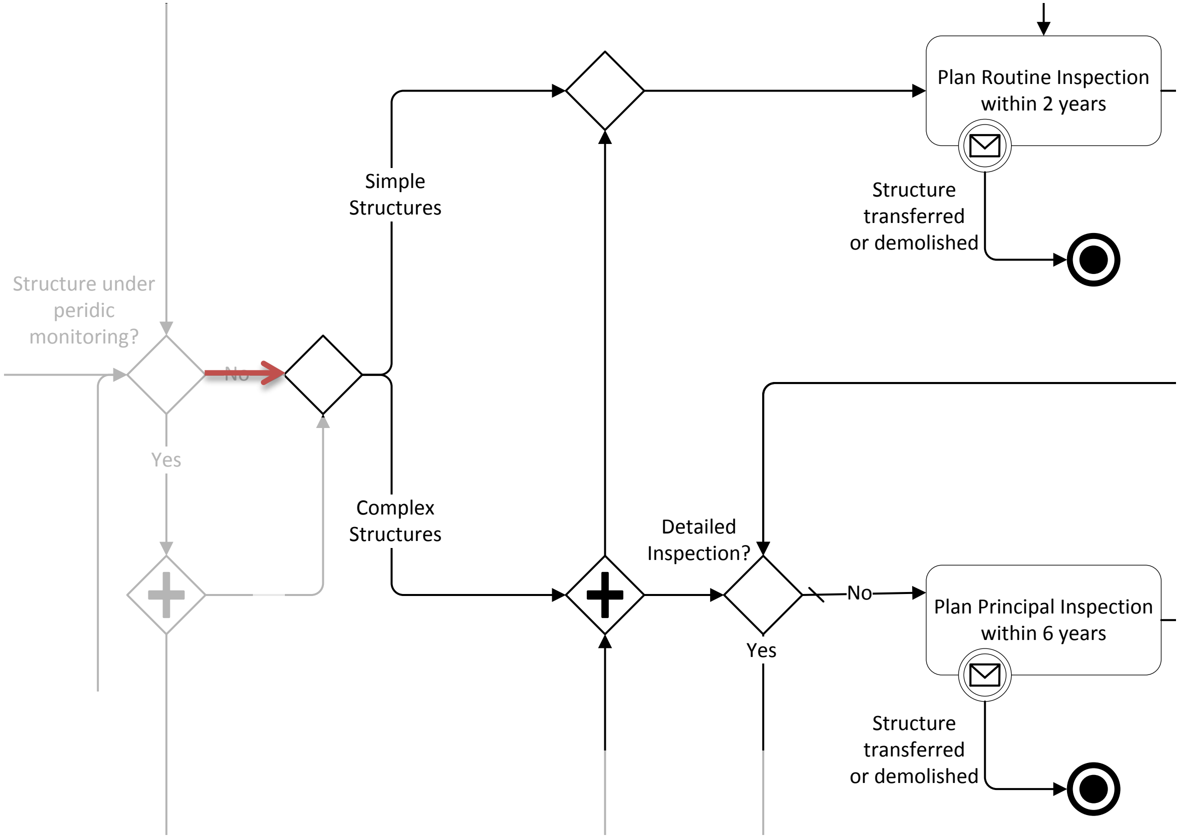
THE QCP PROCESS



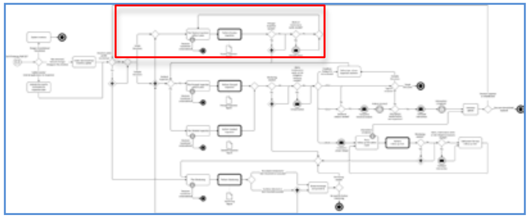


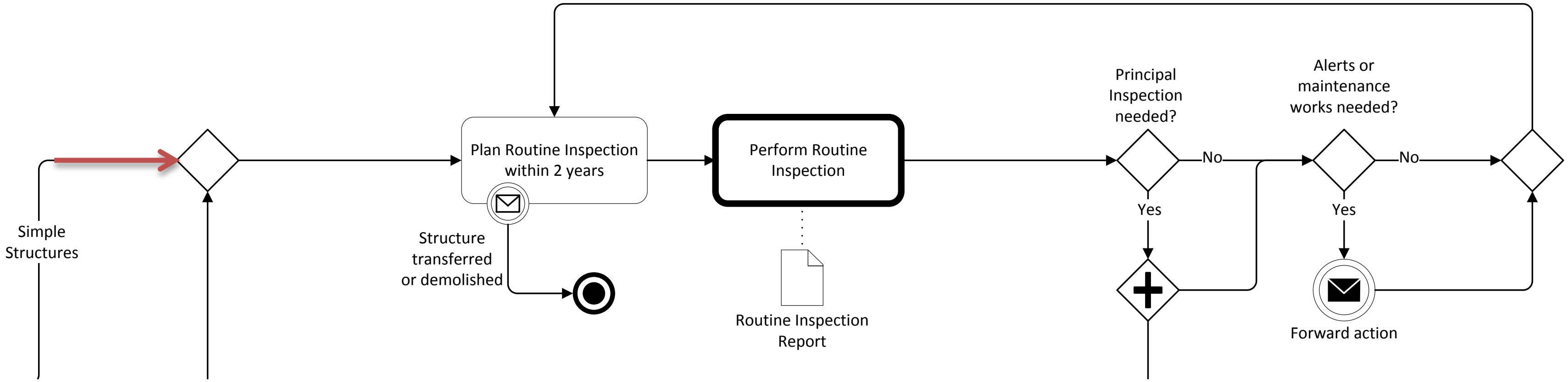
THE QCP PROCESS





THE QCP PROCESS





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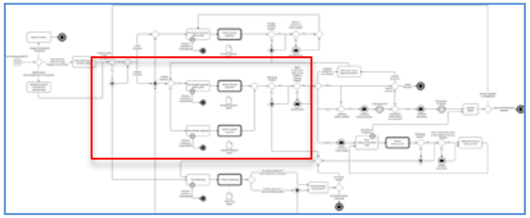
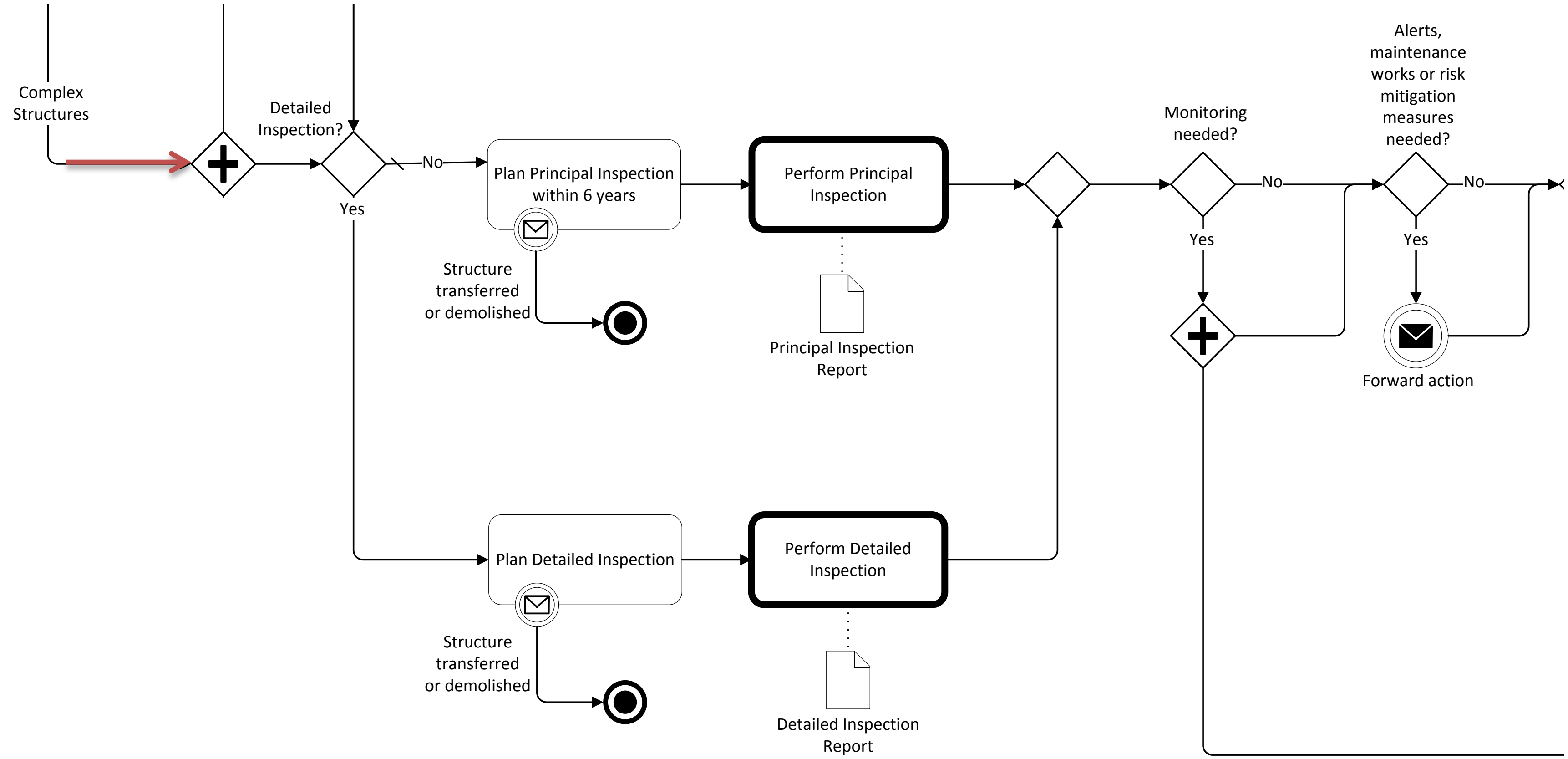
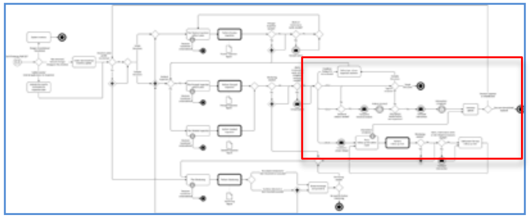
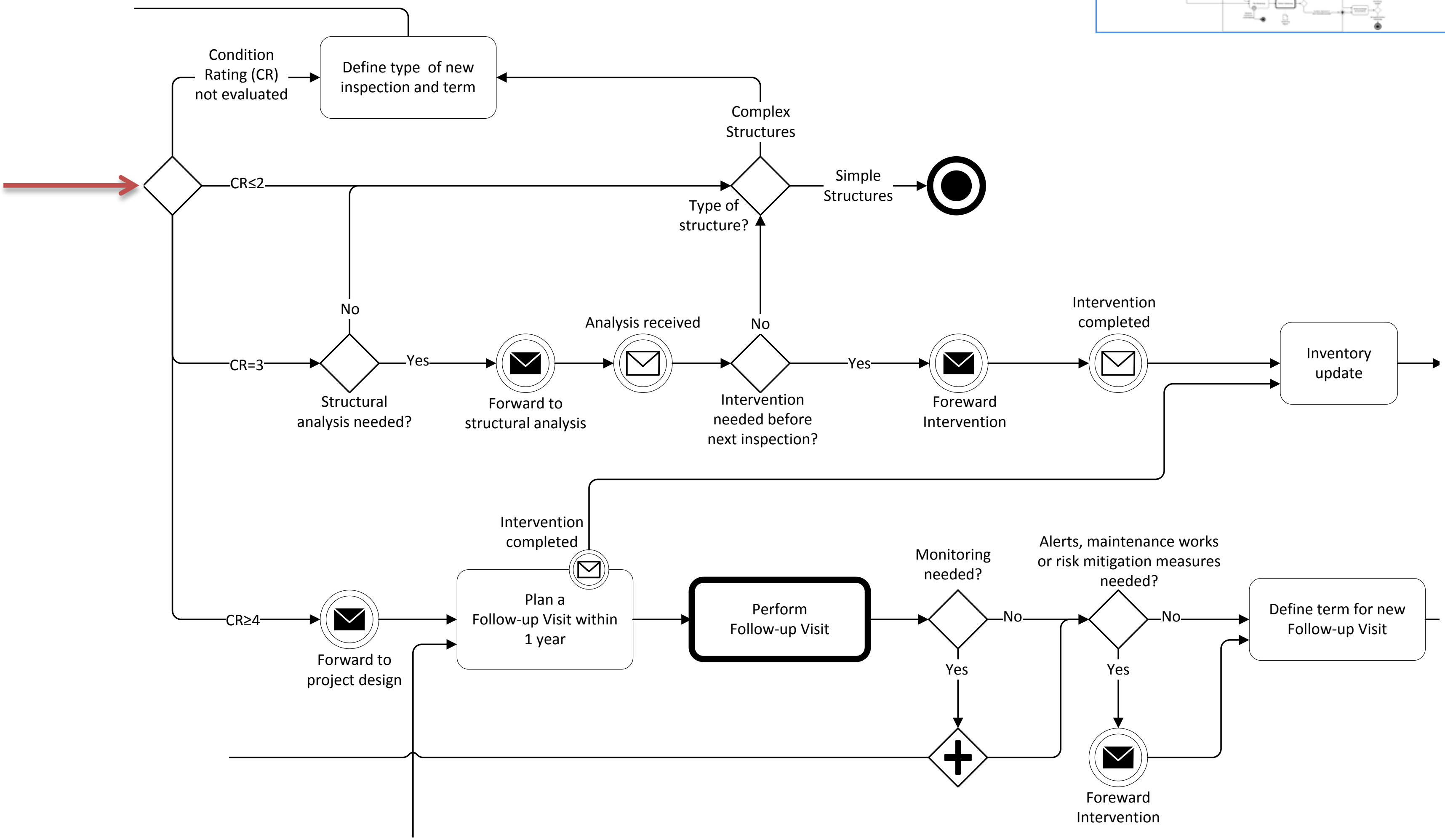


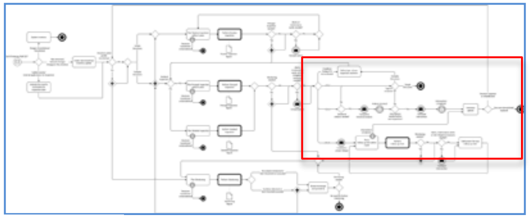


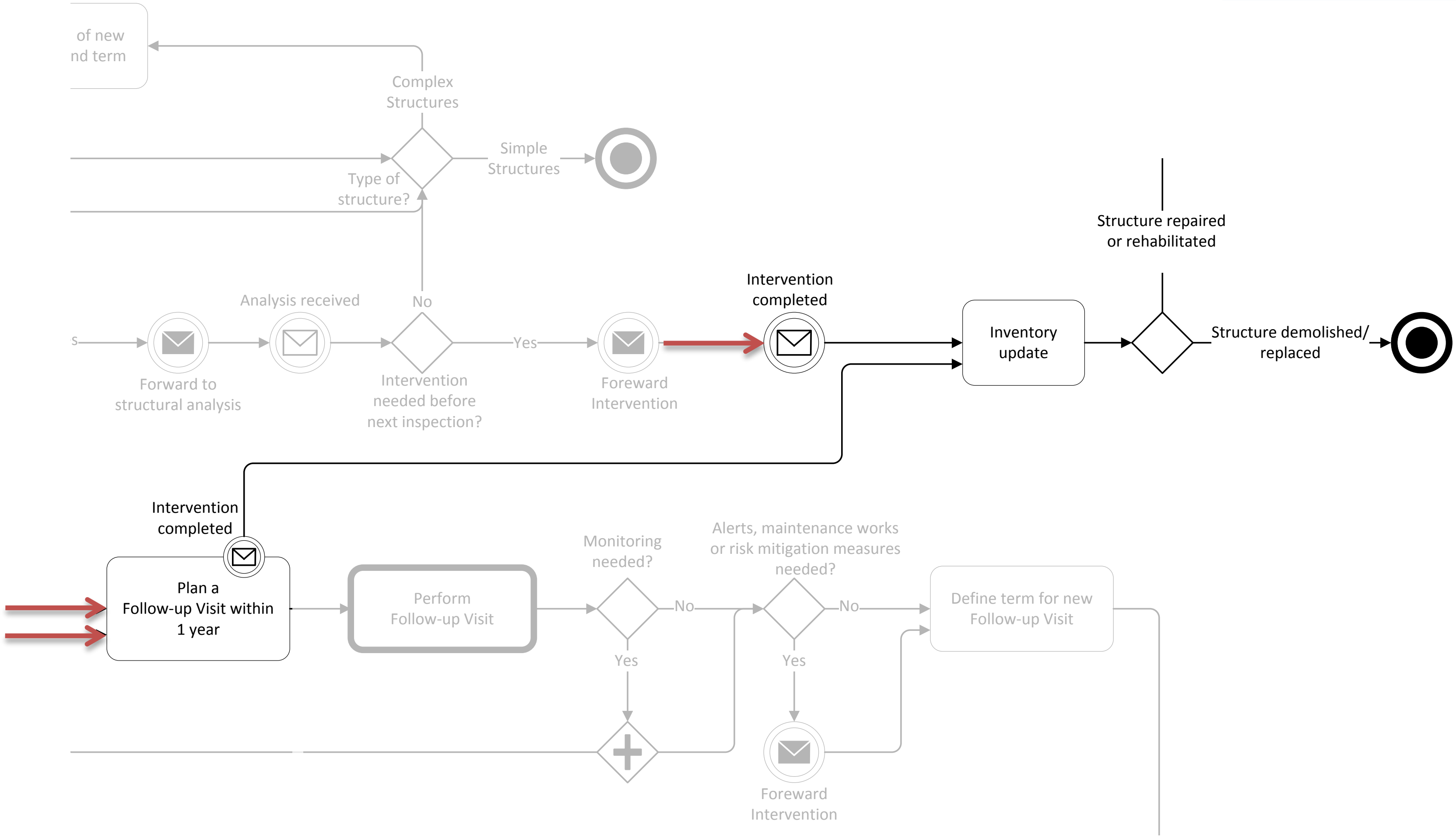
THE QCP PROCESS





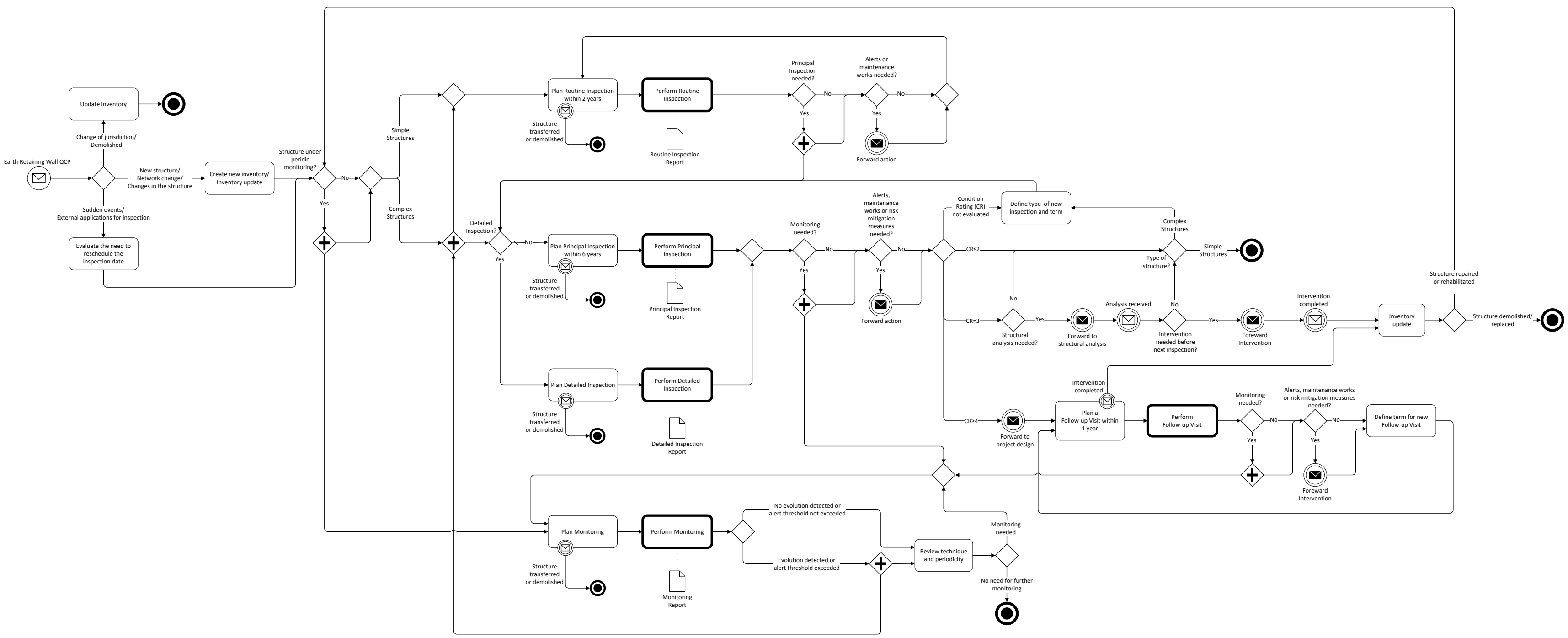
THE QCP PROCESS





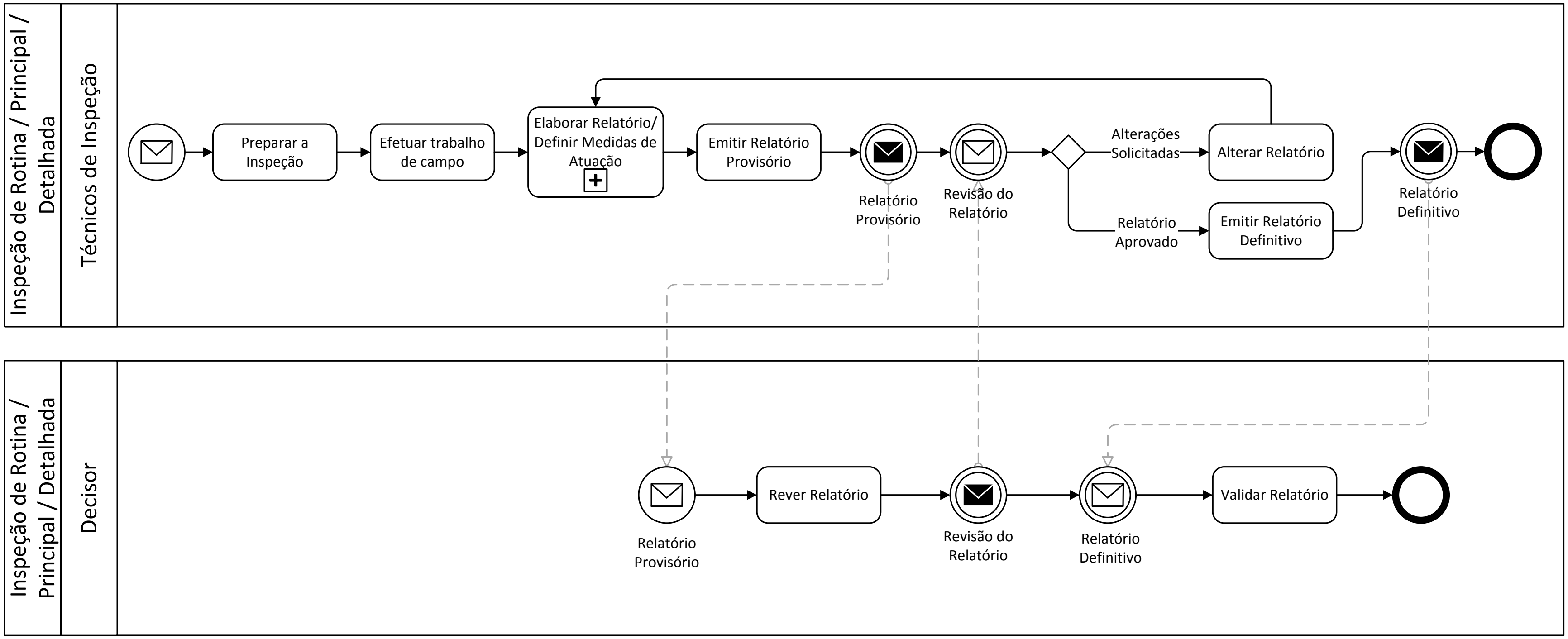
THE QCP PROCESS

Non-Operational model:



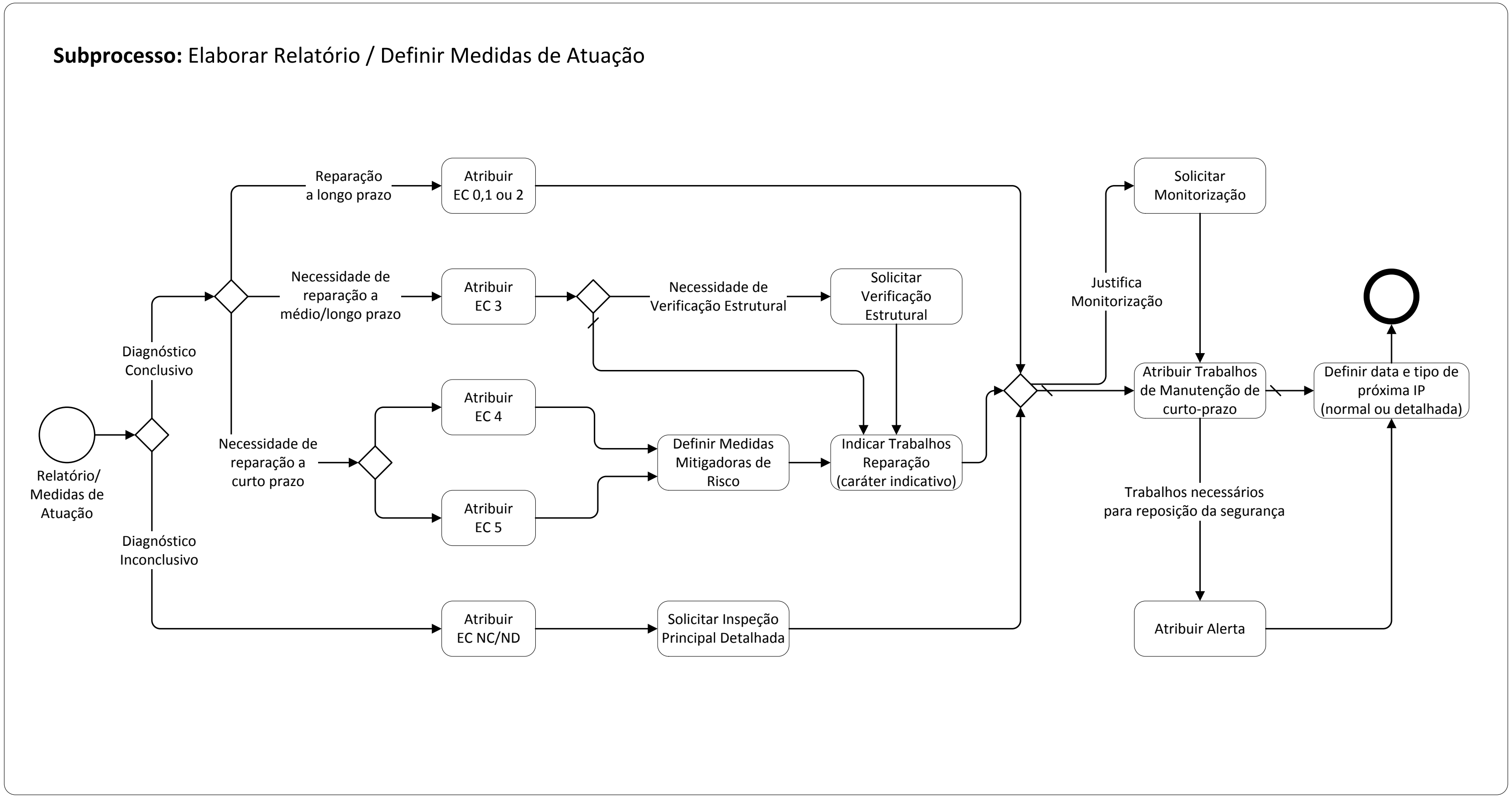
THE QCP PROCESS

Operational model (example)



THE QCP PROCESS

Subprocess (example)



CONCLUSIONS

- Earth retaining structures are suitable to manage according to a quality control plan based on the bridge management systems experience, taking advantage from the alignment of practices relative to these different assets.

- The development of a QCP will take advantage if based on the results of inventory campaigns, allowing to suit the activities, periodicities and resources to the subjacent reality of the set of assets and the company itself.

- Finally, the use of the management disciplines in the scope of structural engineering broadens the concepts, introducing new tools such as the BPMN and the idea of continuous improvement of the processes.

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Value-based method for condition assessment and management of bridges

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infra estructuras

Table of Contents

INTRODUCTION

- Value-based method and tool for assessment condition of a bridge based on visual inspection.
- This tool reduces the subjectivity derived from the inspector during inspection work.
- It aids decision making

Activities. Management Modules

The following activities are usually considered for bridge management:

- 1. Inventory of all structures and general data of Project
- 2. Inspections based on systematic tasks
- 3. Evaluation of structure condition index
- 4. Estimated cost of repair work
- 5. Strategic prioritization for intervention

The bridge assessment tool has been developed following the Integrated Value Model for Sustainability Assessment or **MIVES**, a multi-criteria methodology for decision making that evaluates each of the alternatives that can solve a defined generic problem, through an index value.

Basics for identification and location of bridge

1. Inventory of all structures and general project data

- **LOCATION**
  Location of the bridge including graphical information

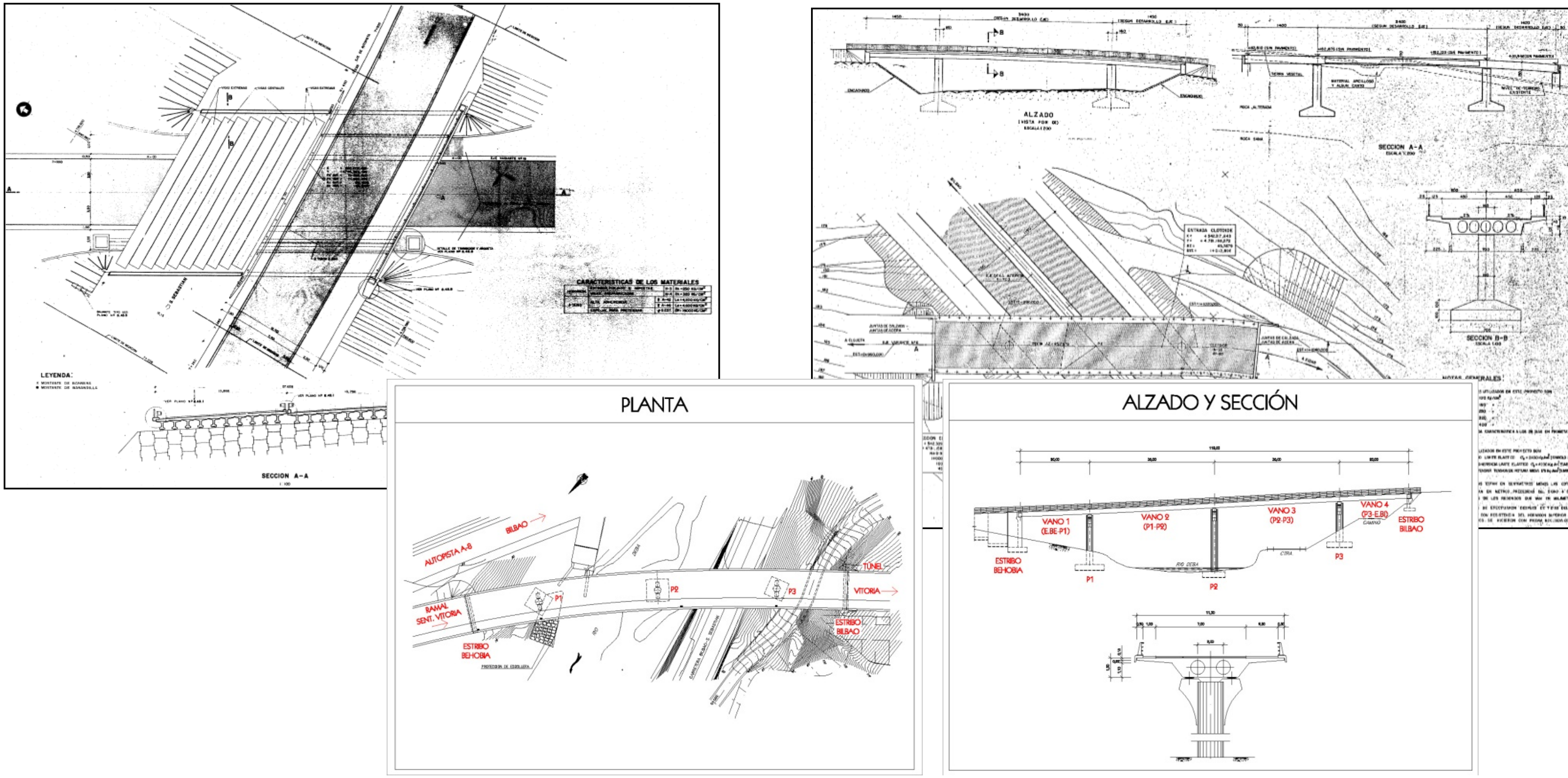
- **FUNCTION**
  Type of traffic allowed + element crossed (river, road,etc.)

- **DESCRIPTION**
  Type + material + geometrical characteristics

- **REFERENCES / INSPECTION**
  Features from previous inspection + inspection results

Basics for identification and location of bridge

1. Inventory of all structures and general project data



Basics for identification and location of bridge

Type of structure and materials:

Basics for identification and location of bridge

Dimensions:

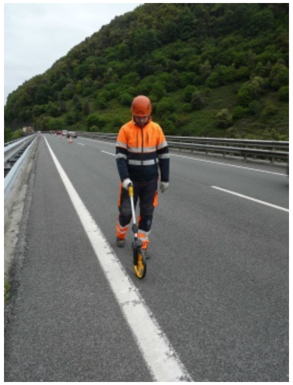
Basics for identification and location of bridge

Elements crossed:

Fieldwork

2. Inspections based on systematic tasks



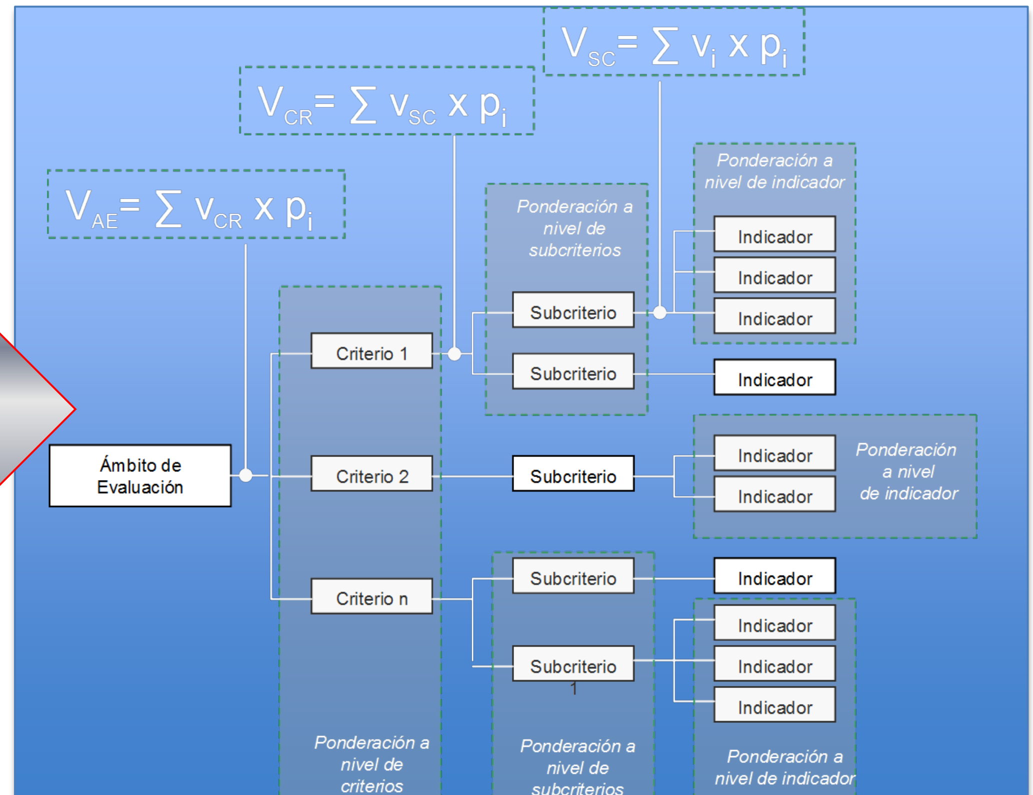



- Damage is visually identified and it is based on bridge inspector experience.

- The importance attributed to the same damage by different inspectors, even if experts, is different.

New Assessment Approach

- 3. Evaluation of structure condition index



DESCRIPTION OF THE ASSESSMENT METHODOLOGY

DAMAGE LIBRARY of more than 270 damages classified by materials and bridge component.

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

- Each damage is defined by **FOUR INDICATORS**:

SEVERITY **EVOLUTION** **IMPLICATION** → Predefined for each damage

EXTENSION → Depends on the on-site verification

Damage indicators have been defined based on the "Spanish guidelines for main roadway bridge inspections", which considers three different indicators for each damage type: *damage extension*, *damage severity* and *damage evolution*.

A forth indicator has been proposed here, *damage implication,* to evaluate how easily a damage can affect other elements of a bridge or can trigger other damage mechanisms.

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

- **Damage severity:** evaluates the capacity reduction of an element to fulfill its specific function as a result of the damage.

- **Damage implication:** evaluates the repercussion of the damage in an element into other types of damage or elements.

- **Damage evolution:** evaluates the probability that the damage will rapidly worsen.

- **Damage extension:** evaluates the surface or volume affected by the damage with respect to the total surface/volume of the element.

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Relative weight assignation

Usual practice → The weights assigned to each indicator are equal
AHP (Analytical Hierarchy Process **) → Pairwise comparison → Matrix decision building by more than 20 experts**

Relative importance R_{ij} (i compared to j)	
Verbal scale	*Numerical scale*
Extremely more important	9
Much more important	7
More important	5
Slightly more important	3
Equally important	1
Slightly less important	1/3
Less important	1/5
Much less important	1/7
Extremely less important	1/9

Pairwise comparison to build decision matrix

Example of pairwise comparison matrix

Verbal Scale	*Severity*	*Extension*	*Evolution*	*Implication*
Severity	1 (equally important)	7 (much more important)	5 (more important)	3 (slightly more important)
Extension		1 (equally important)	1 (equally important)	1/3 (slightly less important)
Evolution			1 (equally important)	1/3 (slightly less important)
Implication				1 (equally important)

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Relative weight assignation

After performing matrix calculations and checking consistency of the assessments of each decider → averaging results → RELATIVE WEIGHT OF EACH INDICATOR

Indicator	*Weight*	*Weight*
Damage degree	25%	>50%
Damage extension	25%	¿?
Damage evolution	25%	¿?
Damage implication	25%	¿?

Relative weights of damage indicators equal relative weight

Alternative options for a given indicator

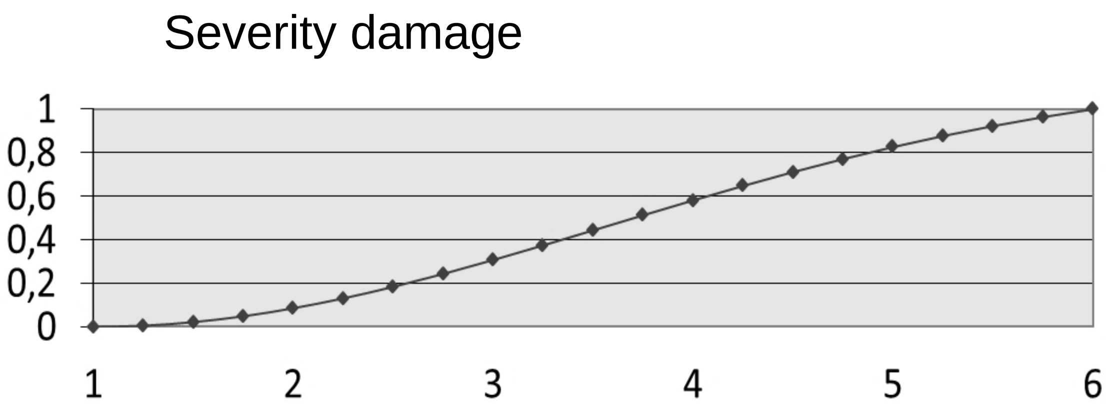
EXAMPLE

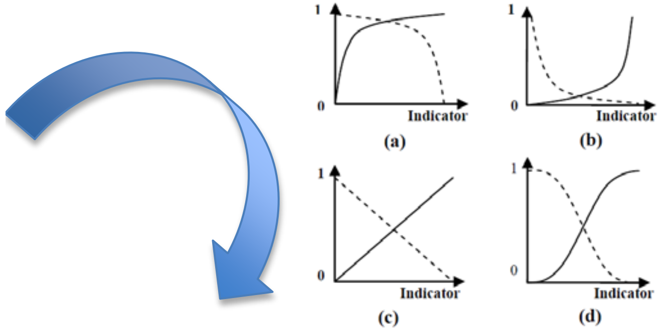
SEVERITY	**EVOLUTION**	**IMPLICATION**	**EXTENSIÓN**
Null	*Null*	*Null*	*<10%*
Very Low	*Slow*	*Low*	*11-30%*
Low	*Medium*	*Medium*	*31-50%*
Medium	*Fast*	*High*	*51-75%*
High			*>75%*
Very High			

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Value function assignation to each alternative

In order to perform the weighted sum between 4 indicators is necessary to transform them from a quantify or qualitative measure into a dimensionless variable between 0 and 1 through **DIFFERENT TYPES OF VALUE FUNCTIONS** (linear, convex, concave or shaped)





EXAMPLE

Severity	Values
Nulll	0
Very Low	0,1
Low	0,3
Medium	0,6
High	0,85
Very high	1,00

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

– Bridge component classification

The assessment methodology classifies the elements of a bridge in four different levels, as shown.

LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4	Damage
BRIDGE	FOUNDATION	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
	SUBSTRUCTURE	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
		Arch	Arch 1, Arch 2, etc.	
	SUPERSTRUCTURE	Deck	Span 1 Span 2, etc....	
		Arch	Arch 1, Arch 2, etc.	
	CONNECTING ELEMENTS	Bearings	Bearings in Abutment 1, Bearings in Pier 1. etc.	
		Dilatation joints	Transverse, longitudinal	
		Cables	Cables in Pilon 1, Cables in Pilon 2, etc.	
	EQUIPMENT	Pavement	Direction A, Direction B.	
		Drainage system	Substructure, superstructure	
		Safety Elements	Railings, protection elements , etc.	
		Protection Elements	Substructure, superstructure	

Bridge inspector reports :

- Type of damage
- Location of damage
- Extension of damage

Classification of bridge elements in levels

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Assessment method

LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4	Damage
BRIDGE	FOUNDATION	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
	SUBSTRUCTURE	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
		Arch	Arch 1, Arch 2, etc.	
	SUPERSTRUCTURE	Deck	Span 1 Span 2, etc.…	
		Arch	Arch 1, Arch 2, etc.	
	CONNECTING ELEMENTS	Bearings	Bearings in Abutment 1, Bearings in Pier 1. etc.	
		Dilatation joints	Transverse, longitudinal	
		Cables	Cables in Pilon 1, Cables in Pilon 2, etc.	
	EQUIPMENT	Pavement	Direction A, Direction B.	
		Drainage system	Substructure, superstructure	
		Safety Elements	Railings, protection elements , etc.	
		Protection Elements	Substructure, superstructure	

$$V_{damage} = \sum_{i=1}^{n} V_{alternative} \times Weight_{indicator} = 0 - 1$$

- **Individual indexes for bridge components**
- **Warnings and recommended repair actions**
- **Global condition index**

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Assessment method

Multi-level assessment (4 Levels)

Element index (Level 4) is determined by the damage with higher index.

Higher element index is carried to Level 3, and so on and so forth until Level 1 (Bridge).

GROUP OF ELEMENTS	ÍNDEX	SUBGROUP OF ELEMENTS	ÍNDEX	ELEMENTS	ÍNDEX
SUPERESTRUCTURE	4	Spans	4	Span 1	4
		Archs	-	Span 2	3
		Pylons	4	Pylon 1	4
		Others	-	Pylon 2	3
SUBESTRUCTURE	3	Abutments	3	Abutment 1	3
		Piers	3	Abutment 2	3
		Archs	-	Pier 1	2
		Others	-	Pier 2	3
				Pier 3	2
CONNECTING ELEMENTS	4	Bearings in Abutments	4	Bearings in Abutment 1	4
		Bearings in Piers	3	Bearings in Abutment 2	3
		Transverse Dilatation Joints	4	Bearings in Pier 1	2
		Longitudinal Dilatation Joint	2	Bearings in Pier 2	2
		Cables	2	Bearings in Pier 3	2
		Others	-	Transv. Dilat. Joint - A1	4
				Transv. Dilat. Joint - A2	4
				Longitudinal Dilatation Joint	2
				Cables	2
EQUIPMENT	3	Pavement	2	Direction A & B	2
		Drainaje system	3	Drainaje system	3
		Safety elements	2	Railings	2
		Protection elements	2	Gauge	2
		Others	-	Otros	-

Last Inspection:	2015	GLOBAL BRIDGE CONDITION INDEX	4
Index:	4		
Company:	TECNALIA		

DESCRIPTION OF THE ASSESSMENT METHODOLOGY

Global bridge condition index:

$$V_{damage} = \sum_{i=1}^{n} V_{alternative} \times Weight_{indicator} = 0\text{ -}1$$

INDEX	DEFINITION
1	Apparently undamaged structure or minor defects with no consequences
2	The structure has defects that can evolve into structural damage, or might need to be repaired in the short or medium term
3	The structure has defects that indicate onset of structural damage
4	Defects in the structure indicate that there is an ongoing process of structural damage. This situation requires a more detailed inspection in the short term, or a repair action in the short or medium term.
5	The bridge has damages that cause a modification of the structural behavior. A special inspection or repair action is needed in the short term.
6	The damages are such that the structure is approaching its serviceability limit state. The bridge has to be closed or its use restricted. A special inspection and an urgent repair action are required.

ECONOMIC EVALUATION

6. Estimated cost of repair work

Maintenance operations should ensure that, during a finite service life, the level of benefit (structural safety, service performance, user safety and durability) of the structure remains above pre-defined acceptable limits.

- The tool calculates the estimated cost of repairing the bridge depending on the damage observed during the inspection.
- The cost of the auxiliary machinery required to perform the repairing work is added separately.

OPTIMIZING AVAILABLE BUDGET

1. Strategic prioritization for intervention

THE BRIDGE CONDITION INDEX is an IMPORTANT factor but NOT DEFINITIVE

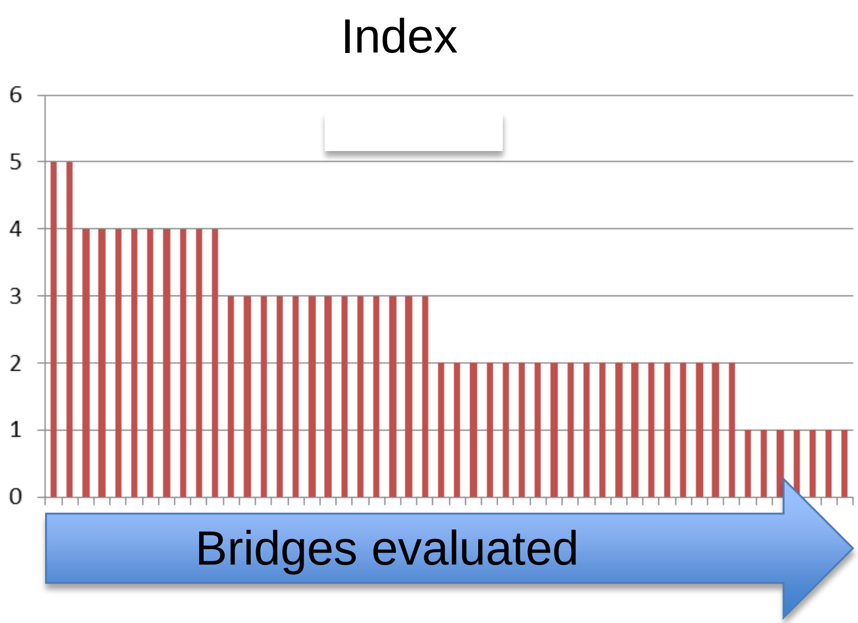
- Global bridge condition index
- Estimated cost of repair work
- Other factors

DECISION SUPPORT

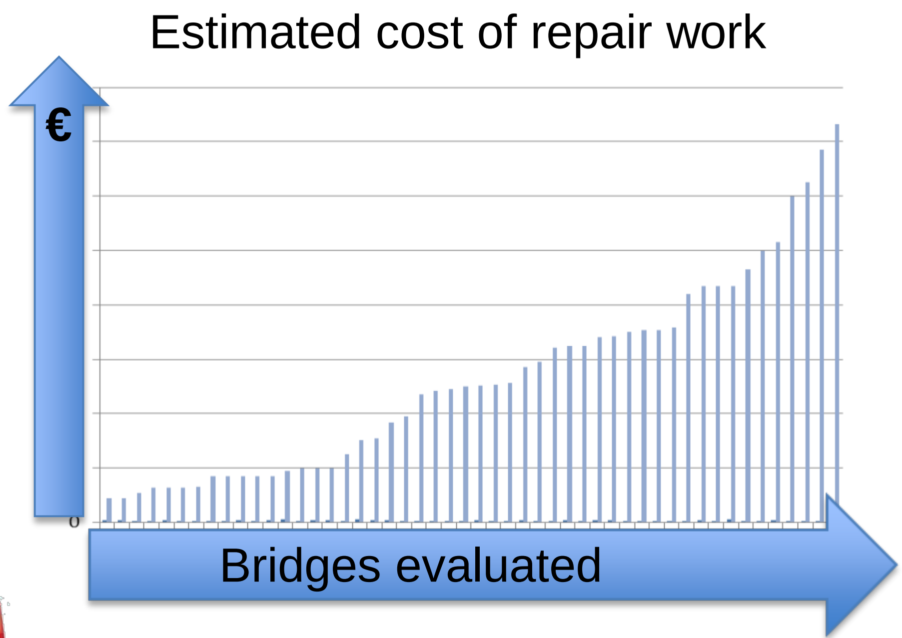
- Architectural or cultural value
- Routes with high traffic density
- Specific repairs

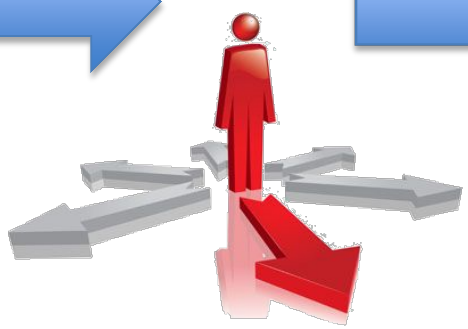
OPTIMIZING AVAILABLE BUDGET

1. Strategic prioritization for intervention



VS





Helps decision making

CONCLUSIONS

The conclusions of this bridge assessment tool and overall management system are:

- Possess operational, consistent and easily accessible information on the characteristics and status of deterioration of all structures.
- Assess the safety and condition of structures with an objective method.
- Optimizing the use of limited budget.

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

QUALITY OF DATA ACQUISITION (EQUIPMENT)

Matej Kušar – Faculty of Civil and Geodetic Engineering, University of Ljubljana, Slovenia
Dušan Cmok – DARS, National motorway operator, Celje, Slovenia

20th – 21st October 2016
Delft, Netherlands

EQUIPMENT

Inspectors need to be aware of possible additional investigations when in doubt determining damage state/intensity/…

Prescription of suitable investigation method is demanding

↓

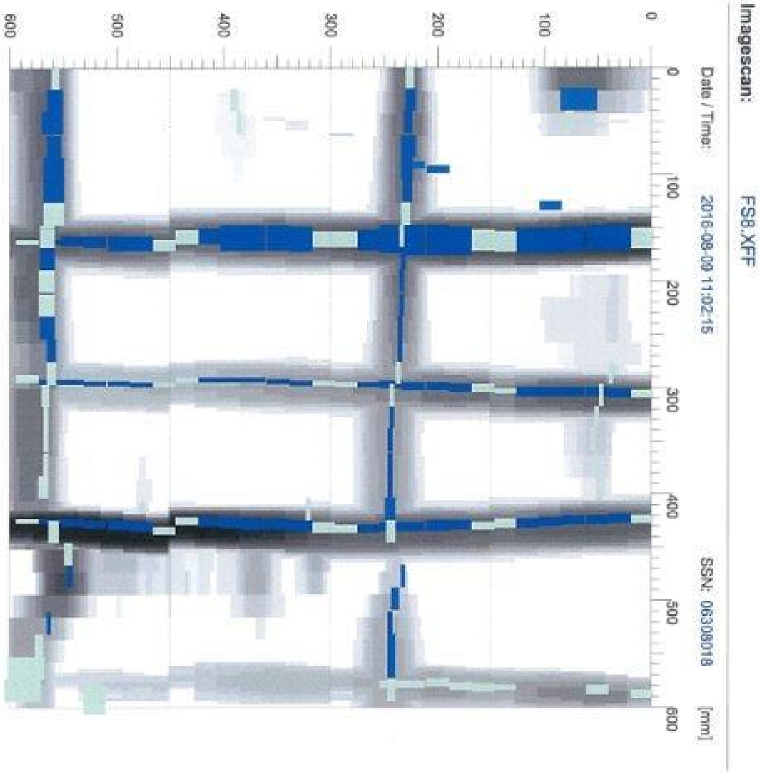
Methods overview needed

↓

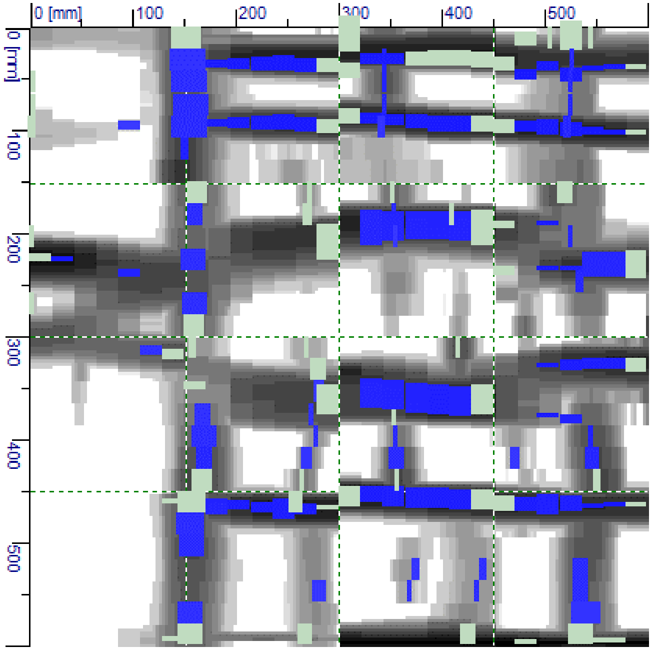
Advantages and disadvantages indicated

CHOOSING THE RIGHT EQUIPMENT

CHOOSING THE RIGHT EQUIPMENT



CHOOSING THE RIGHT EQUIPMENT



DATA RELIABILITY VARIATION

The most suitable method for performing NDT can change from element to element (even for the same structural type)

NDT METHODS

Measuring types
1. Surface damage (cracks, mechanical damage, segregation,…)
2. Compreshive strenght, hardnes, adhesion,…
3. Chloride concentration
4. Corrosion
5. Carbonation
6. Internal damage and defects, delamination
7. Porosity, water absorption

EQUIPMENT

NDT methods for surface damage (sample):

NDT Method	ADVANTAGES	DISADVATAGES	Method BASED on
Visual inspection	Fast, economical	Quality depends on inspectors competence	Eye sight
Image Pro Plus (IPP)	Easy to use, economical	Prolonged acquisition of results	Color comparison
Acoustic emission (AE)	Fast	Expensive	Transient elastic waves
Impact echo	Fast, reliable for thinner elements	Reliability decreases with thickness	Electromagnetic waves transmission
Infrared thermography	Easy to use	No information regarding the depth of a damage	Change in surface temperature
Impulse response	Fast, easy to use	Reliability depends on inspectors experience	Stress waves method
Radiography	Determines thickness, irregularities…	Reliability decreases with thickness	X-rays and gamma radiation
Petrography	Easy to use	No information regarding the depth of a damage	Change in surface temperature
Lamb wave Theory	Precise results	Demanding interpretation of results	Guided waves theory

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

QUALITY OF DATA ACQUISITION (INSPECTORS)

Matej Kušar- Faculty of Civil and Geodetic Engineering, University of Ljubljana, Slovenia

20th – 21st October 2016
Delft, Netherlands

FIELD INVESTIGATION CONDUCTED

Inspection groups (3):	**1. no bridge inspection experience** **2. 2~5 year inspection experience** **3. >5 year inspection experience**
Structures examined (3):	**1. overpass (low damage intensity)** **2. bridge (medium damage intensity)** **3. viaduct (high damage intensity)**
Weather condition (2):	**1. wet conditions** **2. dry conditions**

*** No instruction manual was given.**

STRUCTURES EXAMINED

overpass	bridge	viaduct
		
		

SURVEY SAMPLE

ELEMENT	wing wall
DAMAGE TYPE	cracks

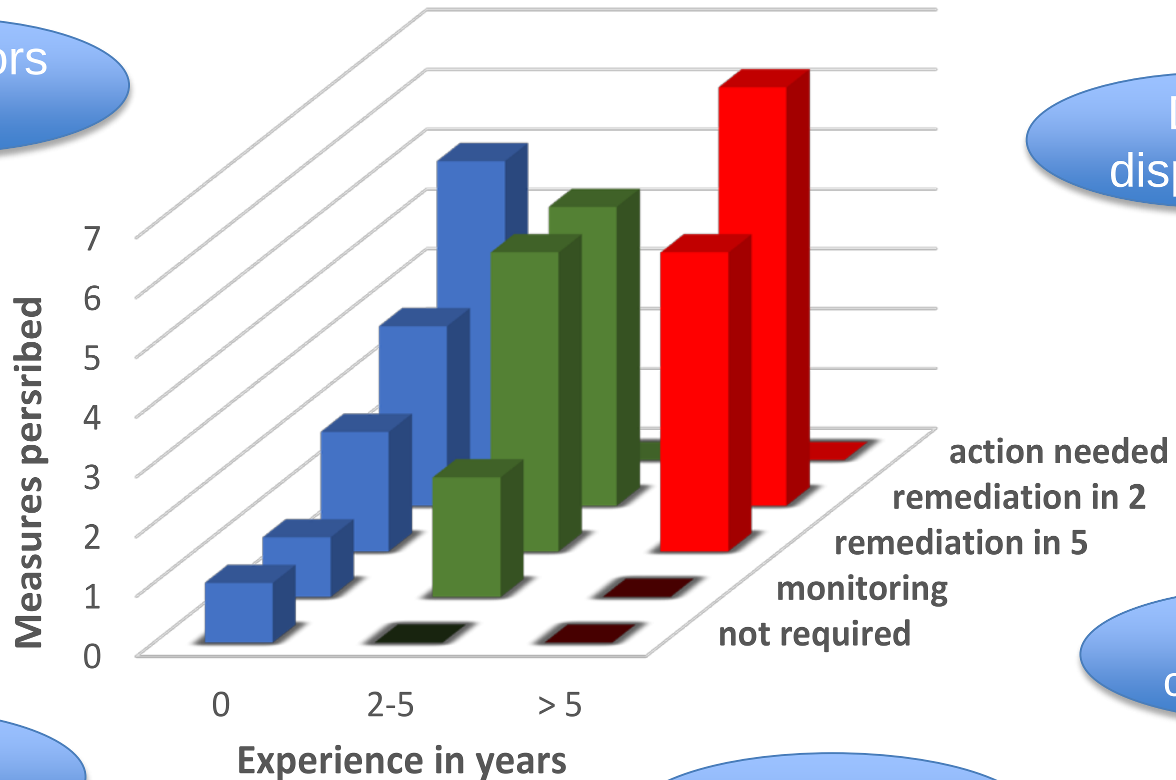
		INTENSITY				
		measures not required	monitoring further development	remediation required in 5 years	remediation required in 2 years	immediate action needed
EXTENT (percentage of damaged area) [%]	no damage					
	1 - 10					
	11 - 20			X		
	21 - 40					
	41 in več					

DATA ANALYSIS

Wet-dry comparison

Inspectors unity

Data dispersion



Time consumption

Remedial actions

...

MAIN CONCLUSIONS

1. **Condition assessments similar in wet and dry**
2. **At low level damage all inspection groups are unanimous**
3. **Semiexperienced inspectors prescribe remedial measures least frequently**
4. **Experienced inspectors ratings are most unified**
5. **Inspection time similar for all groups**
6. **According to the experienced evaluators fatigue and mood influence their evaluation the most**

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Structural health monitoring based on static measurements with temperature compensation to detect stiffness reductions

Viet Ha Nguyen, Sebastian Schommer, Stefan Maas - University of Luxembourg
Arno Zürbes - Fachhochschule Bingen, Germany

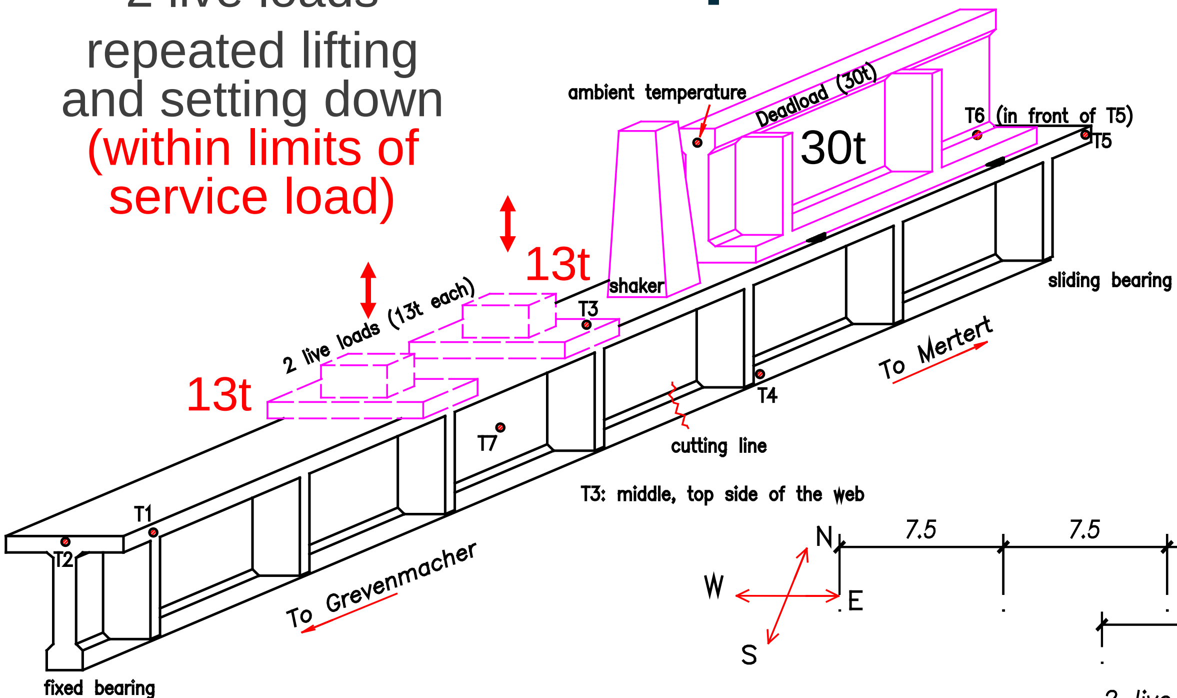
20th – 21st October 2016
Delft, Netherlands

ABSTRACT

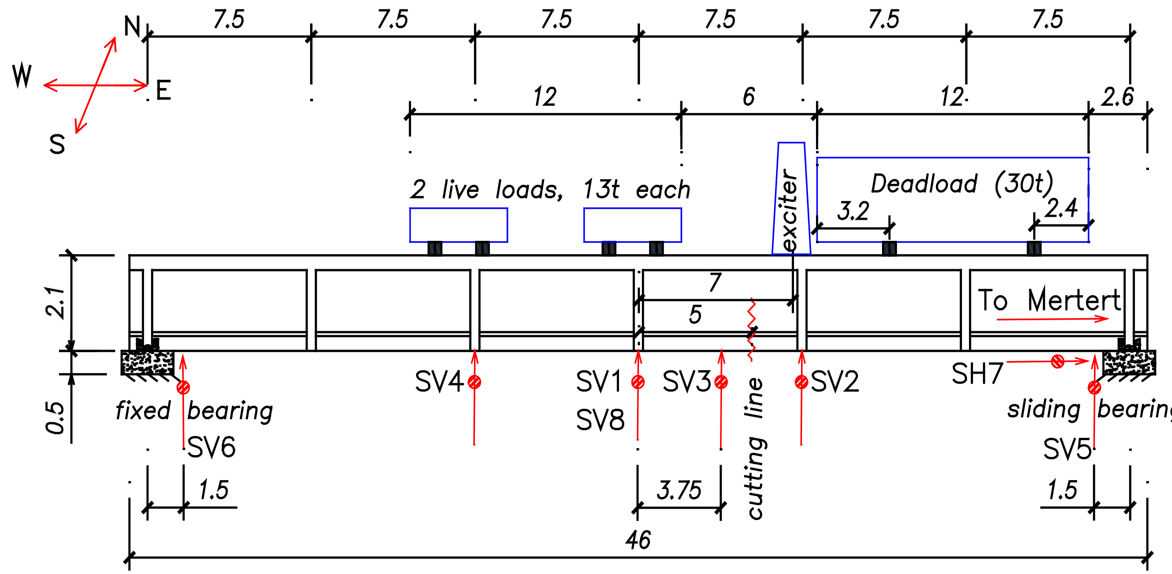
- Experimental results from repeated static loading tests of a prestressed concrete beam, a part from a real bridge
- Stepwise artificial damage was induced by successively cutting prestressed tendons
- Damage detection based on the monitoring of deflection line
- Displacements measured were clearly influenced by outside temperature variation
- Temperature compensation algorithm is proposed to reduce this influence

Experimental set-up

2 live loads
repeated lifting and setting down
(within limits of service load)

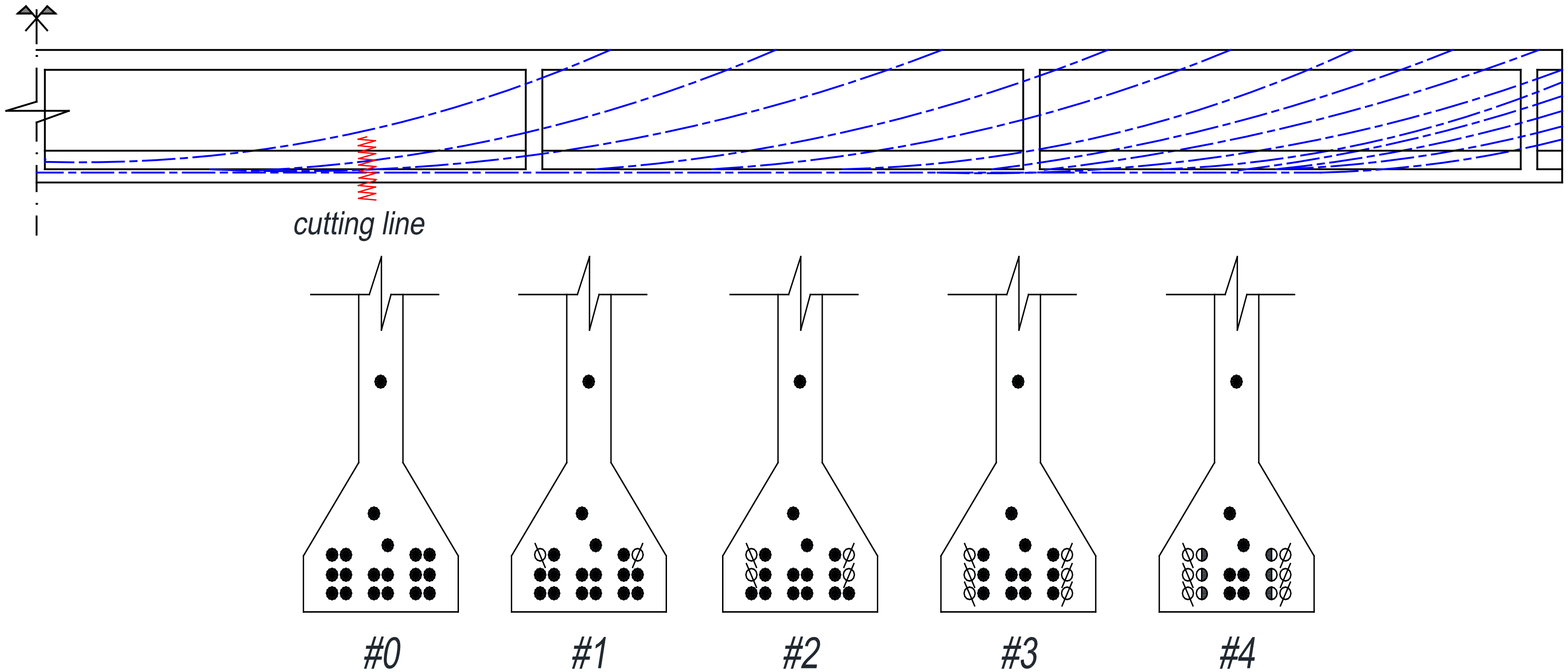


prestressed concrete beam
46m, 120t
part of a demolished bridge
jacked on two artificial supports



Damage scenarios

Increased artificial damage by successively cutting tendons

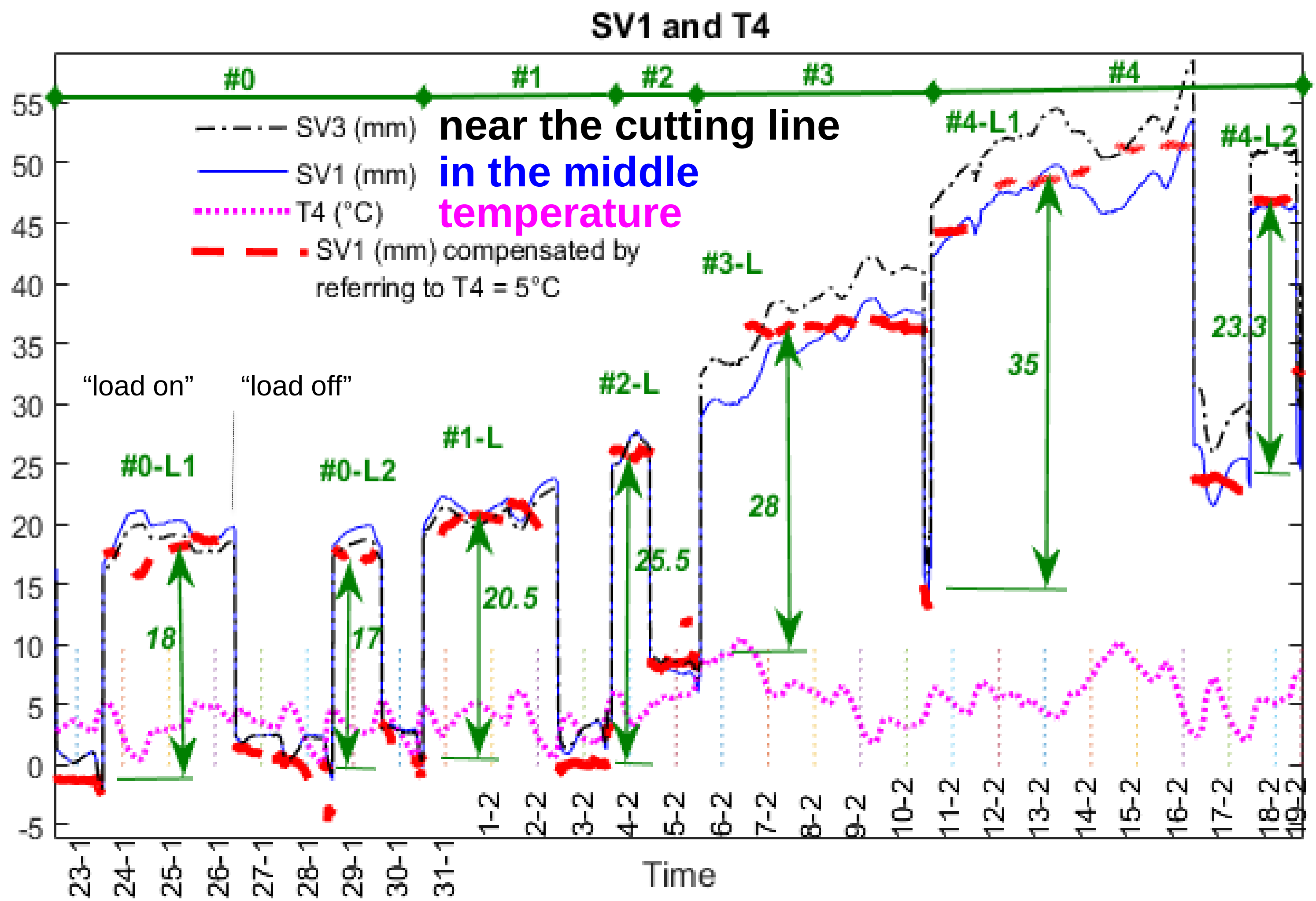


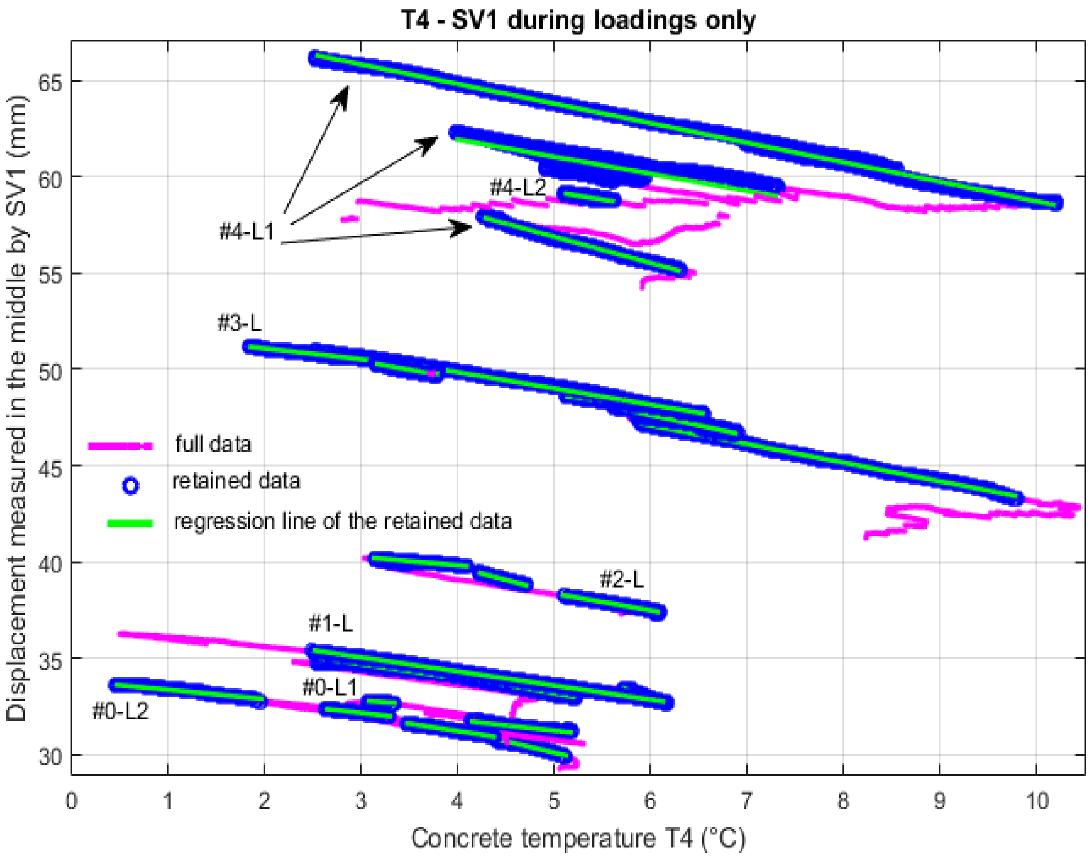
intact

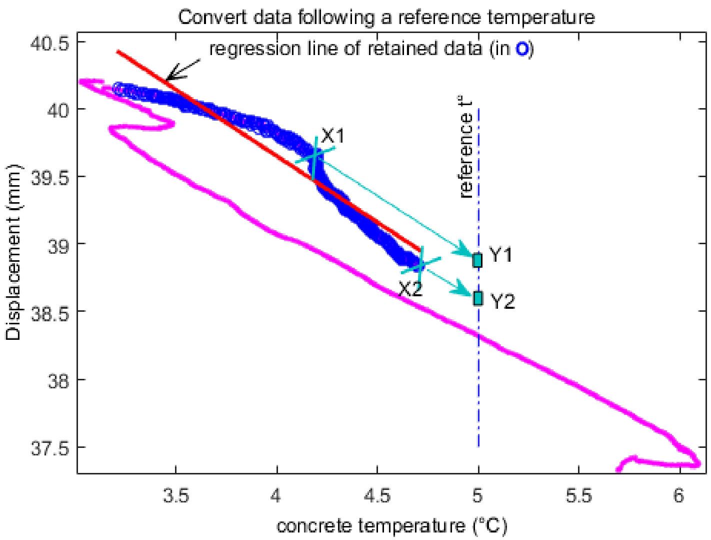
vertical hairline cracks

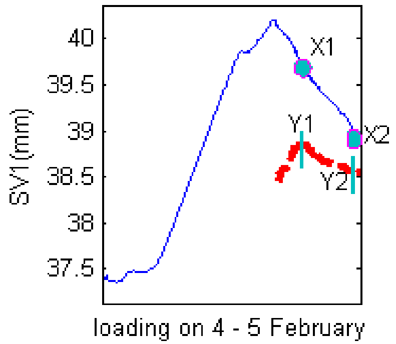
visibly cracked

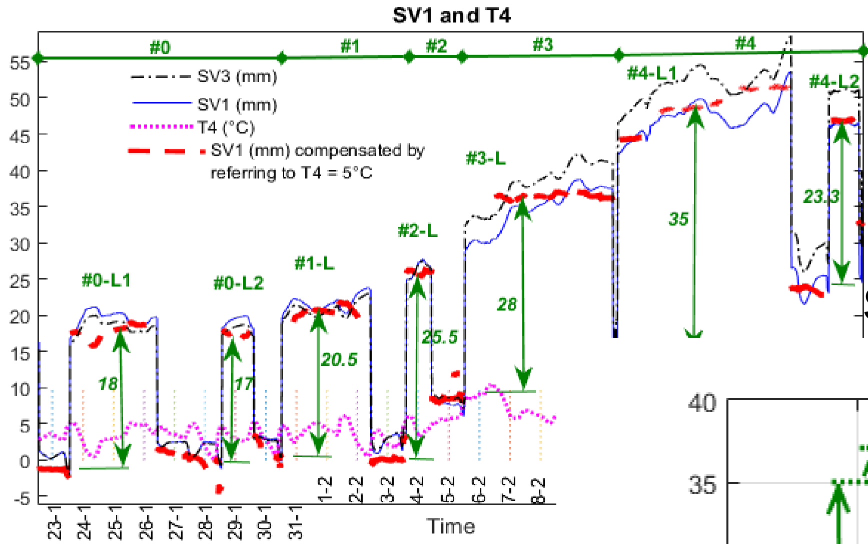
Deflection Measurements and Temperature Compensation

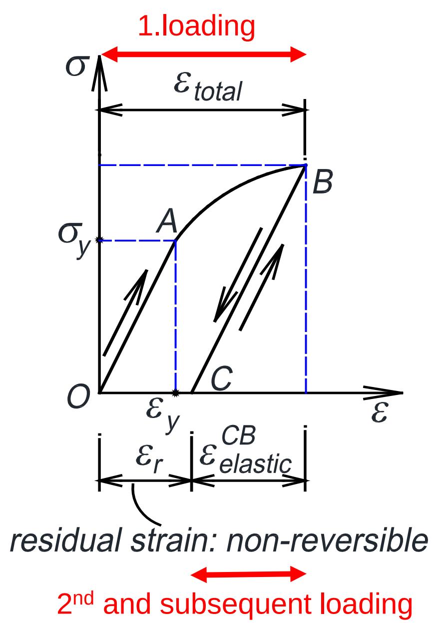



T4 - SV1 during loadings only
Displacement measured in the middle by SV1 (mm)
Concrete temperature T4 (°C)
full data
retained data
regression line of the retained data
#4-L1
#4-L2
#3-L
#2-L
#1-L
#0-L1
#0-L2

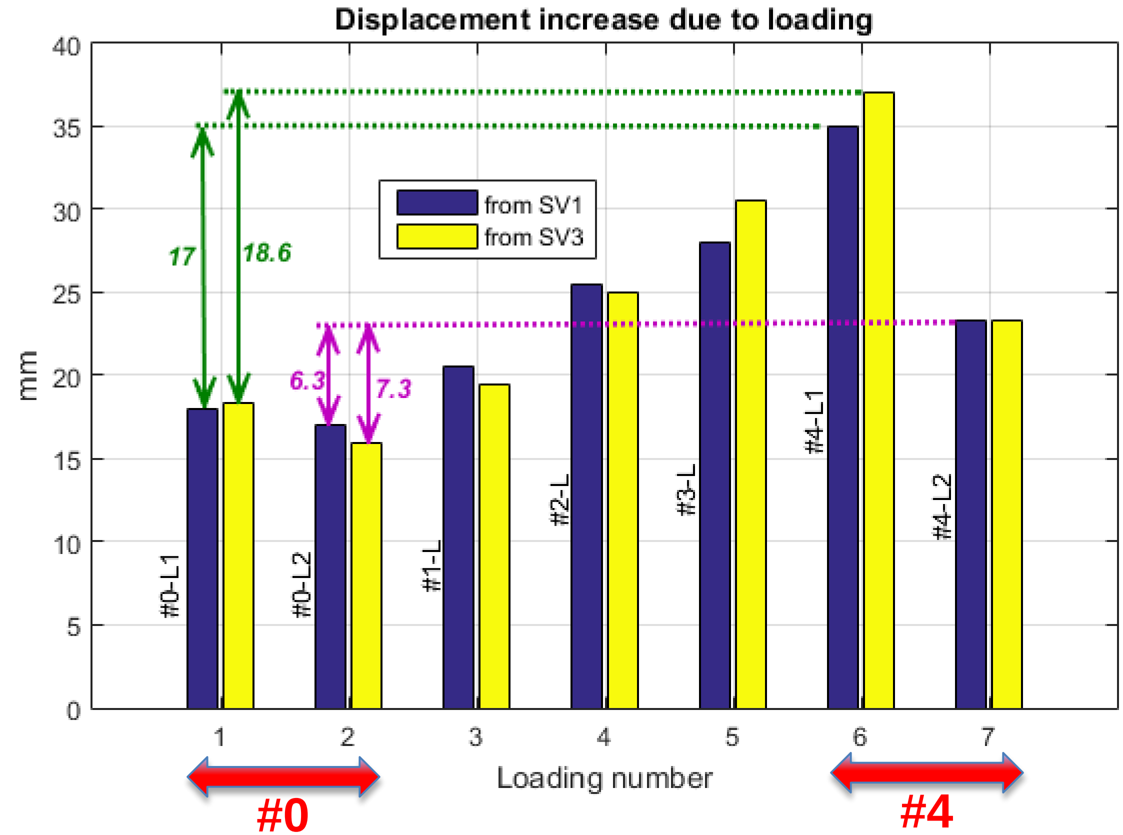

Convert data following a reference temperature
regression line of retained data (in O)
reference t°
Displacement (mm)
concrete temperature (°C)
X1
X2
Y1
Y2


SV1 (mm)
loading on 4 - 5 February
X1
X2
Y1
Y2

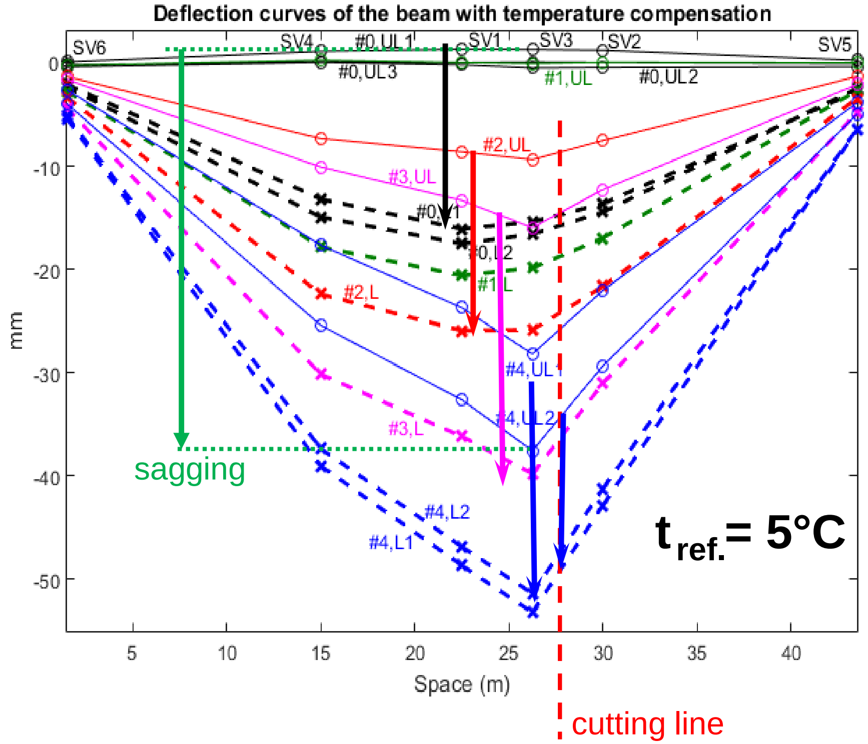




Step-height of compensated deflection



All deflection lines after temperature compensation



Abbreviations:

Damage scenario	#1, #2, ..	
Unloaded	UL	————————
Loaded	L	- - - - - - - -

Observations:

1. 38 mm of sagging in unloaded condition (UL)
2. maximum deflections shifts from the middle to the cutting line ➔ forming of a corner at the cutting line
3. Increasing deflection from unloaded (UL) to loaded (L) condition, except in #4, L2

Conclusions

- Load at least twice to separate plastic and elastic phenomena
- Temperature compensation algorithm is possible based on the slope of the deflection-temperature curve
- Sagging under own weight, shape and maximum of the deflection lines are the most important parameters
- Upcoming new photogrammetric and GPS measurement technology are helpful to measure the deflection line at one glance

➔ ***Stiffness reduction and hence damage detection is possible by repeated static load testing within the limits of service load***

Reference for further details:

Viet Ha Nguyen, Sebastian Schommer, Stefan Maas, Arno Zürbes, « Static load testing with temperature compensation for structural health monitoring of bridges » Engineering Structures 127 (2016) 700–718

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Sustainability score for roadway bridges

Giel Klanker – Rijkswaterstaat, Ministry of Infrastructure and the Environment, the Netherlands

Jaap Bakker– Rijkswaterstaat, Ministry of Infrastructure and the Environment, the Netherlands

Rijkswaterstaat
Ministry of Infrastructure and the Environment

20th – 21st October 2016
Delft, Netherlands

INTRODUCTION

Rijkswaterstaat: asset manager

Ministry of Infrastructure and the Environment: asset owner

Focus shifted from construction of new infrastructure to optimization of existing road network

Focus on required network performance

Aging bridge stock

Life cycle considerations are increasingly important

Financial and other performance aspects

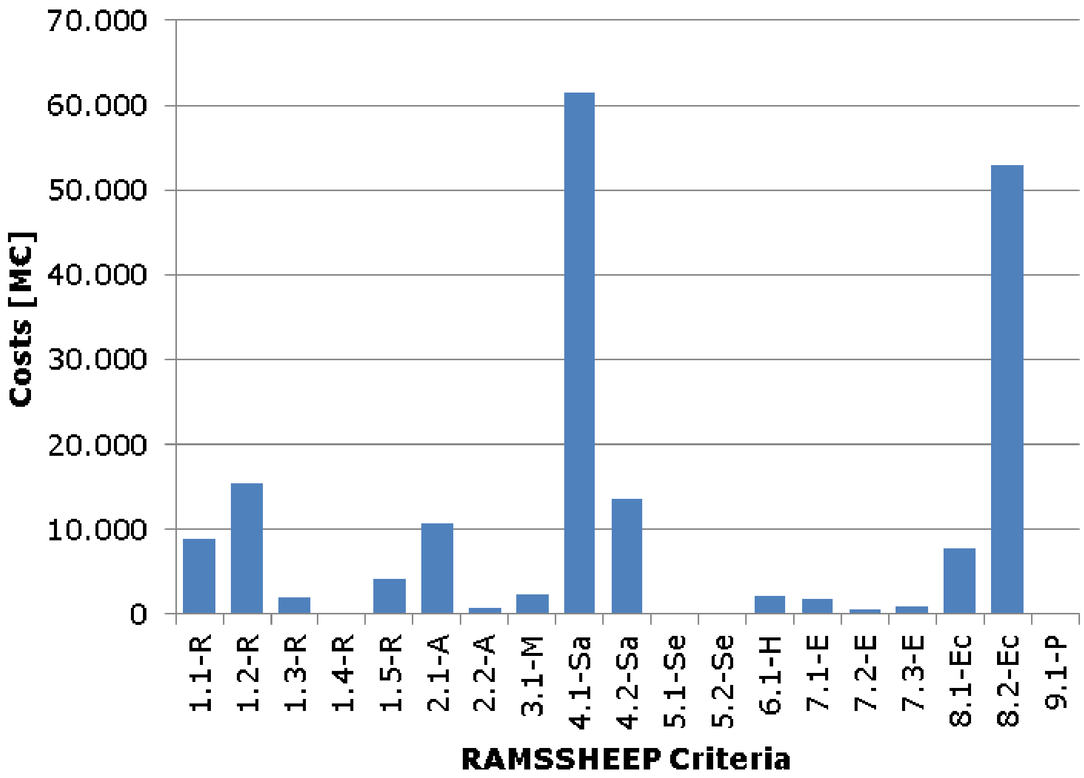
- Does the bridge still fulfill its intended purpose?
- Does the intended purpose meet today's (or tomorrows) requirements?

PERFORMANCE CRITERIA

- Reliability, Availability, Maintainability, Safety, Security, Health, Environment, Economics, Politics
- Examples:
  - Reliability: meet structural reliability requirements
  - Availability: meet object specific requirements
  - Environment: meet esthetical requirements
- Expert judgement by bridge inspector + semi-quantified tools (example: CRIAM score for structural reliability)

USE OF PERFORMANCE CRITERIA

- ✓ More objective risk assessment during inspection processes
- ✓ Maintenance planning based on performance criteria
- ✓ Network level reports
- ❑ Does not take bridge specific information explicitly into account (exception: CRIAM)



BRIDGE SPECIFIC INDICATORS

- RAMSSHEEP indicators can (in theory) be specified for a specific bridge
- However: information is often lacking

BRIDGE ‘semi’ SPECIFIC INDICATORS

- Proposed solution: ‘standard’ performance indicators for different performance criteria
- Default values could be assigned for some indicators (for example based on design life)

Criteria	P.I.		As built	As is	Risk level
Reliability	CRIAM score		-	Orange	4-High
Availability	Clearance	[m]	4,20	4,20	1-Negligible
Safety	Lane width	[m]	3,25	3,25	1-Negligible

- For other indicators, this will likely not be possible...

SUSTAINABILITY SCORE

- From intended to desired performance: ‘sustainability score’

Level	Description
1	Meets current design requirements
2	Current design requirements are not met, network performance is not compromised
3	Current design requirements are not met, network performance is compromised at object level
4	Current design requirements are not met, network performance is compromised at road link level
5	Current design requirements are not met, network performance is compromised at network level

Criteria	P.I.		As built	As is	As desired
Safety	Lane width	[m]	3,25	3,25	3,50

- Is the bridge still fit for purpose?

SUMMARY / CONCLUSIONS

- Integration of bridge management in asset or network management asks for indicators related to network performance
- Standard / generic indicators may be sufficient for a standard inspection process and network level reporting
- Decision making may be enhanced by bridge (semi) specific indicators, comparing the 'as is' with the 'as built' situation
- Furthermore, comparing 'as is' with 'as desired' situation, may be expressed in a 'sustainability score'

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

ENVIRONMENTAL PERFORMANCE FRAMEWORK FOR BRIDGE INFRASTRUCTURE MAINTENANCE

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Hooi Lee - Department of Civil & Environmental Engineering, University of Surrey, UK

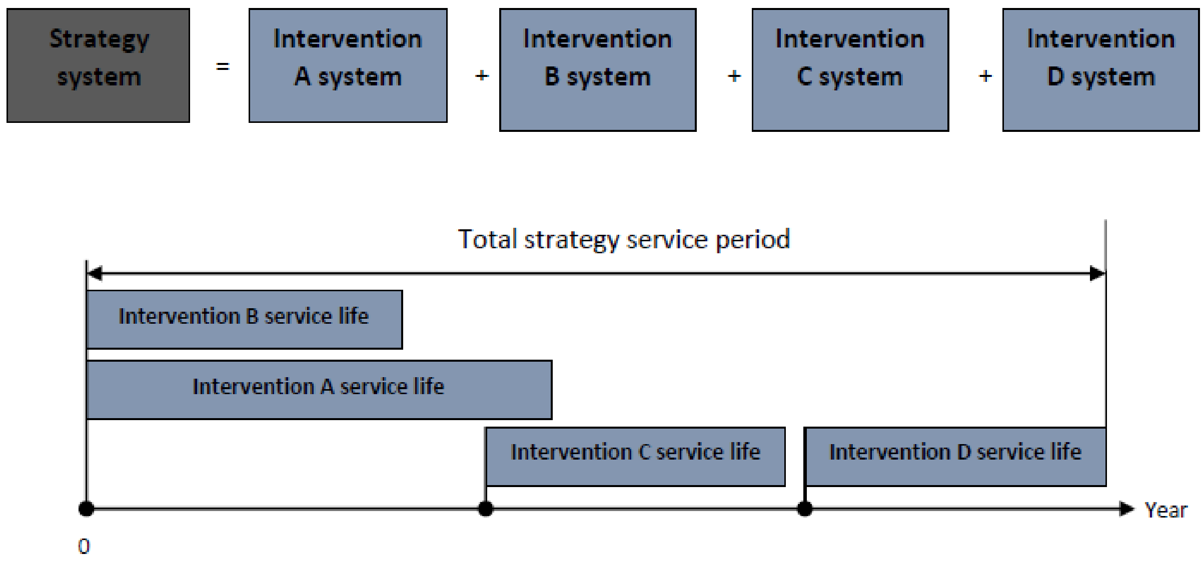
UNIVERSITY OF SURREY

20th – 21st October 2016
Delft, Netherlands

INTRODUCTION

- Aging infrastructure significant part of European networks
- Increasing number of assets to be fully replaced
- Maintenance to prolong service life is often preferred
- Growing importance of environmental considerations (*resource use, waste, climate change*)
- Little work carried out on environmental impacts of maintenance in comparison with asset renewal
- Very few studies on impacts from whole strategy performance and structure life-cycle perspectives
- Unequal life extensions provided by different strategies
- Optimised choice should consider both economic and environmental consequences of maintenance strategies and their trade-offs

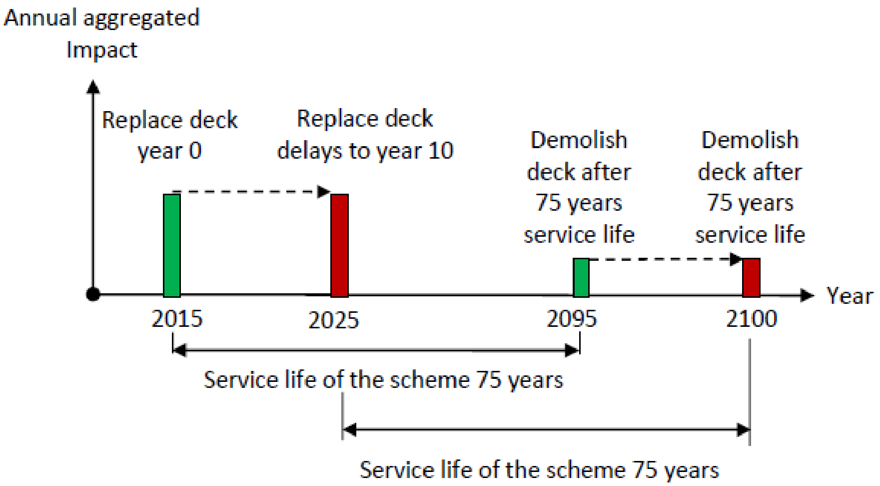
MAINTENANCE STRATEGIES

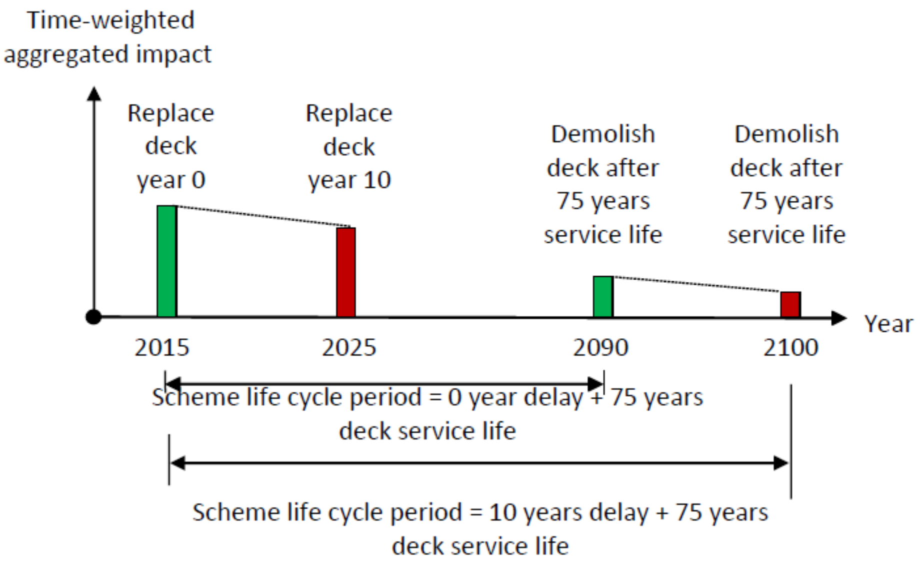


Intervention Examples for Metallic Bridges		
Maintenance interventions	Expected service life (years)	`Source
Protective paint application using M20 epoxy rich paint system	18-22	NR/GN/CIV/002 The use of protective paint and sealant (2009)
Steel repair system	30	NR/GN/CIV/002 The use of protective paint and sealant (2009)
Steel plate strengthening system	60	NR/GN/CIV/002 The use of protective paint and sealant (2009)
New deck system for replacement	120	NR/GN/CIV/002 The use of protective paint and sealant (2009)

MAINTENANCE STRATEGIES

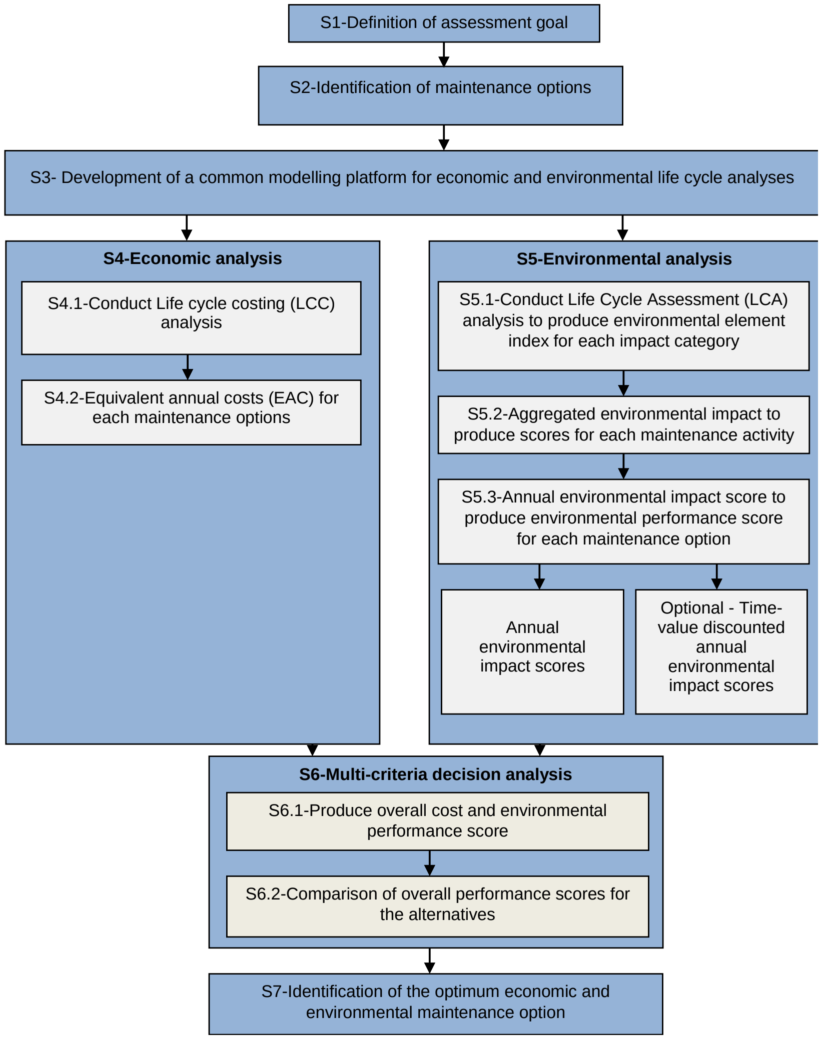
Deferring maintenance



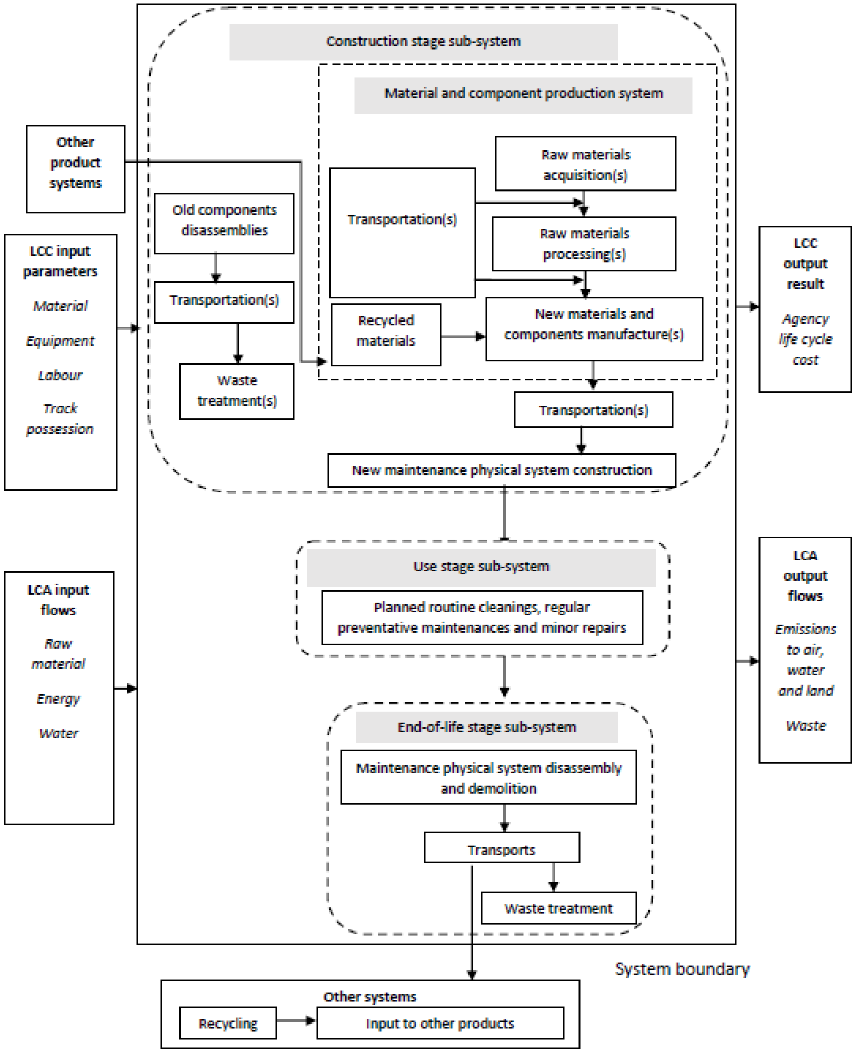


Time-weighting of environmental impacts

DECISION-SUPPORT FRAMEWORK



LIFE CYCLE ENVIRONMENTAL ASSESSMENT



LIFE CYCLE COST ANALYSIS

Equivalent Annual Cost (EAC)

$$\mathrm{EAC} = (\mathrm{NPV}_{\text{activities ,year j}} + \mathrm{NPV}_{\text{activities ,year k}} + \mathrm{NPV}_{\text{activities ,year m}} + \cdots)\ \mathrm{x\ AF}$$

Net Present Value (NPV)

$$NPV = \left(\frac{\mathrm{Cn}}{(1+r)^{\mathrm{n}}}\right)$$

Annuity Factor (AF)

$$AF = \frac{r}{1-\frac{1}{(1+r)^{t}}}$$

t is the total extended service period of the maintenance intervention or strategy;
C is the total activity cost;
r is the discount rate in percentage.

LIFE CYCLE ENVIRONMENTAL ASSESSMENT

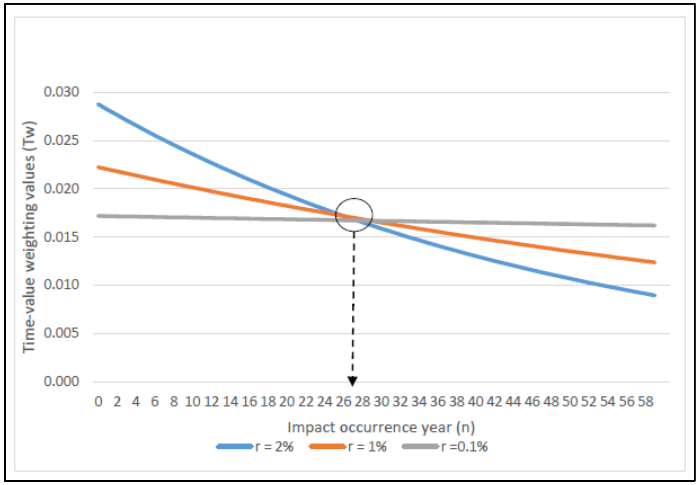
Annual Aggregated Environmental Impact

$$\text{Ann. Agg. EI (in units/year)} = (\sum \text{Agg.EI}_{prod,year\ i} \times Tw_{year\ i}) + (\sum \text{Agg.EI}_{constr,year\ j} \times Tw_{year\ j}) + (\sum \text{Agg.EI}_{use,year\ k} \times Tw_{year\ k}) + (\sum \text{Agg.EI}_{end,year\ m} \times Tw_{year\ m})$$

Time Weighting Factor (Tw)

$$Tw = \text{Discount Factor} \times \text{Annuity Factor} = \frac{1}{(1+r)^n} \times \frac{r}{1-\frac{1}{(1+r)^T}} = \frac{r(1+r)^T}{(1+r)^n[(1+r)^T-1]}$$

r is the environmental discount rate in percentage;
n is the aggregated impact occurred due to activity at a specific year i, j ,or k;
T is the total residual service life duration of the structure; $T=t+t_d$, where t = extended years duration and t_d = delayed year duration



COMBINED ECONOMIC & ENVIRONMENTAL ASSESSMENT

$$\text{CEE score} = (W_{eco} \times \text{EAC}) + (W_{env} \times \text{Ann. Agg.EI})$$

CEE Score	is the combined economic and environmental score
W_{eco}	is the economic weighting factor
EAC	is the Equivalent Annual Costs
W_{env}	is the environmental weighting factor
Ann. Agg. EI	is the Annual Aggregated Environmental Impact score

- Derivation of scores through the Simple Multi-Attribute Rating Technique using Swings (SMARTS) technique
- MCDA analysis
- LCC costs and environmental impact values are normalised into a 0-100 score performance, where 0 represents the most favourable and 100 the least favourable performance.

LIFE CYCLE ENVIRONMENTAL ASSESSMENT

Annual Aggregated Environmental Impact

Paint maintenances activities		**Alternative 1: High durable paint**	**Alternative 2: conventional paint**
Assumed aggregated impact for production of the paint material	$\sum Agg.EI_{prod}$	98 units	98units
Assumed aggregated impact for new paint application onto the structure	$\sum Agg.EI_{constr}$	30 units	30 units
Assumed aggregated impact for carrying out routine touch up to the paint	$\sum Agg.EI_{use}$	0 units	18 units
Assumed aggregated impact for removing and disposing the paint	$\sum Agg.EI_{end}$	20 units	20 units
Assumed extended service years duration given by the paint	t	30 years	15 years
Calculated non-discounted time-value weighting	Tw_c	0.03	0.07
Calculated non-discounted Annual Aggregated Environmental Impact	**Non-discounted Ann. Agg. EI**	**5 unit scores/year**	**11 unit scores/ year**
Environmental performance indicator	**EP ratio**	**1: 2**	

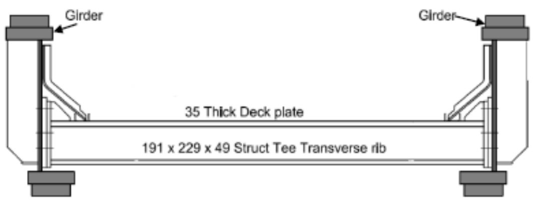
LIFE CYCLE ENVIRONMENTAL ASSESSMENT

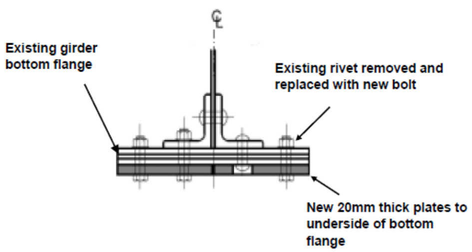
Time-delay Effect

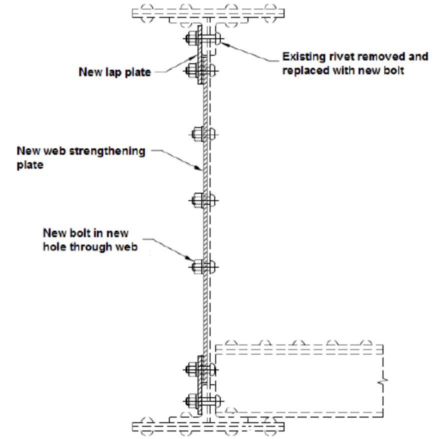
Paint maintenances activities		Alternative 1 Delayed steel repair	Alternative 2 Immediate steel repair
Assumed aggregated impact for production of steel plates, year i= 0	$\sum Agg.EI_{prod,year\ i}$	980 units	980 units
Assumed aggregated impact for repairing the structure, year j = 5	$\sum Agg.EI_{constr}$	300 units	300 units
Assumed aggregated impact for minor maintenance to the steel repair system during its service period, year k = 30	$\sum Agg.EI_{use}$	0 units	0 units
Assumed aggregated impact for deconstructing and disposal of the steel system, year m = 59	$\sum Agg.EI_{end}$	200 units	200 units
Assumed discount rate r = 1%	r	1%	1%
Alternative 1 to delay for 5 years	t_d	5 years	0 year
Assumed extended service years duration given by the paint	t	55 years	55 years
Total service life extension	T	60 years	55 years
Calculated discounted time-value weighting for year i = 0	$TW_{year\ i}$	0.022	0.024
Calculated discounted time-value weighting for year j = 5	$TW_{year\ j}$	0.021*	0.023
Calculated discounted time-value weighting for year k = 30	$TW_{year\ k}$	0.016*	0.018
Calculated discounted time-value weighting for year m = 59	$TW_{year\ m}$	0.012*	0.013
Calculated discounted Annual Aggregated Environmental Impact	**Discounted Ann. Agg. EI**	**29 unit scores/year**	**33 unit scores/ year**
Environmental performance indicator	**EP ratio**	**0.87 : 1.0**	

CASE STUDY - STRATEGIES

Year	Maintenance strategy		
	(i) Deck replacement	(ii) Standard deck restoration	(iii) Minimal deck restoration
0	Deck reconstruction	Steel repair and paint treatment	Steel repair
5			Steel repair
10			Steel repair
15			Steel repair
21			Repaired system removal and disposal
27		Minor repair and patch repainting	
32		Repaired system removal and disposal	
75	Deck removal and disposal		

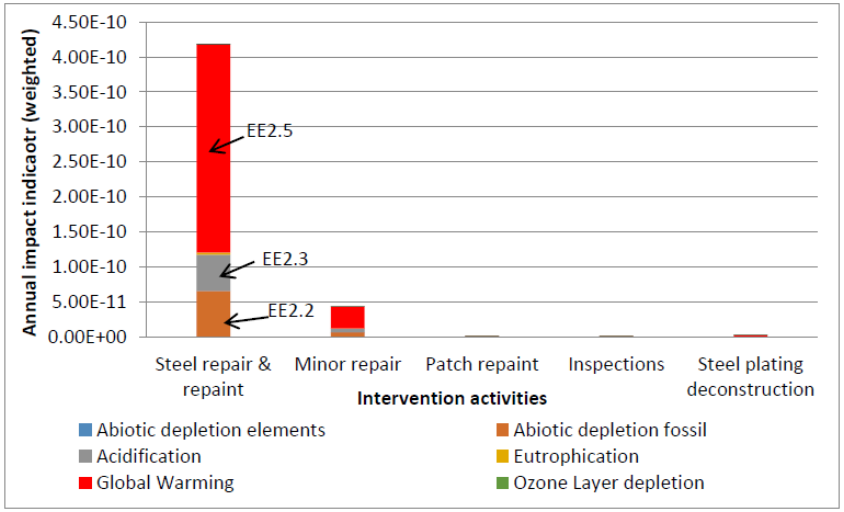


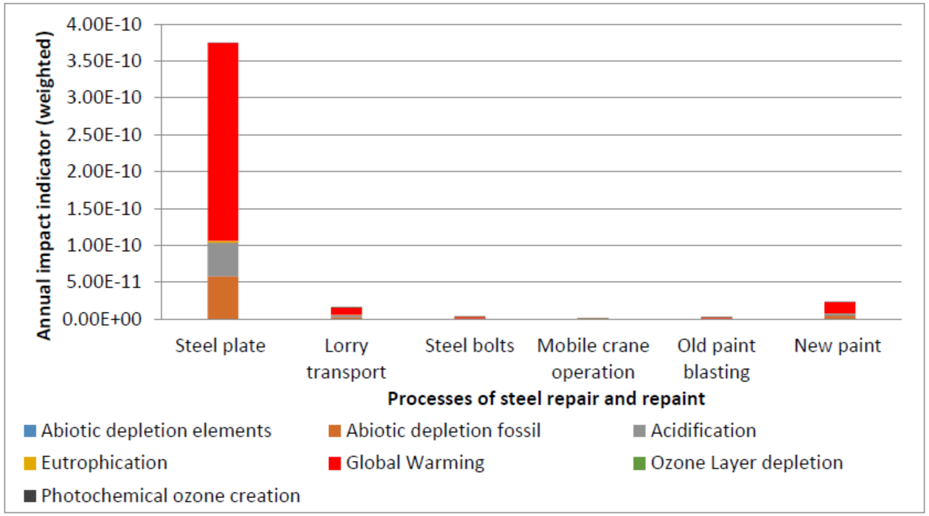




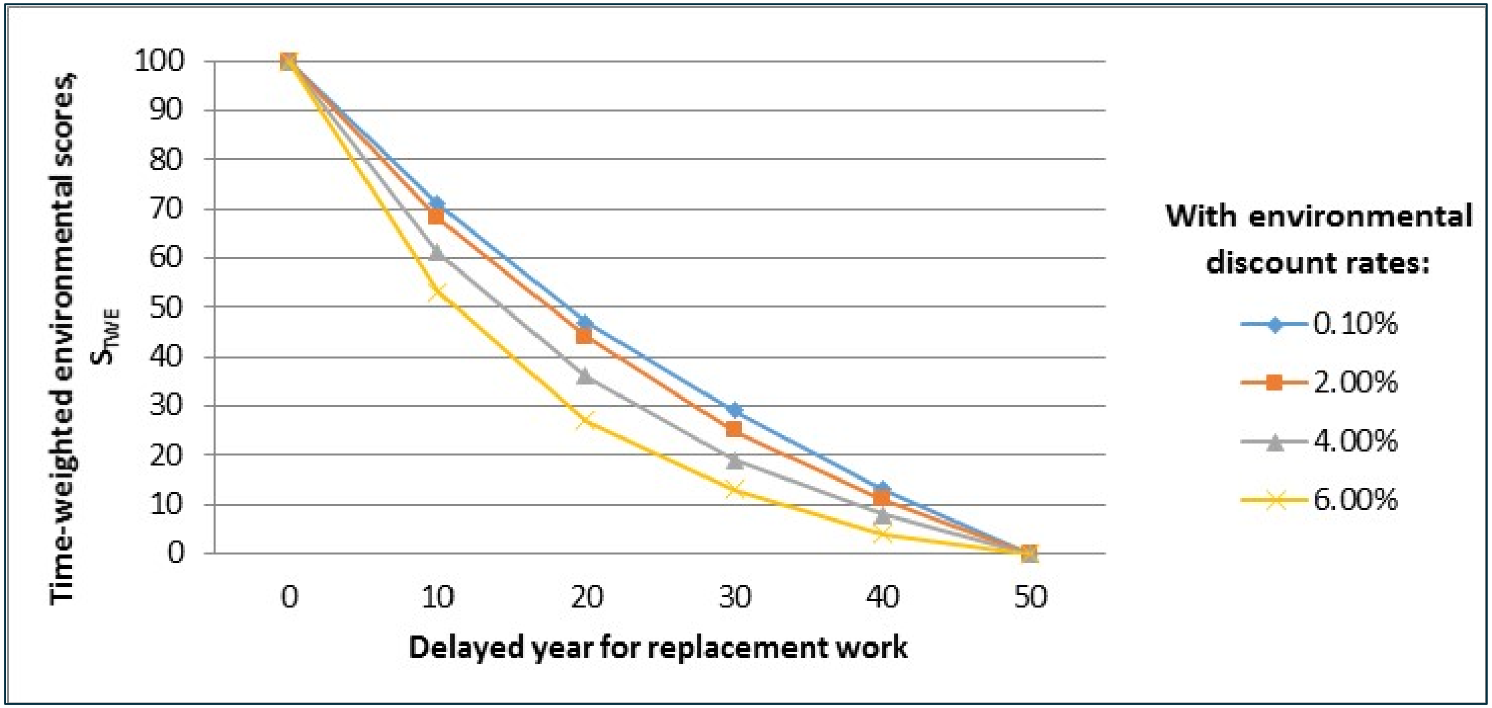
CASE STUDY - RESULTS

Strategy Option	Total NPV (£)	Working Life (years)	EAC (£)	Total Agg. impact	Ann. Av. Agg. impact	Economic score (S_{AC})	Environmental score (S_{AE})
(i)	308,540	75	11,684	7.3×10^{-8}	9.7×10^{-10}	100 points	100 points
(ii)	212,260	32	11,131	1.5×10^{-8}	4.6×10^{-10}	4 points	0 points
(iii)	165,952	21	11,106	1.3×10^{-8}	6.4×10^{-10}	0 points	34 points





CASE STUDY - RESULTS



Time-weighted environmental scores

CONCLUDING REMARKS

- Decision-support framework for assessing economic and environmental performance of maintenance strategies
- Unequal working lives
- Equivalent Annual Cost and Annual Aggregrate Environmental Impact concepts
- Capturing the effects of delaying maintenance activities
- Time-weighting factor for delayed environmental impacts – leads to environmental benefits
- Working life of maintenance strategy has a large influence on economic and environmental performance
- Explicit quantification of standard maintenance interventions for metallic bridges

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

LIFE CYCLE COST OPTIMISATION ON A SET OF BRIDGES

Joana Almeida - Polytechnic Institute of Viana do Castelo, Portugal

Raimundo Delgado - University of Porto, Portugal

Paulo Teixeira - University of Lisboa, Portugal

20th – 21st October 2016
Delft, Netherlands

INDEX

Introduction

Methodology to support bridge management decisions based on LCC optimization

- Degradation model
- Life cycle cost estimation
- Optimisation process

Results for a set of concrete roadway bridges

Conclusions

INTRODUCTION

LIFE CYCLE COST optimisation methodology

for a set of BRIDGEs

during a medium or a long period of time

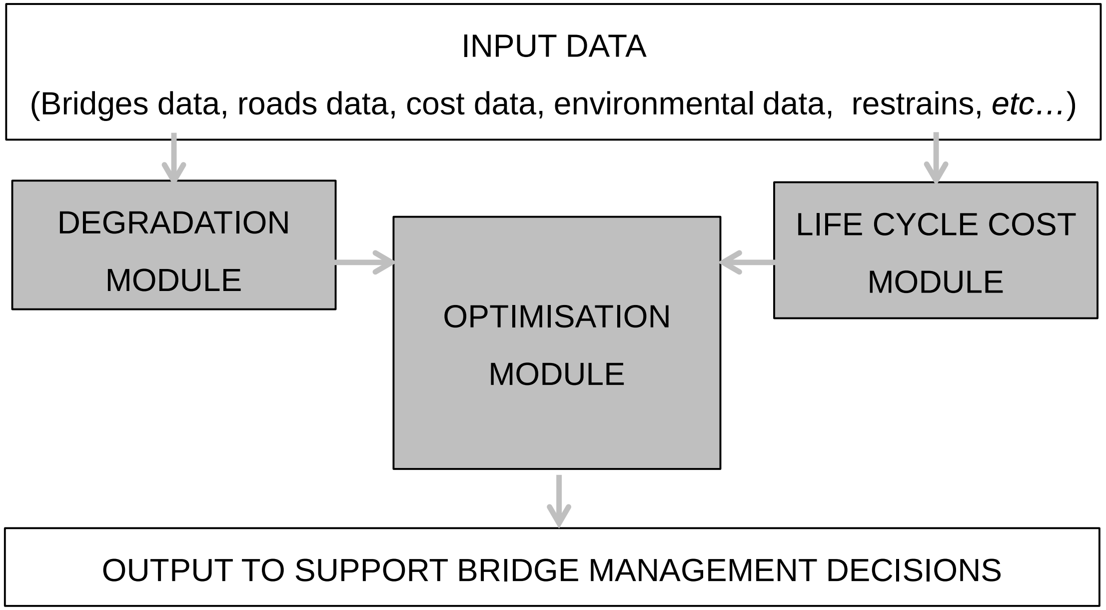
SUPPORT DECISIONS

related to the **scheduling** of the necessary interventions over time

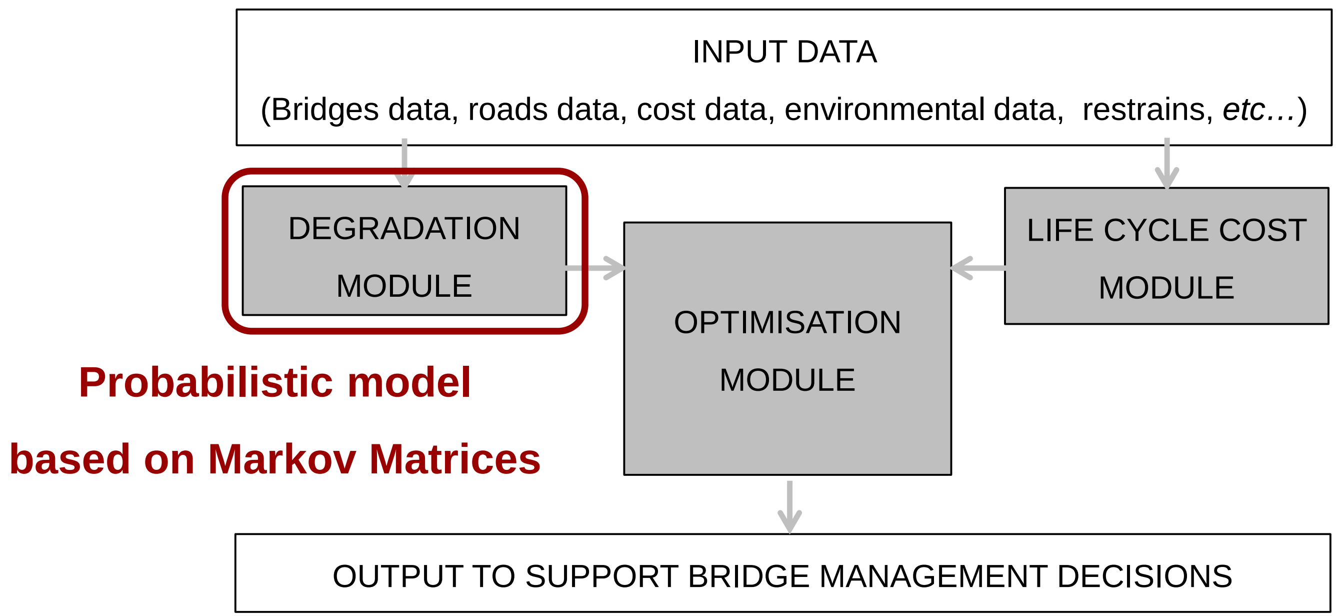
ensuring the required **performance** level

and taking the available **budget** into consideration

METHODOLOGY STRUCTURE



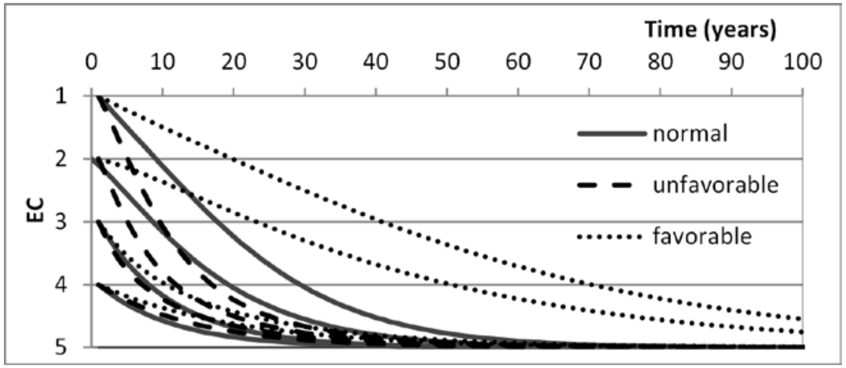
METHODOLOGY STRUCTURE



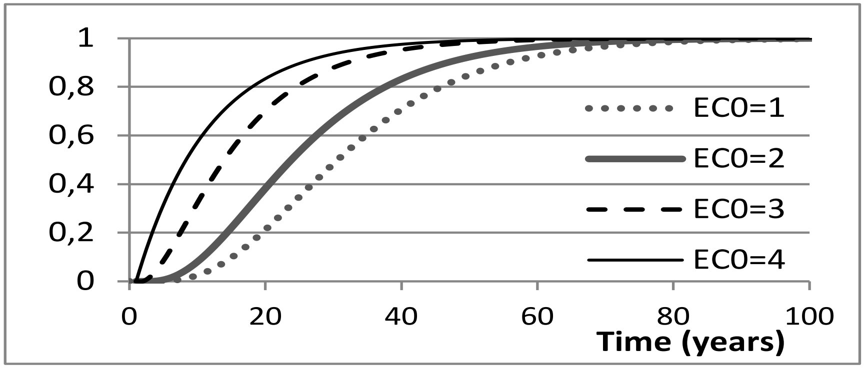
DEGRADATION MODEL

Probabilistic model based on Markov Matrices

Performance index: condition state with 5 stages scale



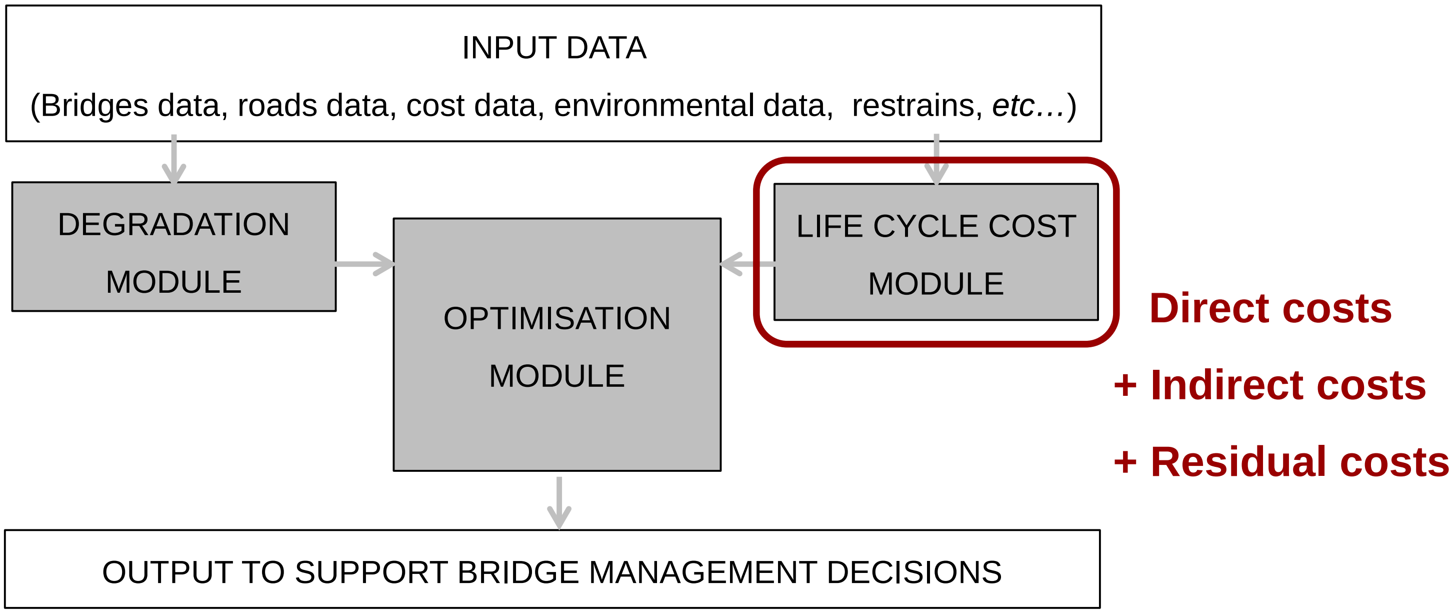
<u>Condition State</u> (EC) temporal evolution for different initial condition states for unfavorable, normal and favorable conditions.



<u>Probability of being in the worst condition</u> (EC5) temporal evolution for different initial states, for normal environmental conditions.

CONCRETE RODWAY BRIDGES
Markov matrices proposed in Roelfstra, G., Modele d'evolution de l'etat des ponts-routes en beton. PhD thesis, École Polytechnique Fédéral de Lausanne (2001).

METHODOLOGY STRUCTURE



LIFE CYCLE COST ESTIMATION

for a set of bridges

Direct costs + Indirect costs + Residual costs

$$LCC_{set\ of\ bridges} = \sum_{b=1}^{nb} LCC_b = \sum_{b=1}^{nb}\left[\sum_{t=t0}^{tu}\left(\frac{DC_{b,t,a}+IC_{b,t,a}}{(1+DR)^{(t-t0)}}\right)+\frac{RC_p}{(1+DR)^{(tu-t0)}}\right]$$

LCC – life cycle cost

b – bridge number

nb – total number of bridges

a – type of intervention

t – year

$t0$ – first year

tu – last year

DR – discount rate

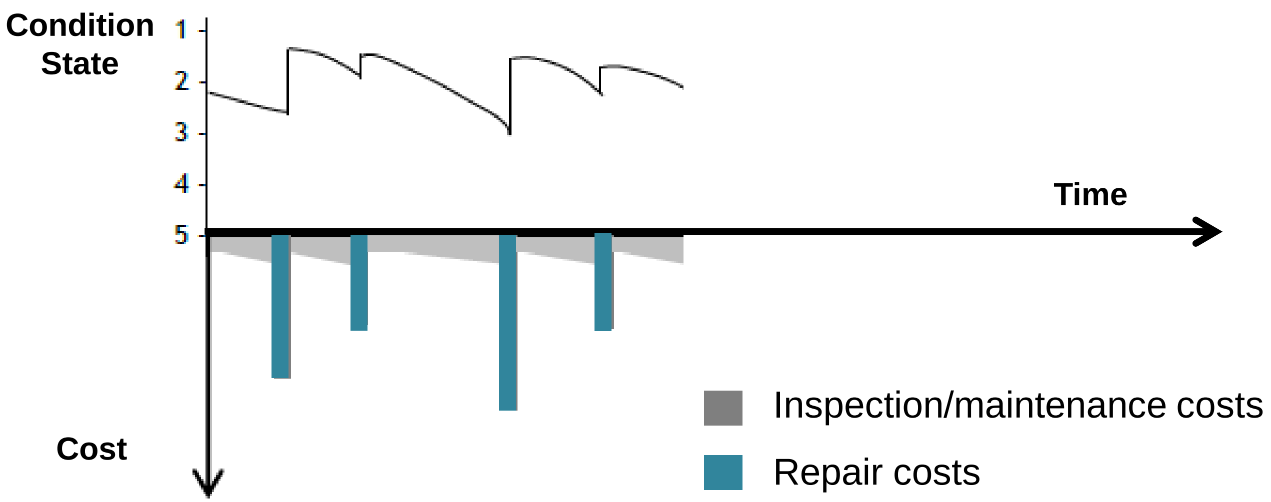
DC – direct cost

IC – indirect cost

RC – residual cost

LIFE CYCLE COST ESTIMATION

for each bridge



Costs for different periods of time

→updated to the initial moment through an annual **Discount Rate**

LIFE CYCLE COST ESTIMATION

Direct costs

for the administration

$$DC_{b,t,a} = VE_{b,t}.VC_a.UC_{a,b}.A_b$$

DC – direct cost

b – bridge number

t – year

a – kind of intervention

A – bridge deck area

UC - intervention unitary cost by area for such type of bridge

VE – bridge condition state at the time of the intervention;

VC - multiplicative condition factor to correct the cost of the maintenance and repairs for different condition states

LIFE CYCLE COST ESTIMATION

Indirect costs

for the users

$$IC_{b,t,a} = VE_{b,t}.VC_a.(TC_{b,t,a}+CC_{b,t,a})$$

IC – indirect cost

b – bridge number

t – year

a – kind of intervention

TC – time costs related to the passengers delay

CC - extraordinary vehicle circulation costs

VE – bridge condition state at the time of the intervention

VC - multiplicative condition factor to correct the cost of the maintenance and repairs for different condition states

LIFE CYCLE COST ESTIMATION

Residual costs

to ensure that the results are independent of the period of time

$$RC_b = \frac{LT[AG(tu); CS = 1] \quad - \; LT[AG(tu); CS(tu)]}{LT[AG(tu); CS = 1]} . \mathrm{UC}_{\mathrm{a=2,b}} . \mathrm{A}_b$$

RC - residual cost

b – bridge number

tu – last year

LT – lifetime

AG – age of the bridge

a=2 – replacement

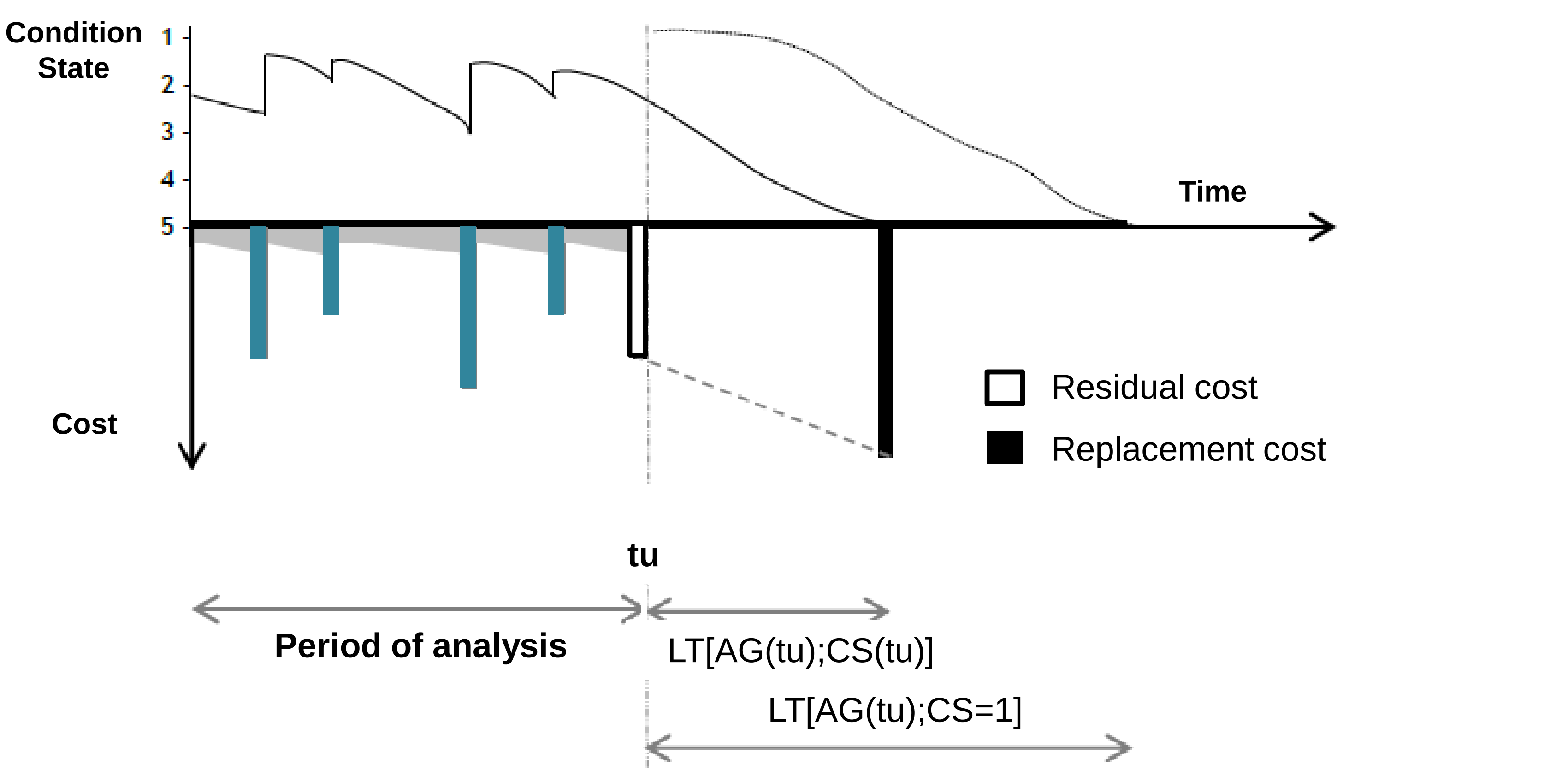
CS – condition state

UC - intervention unitary cost by area for the kind of bridge

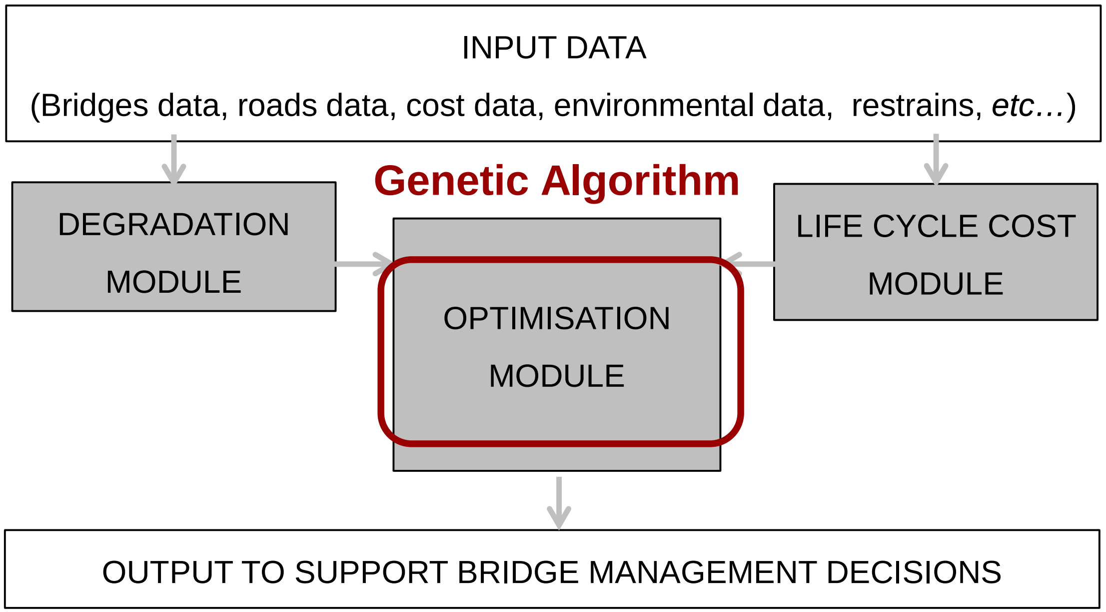
A – bridge deck area

LIFE CYCLE COST ESTIMATION

Residual costs



METHODOLOGY STRUCTURE

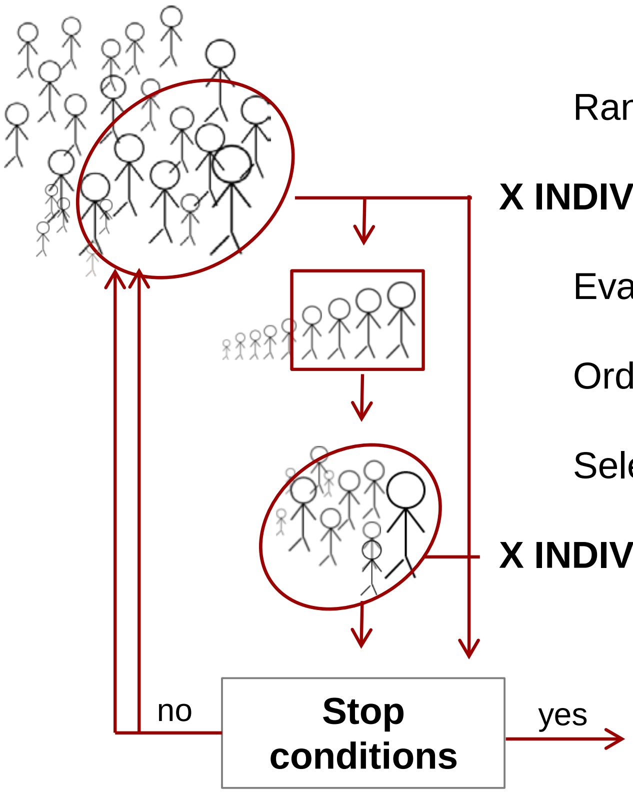


OPTIMISATION PROCESS

Final objectives:	identify the optimized plan for interventions on a set of bridges during a medium/long period of time
Restrictions:	costs and performance limits (per bridge or for the bridges network)
Optimization goal:	life cycle cost minimization

OPTIMISATION PROCESS

genetic algorithm



Random creation of X individuals (Individual = intervention scenario)

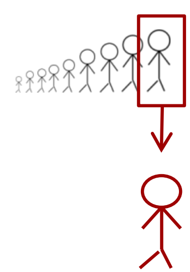
X INDIVIDUALS POPULATION

Evaluate each individual fitness (according to objectives and restrictions)

Order the population individuals according to its fitness

Select the best individuals to generate the next population

X INDIVIDUALS POPULATION



Evaluate each individual fitness

Chose the best fitness individual

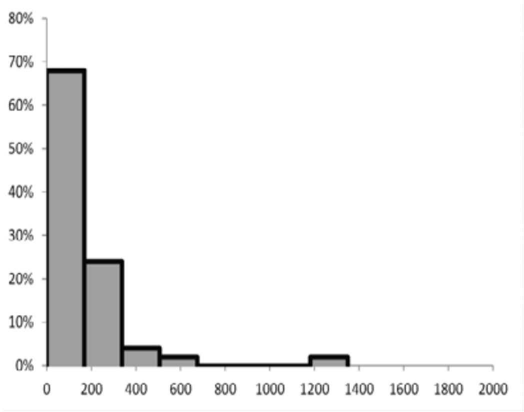
Optimum scenario of intervention

RESULTS

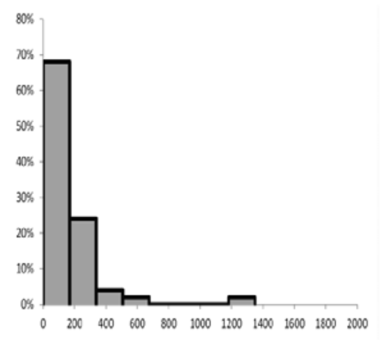
for a set of concrete roadway bridges

Period of analysis: 20 years

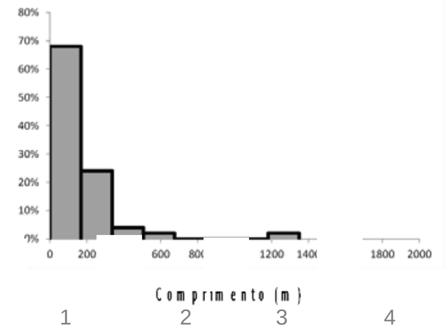
Sample: 100 portuguese bridges



total length 50-1500m



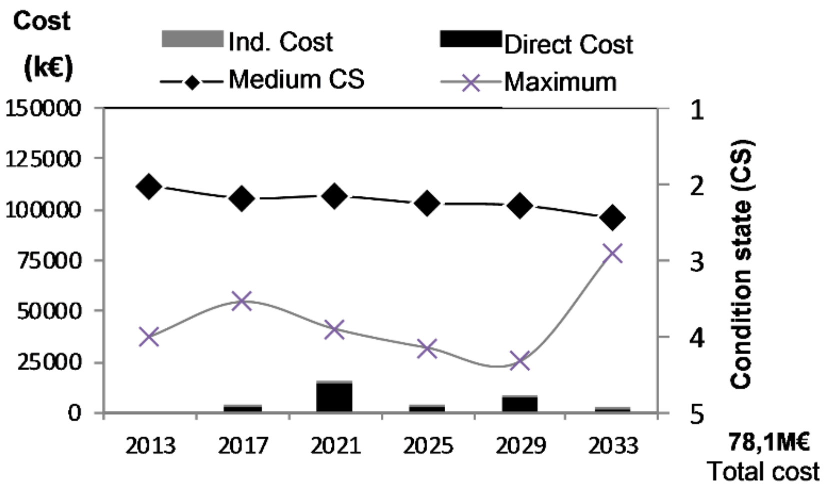
several ages



several condition states
(between 1-4; average=2)

RESULTS

deterministic analysis

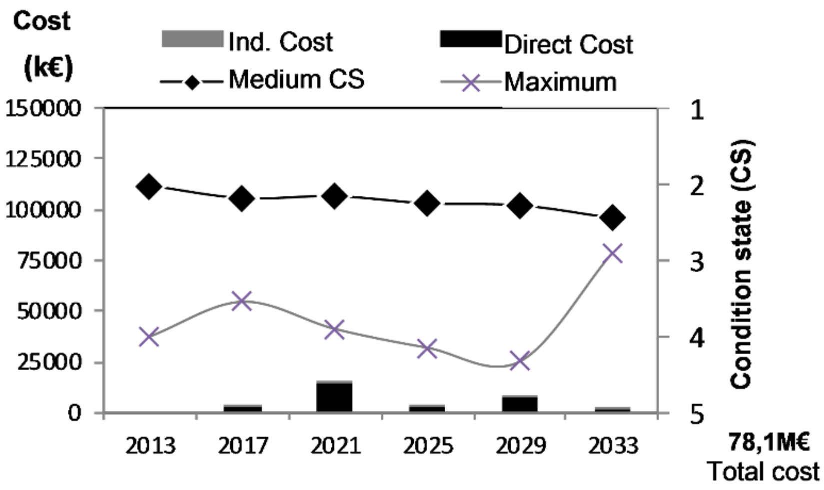


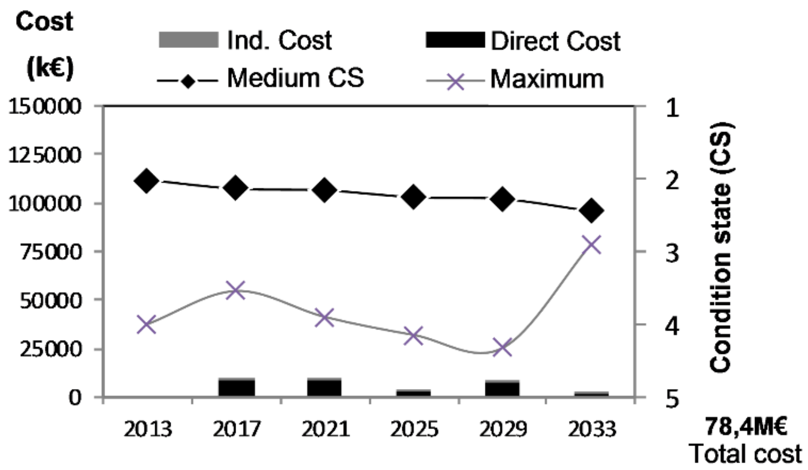
Optimal solution

- Actions
- Costs

RESULTS

imposing restrictions





Optimal solution

without cost constraints

Total cost: 78,1M€

Imposing a direct cost limit

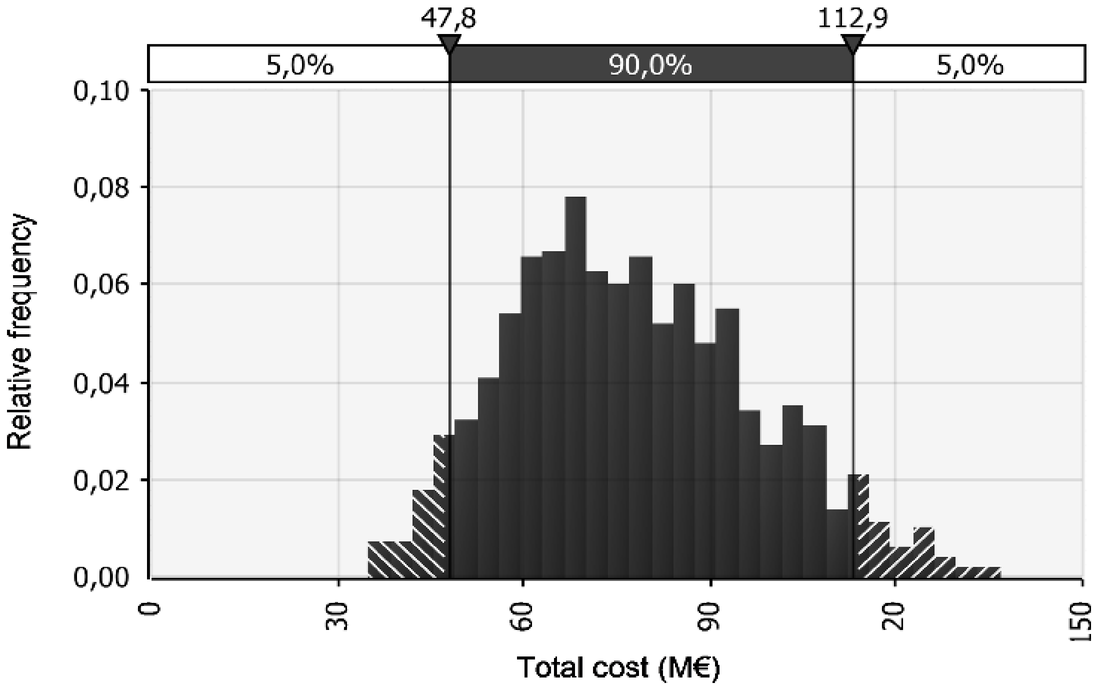
of 10M€ per quadrennial

Total cost: 78,4M€

RESULTS

probabilistic analysis

Monte Carlo method - 1000 simulations results



100 bridges

20 years

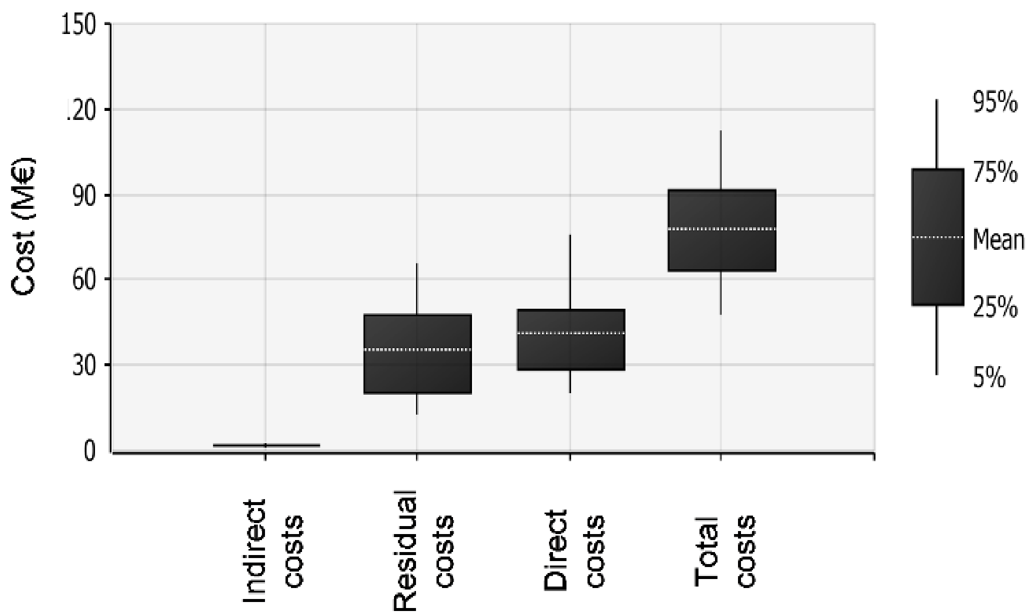
Average total cost: 78M€

Triangular shape (as the variables)

RESULTS

probabilistic analysis

Monte Carlo method - 1000 simulations results



Total cost variation:
60 – 90M€

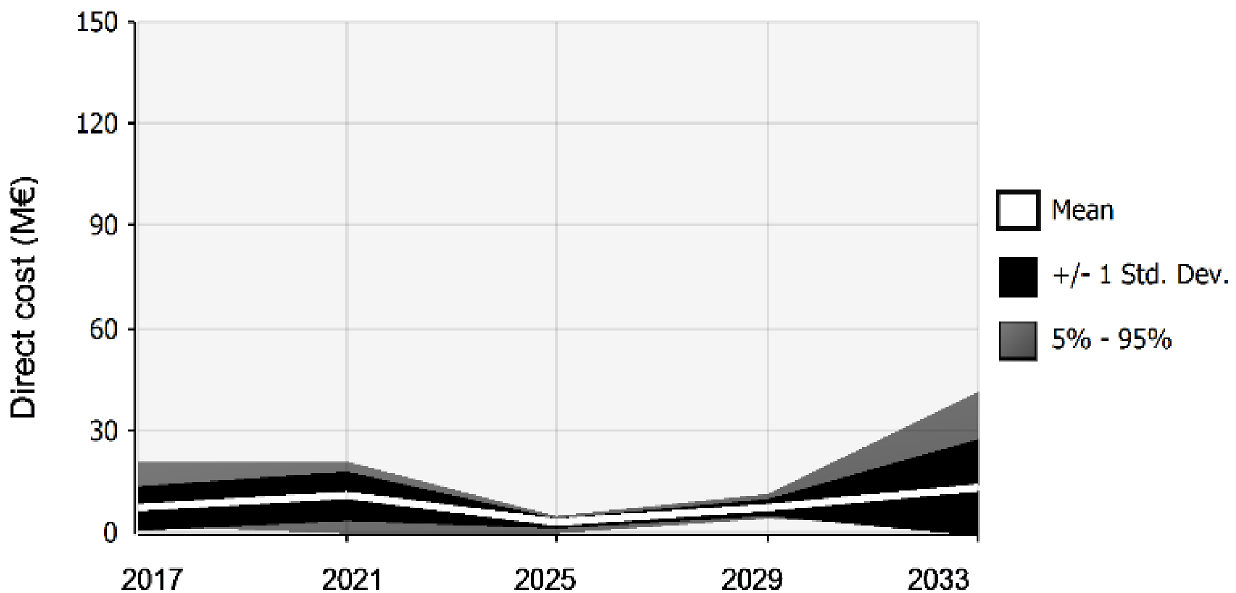
Indirect costs (traffic costs):
Little expressive because only repair works were planned

Residual costs:
Not negligible

RESULTS

probabilistic analysis

Monte Carlo method - 1000 simulations results



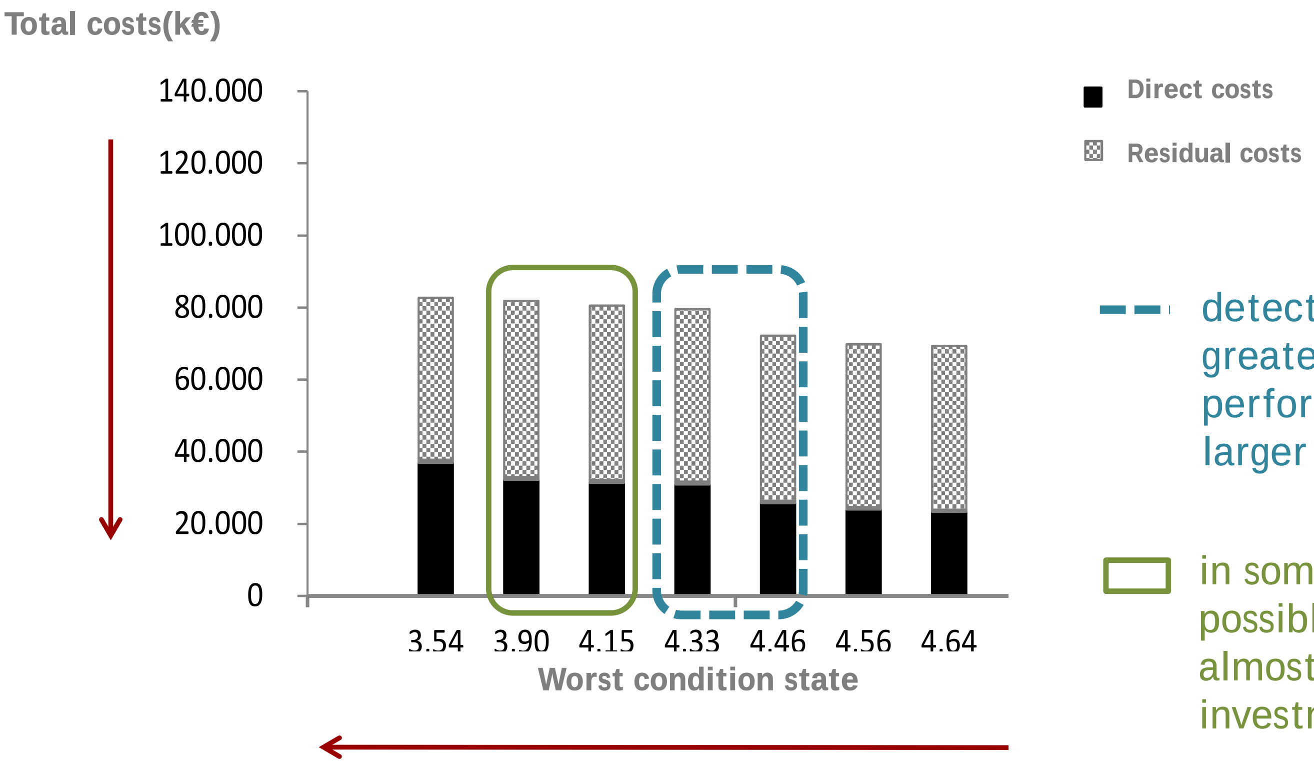
Direct costs variation with time

The variability tends to be higher at the end of the period of analysis

RESULTS

multi-objective optimisation

Based on non dominated Pareto fronts



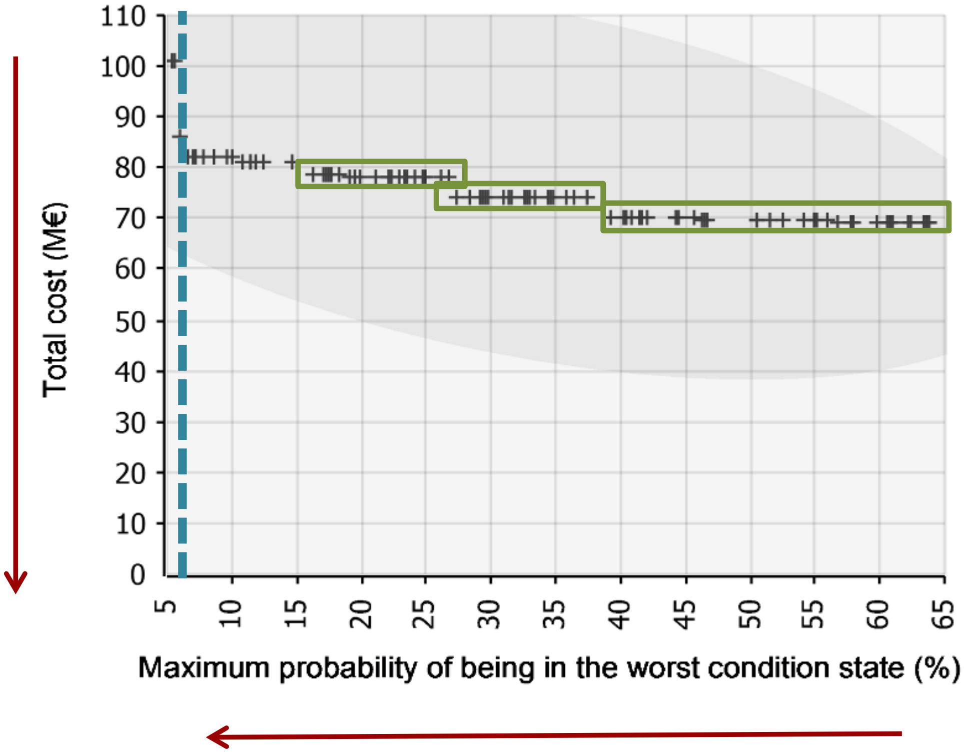
detect situations where a greater demand for performance requires a much larger investment

in some cost levels it is possible to reduce the risk almost without increasing the investment

RESULTS

multi-objective optimisation

Based on non dominated Pareto fronts



detect situations where a greater demand for performance requires a much larger investment

in some cost levels it is possible to reduce the risk almost without increasing the investment

CONCLUSIONS

The proposed life cycle cost estimation:

- May consider different kinds of interventions and the condition state predicted for the bridge at the time of its implementation
- Is made considering both direct and indirect costs
- Uses a residual cost value that showed to be important to ensure that the determined optimal plan of intervention is independent of the considered analysis period

The results for a set of Portuguese concrete bridges:

- Show that the considered indirect costs, related to the traffic restrictions, are little expressive in total costs because only repair works were planned, but that can change for bridges in the worst conditions

CONCLUSIONS

The presented methodology:

- Estimates the bridges' performance and costs over time considering different scenarios of intervention
- Is appropriate for schedule different kinds of actions in a set of bridges
  - in a medium or long time term
  - considering different kinds of restrictions (in performance and cost)
  - maximizing the performance level
  - minimizing the investment
- Allows to make a probabilistic estimation of the future needs in terms of intervention and budget
- Can be used by bridges administration to support their decisions under multiple points of view

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THANKS FOR YOUR ATTENTION!

Special acknowledgements:

FCT Fundação para a Ciência e a Tecnologia

Infraestruturas de Portugal

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

Bridge management practice & methodologies related to flooding hazards

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20^{th} – 21^{st} October 2016
Delft, Netherlands

Outline

- Consideration of a flooding threat in bridge management
- Vulnerability – the top level Performance Indicator for hazards
- Required quality levels
- Adequate actions for bridges vulnerable to flooding events
- Conclusions

Consideration of a threat of flooding in BM practice

- Sudden events i.e. non-interceptable processes
  - Bridges fail regardless of age, static system or materials
  - Adequate maintenance actions ? → Quality Control Plans
- Main topics in BM practice and literature (flooding events)
  - Previous failures
  - Exposure & Condition ratings from visual inspections
- Qualitative assessments (USA) to schedule specific plan of action
  - **N**ational **B**ridge **I**nventory Item No. 113 / Scour Critical bridges
  - Bridge sufficiency index
  - NYSDOT – Hydraulic Vulnerability Score

Consideration of a threat of flooding in BM practice

- **A few** software for risk analysis (HAZUS-MH, FEMA, FEDRO)
  - Exposure is considered (GIS)
  - Only direct costs/consequences are accounted
  - **Resistance of infrastructure is NOT considered** !

↓

Novel approaches are necessary

- Quantitative instead of qualitative
  - Performance Indicators

Risk, Vulnerability, Robustness, Resilience…

…so far only general guidance on application of these indicators

Consideration of a threat of flooding in BM practice

- **Q**uality **C**ontrol **P**lans of the **COST TU1406**
  - **W**ork **G**roup **3**, Task no. 4
    - ✓ Dynamics and uncertainty of sudden processes
    - ✓ Required quality levels (i.e. Performance Goals)
    - ✓ Triggering criteria (i.e. Thresholds) for inspections and maintenance

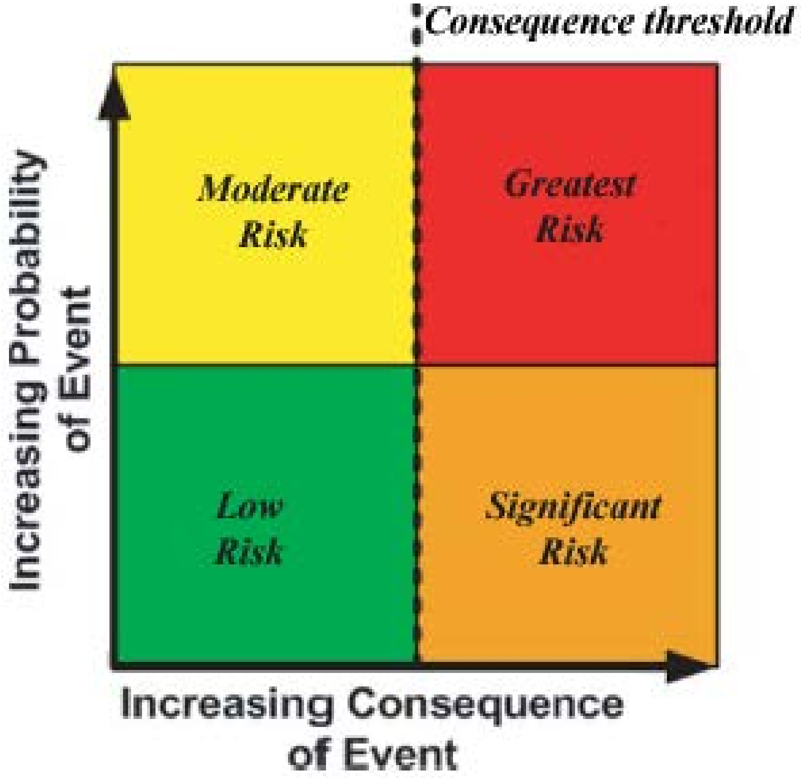
Waco, Texas, USA - June 1. 2016

- **Flooding hazard**
  - Washing away of access roads
  - Local scour at piers and abutments

…road will remain closed until the county bridge is repaired, <u>and there is no prediction as to when that might be</u>. (wacotrib.com)

Top Level Performance Indicator for hazards

- **Risk**
  - Qualitative approaches (e.g. Likelihood & Consequences Matrix)
  - Included only in several BMS !
  - Easy ranking?
  - How to evaluate:
    - *Likelihood* of event?
    - Consequences ?
  - Thresholds ?
  - Event =
    Hazard Scenario & Failure mode



Vulnerability – Top Level Performance Indicator for hazards

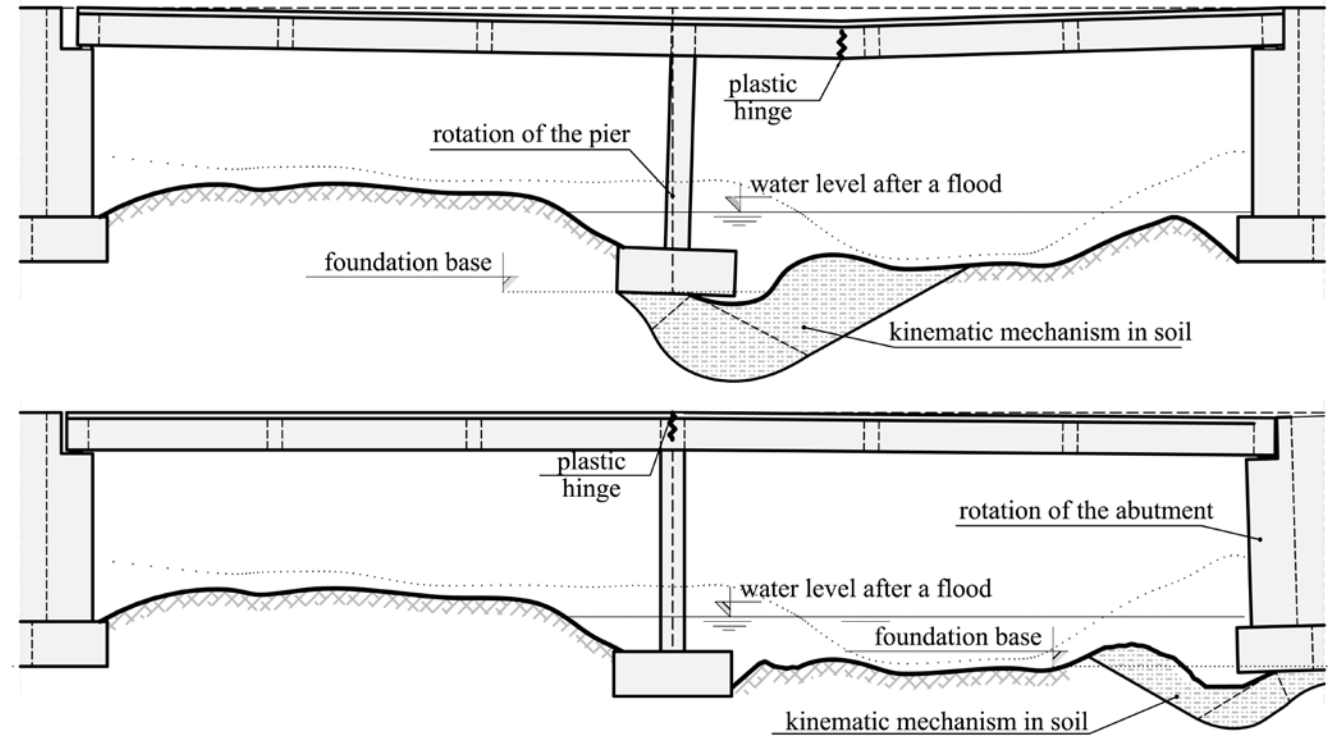
- **Vulnerability** is more convenient to use

$$V_n^s = P_n^s \cdot (DC_n + IC_n)$$

  - Related to a given hazard magnitude (e.g. 100-year flood)
  - Scenario assumed (e.g. local scour at a pier or abutment)
  - Failure modes (e.g. combined soil-bridge kinematic mechanism)
    - ✓ Resistance of the infrastructure can be accounted !
  - Total consequences (direct and indirect) are monetized

Vulnerability – Top Level Performance Indicator for hazards

- Methodology for **quantitative vulnerability assessment**
  - Conditional probability of a bridge failure
    - Flooding magnitudes and related local scour action
    - Combined soil-bridge failure modes



Required quality levels in respect to flooding hazard

- **Required quality levels = Performance Goals for safety and maintenance**
- **Set of thresholds** for pref. goals are defined for: a network, a road section and a single bridge
- **Thresholds are governed by stakeholders** (Users, Society, road Operators/Owners…)
  - no deaths/injuries occur in an extreme flooding event
  - a number or a percent of bridges must not fail in an extreme flooding event
- **Common ground - a monetized threshold:**
  - Vulnerability of a bridge failure to a specific flooding magnitude is limited

Required quality levels in respect to flooding hazard

- **HYRISK** approach (vulnerability & unknown foundations**):**
  - **M**in. **P**erf. **L**evels are not met – immediate foundation survey
    - **MPL** = annual probabilities of failure governed by the frequency of **observed** failures & overtopping in the light of NBI scores
  - Automated scour monitoring if:
    lifetime **risk of death >** cost of automated monitoring
  - Scour countermeasures if:
    lifetime **risk of failure >** estimated cost of countermeasures
- **Drawbacks**
  - Definition of MPL
  - Flooding magnitudes and soil resistance are not considered

Required quality levels in respect to flooding hazard

- **FHWA (H**ydraulic **E**ngineering **C**ircular**-18** & **HEC-23)**
  - Additional hydraulic analysis and/or countermeasures if: evaluated local scour depth > = foundation depth
- **Drawbacks**
  - Conservative local scour evaluation formulas
  - Bridge resistance is not considered

Vulnerability – the minimum set of information/findings

- **Hazard magnitude (flooding event)**
  - Monitoring of water levels
- **Failure mode analysis**
  - Failure scenario plausibility (e.g. abutment/pier exposure)
  - Soil & Bridge combined resistance to local scour in a flooding event

	SOIL	BRIDGE	
Reg./spec. inspection	Soli cover at pier	Bearings condition	**Regular inspection**
One-time inspection	Soil geotechnical properties	Detailing of joints	
	Soil erodibility	Observed damages	

- **Consequences**
  - Damage catalogues
  - Traffic simulations and monitoring

Adequate maintenance actions & mitigation procedures

- **Regular inspection and special inspections/monitoring**
  – data update
- **Soil works on river bank, channel and embankment**
  – alleviating the adverse impacts of a flooding event
- **Bearings, joint or member strengthening/repair/replacement**
  – bridge resistance to a failure mode is increased
- **Scour countermeasures**
  – eliminating the threat of failures due local scour at a pier/abutment
- **Scour monitoring**
  – timely warnings and closures

- **Thresholds of Vulnerability for these need to be distinguished !**

Conclusions

- Hazards are rarely considered in the Bridge Management Systems
- Knowledge on factual Risk/Vulnerability is necessary
- **WG 3** Aim – Performance Thresholds
- **<u>Questionnaire suggested</u>**
  - *Lesson learned*
    Reveal/confirm possible failure modes
  - *BM practice in Europe regarding hazards and climate change*
    What are the needs and shortcomings
  - *Evaluate Vulnerability*
    Possibility to apply quantitative assessments

Questions ?

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WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

ASSESSMENT OF BRIDGE PERFORMANCE BY LOAD TESTING AFTER RECONSTRUCTION

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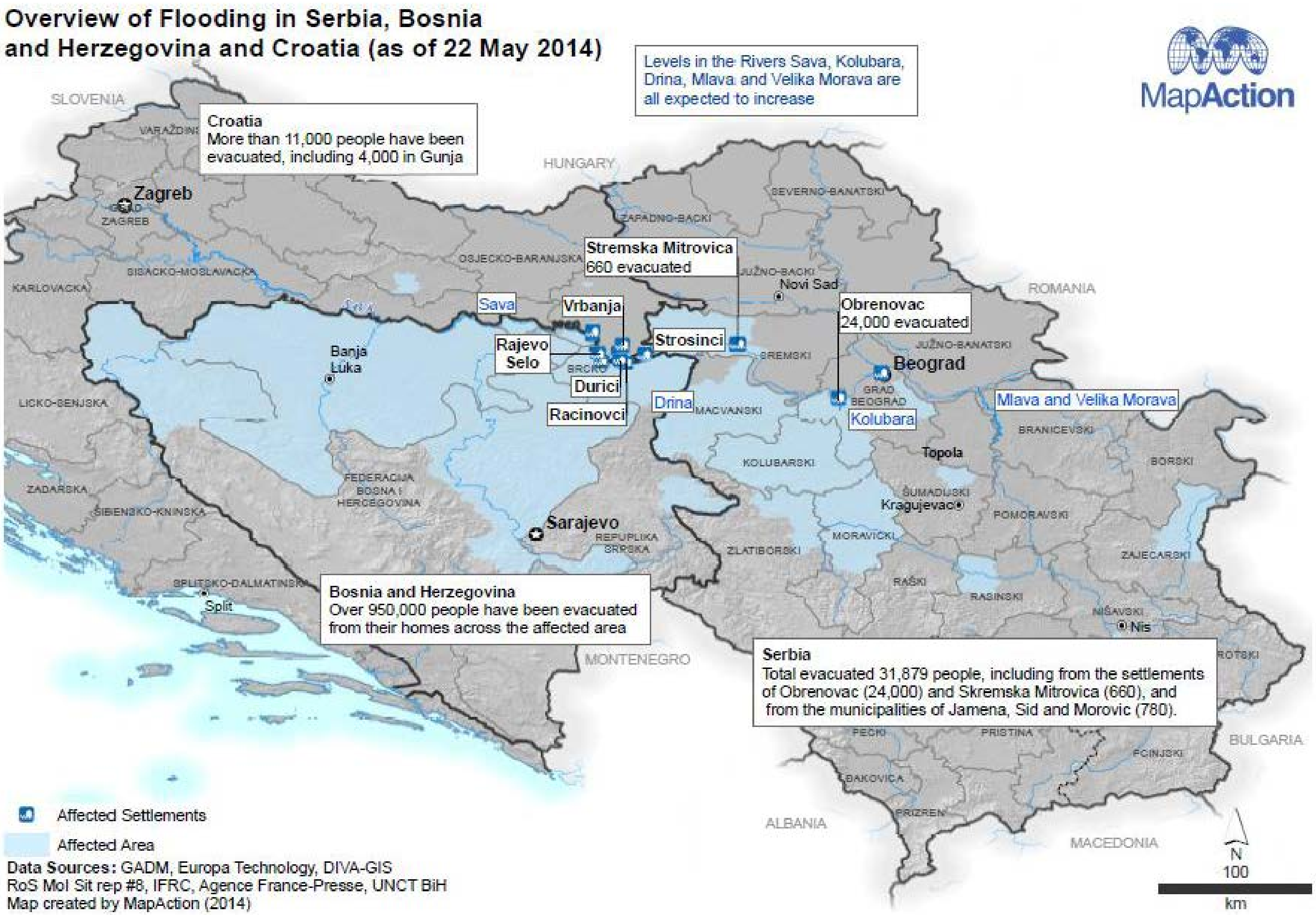
20th – 21st October 2016
Delft, Netherlands

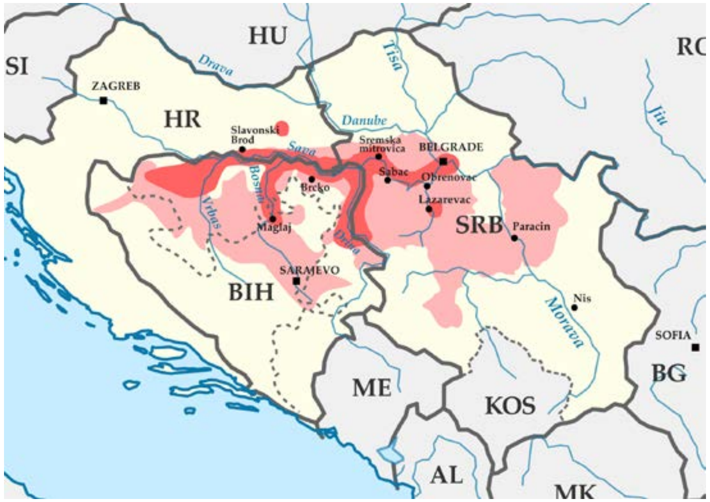
Introduction

- Extraordinarily heavy rains fell over Bosnia and Herzegovina during the third week of **13th -16th May 2014**, causing massive flooding in the northern, eastern and central parts of the territories bordering Croatia and Serbia.
- Three months' worth of rain fell in only three days; it is the heaviest rainfall in BiH since records began in 1894. – 120 years
- 250 (300) liters of rain/m^3
- In BiH, an estimated 1.5 million people were affected (39% of the population). The most affected areas are Bosanski Šamac, Odžak, Orašje, Doboj, Bijeljina, Brčko, Maglaj(rivers Bosna, Drina, Krivaja, Una, Sava, Sana, Vrbas)
- As a result many bridges were destroyed and they had to be rebuilt.

Introduction



Introduction



Introduction

Landslines

Introduction

Total bridge destruction-Zavidovići river Bosnia

The river Bosnia **threatens to inundate** a bridge in the town of Zavidovici.

Bridge Load Testing

- Nondestructive load testing:
  - optical surveying system
  - sensor dilatation measurement on steel and concrete parts of the structure
  - and deflection analysis by the use of Inductive Displacement Transducer
- The bridge is exposed to static and dynamic loading
- Load testing in Bosnia and Herzegovina is defined by (**BAS U.M1.046, 2005**)
- **Purpose:** evaluate structural response of bridges without causing damages
- **Testing procedure**: a clear program needs to be set with clear testing objectives and load configurations, selection and placement of instrumentation, analysis technique, evaluation and comparison of test results and analytical results

Bridge Load Testing- Static

- Placement of vehicles in certian locations- induce maximum effects
- The strength of construction elements is generally determined by placing strain or deflection-transducer gages at critical locations along the elements.
- Deflections measured at critical sections in order to obtain data on the actual behavior of the bridge structure.
- At the end the measured data (deformations, strains-calculated stresses) was compared with the analytical results and adequate conclusions and recommendations were given.

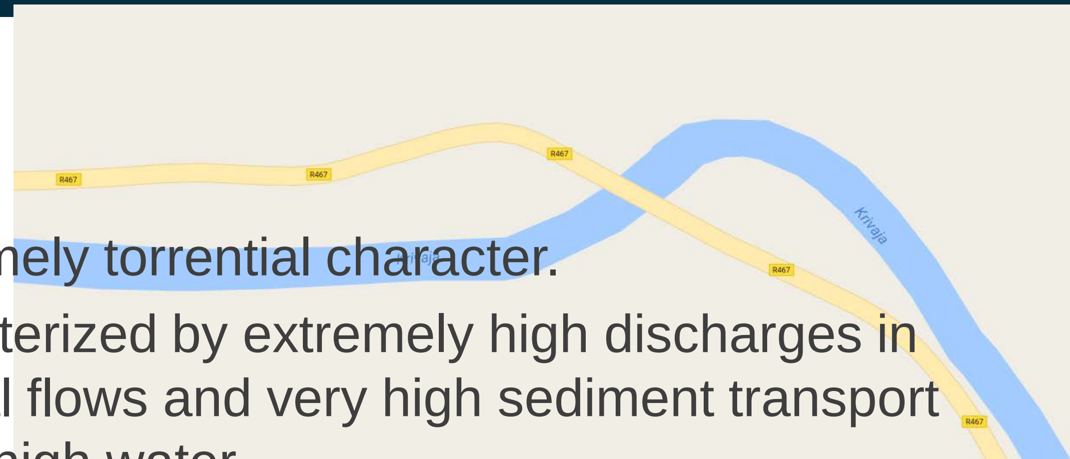
Bridge Load Testing- Dynamic

- The purpose of the dynamic load test is to determine the controlling parameters of the dynamic behavior of the bridges.
- The main dynamic characteristics of the structure are the fundamental vibration frequency, the dynamic amplification factor and the logarithmic decrement.
- Acceleration transducers were used in the dynamic test to measure the acceleration of the bridge.
- Dynamic load testing of the bridge was conducted by one truck passing over a plank of 5 cm thick at different speeds and in different locations
- A dynamic data acquisition system with integrated Fast-Fourier Transform (FFT) analyzer utilizing was used
- At the end the obtained data were compared with the analytical results and adequate conclusions and recommendations were given.

Case Study

- Bridge on the regional road Zavidovići-Olovo crossing over river Krivaja completely destroyed.
- During the floods the superstructure of the bridge was completely destroyed while the substructure was damaged.
- The most significant cause of flooding and damage to the piers was caused by wooden floating objects in the form of logs, branches and other form of floating debris which drastically reduced and jammed the entire river cross-section
- Width of the river bed at the location of the bridge is the smallest which caused blocking of the debris and dramatically increased destructive power of the flood.
- The transported wooden and other floating material during floods caused accumulation and destruction of the upper structure of the bridge.
- As the water levels continued to rise and flow over the bridge “overtopping” was evident and at the end due to the combined effect of water flow and debris the super structure of the bridge was completely destroyed.
- As some sediment was removed around bridge piers there was a fear of sour formation.
- In that respect the piers of the bridge were strengthened while the upper structure was completely rebuilt.

Case Study



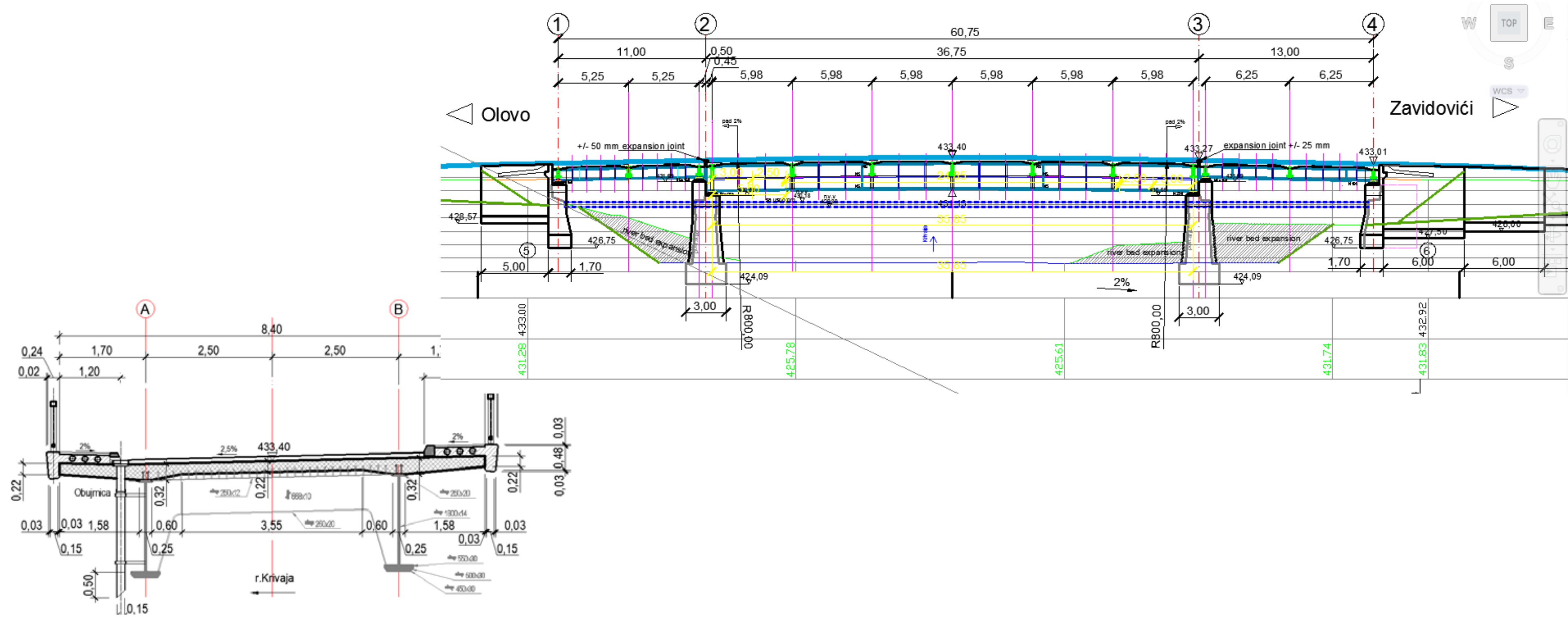
- River Krivaja is of an extremely torrential character.
- Torrential floods are characterized by extremely high discharges in comparison to mean annual flows and very high sediment transport rates and short duration of high water.

Case Study

- River Krivaja is of an extremely torrential character.
- Torrential floods are characterized by extremely high discharges in comparison to mean annual flows and very high sediment transport rates and short duration of high water.

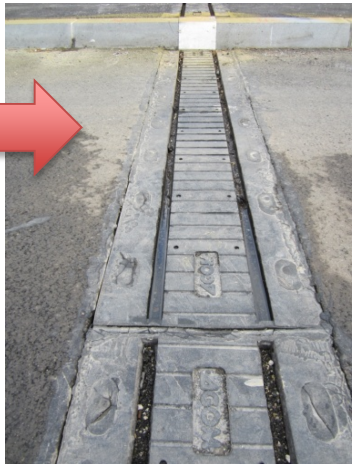
Case Study

- Steel-concrete composite bridge-two-girders
- Three single spans (11,00+36,75+13,00)
- Total length of the bridge 60,75m



1.Visual inspection

- Several defects were observed:
  - Damage of expansion device



  - Significant leakage of water
  - Caused corrosion of the bearings

1.Visual inspection

- Several defects were observed-cont.:
  - direct contact of the steel girder with the back abutment wall

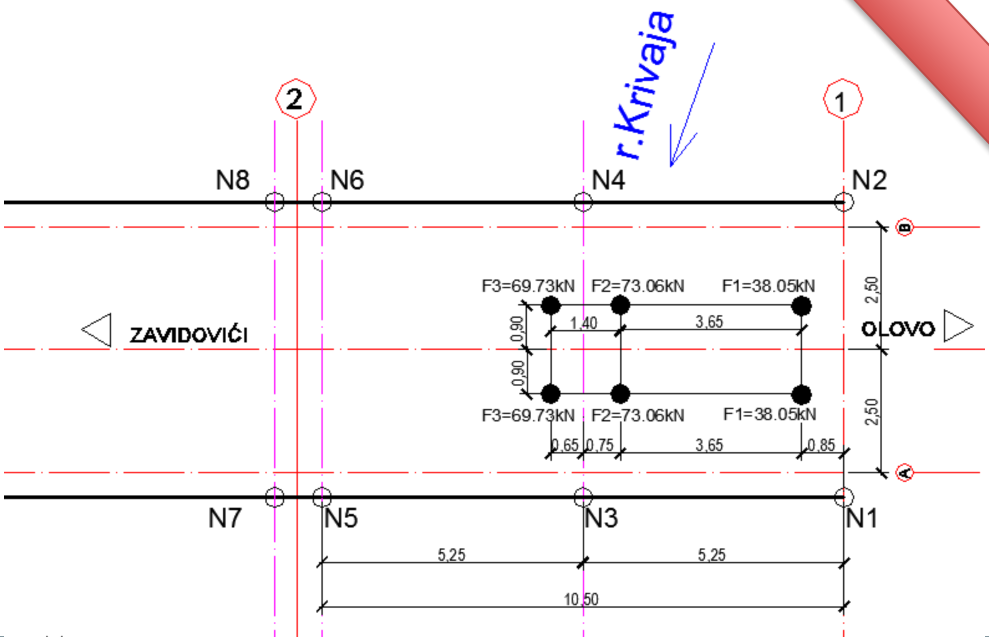
2.Static testing

- Symmetric and asymmetric

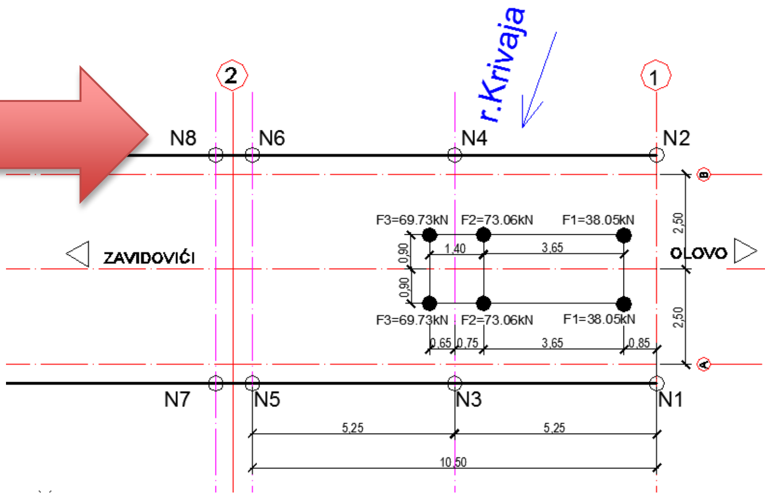
- First span-11,00m
  - symmetric loading in the first span:

2.Static testing

- First span-11,00 m
  - symmetric loading in the first span:
    - the deflections: N3≠N4 - PROBLEM
    - Uplift of N2=2cm -PROBLEM



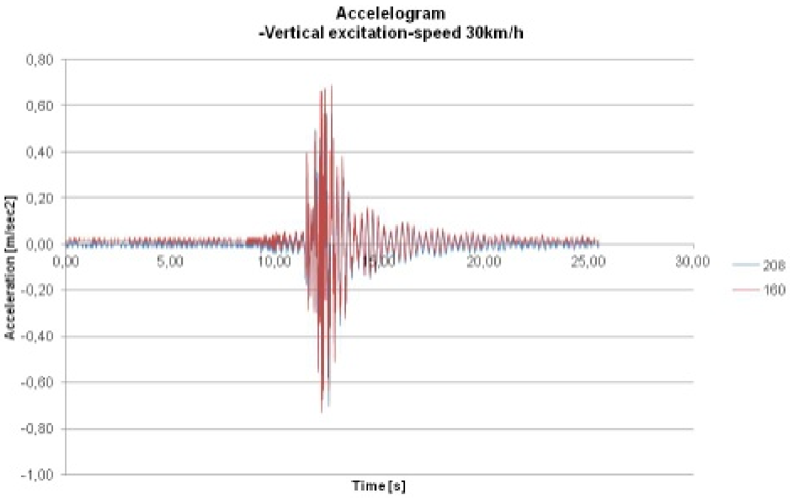
2.Static testing-cont

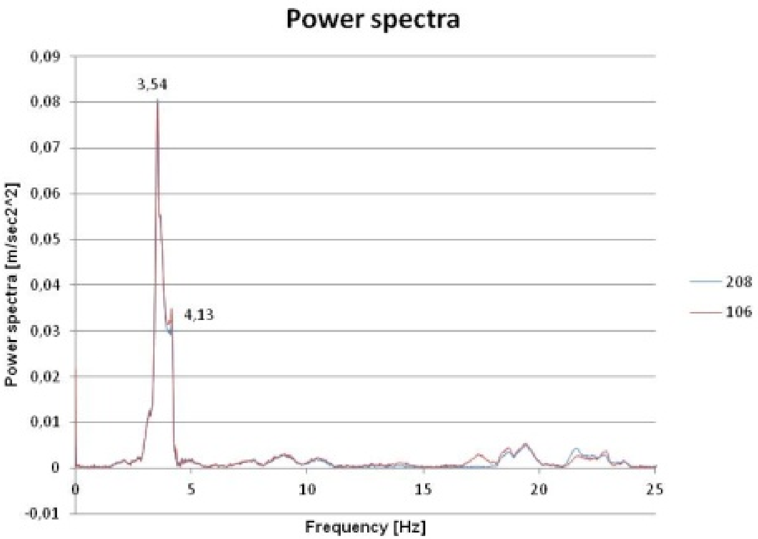


- Second span-36,75 m
  - N8 = 3.8 cm uplift -PROBLEM

3.Dynamic testing

- Truck passing over a plank of 5cm thick





3.Dynamic testing

- The concept of a dynamic amplification factor (DAF) is used to describe the ratio between the maximum load effect when a bridge is loaded dynamically, and the maximum load effect when the same load is applied statically to the bridge.

$$k_d = \frac{f_{dyn}}{f_{stat}}$$

- Average measured value of the DAF amounts to **1.095**, while the calculated value is **1.11**, which is more than satisfactory.

4.Conclusion

- Bad construction of some elements clear discrepancies were notices in respect to the numerical calculations of the structure.
- Unexpected movement of the bearing points- possible causes of such damage
- It was stated that the bridge “partially” satisfies the requirements of the enforced rules and standards behaves BAS U.M1.046.
- Bearing resistance of the bridge to take over the foreseen traffic loads is satisfied; however this cannot be stated for its durability.
- Suggestions were given to the Client regarding expansion joints, bearings and direct contact between the steel girder and the abutment wall.
- Clear connections between performance indicators and damaging effects.

TU1406

COST ACTION

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TU1406
COST ACTION

WG2 and WG3 WORKSHOP

Bridge performance goals and quality control plans

INTERCEPTABLE DECAYING PROCESSES IN ARCH BRIDGES

João Amado - Infraestruturas de Portugal, Lisbon

Infraestruturas de Portugal

20th – 21st October 2016
Delft, Netherlands

Contents

Introduction

- 1st WG3 meeting, held in Belgrade, agreed on the following framework:

Interceptable (observable) processes	Damaging processes	Arch bridges	Girder bridges	Frame bridges
	Demand			
Non-interceptable processes	Sudden events	*Common framework*		
	Non-observable			

- Literature survey based on the works of the International Union of Railways:
  - Catalogue of damages for masonry arch bridges (2006);
  - Recommendations for the inspection, assessment and maintenance of masonry arch bridges (2011)

- Guide for the Assessment of Masonry Bridges, presented in TU1406 workshop held in Belgrade

Deterioration processes in masonry arch bridges

Deterioration processes in masonry arch bridges according to UIC (2006)
1) Mechanical erosive action of water
2) Superstructure problems arising from foundation problems (foundation movement)
3) Problems caused by resistant behavior of skewed bridges
4) Problems caused by dynamic behavior
5) Problems caused by rotation of abutment and wing walls due to excessive earth pressure
6) Problems arising from the use of the structure
7) Problems caused by earth pressure on spandrels
8) Problems caused by differences in relative stiffness between elements
9) Problems caused by the construction process
10) Problems arising from previous interventions
11) Changes in the durability without change in the chemical composition of the material
12) Changes in durability due to changes to the chemical composition of the material

Deterioration processes according to UIC, 2006

1) Mechanical erosive action of water, **which leads to the following problems:**

- Mechanical degradation of foundation
- Loss of scour protection

- Chemical degradation by water containing sulphate or other salts

Deterioration processes according to UIC, 2006

2) Superstructure problems arising from foundation problems:

- Differential transverse settlement of pier or abutment

- Longitudinal rotation of pier or abutment

Deterioration processes according to UIC, 2006

***2) Superstructure problems arising from foundation problems*:**

- Longitudinal settlement and longitudinal rotation of piers or abutment

Deterioration processes according to UIC, 2006

2) Superstructure problems arising from foundation problems:

- Transverse rotation along the longitudinal axis of a pier

- Relative movement between ends and center of piers or abutment
- Differential longitudinal settlement between springing and arch ring
- Longitudinal rotation of pier or abutment

Deterioration processes according to UIC, 2006

3) Problems caused by resistant behavior of skewed bridges

Deterioration processes according to UIC, 2006

5) Problems caused by rotation of abutment and wing walls due to excessive earth pressure

Deterioration processes according to UIC, 2006

6) Problems arising from the use of the structure

Deterioration processes according to UIC, 2006

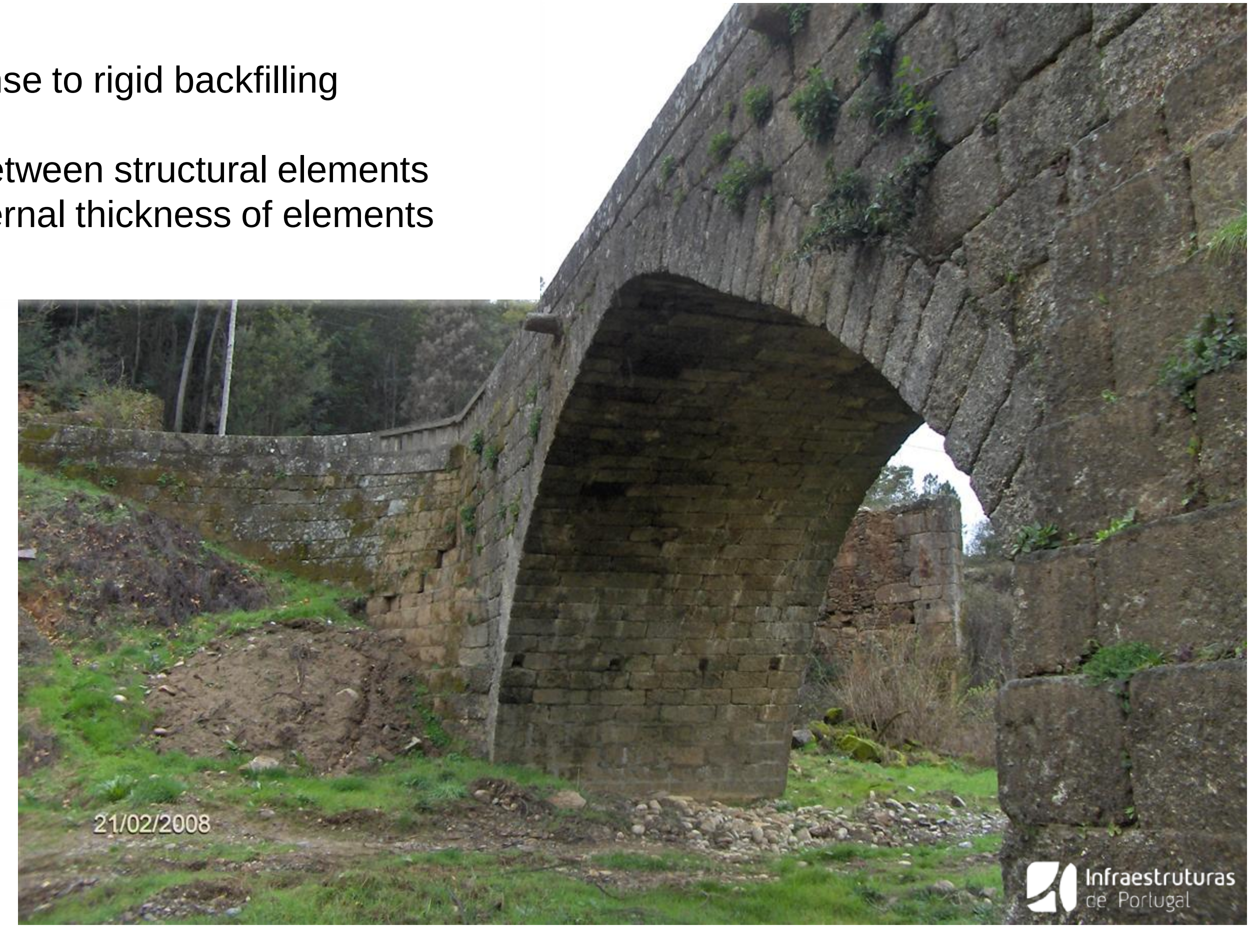
7) Problems caused by earth pressure on spandrels

Deterioration processes according to UIC, 2006

8) Problems caused by differences in relative stiffness between elements

9) Problems caused by the construction process, which can be related to:

- Differential response to rigid backfilling
- Waterproofing
- Different bonds between structural elements
- Change in the internal thickness of elements



- Three-hinges mechanism due to the removal of centering following construction

10) Problems arising from previous interventions:

- Intrados (shotcrete or concrete coating)

- Use of incompatible materials

- Extensions or repairs which modified the management of water
- Changes which affected the permeability of the masonry

11) Changes in the durability without change in the chemical composition of the material:

- Mechanical weathering by water

- Damages caused by vegetation

- Other mechanical weathering due to thermal variations, crystallization of salt, water expansion, wind and the freeze – thaw cycle.

12) Changes in durability due to changes to the chemical composition of the material:

- Erosion caused by salts
- Erosion caused by decomposition of carbonates
- Erosion caused by levigation of clay

Categorization of the interceptable processes

Effects on structures and triggering criteria

General causes of defects and their effect on the structure (Aníbal Costa et al.)

Triggering criteria for *detailed investigations* or *maintenance actions* (?)

Triggering criteria for *detailed investigations* or *maintenance actions* (?)

References

José Matos et al, editors. E-book of the 2nd Workshop Meeting of the COST Action TU 1406. Faculty of Civil Engineering, University of Belgrade, Serbia, 2016.
Minutes of the 1st WG3 Meeting – COST TU 1406. Belgrade, Serbia, 2016.

Rafael García-Catalán and Jose Álamo. Catalogue of damages for masonry arch bridges. Technical Report WP2/3 Final Draft, International Union of Railways – UIC, 2006.

UIC. Recommendations for the inspection, assessment and maintenance of masonry arch bridges, 2nd edition, Code 778-32011, International Union of Railways – UIC, 2011.

Aníbal Costa, Hugo Perneta, Cristina Costa, António Arêde, Humberto Varum, Guide for the Assessment of Masonry Bridges. *Infraestruturas de Portugal*, forthcoming.

THANKS FOR YOUR ATTENTION!

WWW.TU1406.EU

ZAGREB JOINT WORKSHOP

http://www.grad.unizg.hr/joint-zagreb-workshop

Ana Mandić Ivanković – Faculty of Civili Engineering, University of Zagreb

20th – 21st October 2016
Delft, Netherlands

ZAGREB JOINT WORKSHOP

- Title: **The Value of Structural Health Monitoring for the Reliable Bridge Management**
- Date: **2-3 (+4) March 2017**
- Venue: **Faculty of Civil Engineering, University of Zagreb**

ZAGREB JOINT WORKSHOP

- Organisers:
  COST 1402 - Quantifying the value of structural health monitoring
  COST 1406 - Quality specifications for roadway bridges, standardization at EU level
  IABSE WC1 - Structural Performance, Safety and Analysis

TU1406 COST ACTION

IABSE

TU1402 COST ACTION

DECISION THEORY
VALUE OF STRUCTURAL HEALTH MONITORING
SHM STRATEGIES
STRUCTURAL PERFORMANCE

CALL FOR PAPERS

- 3 topics
- Deadline : **November 20, 2016.**
- Submission: Manuscripts should be developed with the **template** available at the web page http://www.grad.unizg.hr/joint-zagreb-workshop and submitted to joint-zagreb-workshop@grad.hr. Please indicate **corresponding author** and adequate **topic** for your paper.

CALL FOR PAPERS

- 1st topic **Performance assessment of existing bridges for their reliable management** *(COST Action TU 1406)*

This session will incorporate the main achievements from COST Action TU 1406, with a focus on performance assessment of existing bridges, the establishment of performance goals and the development of quality control plans. The aim is also to integrate such topics into existing deterministic and probabilistic-based frameworks for the efficient management of a single and a network of bridges.

CALL FOR PAPERS

- 2nd topic **Framework, Strategies and Tools towards the Quantification of the Value of Structural Health Moni**toring *(COST TU 1402)*

Contributions of high relevance for the quantification of the Value of Structural Health Monitoring in the fields of structural performance and integrity management modelling, SHM strategies and methods and tools are very welcome. The contributions should also provide ideas of how to cooperate with COST TU1406 activities and key performance indicators.

CALL FOR PAPERS

- 3rd topic **Management and Performance-assessment of Existing Structures (IABSE WC1)**

Presentation of contributions related to improved existing performance assessment methods within bridge management systems - for example linear and nonlinear analysis methods, semi and full probabilistic based assessment methods, treatment of uncertainties, Bayesian updating. The presentations can serve as basis for creating procedures applicable in practice based on recent scientific achievements.

PROGRAMME

Thursday, March 2nd, 2017	
08:00 – 08:30	*Registration for meetings & Joint workshop*
08:30 – 09:00	Opening of Meetings and Joint Workshop - Welcome speeches/presentation
09:00 – 12:20	**Session 1: COST Action TU 1406 - Performance assessment of existing Bridges for their reliable management**
09:00 – 09:30	Keynote presentation 1
09:30 – 10:10	Presentations (2*20min)
10:10 – 10:30	*Coffee break*
10:30 – 12:20	Presentations (5*20+10 min)
12:30 – 13:30	*Lunch*
13:30 – 16:50	**Session 2: COST TU1402 Framework, Strategies and Tools towards the Quantification of the Value of Structural Health Monitoring**
13:30 – 14:00	Keynote presentation 2
14:00 – 15:00	Presentations (3*20min)
15:00 – 15:20	*Coffee break*
15:20 – 16:50	Presentations (4*20+10 min)
17:00 – 19:00	Technical visit in Zagreb (Sava bridges)
19:30 – 22:00	*Networking Dinner*

Friday, March 3rd, 2017	
08:00 – 08:30	*Registration for meetings & Joint workshop*
08:30 – 12:00	**Session 3: IABSE WC1 - Management and Performance-assessment of Existing Structures**
08:30 – 09:00	Keynote presentation 3
09:00 – 10:00	Presentations (3*20min)
10:00 – 10:20	*Coffee break*
10:20 – 12:00	Presentations (4*20+10 min)
12:00 – 12:20	*Coffee break*
12:20 – 13:20	**Closing session**
12:20 – 13:10	Discussions, conclusions, further progress
13:10 – 13:20	Closing speech
13:30 – 14:30	*Lunch*
14:30 – 15:30	Steering committee meeting of TU 1402, TU 1406 and IABSE
15:30 – 16:45	TU 1406 WG2+WG3 Meeting (75') / TU 1402 MC Meeting (60') / IABSE WC1 Meeting (75')
16:45 – 17:15	*Coffee break*
17:15 – 18:30	TU 1406 WG4+WG5 Meeting (75')
18:30 – 19:15	TU 1406 MC Meeting (45')
19:30 – 22:00	*Social programme*

Saturday, March 4th	
	Technical visit outside Zagreb (if enough people will show interest)

COMMITTEES

Scientific Comittee

Marios Chryssanthopoulos
Rade Hajdin
Niels Peter Hoj
Ana Mandić Ivanković
Jose Matos
Irina Stipanović
Daniel Straub
Sebastian Thöns

Organising Committee

Niels Peter Höj
Ana Mandić Ivanković
Jose Matos
Sebastian Thön

Local Organisers

Ana Mandic Ivankovic
Alex Kindj
Marija Kušter Marić
Dominik Skokandic
Mladen Srbic
Anđelko Vlašić

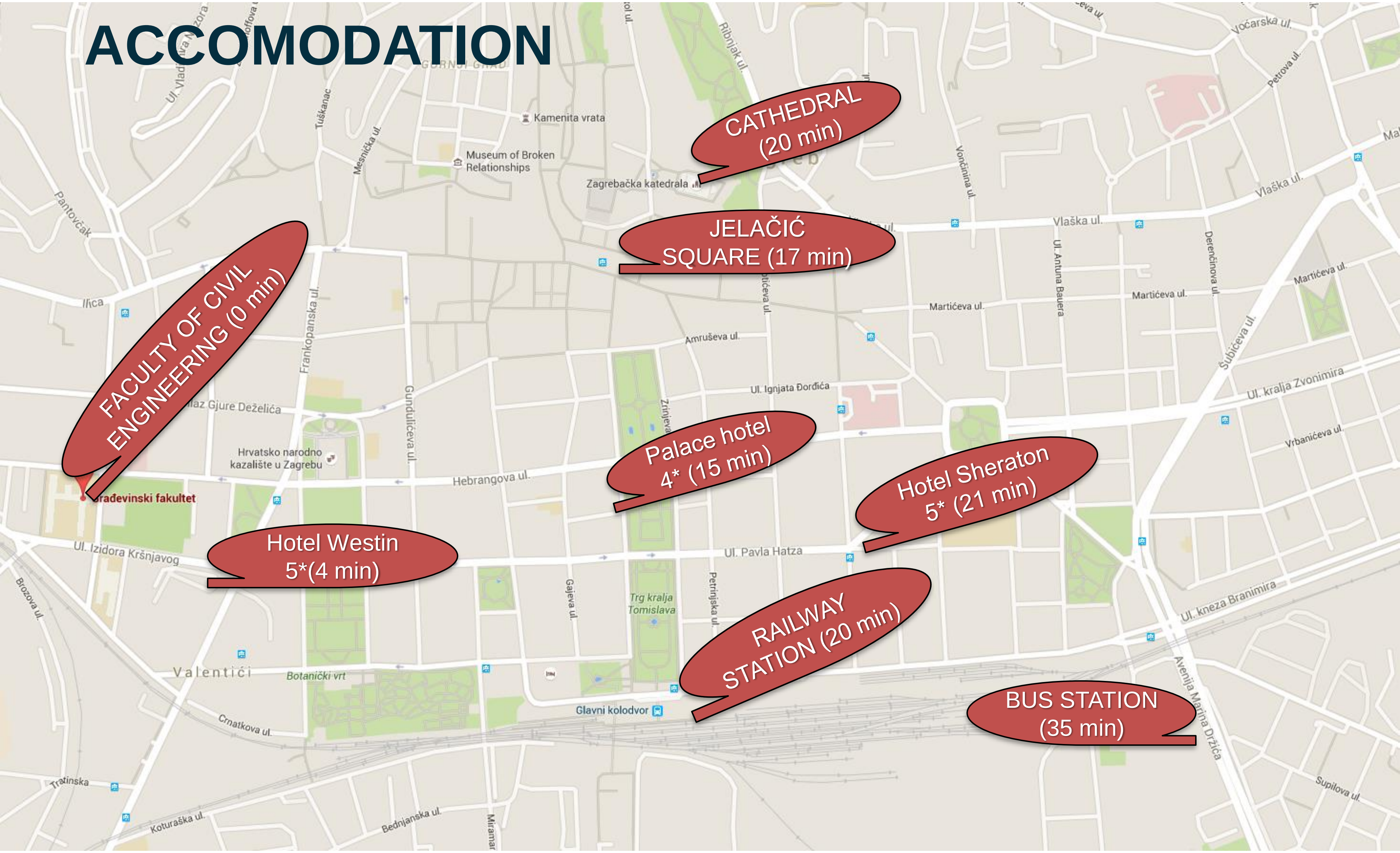
REGISTRATION

- Fees: Meal related fee of **80 Euros** (covering eligible expense for two lunches and two dinners) will be collected at the registration desk. For COST Action participants invited to the workshop, these expenses will be reimbursed within travel reimbursement.

- Form: To register for this event it is necessary to fill up and submit the **form** available at the web page. Invited COST Action members, in addition to submitting this form, need to respond positively to the **e-COST official invitation** to complete their registration.

- Deadline: **December 20th, 2016.**

ACCOMODATION



TECHNICAL VISITS

Sava bridges:	Thursday, 17-19h
Krk bridges (+ Arena?):	Saturday, 9 – 17h

SOCIAL EVENTS

Networking dinner:	Thursday, 19:30-22h
Social event:	Friday, 19:30 – 22h

Welcome to Zagreb!

http://www.grad.unizg.hr/joint-zagreb-workshop

THANK YOU FOR YOUR ATTENTION!

TU1406
COST ACTION

Categorization of indicators / findings
WG 1

A. Strauss, A. Mandic Ivankovic, Ivan Zambon, Lisa Mold

ESF provides the COST Office through a European Commission contract

COST is supported by the EU Framework Programme

COST EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

Main objective is to develop a **guideline** for the establishment of **Quality control** plans in **roadway bridges** at a **European level** and moreover it will also be analyzed the **possibility of incorporation new indicators** related to sustainable performance of roadway bridges.

1st Survey phase

- Questionnaire associated with predefined performance indicator
- Performance indicators indicated in selected attached documents

2nd Survey phase

Nomination of MC members for:

- contacting roadway owners and operators
- uploading I-DOC, E-DOC, B-DOC

Nomination of operating persons per country for:

- processing national documents according to guidelines
- screening I-DOC, E-DOC, B-DOC

Core group WG1

- processed documents
- transferring to Database

Core group WG1-3

- analyzing PI database

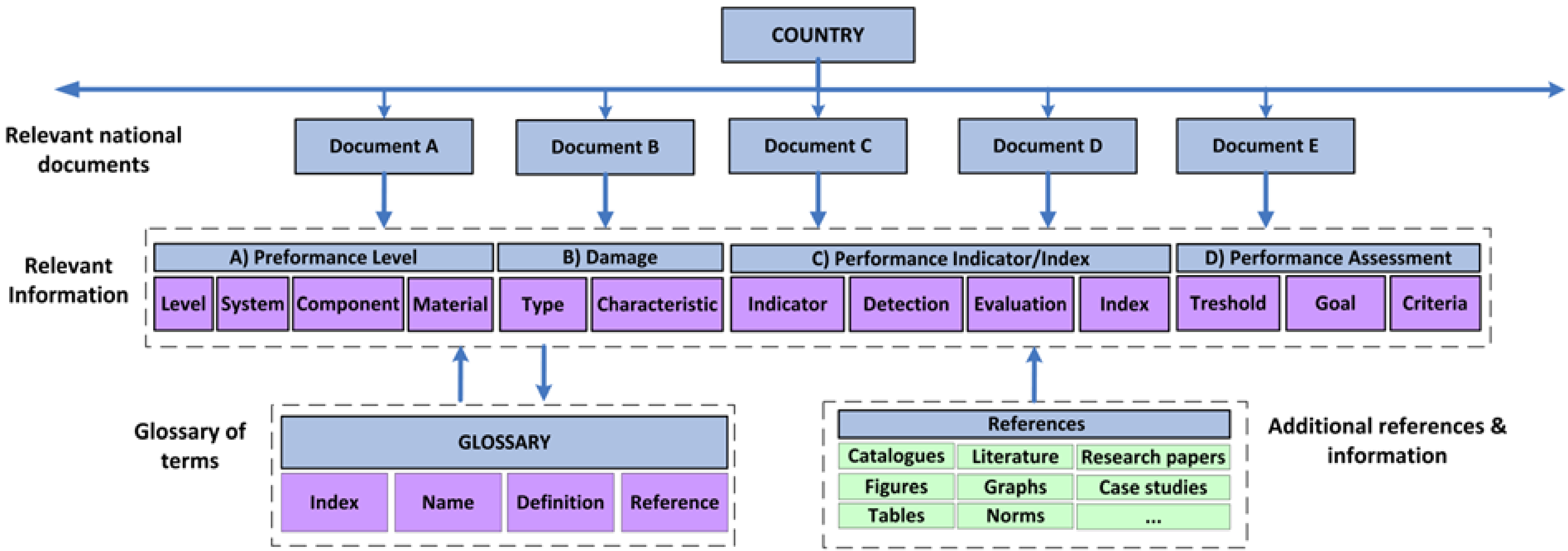
Geneva → **January** → **Belgrade**

- Publishing official COST e-Book WG1 activities
- Preparation of WG1 activity and endreport

Call to researchers for:

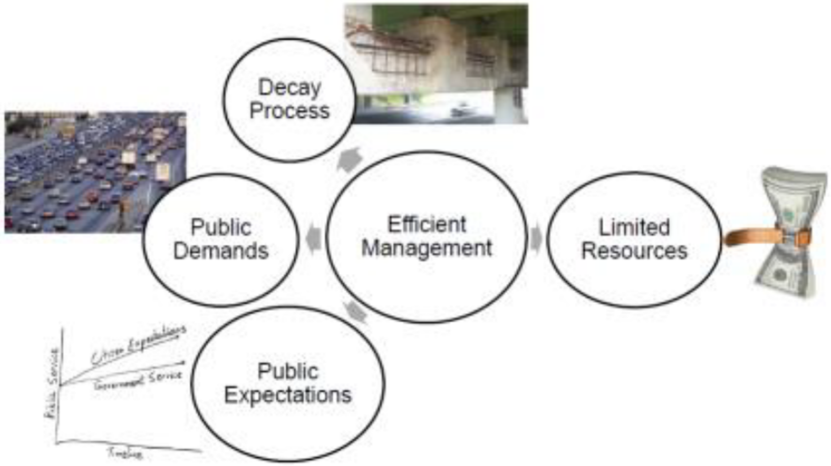
- uploading performance associated documents
- transferring documents to Database
- analyzing PI - Database

Elements and coarse structure of database



DEFINITION OF PERFORMANCE INDICATOR

- Parameter **measurable** and **quantifiable** related to the bridge performance that can
  be compared with a **target measure** of a **performance goal** or can be used for **ranking** purposes among a bridge population
  in the framework of a **Quality Control Plan** or
  life-cycle management (**decisions, actions involving economic resources**)



Screening results *operator* documents / *database*

List of screened documents

SURVEY OF PERFORMANCE INDICATORS

Country	Croatia	New Document

num	Responsible Person	Document	Doc. Type	Author	Year
1	Ana Mandić Ivanković	*Handbook of damages on bridge elements*	Evaluation	Hrvatske ceste d.o.o., dr.sc. Danijel Tenžera	2014
2	Ana Mandić Ivanković	*Guidelines for bridge inspections*	Inspection	Hrvatske ceste d.o.o.	2014
3	Ana Mandić Ivanković	*HRMOS manual – Bridge management*	Inspection	Hrvatske ceste d.o.o.	1999
4	Dominik Skokandić	*HRMOS manual – Bridge management – General bridge inspection*	Inspection	Hrvatske ceste d.o.o.	1999
5	Dominik Skokandić	*Handbook of damages on bridges*	Inspection/ evaluation	Hrvatske Autocesete d.o.o.	2010
6	Ana Mandić Ivanković	*Guideline for bridge evaluation*	Evaluation	Hrvatske Autocesete d.o.o.	2010
7	Ana Mandić Ivanković	*Bridge Management Planning*	Background document	Hrvatske Autocesete d.o.o.	2008
8					
9					
10					
11					
12					

Blank | Names_Table | GeneralData | Cro_1 | Cro_2 | Cro_3 | Cro_4 | Cro_5 | Cro_6 | Cro_7

READY

Croatian example

Handbook of damages on bridge elements

C) Performance Indicator/Index				D) Performance Assessment		
indicator	detection	evaluation	index	threshold	goal	criteria
Damage degree	Direct_Measurement			affected area	Damage Assessment	
Damage degree	Direct_Measurement			crack width (	Damage Assessment	

							C) Performance Indicator/Index				D) Performance Assessment		
							indicator	detection	evaluation	index	threshold	goal	criteria
							Damage degree	Direct_Measurement			affected area	Damage Assessment	
							Damage degree	Direct_Measurement			affected area	Damage Assessment	
17		types					Damage degree	Direct_Measurement			affected dep	Damage Assessment	
18	Element	All bridge types	Foundations		Damage_State	Settlements	Damage degree	Direct_Measurement			sag (cm)	Damage Assessment	
19	Element	All bridge types	Foundations		Damage_State	Degradation	Damage degree	Direct_Measurement			affected area	Damage Assessment	
20	Element	All bridge types	Foundations	Concrete	Damage_State	Spalling	Damage degree	Direct_Measurement			affected area	Damage Assessment	
	Element	All bridge	Foundations	Concrete	Damage_State	Spalling	Damage degree	Direct_Measurement			affected dep	Damage	

Blank | Names_Table | GeneralData | Cro_1 | Cro_2 | Cro_3 | Cro_4 | Cro_5 | Cro_6 | Cro_7

READY

Screening results *research* documents / *database*

Austrian example

SURVEY OF RESEARCH PERFORMANCE INDICATORS

Article	**Performance assessment of concrete s**
Author	Strauss, Zan
Year	
Abstract	An efficient evaluation and prediction of fundamental requirement for life-cycle a structures. Important tools and valuable methods. Unfortunately, due to their pra information gathered with inspection and monitoring methods need to be used in the most effective manner possible. The aim of this contribution is to present a framework performance indicators of concrete structures prone to fatigue, A theoretical background with selected indicators is presented t methods including inspection and monitoring information with i
Journal	IABSE Conference – Structural Engineering: Providing Solutions Geneva, Switzerlan
Keywords	life-cycle analysis; performance indicators; probabilistic perf

Performance Indicator	**Young modulus**
Type of Indicator	Material property
Mathematical Formulation	
Threshold	
Intentions (where to apply)	In order to evaluate the fatigue performance of the critical cross
Level of maturity	Research stage
Case study	STRABAG test foundation in Cuxhaven
Performance Indicator	**Reliability index**
Type of Indicator	Reliability
Mathematical Formulation	
Threshold	
Intentions (where to apply)	In order to evaluate the fatigue performance of the critical cross
Level of maturity	Research stage
Case study	STRABAG test foundation in Cuxhaven

References
[1] Zhao, Y.-G., Zhong, W.-Q., Ang, A.H.-S., 2007. Estimating joint failure probability of series structural systems. J. Eng. Mech. 133, 588–596.
[2] Strauss A, Vidovic A, Zambon I, Grossberger H, Bergmeister K. Monitoring information and probabilistic based prediction models for the
[3] Mark, P., Stangenberg, F., Bergmeister, K., Strauss, A., Ahrens, M.A., 2013. Lebensdauerorientierter Entwurf, Konstruktion, Nachrechnung

SURVEY OF PERFORMANCE INDICATORS

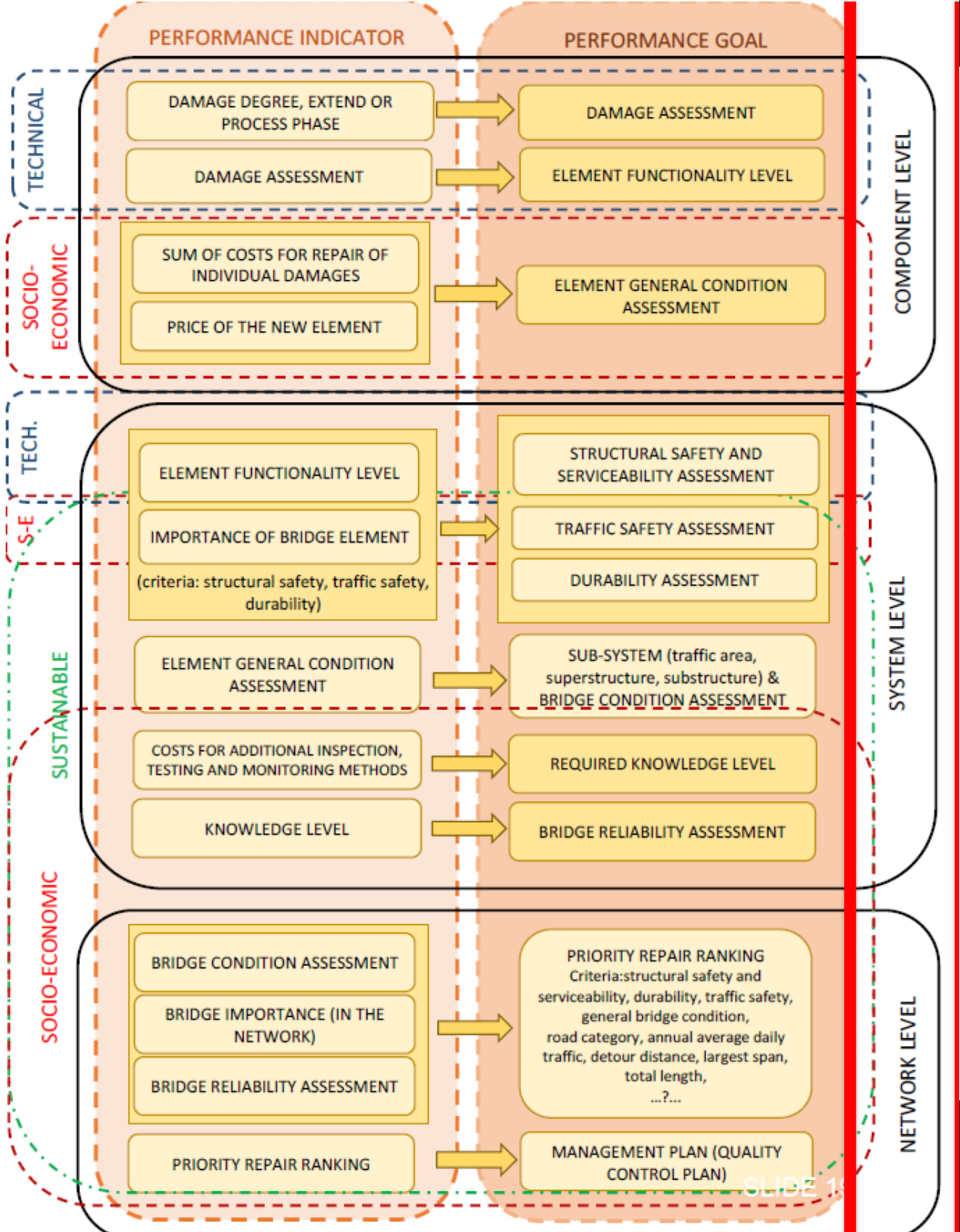
Country	Austria

Add Article

num	Responsible Person	Article	Author	Year
1	Ivan Zambon	*Performance assessment of concrete structures based on probabilistic prediction models and monitoring information*	Strauss, Zambon, Vidovic, Grossberger, Bergmeister	2015
2				
3				
4				

Indicators and goals

- Interactions between KPI and PG are contemplated, as they are crucial for optimal quality control and management of road bridges



SURVEYING → CLUSTERING → HOMOGENISATION

A) Performance Level				B) Damage		C) Performance Indicator/Index		D) Performance Assessment	
level	system	Compo-nent	material	type	characteristic	indicator	detection	threshold	goal
Sub_ System	All bridge types	Super Structure	Concrete	Damage_ State	Cracks	Damage degree	Direct_ Measurement	crack width (mm)	Damage Assessment
Sub_ System	All bridge types	Super Structure	Concrete	Damage_ State	Honeycombing	Damage degree	Direct_ Measurement	affected area (m2)	Damage Assessment
Sub_ System	All bridge types	Super Structure		Damage_ State	Freeze-thaw	Damage degree	Direct_ Measurement	affected area (m2)	Damage Assessment
Sub_ System	All bridge types	Super Structure	Brick	Damage_ State	Disintegration of mortar	Damage	Visual_ Inspection		Damage Assessment
Sub_ System	All bridge types	Railings	Steel	Damage_ State	Missing parts	Damage degree	Visual_ Inspection		Damage Assessment

→ DEFECTS
Crack width

→ MATERIAL PROPERTIES
bad concrete compaction

→ ENVIRONMENTAL BASED
freeze-thaw

→ STRUCTURAL INTEGRITY & JOINTS
disintegration of mortar

→ EQUIPMENT AND PROTECTION
absence of equipment component

SURVEYING → CLUSTERING → HOMOGENISATION

A) Performance Level				B) Damage		C) Performance Indicator/Index		D) Performance Assessment	
level	system	Compo-nent	material	type	characteristic	indicator	detection	threshold	goal
System	All bridge types			Damage_ State	Buckling	Damage degree	Visual_ Inspection		Damage Assessment
System	All bridge types		Concrete	Damage_ State	Execution defects	Damage degree	Direct_ Measurement	affected area (m2)	Damage Assessment
Element				Damaging_ Process	low demage degree (first phase)	Damage degree	Visual_ Inspection	Upper limit + Duration of damage phase	Damage Assessment
Element						importance of bridge element		Quantitative scale of values	Element importance assessment

→ GEOMETRY CHANGES
Buckling

→ ORIGINAL CONSTRUCTION & DESIGN
execution/construction defects

→ RATING
damage degree
+damage evolution

→ COST & IMPORTANCE
importance of bridge element

BASE

Estonia
- List of documents
- Homogenized database

Finland
- List of documents
- Non-homogenized database

France
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Greece
- List of documents
- Homogenized database

ESF provides the COST Office through a European Commission contract

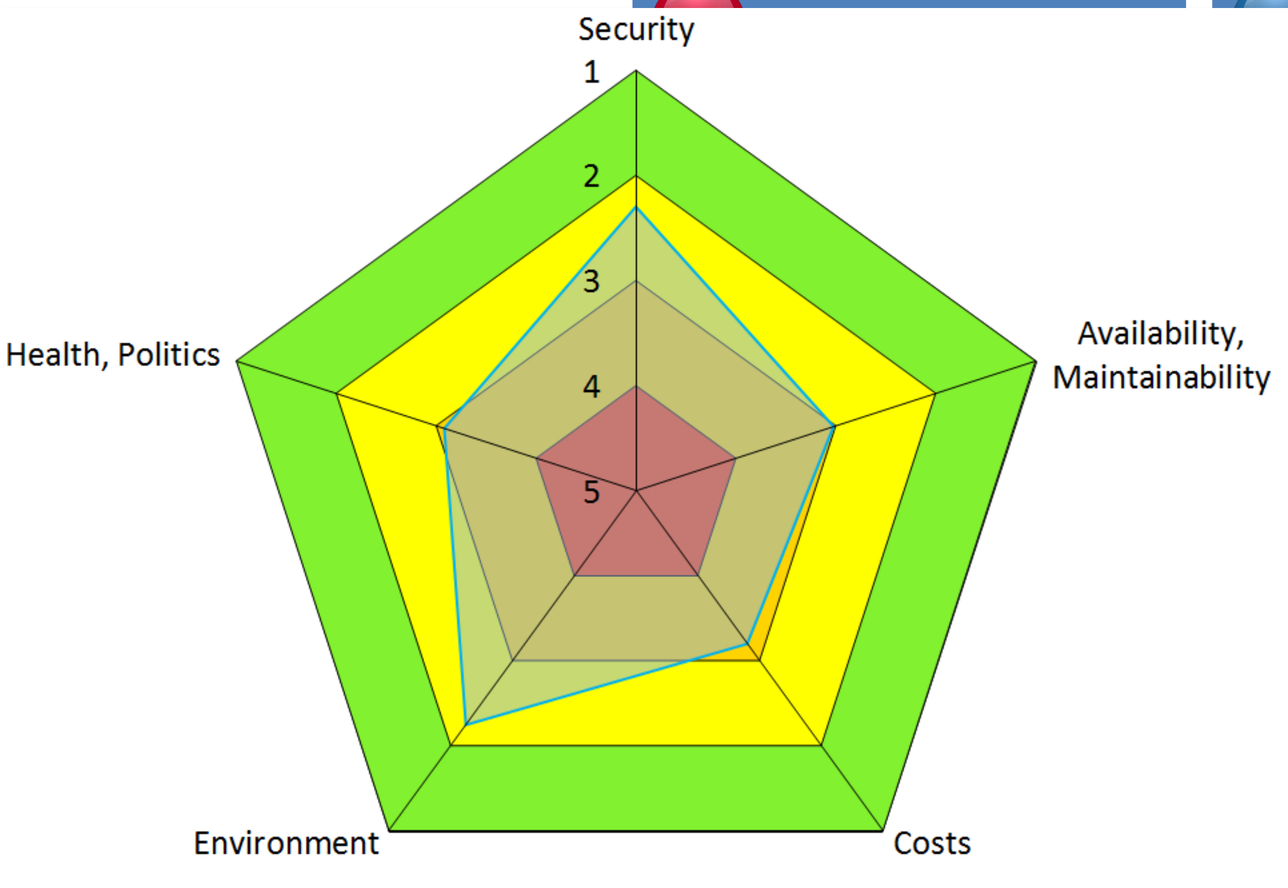
COST is supported by the EU Framework Programme

cost EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

from Performance indicators PI to KPI – WG2+WG3

OPERATORS’ DATABASE

Austria



Estonia
- List of documents
- Homogenized database

Finland
- List of documents
- Non-homogenized database

France
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Greece
- List of documents
- Homogenized database

from Performance indicators PI to KPI

In order to move on with the reduction of the list of Performance Indicators, an Expert Group was asked to **specify a reduced list of 108 PIs according to the following points**:

PERFORMANCE INDICATOR PI: Measurable?, Quantifiable?, Target value available?, Valid for ranking?; Allow decision with economic implications? (YES/NO); Technical (Tech), Socio Economical (SoEc), Sustainable (Sust)

KEY-PERFORMANCE INDICATOR(S) KPI: Reliability (R), Availability (A), Maintainability (M), Safety (S), Security (Se), Environment (E), Costs (C), Health (H), Politics (P), Rating/Inspection (I)

SAFETY, RELIABILITY, SECURITY			
PERFORMANCE INDICATOR (108 PIs)	LEVEL	**PERFORMANCE** INDICATOR PI	**KEY-PERFORMANCE** INDICATOR(S) KPI
	Component Level (CL) System Level (SL) Network Level (NL)	Measurable? Quantifiable? Target value available? Valid for ranking?; Allow decision with economic implications? (YES/NO) Technical (Tech), Socio Economical (SoEc), Sustainable (Sust)	Reliability (R), Availability (A), Maintainability (M), Safety (S), Security (Se), Environment (E), Costs (C), Health (H), Politics (P), Rating/Inspection (I)
CONCRETE COVER (INSUFFICIENT)	CL	YES, TECH, SUST	R, A, (C, I)
CRACKS RELATED….	CL, SL	YES, TECH	R, A, S, (C, I)
FATIGUE CRACKING	CL, SL	YES, TECH	R, A, S, (C, I)
SETTLEMENT	SL	YES, TECH	R, A, S, (C, I)

SAFETY, RELIABILITY, SECURITY		
PERFORMANCE INDICATOR (108 PIs)	**KEY-PERFORMANCE** INDICATOR(S) KPI	**ASSESSMENT**
	Reliability (R), Availability (A), Maintainability (M), Safety (S), Security (Se), Environment (E), Costs (C), Health (H), Politics (P), Rating/Inspection (I)	Threshold (T = …..) Goal (G = ….) Rating (R = ….)
CONCRETE COVER (INSUFFICIENT)	R, A, (C, I)	T= THICKNESS (MM), G= ASSESSMENT OF DAMAGE AND AFFECTED AREA (M2), R=IMPORTANT FOR DURABILITY
CRACKS RELATED….	R, A, S, (C, I)	T=WIDTH (MM), G=UNDERSTAND ORIGIN THROUGH THE CORRELATION OF THE OBSERVED THICKNESS (MM), LENGTH (CM), LOCATION/ORIENTATION AND SPACING/PATTERN, R=KEY PI TO ACCESS RELIABILITY

from Performance indicators PI to KPI

SAFETY, RELIABILITY, SECURITY		
PI	RATING (1-5)	WEIGHTING
CRACK WIDTH	2	0,8
CORROSION	3	0,5
LACK OF BOLTS	5	0,3
SUPPORT DAMAGE	2	1
DRAINAGE SYSTEM	2	0,8
FUNGUS APPEARANCE (WOODEN ELEMENTS)	3	0,5
BUGS ATTACK (WOODEN ELEMENTS)	5	0,3
ROTTING (WOODEN ELEMENTS)	2	1
OVERWEIGHT TRAFFIC	1	1
SEDIMENT ACCUMULATION	2	0,8
VANDALISM	3	0,8

DATABASE

ase

zegovina

latabase

ase

latabase

ase

Estonia
- List of documents
- Homogenized database

Finland
- List of documents
- Non-homogenized database

France
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Greece
- List of documents
- Homogenized database

from Performance indicators PI to KPI

SAFETY, RELIABILITY, SECURITY		
PI	RATING (1-5)	WEIGHTING
CRACK		
CORR		
LACK OI		
SUPPORT		
DRAINAGI		
FUN APPEA (WOO ELEM		
BUGS A (WOO ELEM		
ROTTING ELEM		
OVERW TRA		
SEDI ACCUMU		
VAND		

AVAILABILITY, MAINTAINABILITY		
PI	RATING (1-5)	WEIGHTING
CONCRETE FLORESCENCE	3	0,2
LACK OF BOLTS	5	0,3
SUPPORT DAMAGE	3	1
DRAINAGE SYSTEM	2	0,8
FUNGUS APPEARANCE (WOODEN ELEMENTS)	5	0,5
BUGS ATTACK (WOODEN ELEMENTS)	1	0,3
ROTTING (WOODEN ELEMENTS)	3	1
SEDIMENT ACCUMULATION	1	1
VANDALISM	3	0,8
	total rating	2,60

DATABASE

Estonia
- List of documents
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Finland
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France
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Germany
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- Homogenized database

Greece
- List of documents
- Homogenized database

from Performance indicators PI to KPI

SAFETY, RELIABILITY, SECURITY		
PI	RATING (1-5)	WEIGHTING
CRACK		
CORR		
LACK OI		
SUPPORT		
DRAINAGI		
FUN APPEA (WOO ELEM		
BUGS A (WOO ELEM		
ROTTING ELEM		
OVERW TRA		
SEDI ACCUMU		
VAND		

AVAILABILITY, MAINTAINABILITY		
PI	RATING (1-5)	WEIGHTING
CONCRETE FLORESCE		
LACK OF BO		
SUPPORT DAMAGE	3	1
DRAINAGE SYSTEM		
FUNGUS APPEARANCE (WOODEN ELEMENTS)	3	
BUGS ATTACK (WOODEN ELEMENTS)	1	
ROTTING (WOODEN ELEMENTS)	3	1
SEDIMENT ACCUMULATION	1	1
VANDALISM	3	0,8
	total rating	2,69

COSTS		
PI	RATING (1-5)	WEIGHTING
….	…	…
	total rating	2,69

ENVIRONMENT		
PI	RATING (1-5)	WEIGHTING
….	…	…
	total rating	2,25

HEALTH, POLITICS		
PI	RATING (1-5)	WEIGHTING
….	…	…
	total rating	3,08

from Performance indicators PI to KPI

K-PI	
SAFETY, RELIABILITY, SECURITY	2,30
AVAILABILITY, MAINTAINABILITY	3,02
COST	
ENVIRONMENT	
HEALTH, POLITICS	

’ DATABASE

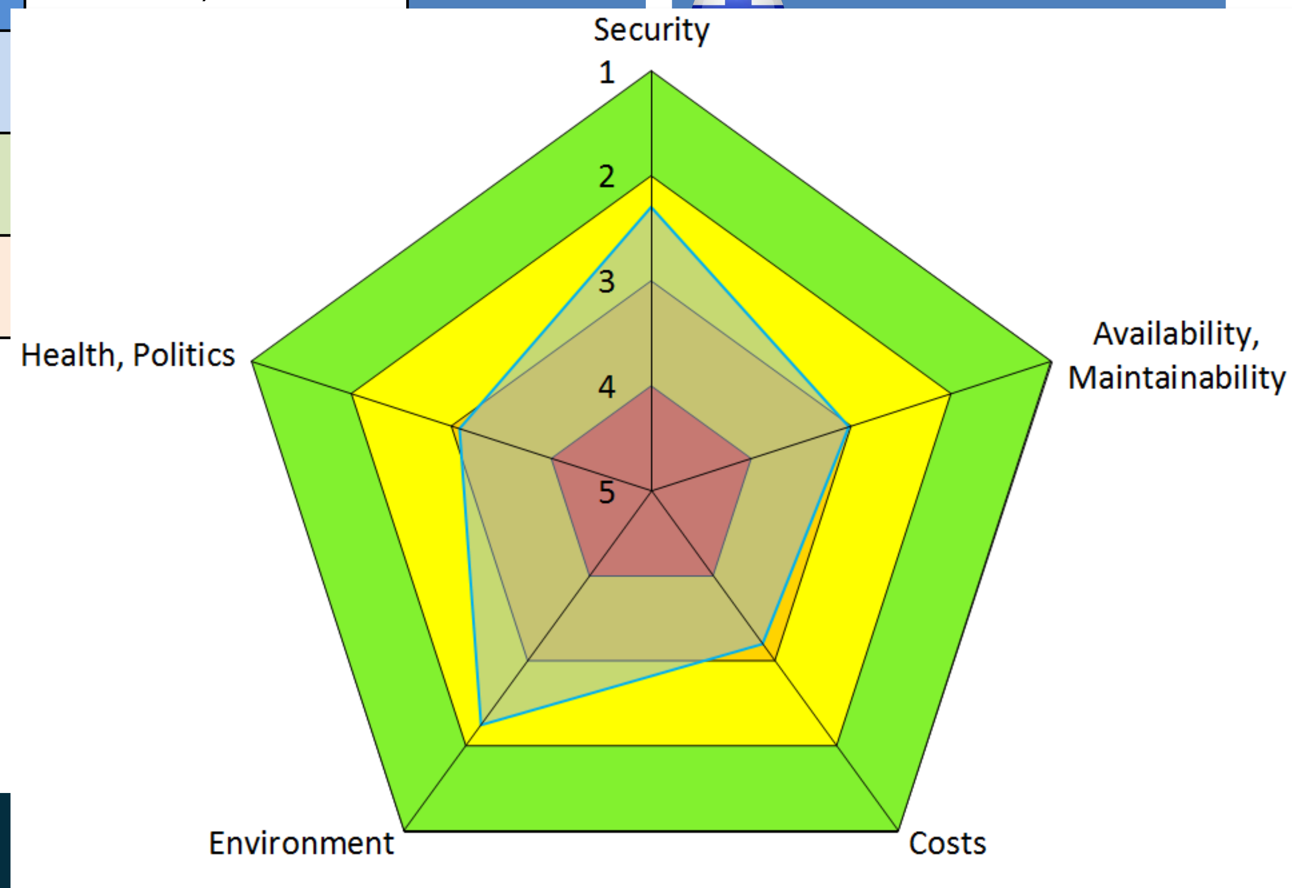
abase

rzegovina

Estonia

- List of documents
- Homogenized database

Finland




WG 1 REPORT

TU1406

COST ACTION

Quality specifications for roadway bridges, standardization at a European level

BASE

Estonia
- List of documents
- Homogenized database

Finland
- List of documents
- Non-homogenized database

France
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Greece
- List of documents
- Homogenized database

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COST EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

WG 1 REPORT

OPERATORS' DATABASE

Austria
- List of documents
- Homogenized database

Estonia
- List of documents
- Homogenized database

Bosnia and Herzegovina
- List of documents
- Non-homogenized database

Finland
- List of documents
- Non-homogenized database

Croatia
- List of documents
- Homogenized database

France
- List of documents
- Non-homogenized database

Czech Republic
- List of documents
- Non-homogenized database

Germany
- List of documents
- Homogenized database

Denmark
- List of documents
- Homogenized database

Greece
- List of documents
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Quality specifications for roadway bridges, standardization at a European level

ESF provides the COST Office through a European Commission contract

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EUROPEAN COOPERATION IN SCIENCE AND TECHNOLOGY

CONTENT LIST

01 General

General

Performance Indicators terms after surveying

02 Operators

Operators

Operators list of documents and database per country

03 Research

Research

Research list of documents and database per country

04 Glossary

Glossary

Glossary and specific term sheet per country

Thank you for your attention!

ACKNOWLEDGMENT

This article is based upon work from COST Action TU1406, Quality specification for roadway bridges, standardization at a European level (BridgeSpec), supported by COST (European Cooperation in Science and Technology). The support of the project “LeCIE – Life-cycle assessment for railway construction – strategies and methods” has been acknowledged.

TU1406

COST ACTION

QUALITY SPECIFICATIONS FOR ROADWAY BRIDGES, STANDARDIZATION AT A EUROPEAN LEVEL

WG 2 Performance Goals

Irina Stipanovic, University of Twente

Paper presented at IALCCE 2016 in Delft:

Multi-objective bridge performance goals

I. Stipanovic Oslakovic
University of Twente, Enschede, Netherlands

N. P. Høj
HOJ Consulting, Brunnen, Switzerland

G. Klanker
Rijkswaterstaat, Utrecht, Netherlands

ABSTRACT: During the implementation of asset management strategies, maintenance actions are required in order to keep assets at a desired performance level. It is verified that there is a large disparity in Europe regarding the way performance indicators are quantified and how performance goals are specified. Therefore, COST TU 1406 Action aims to bring together both research and practicing community in order to accelerate the establishment of a European guideline in this subject. This paper presents the structure and basic ideas for the development of a guideline document incorporating different aspects of bridge performance goals, which may vary according to technical, environmental, economic and social factors. It discusses the need for agencies to measure bridge performance against objectives on structure and network level.

Introduction

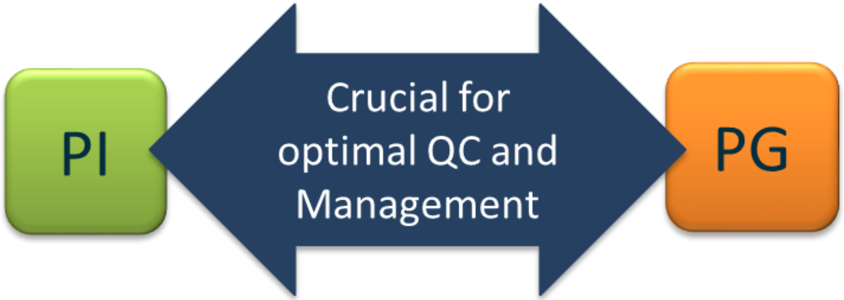
Main objective of WG 2 Performance Goals

- To provide an overview of existing performance goals based on the *performance indicators* previously identified in WG1.
- These goals will vary by technical, environmental, economic and social aspects, and on the component, system and network level.

- Deliver a Report which will specify the performance goals, linked to the Performance Indicators.

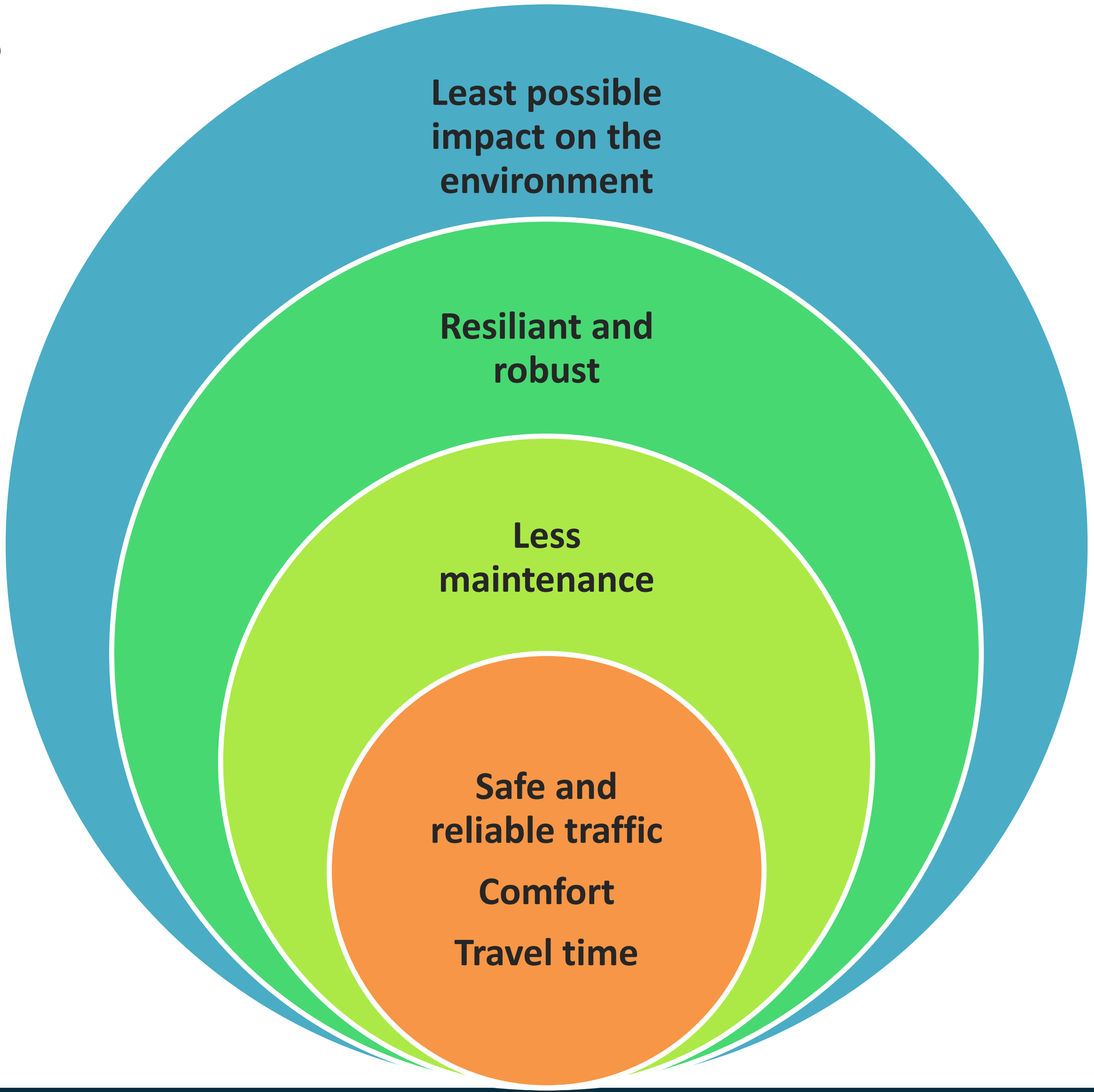
Definition of Performance Indicator

- Parameter **measurable** and **quantifiable** related to the bridge performance that can be compared with a **target measure** of a **performance goal** or can be used for **ranking** purposes among a bridge population in the framework of a **Quality Control Plan** or life-cycle management (**decisions, actions involving economic resources).**

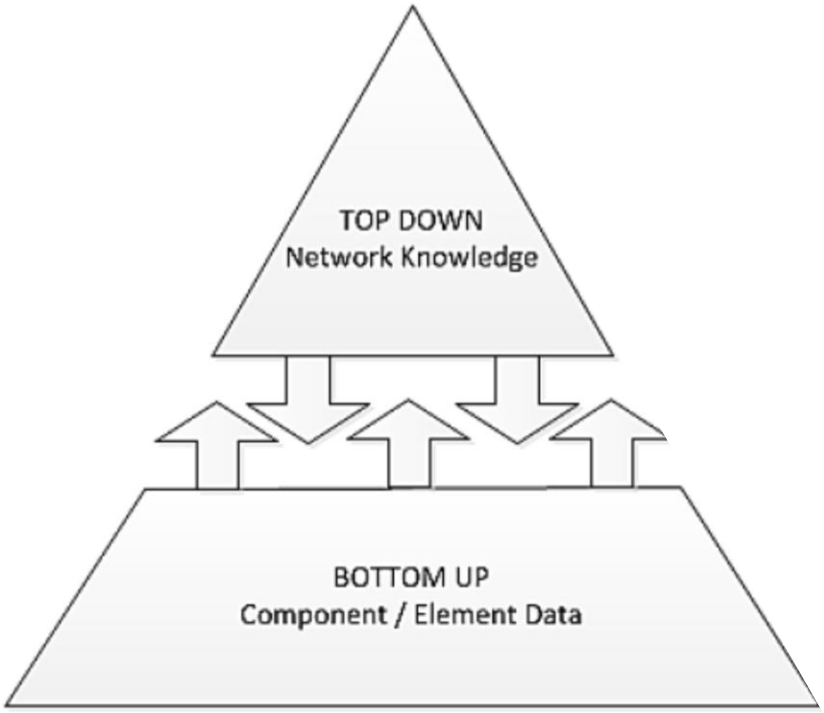


Performance goals
- society / users related

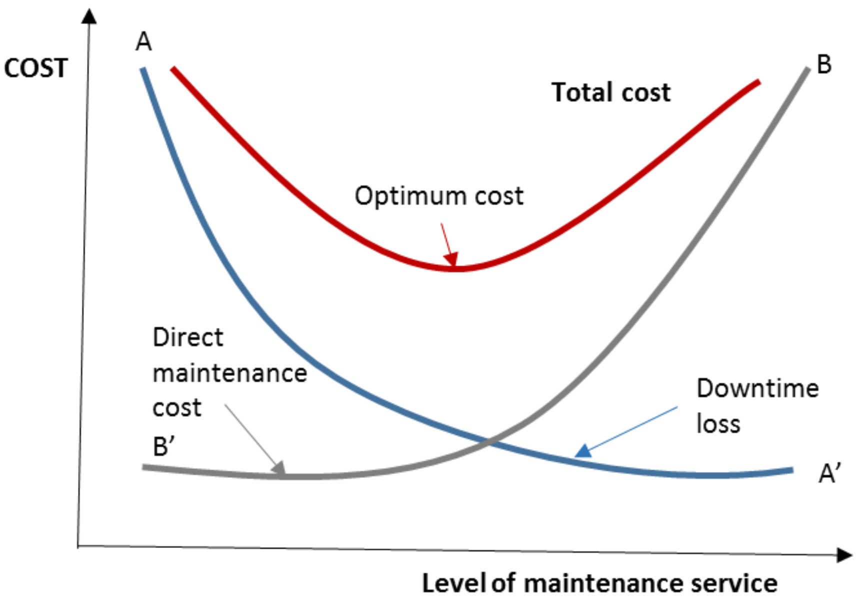
- Technical PGs
  - Reliability and safety related goals
- Sustainable PGs
  - Environmental impact related goals
- Other PGs
  - Economic and social based goals



Asset management approach



- LCC concept



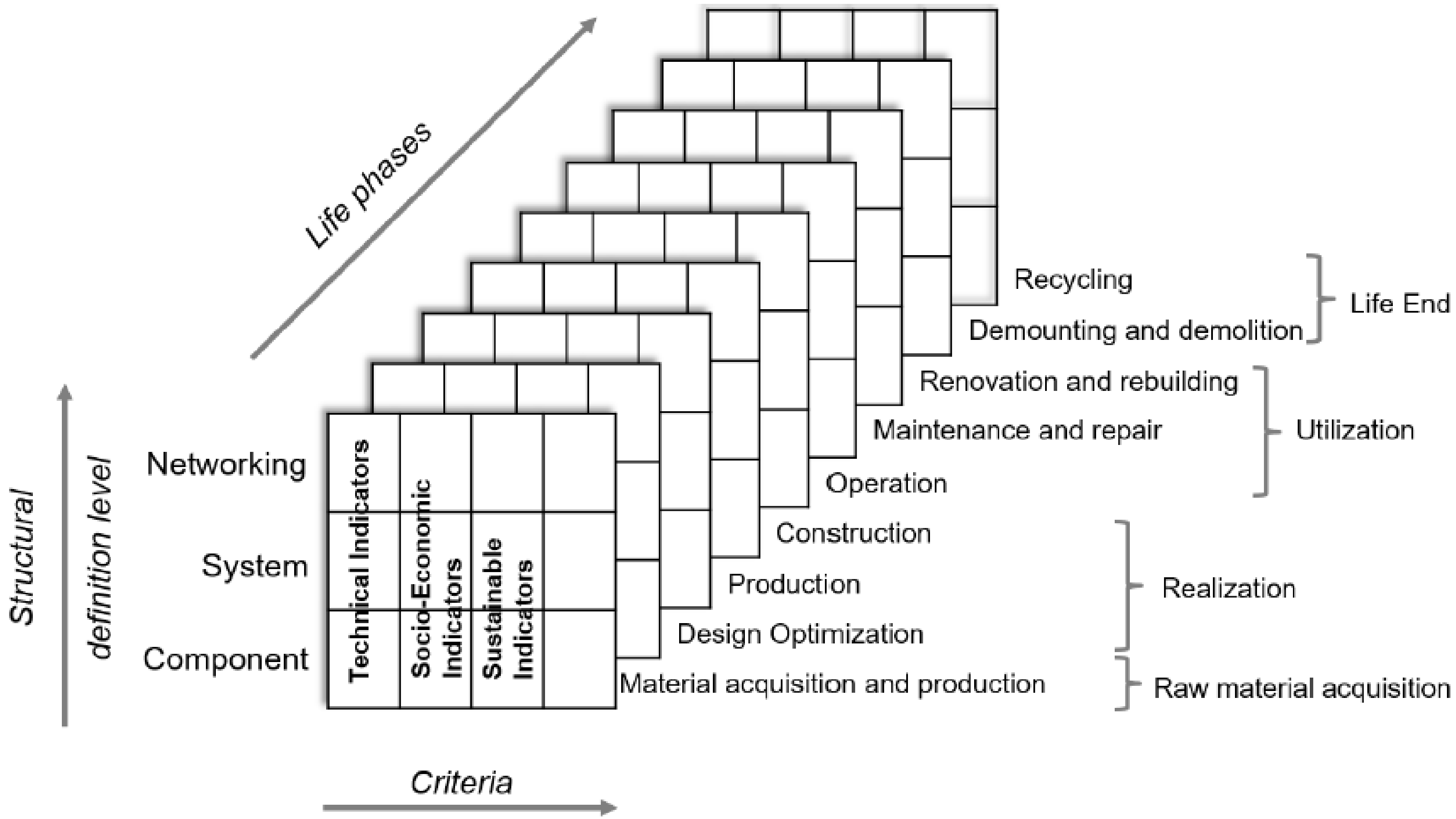
BRIDGE MANAGEMENT APPROACH

- Asset management considers physical assets in relation with other activities to deliver required performance
- Bridge management is to be part of the management of the network
- PAS55 (BSI, 2008) and ISO55000

In the case of the Netherlands network performance is described using nine performance aspects (RAMS SHEEP):

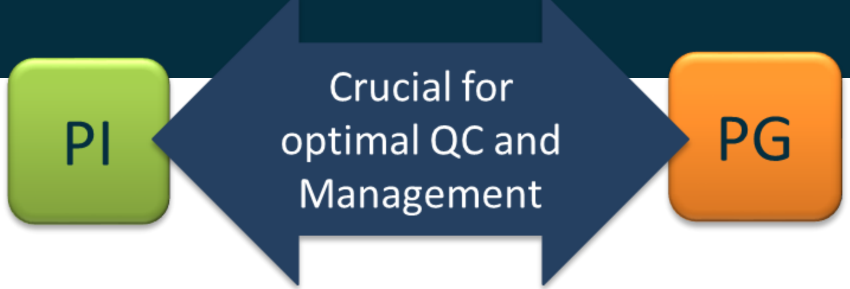
- Reliability, Availability, Maintainability
- Safety
- Security, Health, Environment
- Life Cycle Costs
- Politics

Complexity!

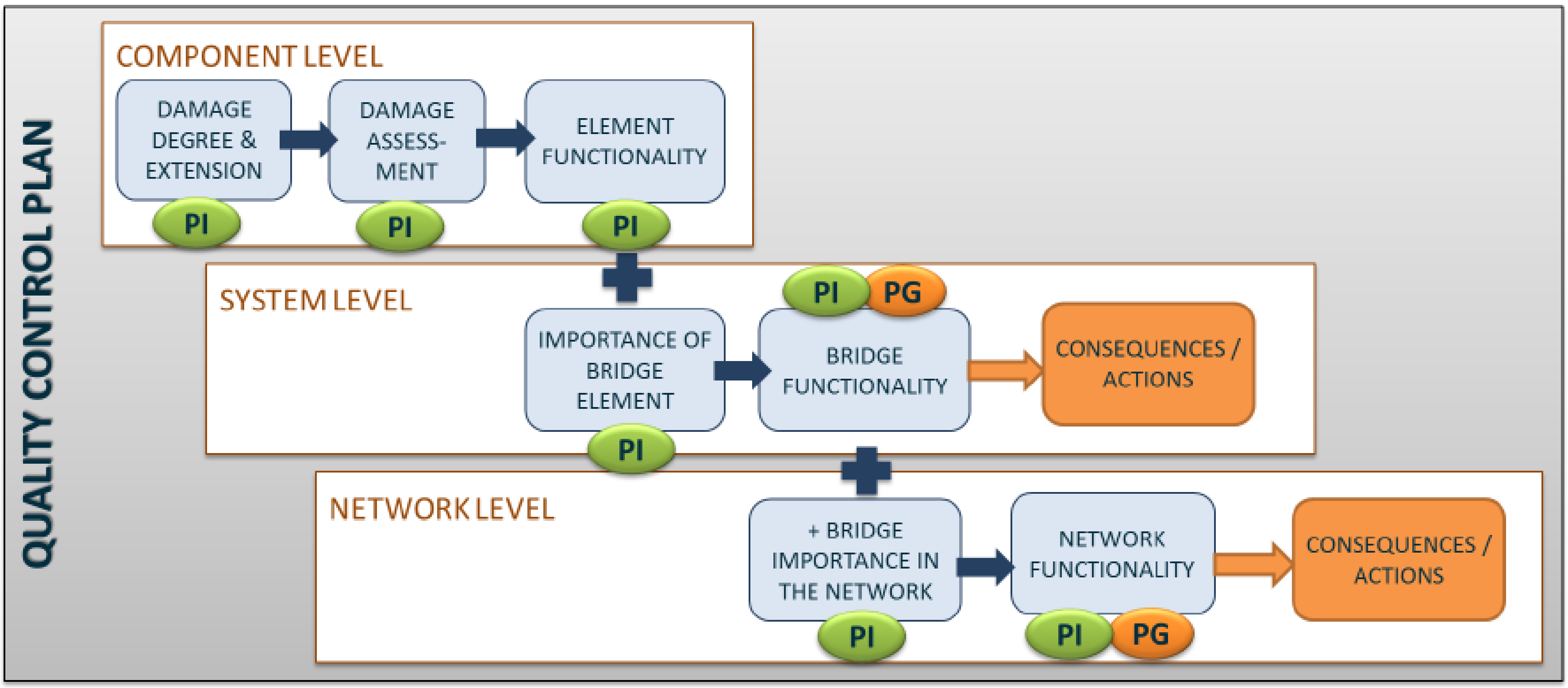


Life phases
Structural
definition level
Networking
System
Component
Technical Indicators
Socio-Economic Indicators
Sustainable Indicators
Criteria
Recycling
Demounting and demolition
Life End
Renovation and rebuilding
Maintenance and repair
Utilization
Operation
Construction
Production
Design Optimization
Realization
Material acquisition and production
Raw material acquisition

Framework

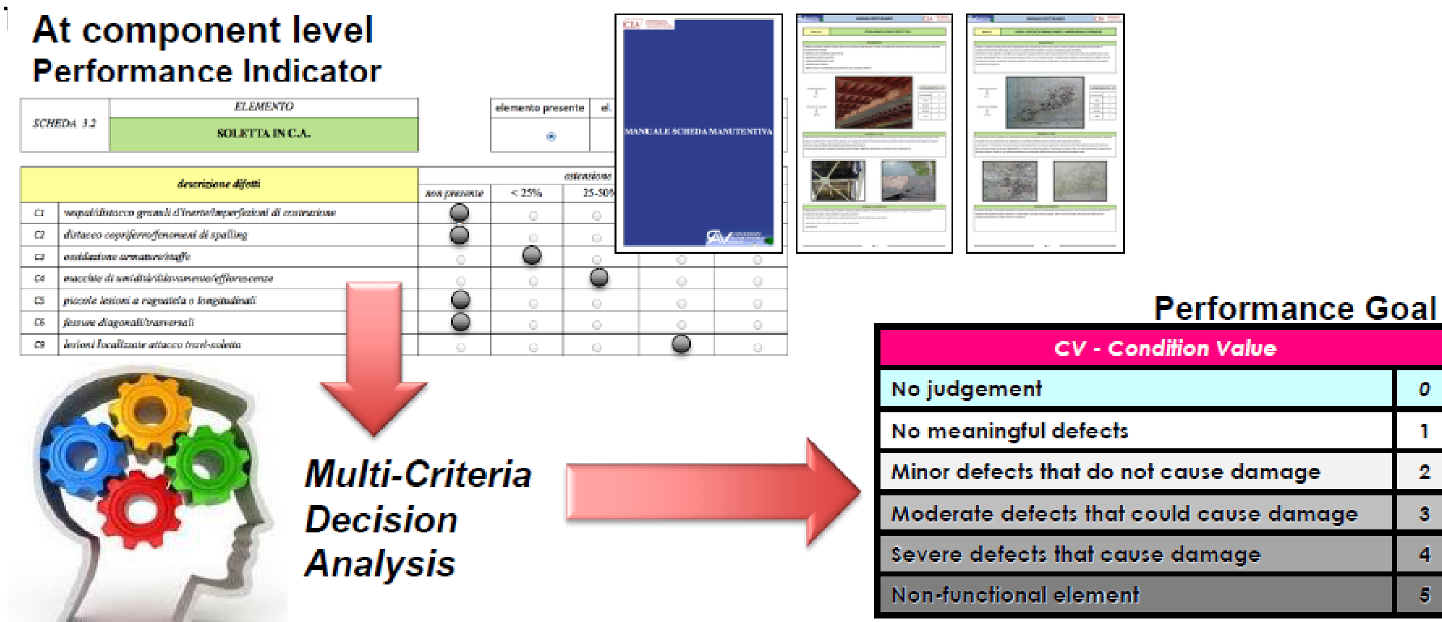


- Clustering the observations and the necessary actions based on the component, system and network level.



Transformation of performance indicators

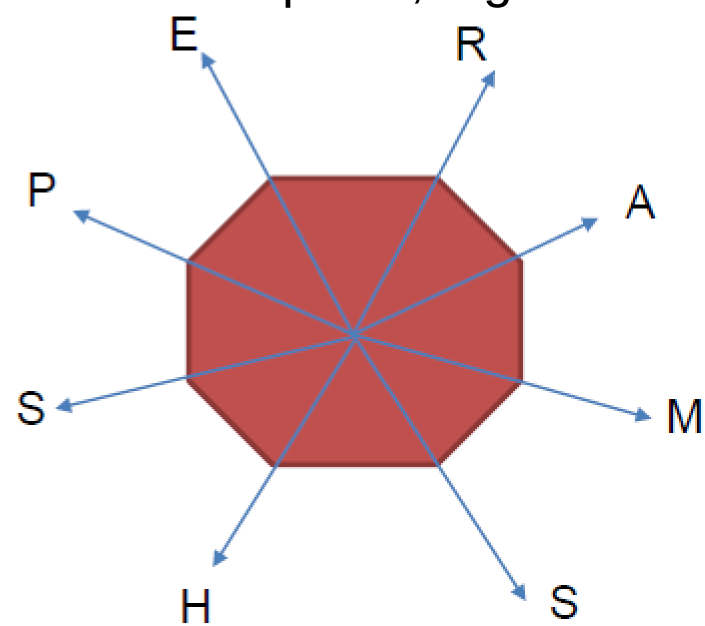
- The multiple performance indicators cannot necessarily be directly compared;
- The PIs must be transformed in order to facilitate decision → one possibility would be to transform the PIs to aggregated condition values
- In order to do so, a “multi-criteria-analysis” or similar will have to be carried out



CV - Condition Value	
No judgement	0
No meaningful defects	1
Minor defects that do not cause damage	2
Moderate defects that could cause damage	3
Severe defects that cause damage	4
Non-functional element	5

Performance goals

- Different aspects of performance should be taken into account for the initial situation and for a situation with remediate measures
  - Technical, sustainability and socio-economic aspects, e.g.
    - Reliability
    - Availability
    - Maintainability
    - Safety
    - Security
    - Environment
    - Costs
    - Health
    - Politics



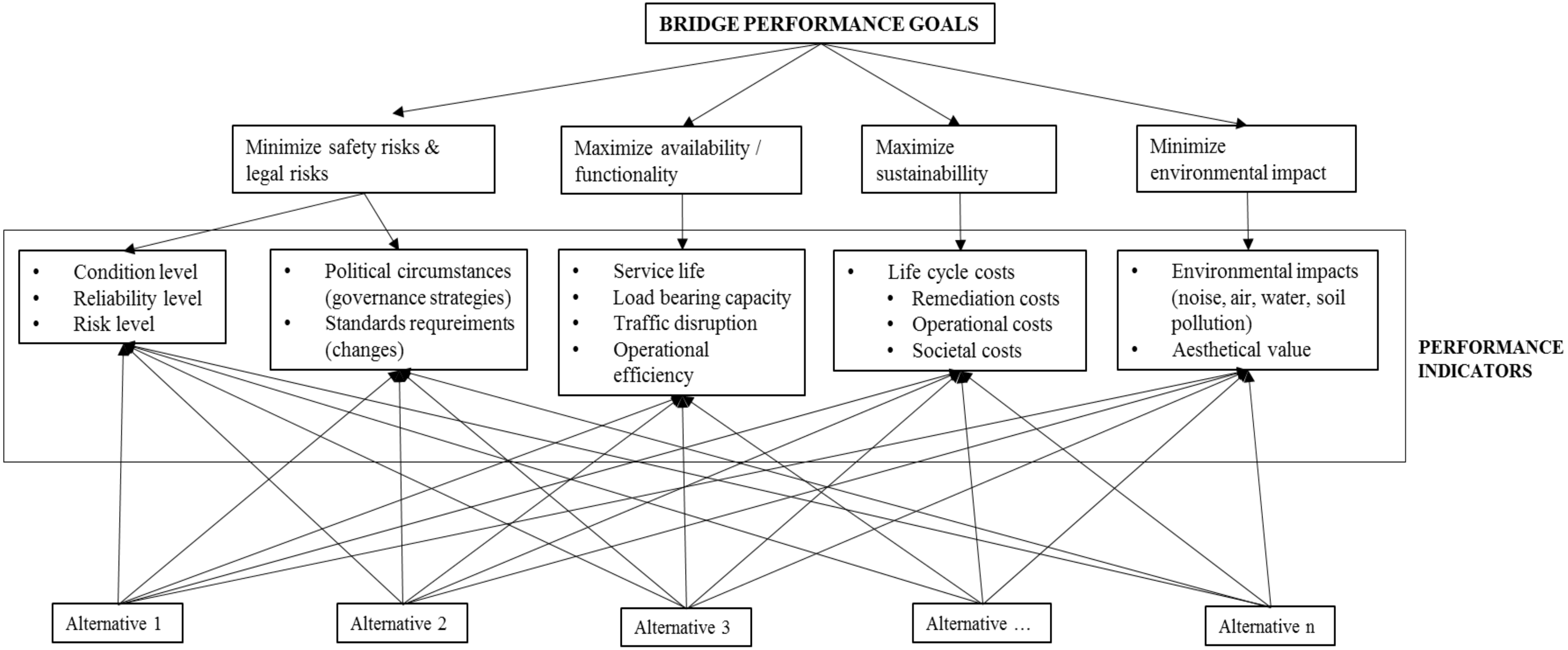
- The ”aspects” will be quantified as a probability and a ”consequence”
- The various ”aspects” will be normalised with ”weight factors”

Level	Performance goals (examples)	Tasks	Performance indicators (examples)
STRATEGIC LEVEL	- To provide safe, responsive and sustainable network; - To provide an efficiently and effectively operated strategic road network; - To support and facilitate economic growth; - To minimize its negative impacts on users, local communities and the environment; - To balance the needs of individuals and businesses that use and rely on it.	Resources allocation at network level, across regions, assets or types of activities for long term, usually for the period of 25 years	- Average availability of the road (% of time) - Long-term trend of traffic jams - Long-term trend of number of traffic accidents - Environmental impacts

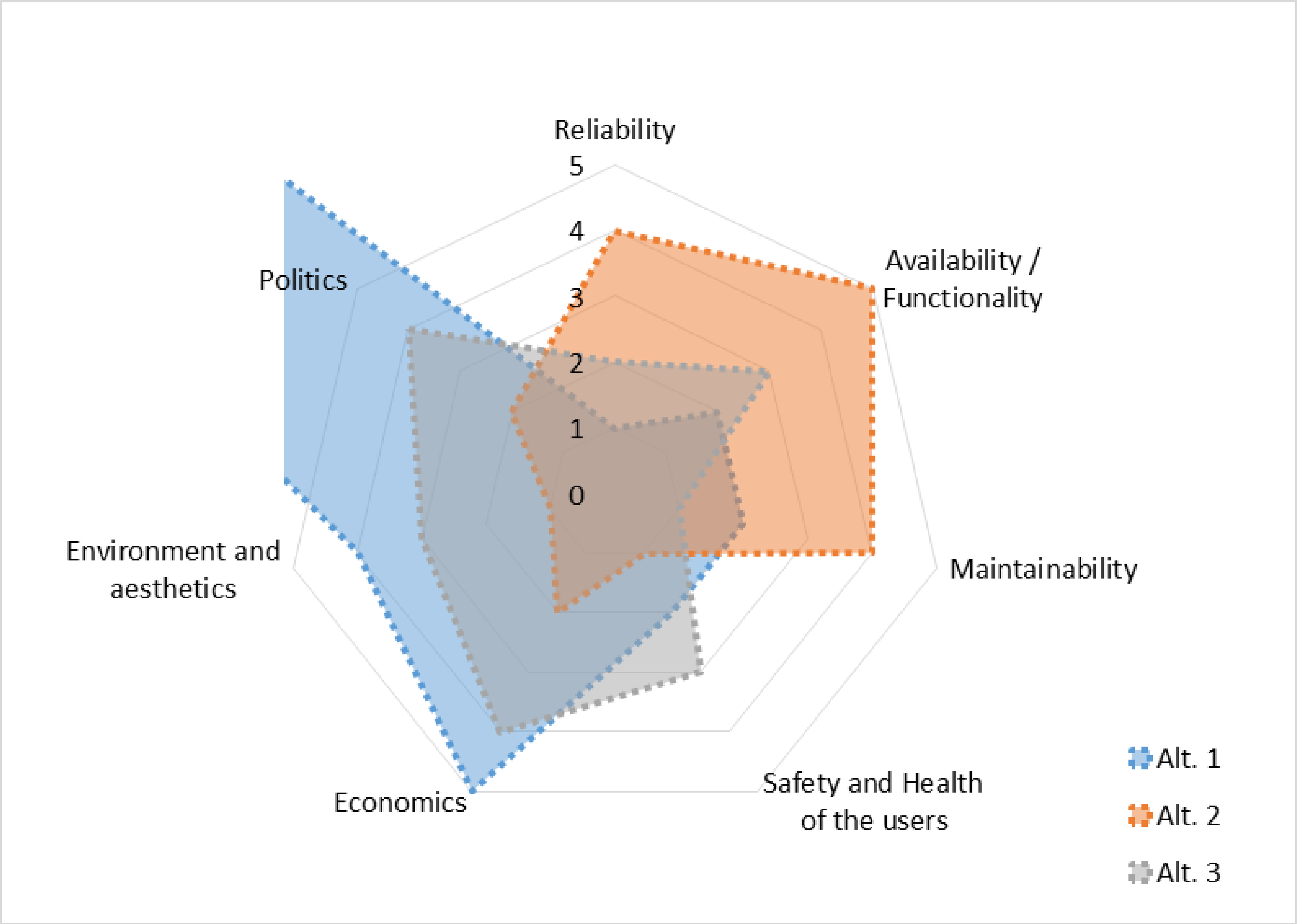
Level	Performance goals (examples)	Tasks	Performance indicators (examples)
TACTICAL LEVEL	Meet technical, socio-economic and sustainability requirements in line with performance goals	Risk assessment Resources allocation across regions, assets or types of activities for medium term, usually for the period of 5 to 10 years	- Availability of the road (% of time) - Traffic delays caused by road works - Availability of the road during rush hour - Traffic jams - Traffic kilometres travelled on the main road network - Number of deaths caused by traffic accidents - Number of hospitalized by traffic accidents - Environmental impacts - Unplanned maintenance / operational costs

Level	Performance goals (examples)	Tasks	Performance indicators (examples)
OPERATIONAL LEVEL	Meet technical, socio-economic and sustainability requirements in line with performance goals	Resources allocation across assets (usually within one type of asset or a part of the network) within 1 to 5 years. Decision on alternative choice on the project / bridge level, e.g. • do nothing and monitor • repair • rehabilitate • replace	Technical indicators: - condition level - reliability index - risk level Socio-economic: - maintenance costs - operational costs - user delay costs - environmental costs Environmental indicators: - air pollution - noise - soil and water pollution

Hierarchy structure for linking multi-objective bridge performance goals and performance indicators

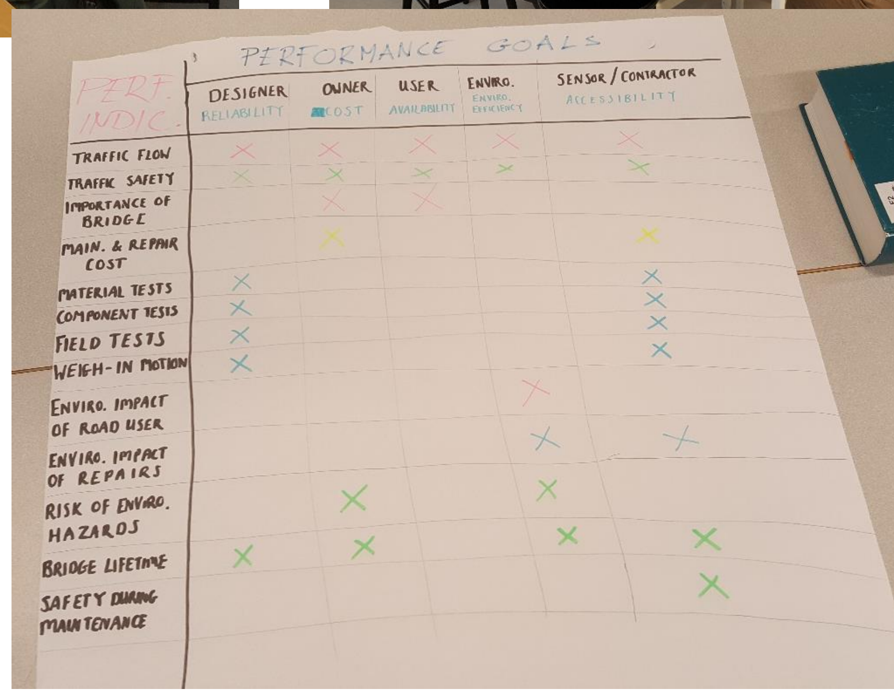


Example


Reliability
5
4
3
2
1
0
Availability / Functionality
Maintainability
Safety and Health of the users
Economics
Environment and aesthetics
Politics
Alt. 1
Alt. 2
Alt. 3

COST and IABSE Summer School

September 2016 in Stockholm, KTH



Next steps

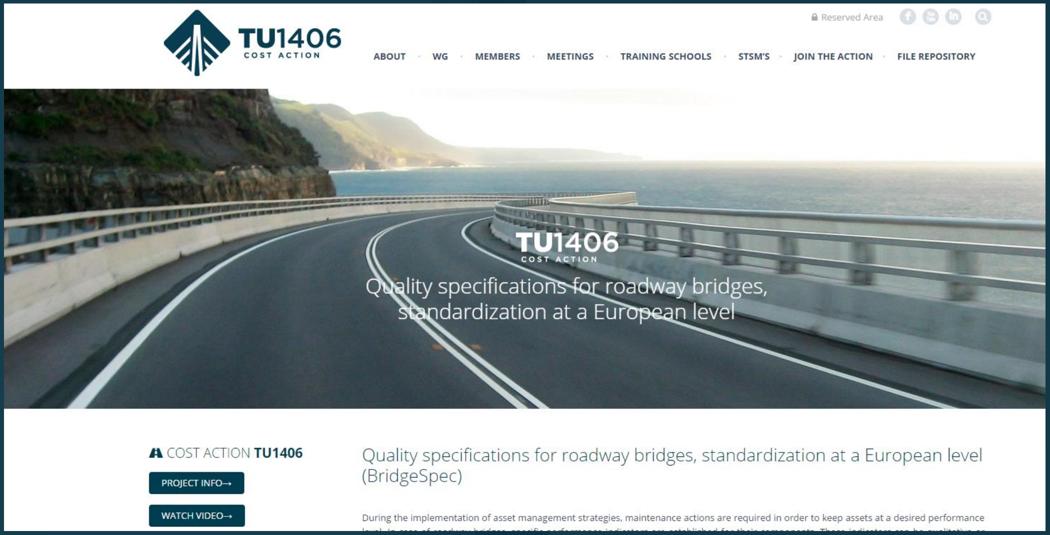
- WG 2 objectives:
  – To develop a

  – To apply proposed framework for multi-criteria decision analysis on system level and network level

Next steps

- Identifying core group members
- Definitions for performance aspects / axes
- Identification of methodologies for determining each of performance aspects (linking PIs with PG assessment)
- Identification of performance thresholds if they exist
- Overview of methodologies for optimizing performance goals
- Examples how to apply framework on system and network level

- Until meeting in Zagreb:
  - Content list of the document
  - Chapter leaders
  - Literature study, focus on COST contributions so far
  - Presentation of the draft report



Thank you for your attention!

TU1406
COST ACTION

WG2 and WG3 WORKSHOP
Bridge performance goals and quality control plans

From performance indicators to performance goals

Rade Hajdin - University of Belgrade, Serbia

Грађевински факултет

Универзитет
у Београду

Challenge

- Connecting observation and other performance indicators with KPI
- KPI are performance indicators that can be directly related to performance goals.

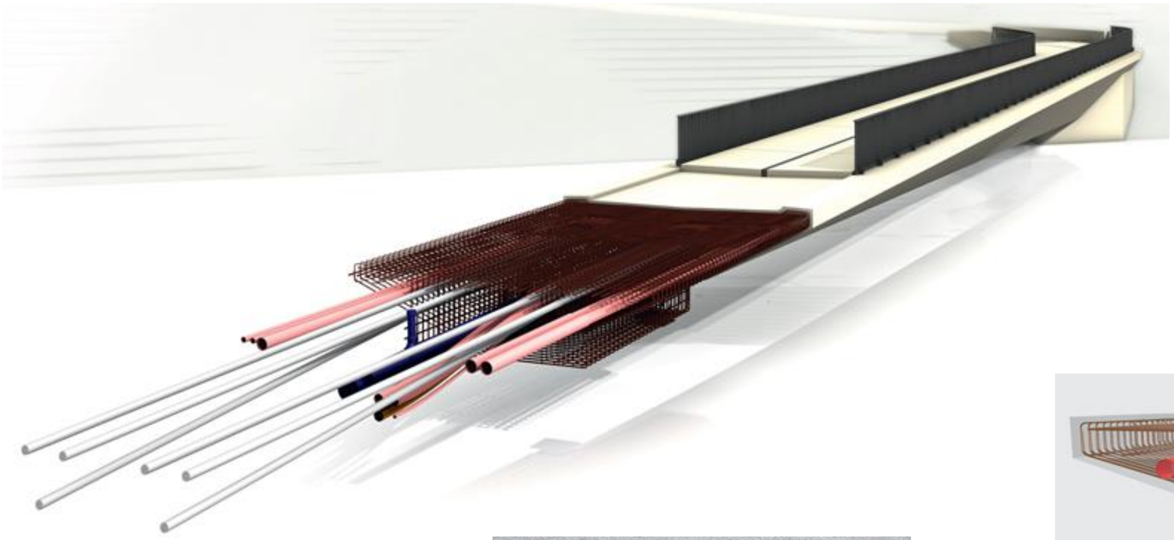
- Defects
- Indicators related to material properties
- Indicators related to equipment and protection
- Geometry changes
- Indicators related to bearing capacity, structural integrity and joints
- Indicators related to original construction and design
- Indicators related to dynamic behavior
- Environmentally based indicators (common appearance)
- Rating
- Cost and importance

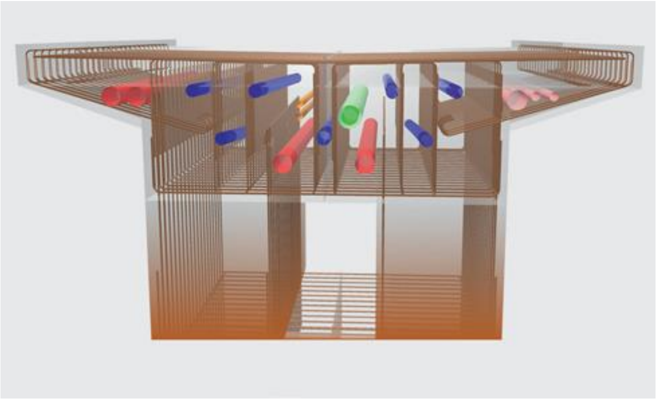
↔

- **S**afety, **R**eliability, **S**ecurity
- **A**vailability, **M**aintainability
- Costs (**€**)
- **E**nvironment
- **H**ealth, **P**olitics

Vision

- Helmut Schmidt: “Anyone who has visions should go to the doctor.”





Modelling Reality

- Modelling reality -> Semantic 3D has the highest fidelity
  - Geometry
  - Material
  - Structural model
  - Deterioration simulation
- Observations can be integrated directly in the model
- Their impact on
  - Safety and Reliability,
  - Availability and
  - Long-term costs

  is immediately available at any time instance.
- These models are still not available and in some cases they might be an overkill!

Adobe Acrobat Document

Taxonomy

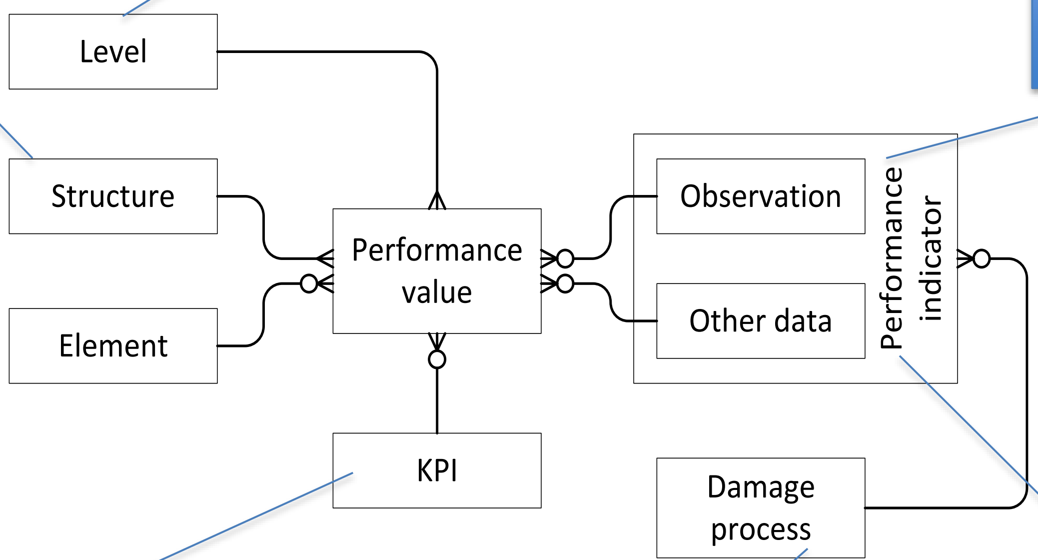
- One tries to find typical situations, the patterns that are well known
- The bridges are typified according to their structural system
- The bridge elements are typified according to their function and7or visual appearance.
- Type is distinguishing feature of an object.
- Objects of the type “column” can differ in size and even in shape but they need to be near vertical and be predominantly stressed in compression.
- The WG1 has essentially established the taxonomy of performance indicators
- Similarly as in 3D models one has to merge the original structure (geometry, materials) with the more recently obtained performance indicators.

Approach

- Network
- System
- Component

- Crack
- Chloride content
- Corrosion pit
- Deformation
- Frequency
- etc.

- Girder bridge
- Arch bridge
- Frame bridge



- **S**afety, **R**eliability, **S**ecurity
- **A**vailability, **M**aintainability
- Costs (€)
- **E**nvironment
- **H**ealth, **P**olitics

- Corrosion
- Alkali-silica reaction
- Fatigue
- Repeated overloading (Low-cycle fatigue)
- etc.

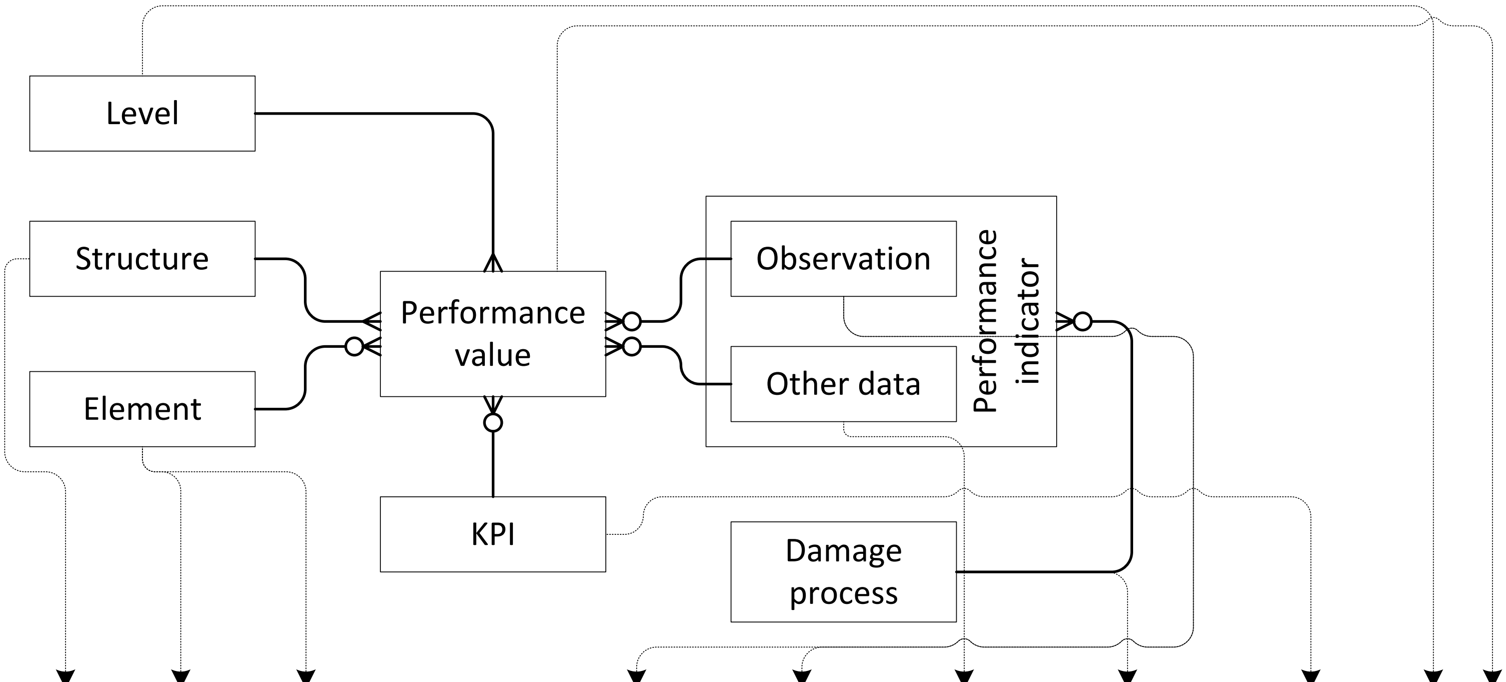
- Construction year
- Construction type
- Designer
- Contractor
- etc.

BREAKDOWN OF TASKS

- Focus on most common bridge types and systems
- No landmark bridges

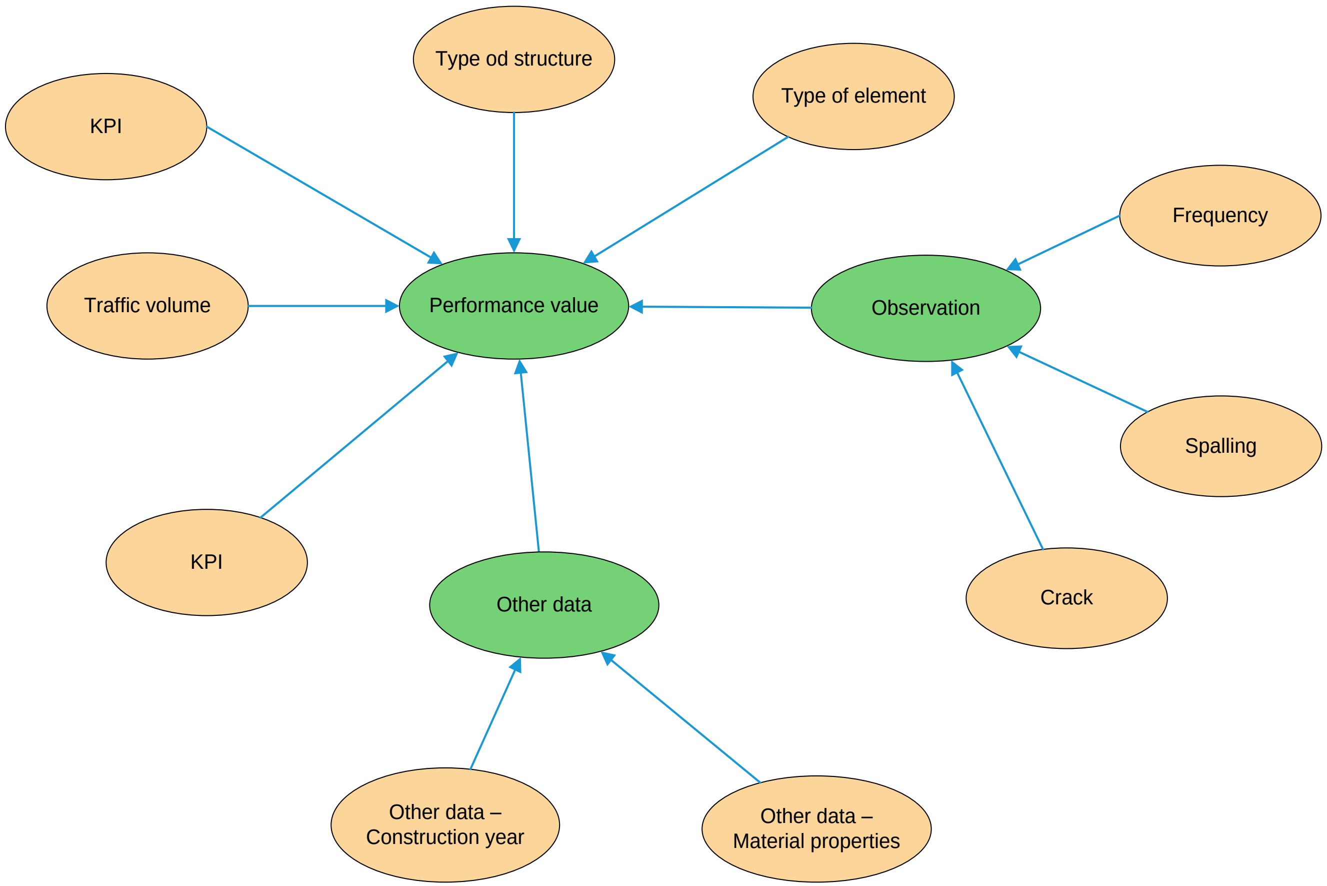
			Girder bridges	Arch bridges	Frame bridges	Etc.
Interceptable processes (Observable processes)	Damaging processes	Corrosion	**Task 2 (Masovic. Linneberg)**	**Task 3 (Amado)**	**Task 2 (Masovic, Linneberg)**	
		Alkali Aggregate				
		Sulphate				
		Fatigue				
	Demand	Traffic volume				
		Traffic loading				
		Climate				
Non-interceptable processes	Sudden events	Earthquake	**Task 4 (Tanasic)**			
		Gravitational hazards				
		Fire				
		Accidents				
	Non-observable	Fatigue				
		Hidden damaging processes				

In more tangible terms ….



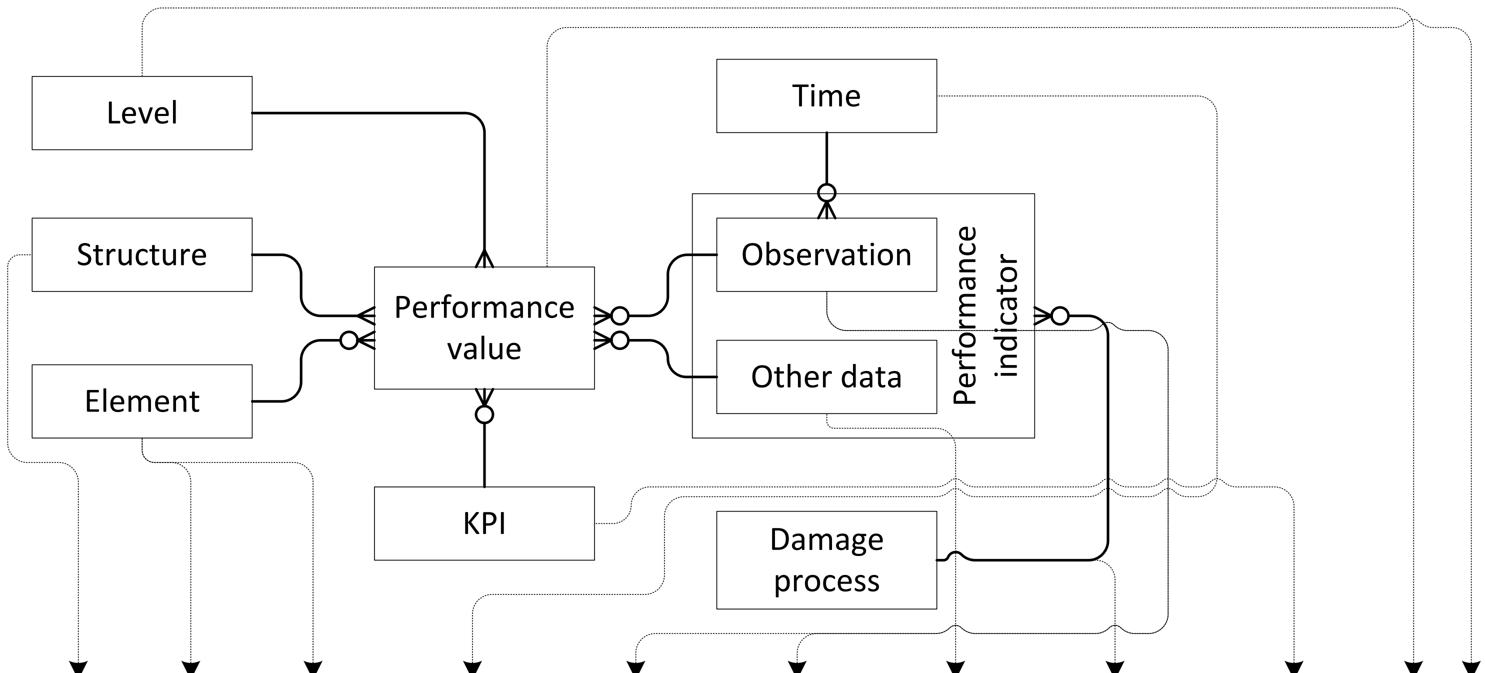
Single Girder Bridge	Beam	Reinforced concrete		Crack	0.4 mm	Construction year	Repeated overloading	Safety, Reliability, Security	C	2
Single Girder Bridge	Beam	Reinforced concrete		Spalling	1.0 m^2	Construction year	Corrosion of Reinforcement	Safety, Reliability, Security	C	3

Bayesian networks?



Condition forecast

- If damage (deterioration) process is known, it governs the development of defects.
- The development of defects – or any other PI – in time affects the KPI indicators.
- The development of KPI in time can in this manner be evaluated.

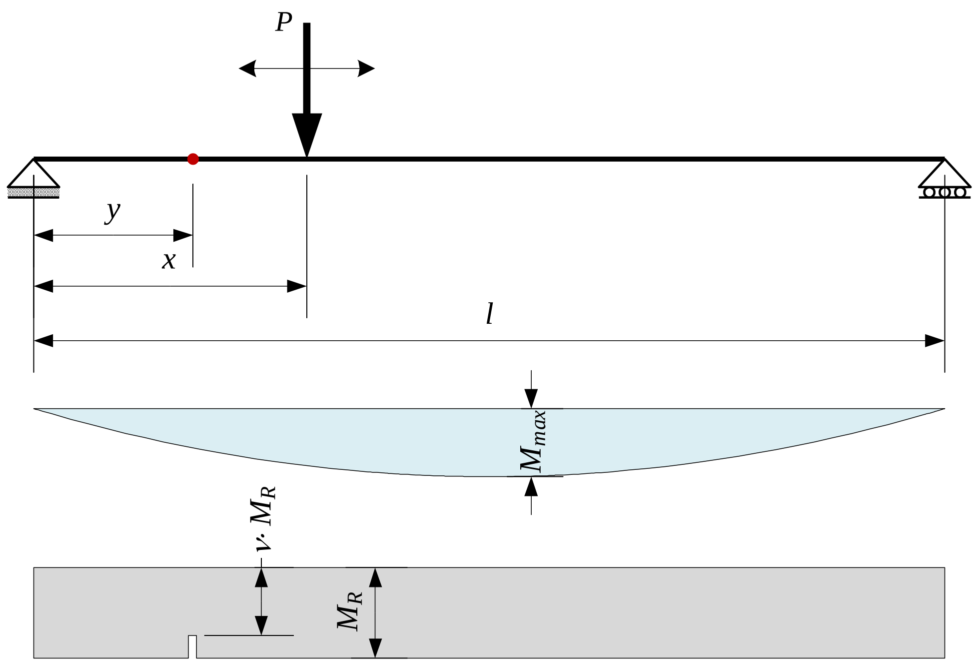


Single Girder Bridge	Beam	Reinforced concrete	*Time of the latest inspection*	Crack	0.4 mm	Construction year	Repeated overloading	Safety, Reliability, Security	C	2
Single Girder Bridge	Beam	Reinforced concrete	*Time of the latest inspection*	Spalling	1.0 m^2	Construction year	Corrosion of Reinforcement	Safety, Reliability, Security	C	3
Single Girder Bridge	Beam	Reinforced concrete	*Forecast in 10 years*	Spalling	2.0	Construction year	**Corrosion of Reinforcement**	Safety, Reliability, Security	C	4

Qualitative vs. quantitative KPI’s

- The chosen approach is fully qualitative.
- This decision was taken in Belgrade.
- However the suggested frameworks is also applicable for quantitative approaches.
- In the suggested qualitative approach the “failure” scenario is implicitly considered.
- In quantitative scenarios the “failure” scenario has to be explicitly defined.
- For this purpose the separate entity “Failure Scenario” has to be added to the ERD (Entity Relationship Diagram).
- Most of the observations are visual however ….
- Can these visual observations be used in evaluation of quantitative KPI’s?

Exploitation of visual observations I



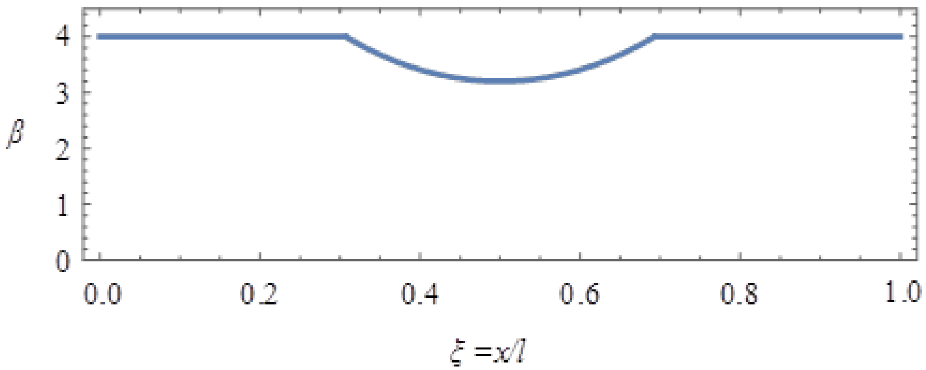
$$\beta = \frac{\mu_{M_R} - \frac{\mu_P \cdot l}{4}}{\sqrt{\sigma_{M_R}^2 + \left(\frac{\sigma_P \cdot l}{4}\right)^2}} = \frac{500 - 250}{\sqrt{50^2 + \left(\frac{150}{4}\right)^2}} = \frac{250}{50 \cdot \sqrt{\frac{25}{16}}} = \frac{1000}{250} = 4.0$$

$$P_f = 3.17 \cdot 10^{-5}$$

Exploitation of visual observations II

$$\beta = Min\left(\frac{\nu \cdot \mu_{M_R} - \mu_P \cdot \xi \cdot (1-\xi) \cdot l}{\sqrt{\sigma_{M_R}^2 + \left(\sigma_P \cdot \xi \cdot (1-\xi) \cdot l\right)^2}}, 4\right)$$

$$P_f = \Phi(-\beta)$$



$$P_f = \int_0^1 \Phi\left(-Min\left(\frac{\nu \cdot \mu_{M_R} - \mu_P \cdot \xi \cdot (1-\xi) \cdot l}{\sqrt{\sigma_{M_R}^2 + \left(\sigma_P \cdot \xi \cdot (1-\xi) \cdot l\right)^2}}, 4\right)\right) \cdot d\xi$$

$$= \int_0^1 \Phi\left(-Min\left(\frac{0.9 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + \left(15 \cdot \xi \cdot (1-\xi) \cdot 10\right)^2}}, 4\right)\right) \cdot d\xi = 15.69 \cdot 10^{-5}$$

$$\beta = -\Phi^{-1}\left(P_f\right) = 3.6$$

Exploitation of visual findings III

Loss of cross-section	5%	10%	15%	20%
Probability	60%	20%	10%	10%

$$P_f = 0.6 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.95 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + (15 \cdot \xi \cdot (1-\xi) \cdot 10)^2}}, 4\right)\right) \cdot d\xi +$$

$$+0.2 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.9 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + (15 \cdot \xi \cdot (1-\xi) \cdot 10)^2}}, 4\right)\right) \cdot d\xi +$$

$$+0.1 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.85 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + (15 \cdot \xi \cdot (1-\xi) \cdot 10)^2}}, 4\right)\right) \cdot d\xi +$$

$$+0.1 \cdot \int_0^1 \Phi\left(-Min\left(\frac{0.8 \cdot \mu_{M_R} - 100 \cdot \xi \cdot (1-\xi) \cdot 10}{\sqrt{20^2 + (15 \cdot \xi \cdot (1-\xi) \cdot 10)^2}}, 4\right)\right) \cdot d\xi$$

$$= 0.6 \cdot 5.134 \cdot 10^{-5} + 0.2 \cdot 15.69 \cdot 10^{-5} + 0.1 \cdot 56.62 \cdot 10^{-5} + 0.1 \cdot 190.55 \cdot 10^{-5} = 30.94 \cdot 10^{-5}$$

$$\beta = 3.42$$

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Performance goal assessment for existing bridges subject to pier local scour

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Abstract. In European roadway and railway networks, many multi-span unreinforced arch bridges were built in the last century. These structures are often characterized by piers with shallow foundations placed in the river bed, which can be subject to hydrodynamic turbulences in case of river floods and can lead to localized scour phenomena. The lack of adequate monitoring can cause over time scour-induced settlements able to involve bridge structural failure. This papers analyses this question focusing on structural behaviour of existing masonry bridges through the failure analysis of a finite element model case study, by numerically simulating the evolution of the structural behaviour and finding a relationship with the local scour profile at the base of pier foundations, useful for the identification of a structurally consistent performance goal.

Keywords: masonry arch bridges; local scour; failure analysis; flooding; pier settlements

1 Introduction

Bridges are key elements in infrastructural networks and their functionality must be preserved. Bridge structures have therefore to be adequately maintained, since structural deterioration combined with the occurrence of hazardous events like earthquakes or floods can compromise structural stability, and lead to bridge failure (Deng et al. 2016). In particular, flooding phenomena can generate scouring at the base of bridge pier foundations located in the river bed, involving a reduction of soil capacity and subsequent settlements that, in the worst case, can induce the pier collapse and consequent bridge closure. Bridge failures due to scouring were increased in the last decades mainly due to scarce maintenance and monitoring activities, and at the same time to climate change effects that in many regions give rise to extreme weather conditions alternating drought periods with sudden heavy rainfalls. Scouring effects are significantly dangerous in particular for ancient bridges, like masonry arch bridges, since shallow foundations characterize them. Local erosion is mainly due to the increase of the hydraulic outflow speed (Hydraulic engineering circular no. 18, 2001) close to bridge piers, which causes turbulences and vortex shedding, generating local scouring at the base of pier foundations. Scour of bridge foundations is one of the most frequent causes of structural collapse in United States, with about 600 bridges failed during the last 30 years (Briaud et al. 2005) and an annual average cost for flood damage repair of highway bridges estimated in about $50 million (Lagasse et al. 1995). Also in Europe scouring is one of the most frequent causes of bridge collapses, particularly in United Kingdom (Maddison 2012) and central Europe (Tanasic 2016). In Italy, during recent flooding phenomena some failures of old masonry bridge structures were observed, *a posteriori* imputable to local scouring of piers sited in the river bed (Ballio et al. 1998; Bergamo et al. 2015). These events caused significant impacts on the traffic mobility in the following months, requiring in some cases the construction of temporary bridges with floating piers. The analysis of the effects of scouring on the structural behavior of bridges is a multidisciplinary issue, since hydraulical, geotechnical and structural competences are intertwined and needed for a clear comprehension of the phenomenon. Few studies were indeed conducted about the effects of local scouring on masonry arch bridges, focusing on the validation of the structural behavior of simple physical laboratory models (Invernizzi et al. 2011). This topic has therefore to be exhaustively deepened with further additional studies. For these reasons, this work aims to study the structural response of multi-span masonry arch bridges subject to pier local scouring. The numerical simulation of the local erosion is reproduced with suitable 2-D finite element models, evaluating stresses for different local scouring stages. The analyses describe the evolution of the masonry arch bridge structural behavior during the transient flooding phenomenon, until the achievement of the threshold scouring able to induce the structural failure, which is representative of the performance goal. Results are useful for the calibration of simplified analytical models for multi-span masonry arch bridges affected by pier local scouring.

2 Case study and scour layouts description

A failure analysis was conducted to evaluate the effects of scouring phenomena on the structural response of a multi-span masonry arch bridge. Since masonry bridges realized in the past centuries are characterized by shallow foundational systems, flooding phenomena can cause scouring and consequent settlements that, due to the inductile behavior of masonry, can lead to structural failure. A six-span bridge was considered as case study with a local scouring localized in the central pier: Figure 1 shows its main geometrical characteristics and details of the adopted finite element model. Table 1 lists main mechanical properties adopted in the finite element model. Tensional stress evolution in the arches was monitored with the aim to identify potential kinematic collapse mechanisms.

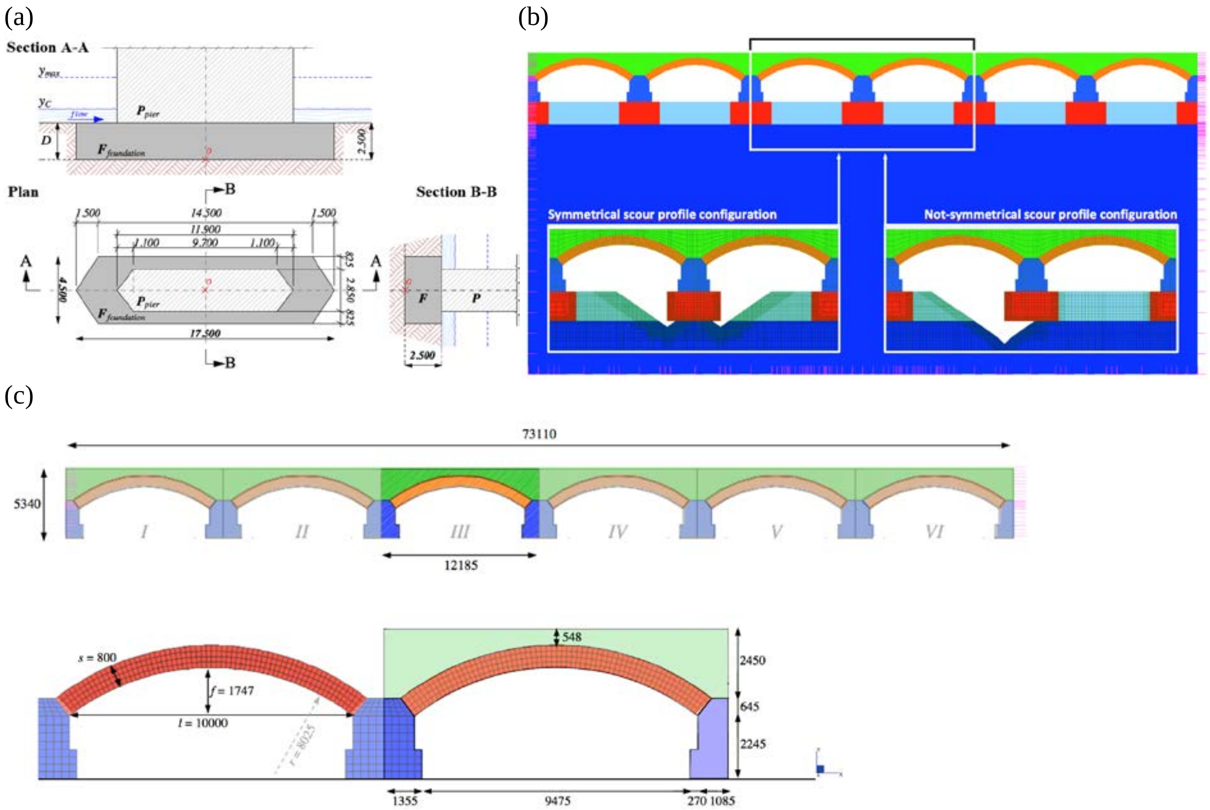

Fig. 1. Main geometrical characteristics of the six-span masonry arch bridge case study (a, c) and investigated scour profile configurations (b).

Table 1. Main mechanical properties for soil, foundation, arches, piers and backfill elements.

Parameter	Soil	Foundation	Arch / Piers	Backfill
Specific weight γ	18 KN/m^3	20 KN/m^3	18 KN/m^3	18 KN/m^3
Elastic modulus E	50 MPa	20000 MPa	4000 MPa	1400 MPa
Poisson's coefficient ν	0.25	0.20	0.20	0.20
Finite element	Plate Quad8	Plate Quad8	Plate Quad8	Plate Quad8
Constitutive law	Elastic-Plastic	Elastic	Elastic-Plastic	Elastic-Plastic
Material model	Mohr-Coulomb	von Mises	Drucker-Prager	Mohr-Coulomb
Friction angle φ	35°	-	64.8°	30°
Cohesion c	0.001 MPa	-	0.43 MPa	0.08 MPa

3 Failure analysis in case of symmetrical local scouring phenomena

Figure 2 shows results obtained for the symmetrical scour case, reporting foundation settlements at the different load steps and evidencing significant scour layout. The first 20 load steps are characterized by invariant load values, whereas in the following load steps scouring induced by flooding is applied by neglecting soil plate elements enclosed by the scour profile, maintaining constant at the same time the service loads. Figure 2 highlights that scour depth is lower than pier foundation height, settlements are negligible (lower than 0.1 mm). When local scour begins to remove soil under the foundation base, settlements starts to be visually detectable (configuration characterized by a 2 mm vertical settlement), reaching the ultimate significant scour profile layout for which a 10 mm foundation settlement was computed. Results were post-processed deriving the relationship between pier vertical reaction and vertical uniform settlement values (Figure 2) and evidencing some significant bridge structural damage configurations. Starting from a foundation vertical reaction of 541 N/mm, a local scour involves a reduction of this value: in particular, the load step 32 is characterized by a negligible foundation settlement of about 0.2 mm involving the null normal stresses and consequent initial cracking at the intrados of the arch sections close to the central scoured pier. Moving to load step 37, foundation settlement is equal to 1.8 mm and cracks extend to the lower half of the arch thickness in the sections previously identified: second cracking phenomena also start to appear at the arches extrados, close to the piers adjacent to the central one. For load steps following the 38th (e.g. the 40th, characterized by a foundation settlement equal to 3.1 mm) cracks at the arch sections close to the central pier are completely affecting arch thickness thus leading to possible sliding phenomena associated to shear actions and the subdivision of the bridge structure into rigid blocks.

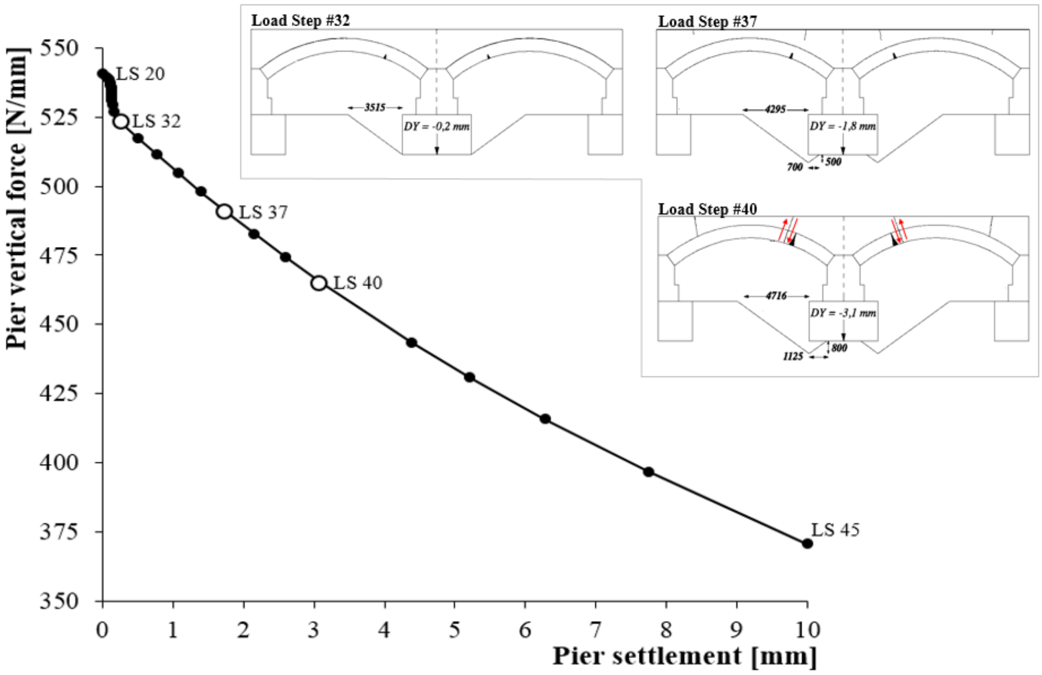

Fig. 2. Incremental scour profile configurations considered for both symmetrical (a) and not-symmetrical (b) scour layouts.

Figure 3 illustrates contour plot and vector main compressive stresses for the 40th symmetrical scouring load step, which evidences three zones subject to specific tensional configurations. Zone 1 is characterized by compressive stress vectors orthogonal to the arch transversal sections, Zone 2 evidences compressive vectors with the highest inclination with respect to arch axis, whereas in Zone 3 compressive stresses are highest at the intrados and null at the arch extrados. Figure 4 summarizes the failure mechanism of the masonry arch bridge derived by the numerical model, which is characterized by two sections close to the settled pier subject to rigid block sliding, other two sections close to the adjacent piers in which hinges are localized at the arch intrados and the pier foundation that losses its vertical restraint.

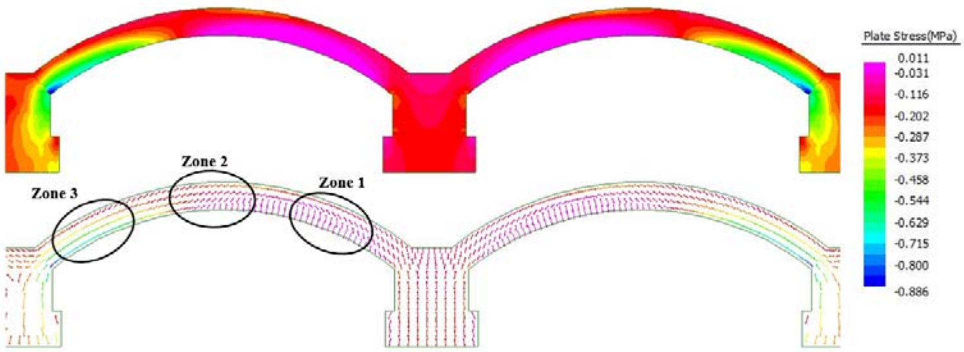

Fig. 3. Contour and main compression stress vectors for the 40th load step of symmetrical local scouring.

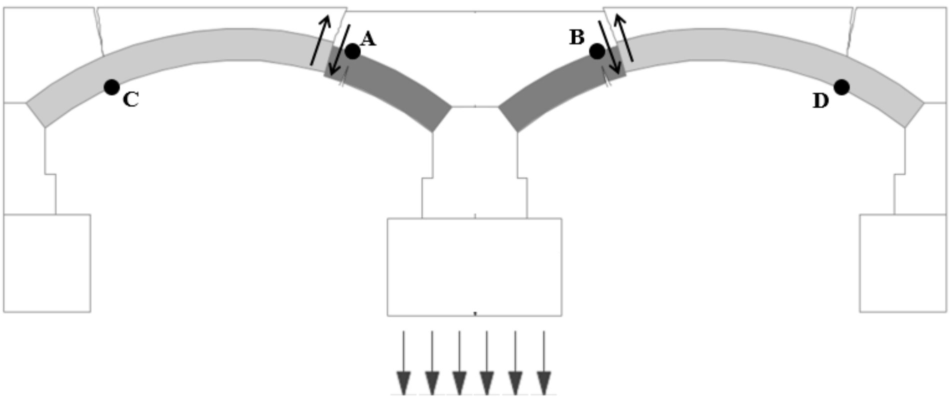

Fig. 4. Failure mechanism for a masonry arch bridge subject to not-symmetrical scouring.

4 Failure analysis in case of not-symmetrical local scouring phenomena

Not-symmetrical scour profiles causes differential settlements that involve vertical displacements and rotations: for this reason, three significant measurement points were considered at the foundation base (left corner, foundation base center and right corner). The same analyses were thus conducted in case of not-symmetrical scour case, reporting foundation settlements related to the foundation base center at different load steps, and evidencing causative scour layouts. If scour depth is lower than pier foundation height, settlements are negligible for all the measurement points (lower than 0.5 mm). When local scour begins to remove soil under the foundation base, settlements starts to be visually detectable: for a 30% reduced foundation base, is characterized by a 5, 4 and 2 mm vertical settlement respectively at left corner, foundation base center and right corner. For the ultimate significant scour profile layout, 10, 7.5 and 4.5 mm foundation settlements were indeed computed respectively at the same measurement points. Results were then post-processed assessing the relationship between pier vertical reaction and vertical uniform settlement values (Figure 5) and evidencing some significant bridge structural damage configurations. Starting also in this case from a foundation vertical reaction of 541 N/mm, a local scour of the left foundation corner causes a reduction of the vertical reaction. The first significant scour profile is represented by the load step 32, characterized by a negligible foundation settlement of about 0.5 mm involving initial asymmetric cracking at the arches intrados. When local scour starts to remove soil under the foundation base left corner, differential settlement become evident and spread first cracks: it is interesting to observe second cracking phenomena at the pier mid-height. Until the 43rd load step the arch is characterized by V/H ratios lower than its threshold value of 0.6. From the 44th load step, cracks are significantly affecting arch thickness and pier section, thus leading to possible sliding phenomena associated to shear actions and the subdivision of the bridge structure into rigid blocks. Figure 6 illustrates contour plot and vector main compressive stresses for the 43rd not-symmetrical scouring load step, which evidences five zones subject to specific tensional configurations. Zone 2 is characterized by compressive stress vectors parallel to the arch transversal sections, Zone 2 evidences compressive stress vectors orthogonal to the arch transversal sections, whereas Zone 3 at the pier left mid-height is characterized by an eccentric axial force with partially resistant cross section.

Figure 7 illustrates the failure mechanism of the masonry arch bridge derived by the numerical model, which is characterized by two asymmetric sections with respect to the settled pier subject to rigid block sliding, two sections close to the adjacent piers in which hinges are localized at the arch intrados, cracking and a consequent hinge at the pier mid-height left. It can be noted how the failure mechanism is quite similar in the arches to the previous symmetrical case, since the vertical component of the differential settlement is predominant with respect to the rotational one: for bridges with slender piers rotational effects can highly affect structural response, leading to a change of the failure mechanism.

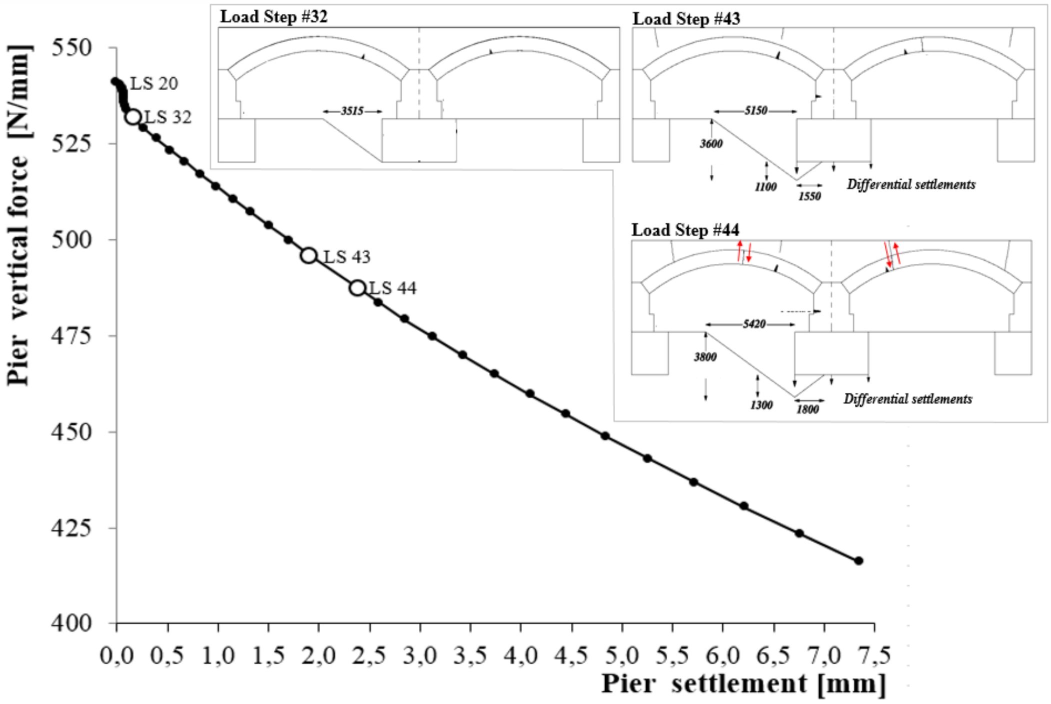

Fig. 5. Pier vertical reaction vs foundational settlement associated to increasing not-symmetrical local scour profiles, for the analyzed soil-structure finite element model.

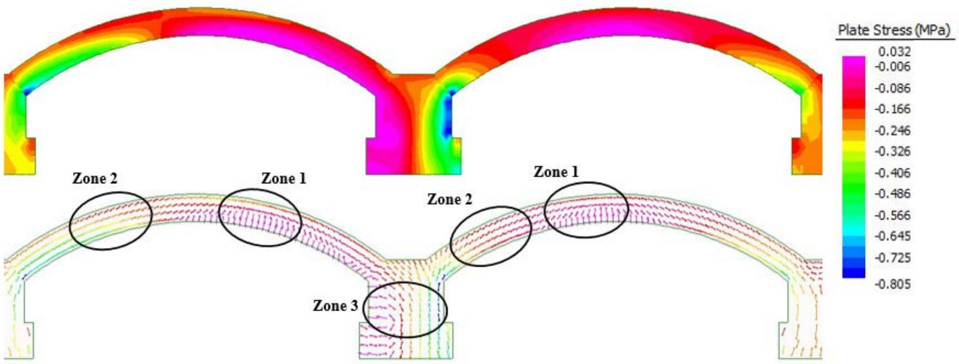

Fig. 6. Contour and main compression stress vectors for the 43th load step of not-symmetrical local scouring.

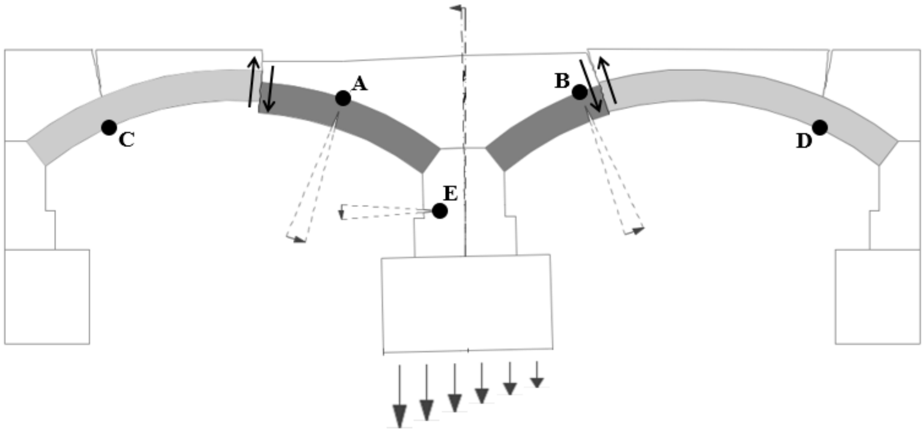

Fig. 7. Failure mechanism for a masonry arch bridge subject to not-symmetrical scouring.

5 Conclusions

In this study a failure analysis was performed on a six-span masonry arch bridge with squat piers subject to local scouring phenomena. The main aim of the work was to try to link consequences of the hydraulic phenomenon of the scour with the structural behavior of the bridge, and to identify the dominant failure mechanism involved by a specific limit local scour profile, considered as performance goal. Suitable 2-D finite element models were developed for the representation of symmetrical and not-symmetrical local erosion profiles, evaluating stresses for different local scouring stages. Non-linear incremental static analyses were performed for increasing local scour load steps, evidencing some significant bridge structural damage configurations. Results showed how for dimensionless scour depth values (d_s/D) lower than 1, scouring has a negligible influence on the structural response of masonry arch bridges, whereas settlements become evident when erosion starts under pier foundation base, inducing cracking phenomena at the arch intrados. The structural system fails when it turns into rigid blocks subject to relative sliding. Cracking phenomena appear at pier mid-height if differential settlements affect pier foundation and are caused by the eccentricity of the pier axial force, whereas in case of uniform settlement distribution at the foundation base, the central rigid block simply vertically yields without pier cracking phenomena.

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Detailed Evaluation of Deteriorated Highway Bridges in Greece

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Abstract. The methods commonly followed for the preliminary evaluation of the condition state of existing structures is that of a Detailed Visual Inspection, based on a nine scale system (Detailed Visual Inspection Manual, Advitam). Condition rating provides a macroscopic visual assessment which enables for further decisions towards detailed assessment or regular maintenance actions. However, in most of the cases, available Detailed Visual Inspection process (DVI) cannot successfully reflect the real in-situ interdependence of multiple degradation (damage) factors and their combined "weighing". Considering the complexity of deterioration modeling of multiple degradation factors, a simplified method for assessing the existing structures that suffer from steel bar corrosion is proposed herein. The method is based on the implementation of predefined Non destructive testing (NDT) for assessing 11 main influencing "material damage (degradation) factors". Their extent is expressed through the Damage Indices (DI) on a four Damage Index scale (1-4). The interdependence of multiple degradation factors can be expressed as the mean value of the summation of all existing DI. Therefore, the weight and the extent of damage due to bar corrosion related degradation is expressed through a final index called Corrosion Damage Index (CDI), expressing the effect but not the cause. Causes of deterioration can be expressed through Exposure Classes according to ELOT EN206-1 or KTS (concrete technology code) 2016. In the same way, each of the exposure classes is presented as a damage index due to environmental aggressivity (EA). Hence, the Structural Index considering the environmental impact (Lifecon can be expressed as the mean value of the summation of CDI and EA. Structural Index expresses the level of damage on a six scale (1-6) system, as well as the value of strength reduction factor a_R (FORM sensitivity factor). The proposed method is presented herein through 6 case studies of existing bridges on a Highway Network.

Keywords: Visual Inspection, Damage Index, Damage Level, Condition Rating, Strength Reduction Factor, Structural Redundancy.

1 Introduction

The aim of the study is to develop a simplified evaluation tool regarding the structural redundancy of deteriorated existing bridges. Considering the difficulties arise from complex reliability models, based on time depended failure function of degradation models, further research has been performed towards an efficient assessment tool proposed for Highway Network in Greece. More than 240 bridges were selected for structural assessment, based on quantitative methods. Such methods utilize instrumented inspections, measuring pre-defined influencing parameters. The main cause of structural damages through degradation processes is that of steel corrosion in concrete. However, corrosion is not a simple degradation process since it can be influenced by many parameters. The following figure presents a common deterioration procedure of a Reinforced Concrete section subjected to rebar corrosion due to (Rodriguez et al. 1995, COIN Project, report 7-2008):

a. Low Concrete Alkalinity.
b. Diffusion of Carbon Dioxide through porous concrete.
c. Diffusion of Chloride Ions.
d. Low Concrete Cover.
e. Bad quality of concrete cover, presence of cracks on surface.
f. Environmental Loads.

It also presents the interdependence of multiple degradation factors causing “damage”(Gonzalez et al. 1995).

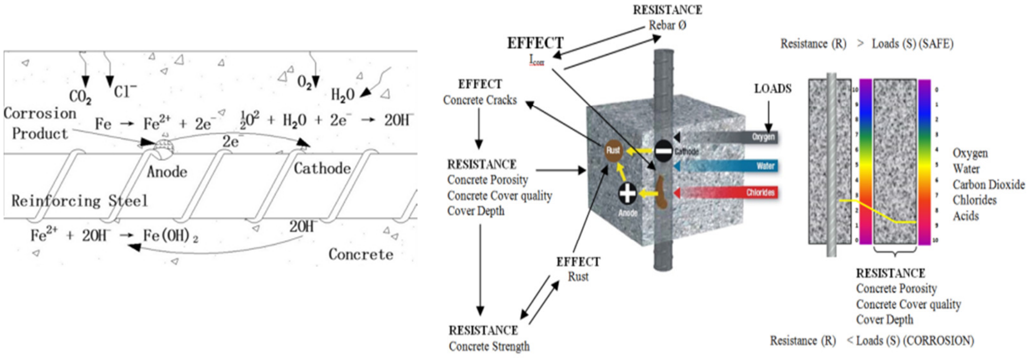


Fig. 1. Interdependence of multiple degradation factors

2 Evaluation Method

The proposed method has been implemented on 6 existing bridges in order to verify the “weighing” of the inspected damages through the Detailed Visual Inspection (Detailed Visual Inspection Manual, Advitam). The existing carrying capacity of the bridges has been determined using an estimated Structural Redundancy Factor (r_{eff}) (Coronelli, 2007) based on the extend of those damages with a service life targeted at 50 years (30 years concession period plus 20 years extension). Damage evaluation has been carried out through a detailed visual inspection by experts (Certified Bridge Inspectors) and all findings were depicted on a detailed damage mapping (dwg file). The initial condition rating of the bridge (CR) was based on a nine scale system (similar to Egnatia Odos Condition Rating Manual) (Condition Rating Manual, Advitam), where the final score comes from the worst damage observed. Yet, the approach of worst damage observed, does not account for the criticality of the component for the overall structure. The proposed method covers the lack of weighing of all damages observed, while the final expression of damage is related to the structural efficiency of the bridge. A detailed study during 2009-2010 has been carried out for more than 220 bridges. All damages were recorded through the detailed visual inspection process on a “dwg” format, providing all necessary information for the condition of the bridge since 2010. Due to lack of previous information regarding the degradation rate of the bridges through time, an initial setting time of degradation monitoring has been set on 2010. The NDT for all bridges covers 11 influencing factors (LIFECON D3.1, 2003) presented in Tables 1 &2.

Table.1. Reinforced Concrete Bridge Damage Indices

No	Damage	Code	Damage Index			
			1	2	3	4
1	Carbonation Depth	CRB	0	<c	=c	>c
2	Chloride Front	CL	0	<c	=c	>c
3	% Cl w.t of concrete (at concrete cover)	CLC	<0.025	0.025<CLC<0.03	0.025<CLC<0.03	CLC>0.04
4	Cracks (RC Concrete)	CR	none	<0.3mm	>0.3	Spalling - cavities - large

						cracks
5	Concrete Resistivity (KΩcm)	**R**	>20	10< R<20	5<R<10	R<5
6	Half - Cell (mV)($Cu/CuSO_4$)	**HC**	0-220	220-350	350-450	>450
7	Corrosion Rate (Icorr, LPR)	**CORR**	<0.1	0.1-0.5	0.5-1	>1
8	Ømm (Rebar Diameter)	**D**	>18mm	18mm-16mm	16mm-12mm	<12mm
9	% rebar mass loss	**RD**	0-2	2-5	5-10	>10
10	Concrete Moisture Content ,EMC (%)	**MST**	<2	2-4	4-6	>6
11	Concrete Strength	**CS**	≥C25/30	C20/25	C16/20	C12/15

Table.2. Prestressed Concrete Bridge Damage Indices.

No	**Damage**	**Code**	**Damage Index**			
			1	**2**	**3**	**4**
1	Carbonation Depth	**CRB**	0	<c	=c	>c
2	Chloride Front	**CL**	0	<c	=c	>c
3	% Cl w.t of concrete (at concrete cover)	**CLC**	<0.025	0.025<CLC<0.03	0.025<CLC<0.03	CLC>0.04
4	Cracks (PS concrete-mm)	**CR**	<0.05mm	0.05-0.3mm	0.3-1mm	1-3mm
5	Concrete Resistivity (KΩcm)	**R**	>20	10< R<20	5<R<10	R<5
6	Half - Cell (mV)($Cu/CuSO_4$)	**HC**	0-220	220-350	350-450	>450
7	Corrosion Rate (Icorr, LPR)	**CORR**	<0.1	0.1-0.5	0.5-1	>1
8	Ømm (Rebar Diameter)	**D**	>18mm	18mm-16mm	16mm-12mm	<12mm
9	% rebar mass loss	**RD**	0-2	2-5	5-10	>10
10	Concrete Moisture Content ,EMC (%)	**MST**	<2	2-4	4-6	>6
11	Concrete Strength	**CS**	≥C25/30	C20/25	C16/20	C12/15
12	Tendon Corrosion	**TC**	Low	Moderate	High	Possible Failure

$$DI = \frac{\sum_{i=1}^{n} Damage\ Weighing}{Damage\ No\ (n)} \ , \qquad (1)$$

Where n – Total number of damages.

Procedure for assessing Tendon Corrosion Probability:

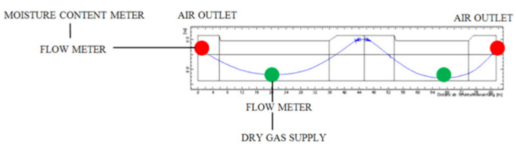
Potential of Corrosion can be identified through Half Cell readings close to tendon Duct (Metallic). Further investigation should be carried out regarding the grouting chloride content by sampling (Specified No of samples according to tendons No, see fig 2). Damage Index due to chloride presence $DI_{(Cl)}$ is defined for different chloride levels in table 3.

Fig.2. Tendon Inspection and Grouting Sampling (Cl^{-})

Table.3. Damage Indices due to Chloride Presence in grouting.

Chlorides (% w.t cement grout)	<0.08	0.08<Cl≤0.20	0.2<Cl≤0.40	Cl>0.40
Damage Index $DI_{(Cl)}$	**1**	**2**	**3**	**4**

The second inspection of Tendons is related to grouting condition (voids, moisture), where a Damage Index defines the combination of possible voids and humidity (presence of water inside ducts). Hence, an inlet is placed on the lower point of tendon profile and two outlets at the upper points at equal distances. A Dry Nitrogen gas (0% RH) is supplied from a nozzle at the lower point, while a flow meter records the flow inside the ducts. Gas moves upwards towards outlets, where a flow and gas moisture content is recorded. The following figure presents the procedure should followed in order to assess internal tendon moisture and voids.



Tendon Corrosion (TC)	Damage Index (TC)
Low	1
Moderate	2
High	3
Possible Failure	4

Fig. 3. Method of Assessing Probability of Tendon Corrosion using Dry Nitrogen

Table.4. Damage Indices PT Evaluation Scale.

$DI_{(PT)}$ Evaluation Scale	Measure	Condition	Potential for Corrosion
1	No Air Flow	No Air Flow	Unknown
2	<0.3%	Dry	Low
3	0.3 -0.7 %	Moist	Moderate
4	>0.7%	Wet	High

The average of Damage Index $DI_{(Cl)}$ and Damage Index $DI_{(PT)}$, gives the Damage Index of Tendon Corrosion $DI_{(TC)}$.

$$DI_{(TC)} = \frac{DI_{(Cl)} + DI_{(PT)}}{2}$$

The cause of damage can be investigated on the micro-environment of each deteriorated element. The weighing of the environmental aggressivity (EA) can be expressed according to exposure classes presented on EN206-1.

Table.5. Exposure Classes according to EN206-1

Class	**X0**	**XC1**	**XC2**	**XC3**	**XC4**	**XD1**	**XD2**	**XD3**	**XS1**	**XS2**	**XS3**
Weight (EA)	0	1	1	2	3	2	3	4	2	3	4

The final calculation of Structural Damage Index (SDI) is made, by averaging the weight of the exposure class with the actual Damage Index (DI) .Therefore, SDI (Table 6, Fig 4) provides sensitivity on damage evaluation by considering both cause and effect of a damage. The mathematical expression of SDI is:

$$SDI = \frac{DI+EA}{2} \tag{2}$$

	Damage Level	Damage Discription	SDI	α_R
	I	No Damage	0-0.65	0.3
	II	Low	0.65-1.20	0.4
	III	Mean	1.20-1.90	0.5
	IV	High	1.90-2.55	0.6
	V	Very High	2.55-3.5	0.7
	VI	Critical Damage Assessment	>3.5	0.8

Fig.4. Damage Levels

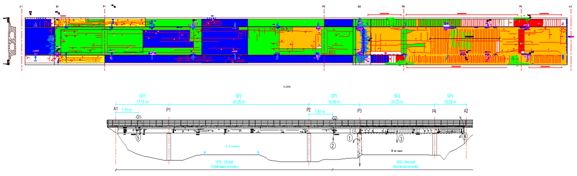


Fig.5. Damage Levels on existing Bridge

According to Fig.4. SDI expressing the damage level and α_R is a sensitivity factor (sensitivity factor FORM(First Order Reliability Analysis) related to damage level (SDI). The determination of structural redundancy due to SDI can be evaluated by the following formula (Coronelli, 2007):

$$r_{eff} = Be^{-\alpha_R \beta V_R} \tag{3}$$

Where B – Overstrength Factor (Existing/Demand), a_R – sensitivity factor FORM, V_R – Coefficient of Variation., β – Target Reliability Index, EN1990 for bridges (Design) β=4.5. Detailed calculations of the SR, SCI, α_R and r_{eff} for case studies 1 to 6 are given below.

3 Case Studies

Case Study 1: Structural Rating (SR) – **2 (9 scale)**

Total Damage Level– Low– SCI 1.15 - α_R=0.4

Initial (SR): 2 (Assessment)
Very poor condition.
Further Assessment.

Calibrated (SR): 7
B = 1.2, r_{eff} = 1.12
Bridge is safe for operation

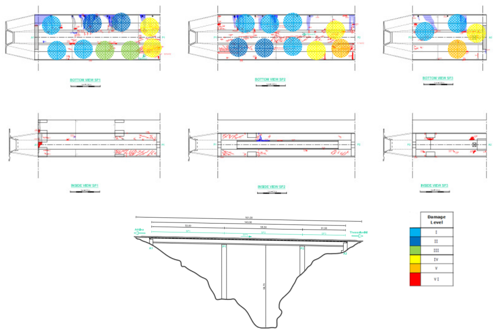


Fig.6. Bridge No1

Case Study 2: Structural Rating (SR) - **3**

Total Damage Level– Medium– SCI 1.65 - α_R=0.5

Initial (SR): 3 (Assessment)
Very poor condition.
Further Assessment.

Calibrated (SR): 5
B = 0.80, r_{eff} = 0.68
Retrofitting of specific members

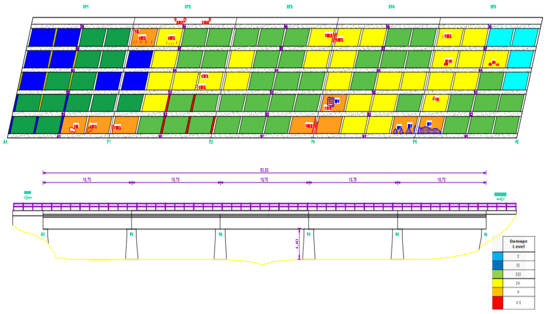
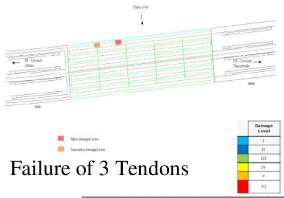
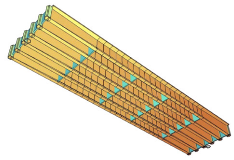


Fig.7. Bridge No2

Case Study 3: Structural Rating (SR) – **2** Total Damage Level– High– SCI 2.45 - α_R=0.6



Failure of 3 Tendons



Model

(safety factors according to EN 1990 / EN 1992-2)						
Capacity	20006	18803	18803	18803	18803	16087
Required (DIN 1072 loads)	19231	18563	17139	16517	15807	15301
utilization factor	96%	99%	91%	88%	84%	95%
Required (military vehicle loads)	17240	16615	15448	15497	15430	15353
utilization factor	86%	88%	82%	82%	82%	95%

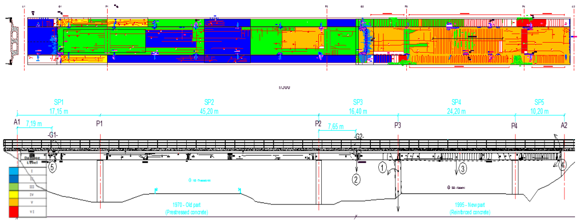
Estimation of the over strength factor B

Initial (SR): 1 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

Calibrated (SR): 5
B = 1.20, r_{eff} = 0.85
Bridge Safe for operation.
Retrofitting under seismic loads.

Fig.8. Bridge No3

Case Study 4: Structural Rating (SR) - **2** Total Damage Level– High– SCI 2.20 - α_R=0.6

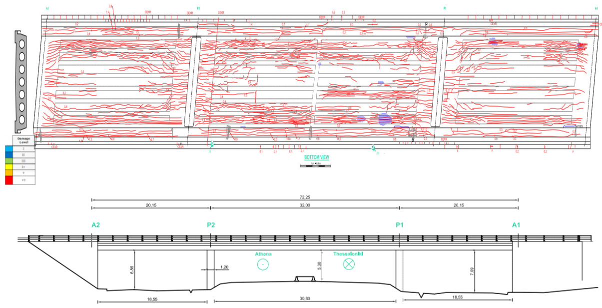


Initial (SR):2 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

Calibrated (SR): 4
B = 1.10, r_{eff} = 0.78
Bridge Safe for Class 30.
Retrofitting of Bridge

Fig.9. Bridge No4

Case Study 5: Structural Rating (SR) - **3** (AAR*) Total Damage Level– Medium– SCI 1.70 - α_R=0.5



Initial (SR): 3 (Assessment)
Very poor condition.
Further Assessment
Traffic Restrictions

Calibrated (SR): 6
B = 2.10, r_{eff} = 1.58
Bridge Safe for operation
Repair AAR cracks

*AAR – Alkali Silica Reaction

Fig.10. Bridge No5

While the overall structural rating may be a tool that enables for decision making and prioritization of structural interventions among large number of bridges, r_{eff} may be used for calculation at the whole bridge level, at critical components level or for local checks (different r_{eff} at each level and region of the bridge). Figures 5 to 10 provide the assessment results for 6 bridges based on the commonly used 9-scale condition rating and on the proposed method.

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Reliability of existing bridges determined with physical models – chloride induced corrosion

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Abstract:

Management of bridges includes activities which aim to optimize the use in a manner that maximizes the benefits while satisfying proscribed requirements over a predetermined period of time. Forecasting long-term performance of bridges with deterioration models is a crucial component in any management strategy. To cope with the performance forecasting, a model in use should describe the process of degradation and allow prediction of condition of structure in time, taking into consideration environmental surrounding, nature of use and maintenance actions. Deterioration models can basically be divided into mathematical (statistical), empirical and physical models. Statistical models are formed by analysing data that describe condition of a greater number of bridges, empirical models are based on experience, while physical models are based on knowledge and modelling of damage-causing processes. The main aim of this manuscript is to present the physical model of chloride ingress in evaluation of performance of existing reinforced concrete bridges. Special attention was given to accessibility of material and environmental parameters which govern chloride ingress formulation. In the end, analysis was performed in order to present the achieved reliability of reinforced concrete bridges for combination of different exposure classes and material characteristics.

Keywords: *bridge performance, physical models, chloride ingress, reliability index*

1 Introduction

The research presented in this manuscript was done as a part of a LeCIE – “Life-Cycle Assessment for Railway Structures” project, currently being realized at the University of Natural Resources and Life Sciences in Vienna under auspices and for the needs of ÖBB-Infrastruktur – Austrian Federal Railways. The objective of the project is to develop a comprehensive concept for an anticipatory life cycle management system for engineering structures in railway systems. This thorough approach is to link inspection results relating to damage symptoms and damage processes with probability-based degradation predictions and forecasting methods and with monitoring and assessment methods. Presented part of research is connected with the probability-based degradation predictions.

The decision of modeling deterioration by using physical models was due the reason that mathematical (statistical) deterioration models were excluded under the assumption of input data based on visual inspections rating being prone to subjectivity of engineers performing inspections. Deterioration processes implemented in LeCIE concept are primarily chloride and carbonation induced corrosion, freeze-thaw, alkali-silica reaction and fatigue. This contribution will focus on chloride ingress as an initiation phase of chloride induced corrosion.

To best describe a physical deterioration process, significant attention and effort needs to be pointed in characterization of parameters which influence particular deterioration process. The quality and accuracy of input parameters is of great significance for the realistic and useful prediction.

Since this study was undertaken in Austria, it is important to mention that the analysis was made only for the exposure classes related to chlorides from sources other than sea water, in other words only for exposure classes XD.

2 Chloride ingress

A full probabilistic design approach for the modelling of chloride induced corrosion in uncracked concrete presented in this manuscript, has been developed within the research project DuraCrete and slightly revised in the research project DARTS, each project was funded by the European Union. It is based on the limit state equation Eq. (1) in which the critical chloride concentration is compared to the actual chloride concentration at the depth of the reinforcing steel. The model is applicable both to marine environment and for de-icing salts on roads/bridges.

$$g(c,t) = C_{c} \quad - C(c,t) \tag{1}$$

The model is based on Fick's 2nd law of diffusion, takin into account that most observations indicate that transport of chlorides in concrete is diffusion controlled. Fick's 2nd law for diffusion was first proposed for application in chloride exposed structures by M. Collepardi in 1970 (Collepardi , Marcialis, & Turriziani, 1972).

The content of chlorides at corresponding depth *x* (usually for *x* the value of the depth of reinforcing steel *c* is considered) is given by Eq. (2):

$$C_x(x,t) = C_0 + \left(C_{S,\Delta x} - C_0\right) \cdot \left[1 - e \quad \frac{x - \Delta x}{2 \cdot \sqrt{k_e \cdot D_{R} \quad _{,0} \cdot \left(\frac{t_0}{t}\right)^{\alpha} \cdot t}}\right] \tag{2}$$

where:

C_0 initial chloride content;
$C_{S,\Delta x}$ chloride content at depth Δx;
x depth with a corresponding content of chlorides ($x = c$ – depth of reinforcement);
Δx depth of convection zone;
k_e environmental variable (considers temperature);
$D_{RCM,0}$ apparent diffusion coefficient (rapid chloride migration test);
t_0 reference point of time;
α ageing exponent;

$$k_e = e \quad \left[b_e\left(\frac{1}{T_r} - \frac{1}{T_r}\right)\right] \tag{3}$$

where:

b_e temperature coefficient (proportional to activation energy) [K]
T_{ref} reference temperature [K]
T_{real} temperature of the structural element or the ambient air [K]

For the quantification of reliability indices the parameters presented in the **Table 1** were used. Fixed parameters were obtained from the literature and/or chosen for specific conditions in Austria. Varied parameters are those which are hard to access and therefore different combinations of them were made.

Table 1. Fixed and varied parameters used for reliability analysis

Fixed parameters						
Parameter	**Unit**	**Distribution**	**Mean**	**Standard Deviation**	**a**	**b**
t_0	years	Constant	0,0767	-		
t	years	Constant	50	-		
T_{ref}	K	Constant	293	-		
T_{real}	K	Normal	281	5		
b_e	K	Normal	4800	700		
C_{crit}	wt.-%/c	Beta	0,60	0,15	0,2	2,0

Varied parameters			XD1	XD2	XD3	XD1	XD2	XD3		
Parameter	**Unit**	**Distribution**	**Mean**			**Standard Deviation**			**a**	**b**
α	-	Beta	0,65	0,30	0,30	0,12			0	1
$C_{S,\Delta x}$	wt.-%/c	Normal	0,5-1,5	2-4	2-4	μ * 0,75				
c	mm	Normal	40-60			10				
Δx	mm	Constant	0	0	10	0	0	5	0	50
$D_{RCM,0}$	m²/s	Normal	8,9-25,0			μ * 0,20				

3 Accessibility of parameters of existing bridges

When performing an evaluation of an existing bridge, even by carrying detailed study of documents, reviews and other evidences it is hard to assess and completely understand the environmental and material characteristics of the bridge. For older bridges in most cases design documents have not been preserved, and when they were they contain scarce data which cannot be used for durability related analysis. One of the future ways to deal with the problem of scarce data is introduction of birth certificate document (BCD). BCD is an extract of as-built documentation which contains information such as: cover thickness to the reinforcement, diffusion coefficient for the concrete cover and so on (fib Model Code for Concrete Structures 2010, 2010).

Hard accessible parameters important for analysis of chloride ingress and thus analysis of chloride induced corrosion are mainly cement type, w/c ratio and chloride content at the surface. The type of cement and w/c ratio are the two parameters which strongly affect the pore structure of concrete and hence, its potential chloride migration coefficient (Benchmarking of deemed-to-satisfy provisions in standards: Durability of reinforced concrete structures exposed to chlorides; fib Bulletin 76, 2015).

The amount of cement, a minimum which provides sufficient guaranteed compaction as well as type, shape and size of aggregate are less important.

The use of cement types in past can be accessed from literature or roughly estimated in cooperation with cement and concrete producers and providers. In the best case this estimation provides notion of extent of use of cement types in particular geographical regions, which still does not indicate which particular cement was used for which bridge.

Originally used w/c ratio for construction of specific bridge can hardly be known. In best case, w/c ratio can be restricted between limiting values. Generally it is considered that in the past, the value of w/c lower than 0,4 was not used due to technical reasons. For the upper limits of w/c ratio the survey of historical guidelines and norms has to be performed. Usually, restriction of upper limits of w/c ratio is connected with specific exposure conditions (mainly in newer norms) and quality of concrete needed due to structural demands.

As indicated in (von Greve-Dierfeld & Gehlen, 2016) the chloride content at exposed concrete surface $C_{s,0}$ as well as the substitute surface content $C_{S,\Delta x}$ are variables that depend on material properties and on geometrical and environmental conditions. An adequate quantification of the potential chloride impact turns out to be very complex, as for bridges that are subjected to chloride impact due to de-icing salt, the variables describing the amount of de-icing salt applied are hard to quantify because of seasonal loading. Under real exposure conditions the chloride surface content varies randomly due to the variations in the chloride concentrations of ambient solution, frequency of application of de-icing salts, temporal and spatial variations in the humidity conditions of the concrete and so forth.

To cover the possible cases of chloride content at the depth of convection zone, the analysis was performed for maximal amount for exposure class XD1 ($C_{S,\Delta x}$ = 1,5), and for minimal, average and maximal amount for exposure classes XD2 and XD3 ($C_{S,\Delta x}$ = 2; $C_{S,\Delta x}$ = 3 and $C_{S,\Delta x}$ = 4).

4 Achieved reliability

In the case of chloride induced corrosion, it is assumed that the depassivation of the reinforcement occurs as soon as a critical, corrosion inducing chloride content C_{crit} has reached the depth of the reinforcement. Or reformulated, if the content of chlorides at the depth of reinforcement (action) $C(c,t)$ is higher than corrosion-inducing chloride content (resistance) C_{crit}.

Actions and resistance are uncertain quantities, not deterministic ones. They are therefore introduced as random variables and contrasted in the limit state function $g(c,t)$, as presented in Eq. (1). The difference between action and resistance will then also be a random variable. Since the action and resistance are not normal distributed random variables, it is not possible to perform an analytical calculation of probability of failure p_f. For this purpose the Monte Carlo method with 10^6 simulations was performed. In this case, the reliability index β was stated instead of the probability of failure p_f.

For obtaining the indices of reliability β the parameters presented in **Table 1** were used, and the results are presented in the **Table 2**.

Table 2. Obtained indices of reliability β

CEM I								
w/c [%]	**c_{nom} [mm]**	**XD1**	**XD2**			**XD3**		
		$C_{S,\Delta x}$ = 1,5	**$C_{S,\Delta x}$ = 2**	**$C_{S,\Delta x}$ = 3**	**$C_{S,\Delta x}$ = 4**	**$C_{S,\Delta x}$ = 2**	**$C_{S,\Delta x}$ = 3**	**$C_{S,\Delta x}$ = 4**
0,40	**50**	3,1	2,8	-0,3	-0,4	1,8	-0,5	-0,6
	45	2,8	2,4	-0,4	-0,5	1,3	-0,5	-0,7
	40	2,4	1,9	-0,4	-0,6	0,9	-0,6	-0,7
	35	2,0	1,5	-0,5	-0,7	0,5	-0,7	-0,8
	30	1,5	1,1	-0,6	-0,8	0,2	-0,8	-0,9
0,45	**50**	3,0	2,6	-0,3	-0,5	1,6	-0,5	-0,6
	45	2,6	2,2	-0,4	-0,5	1,2	-0,6	-0,7
	40	2,2	1,8	-0,5	-0,6	0,8	-0,7	-0,8
	35	1,8	1,3	-0,6	-0,7	0,4	-0,8	-0,8
	30	1,4	0,9	-0,7	-0,8	0,1	-0,8	-0,9
0,50	**50**	2,3	1,8	-0,5	-0,6	1,1	-0,6	-0,8
	45	1,9	1,5	-0,6	-0,7	0,7	-0,7	-0,8
	40	1,6	1,1	-0,6	-0,8	0,4	-0,8	-0,9
	35	1,2	0,8	-0,7	-0,8	0,2	-0,8	-0,9
	30	0,9	0,5	-0,8	-0,9	-0,1	-0,9	-1,0
0,55	**50**	2,0	1,5	-0,6	-0,7	0,8	-0,7	-0,8
	45	1,6	1,2	-0,6	-0,8	0,6	-0,8	-0,9
	40	1,3	0,9	-0,7	-0,8	0,3	-0,8	-0,9
	35	1,0	0,6	-0,8	-0,9	0,1	-0,9	-1,0
	30	0,7	0,3	-0,8	-0,9	-0,2	-0,9	-1,0
0,60	**50**	1,6	1,2	-0,6	-0,8	0,6	-0,8	-0,9
	45	1,3	0,9	-0,7	-0,8	0,4	-0,8	-0,9
	40	1,0	0,7	-0,8	-0,9	0,1	-0,9	-0,9
	35	0,8	0,4	-0,8	-0,9	-0,1	-0,9	-1,0
	30	0,5	0,2	-0,9	-1,0	-0,3	-0,9	-1,0

**Grey assigned cells are the cells with index of reliability lower than targeted*

In current specifications the target reliabilities are between 0,5 ≤ β ≤ 2,3. Most of the current standards and specifications distinguish reliability with respect to the consequence of failure (consequence class CC) and measures for increasing safety (Model Code for Service Life Design of Concrete Structures, *fib* Bulletin 34, 2006).

The consequence of chloride ingress and subsequent depassivation of the reinforcing steel may be assessed by, for example, the corrosion rate after depassivation. For exposure classes XD1 and XD2 (i.e. predominantly dry or wet conditions) the consequence of depassivation is not expected to be significant because corrosion rate is limited either due to the lack of moisture or the lack of oxygen. However, in exposure class XD3 higher corrosion rates are expected because it is likely that sufficient moisture and oxygen are present (von Greve-Dierfeld & Gehlen, 2016).

The following values of target reliability β at the age of 50 years are proposed:

- $\beta \geq 1{,}5$ for bridge elements exposed to exposure class XD3
- $\beta \geq 0{,}5$ for bridge elements exposed to exposure class XD1 and XD2

5 Conclusions

In the presented manuscript the physical model of chloride ingress was used for the evaluation of reliability of existing reinforced concrete bridges. Attention was pointed to problems of accessibility of material and environmental parameters which govern chloride ingress formulation. An analysis was performed in order to present the achieved reliability of reinforced concrete bridges for combination of different exposure classes and material characteristics. Achieved reliability indices were compared with the targeted for the bridges at the age of 50 years for the cement type CEM I. The reliability indices higher than targeted were obtained for almost all of the combinations of exposure class XD1 and for exposure class XD2, for the minimal $C_{S,\Delta x}$. On contrary, the reliability indices lower than targeted were obtained for exposure class XD3 for almost all the combinations of w/c ratio and depth of reinforcement c_{nom} and for exposure class XD2 for the average and maximal $C_{S,\Delta x}$.

From the presented results it can be concluded that for the concrete structures made from concrete with cement type CEM 1 special attention should be given for the exposure classes XD2 and XD3.

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Quality control of masonry bridges based on empirical influence lines of displacements

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Abstract. Quality control of masonry bridges carried out through measurements of arch barrel deformation generated by their typical service loads is analysed. Attention is paid to displacements of characteristic point of the structure i.e. vertical deflection of the arch crown section. In the study measurements of deflection carried out on two masonry arch bridges in Poland are used. On the basis of the direct results empirical influence functions of displacements are being created while considering also various speeds of loading vehicles, the empirical influence lines are developed. Careful comparison of the experimental and numerical outcomes shows potential of field tests carried out under live loads to be a useful tool of masonry bridge numerical models calibration and control of the structures' condition.

Keywords: bridge, live loads, influence line, FEM, quality control

1 Introduction

Masonry arch bridges are complex structures characterised by many parameters being sometimes difficult to determine having however an important influence on their mechanical performance. These technical parameters cover both the material properties of the structural components as well as their geometrical characteristics including those hidden by the backfill and inaccessible for the direct measurements. Determination of them may be done during diverse laboratory tests as well as by means of site NDT or SDT methods. Therefore a reasonable quality control of the masonry bridges becomes a demanding and time-consuming task especially in case of no technical data on the structure available.

A simpler solution to the problem may be a careful study on the structure deformation under live loads. Especially in case of the railway bridges the that kind of analysis can be effective taking into account regularity of the exploitation load scenarios however the road structures may undergo analogical procedure as well. The main idea of the approach is related to measurements of the structure displacements on site. An important advantage is a possibility to carry out the tests during regular exploitation of bridges – without any disturbance to the traffic (Kamiński & Bień 2015). Even if the measurements are limited to a single point of the structure, like the midspan arch deflection presented within this study, the obtained results can provide comprehensive information on the structural response to many independent loading cases. Thus, in this way that kind of tests may be an efficient tool of a masonry bridge model calibration verifying it in a global way. The proposed analysis is presented in two case studies of arch bridges with a similar structure. Effects of various railway vehicle crossing them with different speed are recorded.

2 Experimental analysis of masonry bridge displacements

2.1 Analysed structures

In the analysis two similar single-span brick railway bridges are considered (Fig. 1) with structure, technical condition and age representative for the bridges in this part of Europe. Both of them are located along Polish railway line no. 281: in Oleśnica and Milicz. The bridge in Oleśnica have semi-circular arch barrel with intrados radius R = 4,97 m and thickness h = 0,78 m (assumed theoretical span length L_t = 10,72 m). The bridge in Milicz is differing with intrados radius R = 6,0 m and thickness h = 0,80 m (thus L_t = 12,80 m). The structures have the same width equal to B = 8,55 m and both are dating back about 1875 (Kamiński 2008b).

a) 

b) 

Fig. 1. Analysed bridges during the tests: a) in Oleśnica, b) in Milicz

2.2 Applied live loads

Among all types of transport vehicles a locomotive has the most regular, invariable in time and easy to determine loading parameters including values of axle loads and spaces between the axles. Wagons of trains can be more diverse regarding these parameters, therefore only effects of locomotive's action crossing the bridge are considered within the presented study. The applied locomotive types and their technical parameters are presented in Tab. 1 and described in Fig. 2 for a representative vehicle.

Table 1. Characteristics of locomotives and their passages across the bridges.

Bridge	Passage	Locomotive type	Axle load P [kN]	Bogie type	Space between bogies d [m]	Spaces between axles [m]			Speed v [m/s]
						a	b	c	
Oleśnica	V11	ET22	200	Co'Co'	10,30	1,75	2,720	6,80	10,8
	V10	ET22	200	Co'Co'	10,30	1,75	2,720	6,80	10,0
	V13	EU07	196,2	Bo'Bo'	8,55*	3,05*	2,317	5,50	12,9
Milicz	V16	E31	203	Co'Co'	10,95	2,40	1,525	6,15	15,6
	V20	Dragon	202,2	Co'Co'	10,50	1,95	2,965	6,60	19,8

* 4-axle locomotive

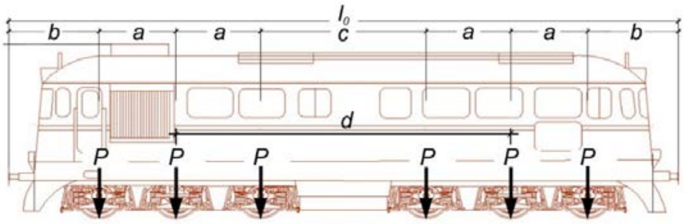

Fig. 2. Dimensions of a representative locomotive.

2.3 Testing procedure

The applied testing procedure is based on measurements of the arch barrel deflection during passages of the locomotives across the bridge (Machelski 2014). Results of deflection measurements used within the study were collected by means of various types of gauges including LVDT gauges, laser distance sensor and microradar equipment (Helmerich et al. 2012). The controlled points are located on intrados of the arch barrel in the midspan (crown cross-section). An example of the recorded deflection $w(x)$ as a function of the front (reference) bogie axis position x against the arch crown section for the bridge in Oleśnica during passage of ET22 locomotive is shown in Fig. 3. The two extremes in the figure indicated when the front and the rear locomotive bogie axes correspondingly are over the arch crown section. Effects of individual three axles of each bogie is invisible due to dispersion of the loads within the track and the bridge structure. When $x = 0$ the reference bogie is located centrally over the arch midspan and the other bogie is in the distance d from the midspan what corresponds to the asymmetrical position (A). For $x = d/2$ both bogies are equally distant from the midspan and the locomotive takes the symmetrical position (S). On the basis of the time difference corresponding to the extremes of $w(x)$ and the distance between bogies the average speed of the locomotive crossing a bridge is evaluated (see Tab. 1).

3 Empirical influence functions of displacements

The diagrams $w(x)$ (given in Fig. 4 and Fig. 5 for the bridge in Oleśnica and Milicz correspondingly) may be further used to create empirical influence functions of the arch crown deflection $\xi(x)$. For this purpose constant axle load values P given in Tab. 1 for each case are assumed. Accordingly, a general relationship between the arch crown deflection $w(x)$ and ordinates $\xi(x)$ of the influence function is expressed by a formula:

$$w(x) = P\sum_{i=1}^{n}\xi(x + x_i) \tag{1}$$

where: x – location of the locomotive reference axle against the arch midspan section, x_i – location of the consecutive locomotive axles i against the reference axle, n – number of locomotive axles.

To find the values of the influence function $\xi(x)$ a progressive calculation procedure is applied starting from the point $x = x_0$, for which the measured deflection is equal to $w(x_0)$, while for all previous points laying at least in distance a away from x_0, it is equal to $w(x_0 - a) = 0$. Thus, at the beginning of the analysis the initial position of the first axle load is considered as follows:

$$w(x_o) = P \cdot \xi(x_0 + a)) \tag{2}$$

From the formula (2) the function value $\xi(x_0+a)$ can be calculated taking into account $\xi(x_0) = 0$. The second function ordinate is being calculated for a point in distance a from the previous one, according to the formula:

$$w(x_o + a) = P\left[\xi(x_o + a) + \xi(x_o + 2a)\right] \tag{3}$$

Further procedure carried out with subsequent positions of the locomotive defined by $x = x_0 + i{\cdot}a$ allows to find the next ordinates of the influence function $\xi(x)$.

Fig. 3 presents the shape of the deflection influence function $\xi(x)$ calculated according to the described procedure for the structure in Oleśnica on the basis of ET22 locomotive crossing the bridge alone. For the compatibility of the units with the diagrams $w(x)$ the ordinates ξ are multiplied by axle load P, treated here as a constant factor. In case of a single locomotive crossing a bridge the confirmation of the influence function $\xi(x)$ correctness should be an agreement between diagram $w_\xi(x)$ developed backward from $\xi(x)$ with the diagram of the directly measured deflections $w(x)$ – as it is presented also in Fig. 3 – according to the formula:

$$w(x) \cong w_\xi(x) = P\sum_{i=1}^{n}\xi(x + x_i) \tag{4}$$

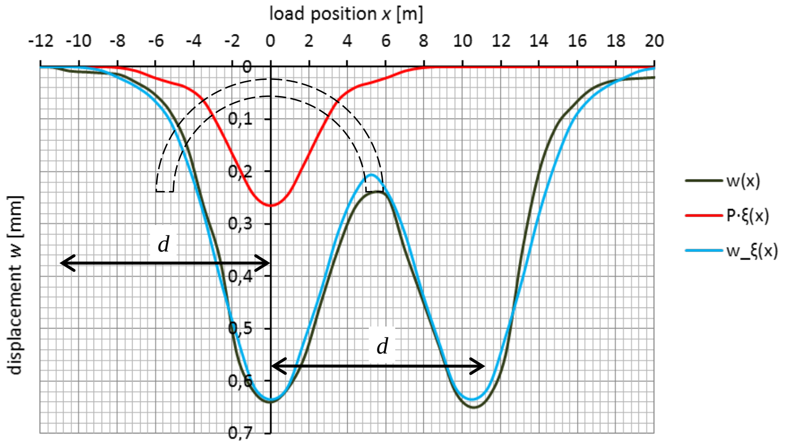

Fig. 3. Comparison of the arch crown deflections of the bridge in Oleśnica (generated by ET22 locomotive) directly measured during the tests $w(x)$ and calculated $w_\xi(x)$ from the influence function $\xi(x)$ (presented rescaled by force P).

The lack of a prefect agreement between the diagrams $w(x)$ and $w_\xi(x)$ can arise from many reasons including: imprecision of measurements, different real axle loads, variable speed of the locomotive or nonlinear behaviour of the real structure (Machelski 2014).

Analogical diagrams are presented in Fig. 4 for the bridge in Milicz also on the basis of ET22 locomotive passage. In this case additional influence of wagons following the locomotive is visible in the diagram $w(x)$ for $x > d/2$.

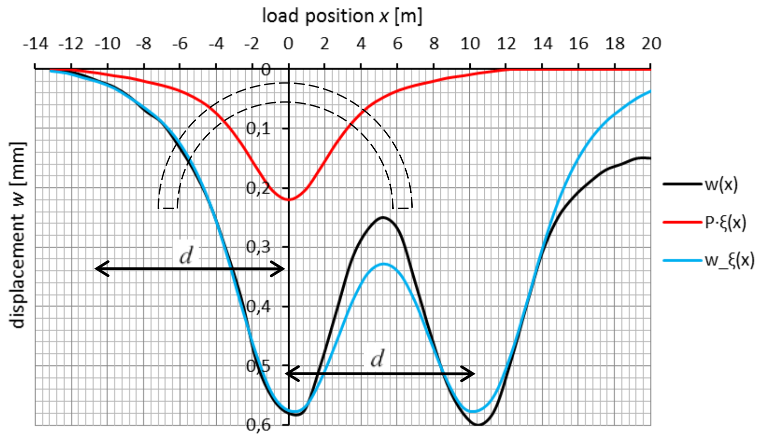

Fig. 4. Comparison of the arch crown deflections of the bridge in Milicz (generated by ET22 locomotive) directly measured during the tests ($w(x)$) and calculated ($w_\xi(x)$) from the influence function $\xi(x)$ (presented rescaled by force P).

4 Influence lines of the arch crown displacements

Influence functions of deflection $\xi(x)$ presented in the previous chapter are calculated on the basis of the arch deflections generated by selected single runs of the vehicles. In Fig. 5 influence functions of deflection $\xi(x)$ determined by means of all locomotives travelling with various speeds are given together for the bridge in Milicz.

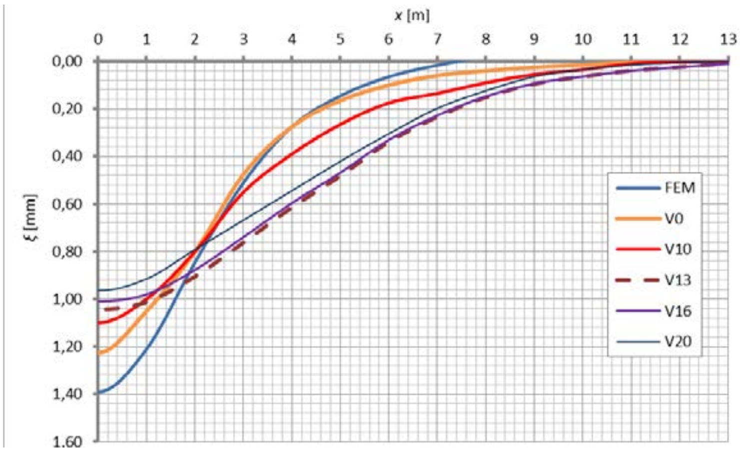

Fig. 5. Influence functions ξ(x) of the arch in Milicz obtained on the basis of measured deflections (for various locomotive speeds V10-V20) and extrapolated (V0) – compared with calculation (FEM).

Diagrams V10-V20 (presented in Fig. 5) corresponding to speeds given in Tab. 1, have different shapes what is not related to various geometry of the locomotives. The crucial feature of the diagrams is their dependence on the speed of the running locomotives. According to the results, the extreme ordinate ξ_{max} equal to $\xi(x=0)$ is getting higher with the decrease of the speed v. When the speed of the vehicle is $v = 0$ the received influence function of deflection $\xi(x)$ corresponds to the influence line of deflection $\eta(x)$ describing static effect of a single force. The difference between $\eta(x)$ and $\xi(x)$ is not caused by the effect of the dynamic vibrations (what was found negligible for the bridge) but rather is related to large inertia of masonry bridges responding with some delay to the loads.

Anyway, separate extrapolation of ordinates of the influence functions of deflection $\xi(x)$ for various speeds but for the same location x gives the influence function of deflection $\xi_0(x)$ corresponding to zero speed which is also presented as a virtual diagram (V0) in Fig. 5. Such extrapolation should eliminate all unknown effects manifesting themselves in speed dependent shapes of the function $\xi(x)$ and finally give the empirical influence line $\eta(x)$.

5 Finite Element analysis

5.1 Modelling technique

Two-dimensional FE models are applied in analysis of both considered bridges representing the effective width of the structures equal to half of the total width. They are composed of a masonry arch barrel, masonry backing, soil backfill and pavement layer (see Fig. 6). The extent of the models covers the area of the soil about 20 m away from the arch to both sides to consider the most distant positions of the live loads.

The masonry arch is modelled with application of so called mezomodelling technique (Kamiński 2008a), related to direct representation of selected radial masonry joints in the model and using average (homogenized) masonry properties for the remaining area of the arch.

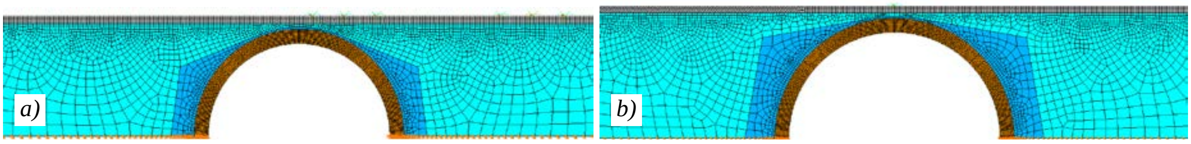

Fig. 6. *FE models of the analysed bridges (central part visible): a) in Oleśnica and b) in Milicz.*

Defined properties of all materials are determined by means of field and laboratory testing as well as on the way of numerical calibration based on loading test results; farther details on it can be found in (Kamiński 2008b). The live loads represent action of the locomotive axles; each axle is applied to the top of the pavement layer as a pressure uniformly distributed over a width equal to 80 cm.

Within the carried out analysis the deflection of the arch barrel intrados in the midspan is controlled. Various loading scenarios are considered including: action of a single axle located in positions along the whole model differing by 1 m – to create a numerical influence line of deflection as well as action of selected locomotives in positions corresponding to those recorded during measurements.

5.2 Results of analysis

The calculated influence line of deflection for the bridge in Milicz is compared in Fig. 6 with the one corresponding to zero speed developed by means of extrapolation procedure (from the measurement results for various speeds) described in the previous chapter. The calculated global response of both bridges triggered by selected positions of the applied locomotives is presented in Fig. 7. Precise control of the values of displacements of the arch mid-span section of the bridge in Milicz triggered by all locomotives – each located in two specific positions A and S – is presented in Tab. 2. These numerical results w_c are compared to the directly measured w_v (at a given speed) and to extrapolated w_0 deflections. The extrapolated deflections are recalculated according to the formula:

$$w_0(x) = P\sum_{i=1}^{n}\xi_0(x + x_i) \tag{5}$$

where: $\xi_0(x)$ – extrapolated influence function of deflection corresponding to zero speed (presented in Fig. 6).

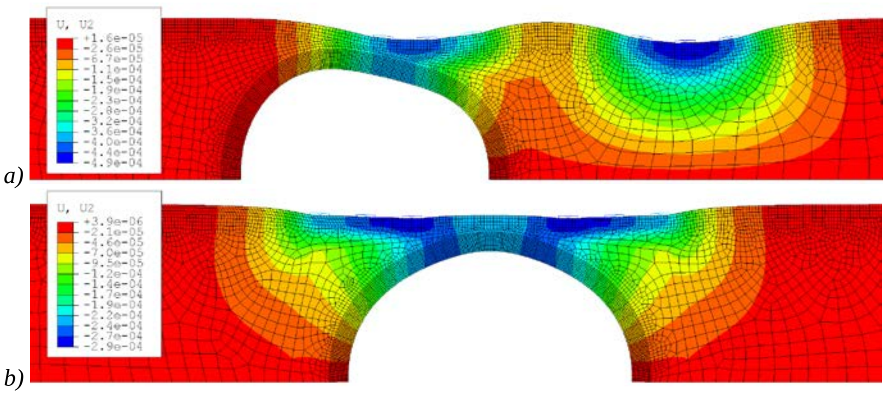

Fig. 7. Deformation calculated in FE models: a) of the bridge in Oleśnica under ET22 locomotive, b) of the bridge in Milicz under symmetric position of DRAGON locomotive..

Only the comparison of the calculated values with the extrapolated measured deflections (corresponding to v=0) shows good compatibility of the results while the results of direct measurements of the train passages significantly differ (especially for higher speeds) from the FE based calculations. The average discrepancy expressed on the percentage basis between w_0 and w_c considering all loading cases is very low being equal to $\overline{\omega}$ = -4.6 %.

Table 2. Comparison of the measured and calculated deflections of the bridge in Milicz.

Passage	Locomotive position	Directly measured deflection w_v [mm]	Extrapolated measured deflection w_0 [mm]	Calculated deflection w_c [mm]	Discrepancy $\omega = (w_c - w_0)/w_0$ [%]
V10	A	0,508	0,595	0,562	-5,5
	S	0,245	0,236	0,233	-1,3
V13	A	0,375	0,385	0,369	-4,2
	S	0,256	0,274	0,261	-4,7
V16	A	0,504	0,522	0,487	-6,7
	S	0,453	0,248	0,235	-5,2
V20	A	0,509	0,573	0,536	-6,5
	S	0,410	0,240	0,233	-2,9
					$\overline{\omega}$ = -4,6%

6 Conclusions

The presented procedure of testing and analysis of masonry arch bridge deflections under live loads may be an effective method of a comprehensive calibration of bridge numerical models including verification of the assumed material properties, invisible geometry or, in case of a 2D model, its effective width. Simultaneously it may be useful in the quality control procedures especially to monitor changes in time of the structural behaviour through e.g. control of structural stiffness (Machelski 2015) of the bridges.

The opportunity to get sufficient results from measurements carried out during regular exploitation of a bridge without any disturbance to the traffic is very attractive and in many situation makes the testing possible at all. The proposed approach is especially useful in analysis of railway bridges undergoing very regular and easily characterised loading vehicles represented by locomotives. However it can be also used in analysis of road bridges. The procedure can be based as well on other mechanical effects (including both vertical and horizontal displacements or strains) in any structural point other than the midspan section presented within this work.

Calibration process carried out on the presented bridge FE models according to the proposed procedure enabled formulation of conclusions about specific mechanical features of the bridges. First, large area of the soil in the approaching zones of the bridge (reaching at least L outside the arch springing) needs to be included in the model to eliminated impact of the side boundary conditions and to enable consideration of live loads affecting the arch even from large distance. Besides, an essential meaning of the surrounding backfill properties (found to be defined by very large modulus of elasticity exceeding 100 MPa) as well as the shape of the masonry backing (visible in Fig. 6) to the behaviour of the arch was discovered. Finally, an evident participation of the railway pavement in the bridge stiffness is visible which also significantly influence distribution of concentrated axle loads and therefore its consideration in the model is indispensable to provide compatibility of the numerical results with the values measured on the real structure.

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Overview and Preliminary Results of Long Term Bridge Performance (LTBP) Program

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Abstract. The FHWA’s Long-Term Bridge Performance (LTBP) Program is a 20-year plus research effort to collect scientific performance field data from a representative sample of bridges nationwide that will help the bridge community better understand bridge deterioration and performance. The products from this program will be data-driven tools, including predictive and forecasting models, which will enhance the abilities of bridge owners to optimize their management of bridges. Concrete bridge deck performance was identified as one of the key bridge performance issues in the LTBP Program. The overall objective of the LTBP Program is to inspect, evaluate, and periodically monitor representative samples of bridges nationwide to collect, document, maintain, and manage high-quality quantitative performance data over an extended period of time by taking advantage of sensing and nondestructive evaluation (NDE) tools and technologies such as impact echo (IE), ground penetrating radar (GPR), half-cell potential (HCP), ultrasonic surface waves (USW), and electrical resistivity (ER). This paper presents the condition change of a bridge deck in Virginia over a 6 year period. The assessment covered corrosive environment and corrosion processes, concrete degradation, and deck delamination. Deterioration progression from periodic NDE surveys is illustrated qualitatively by condition maps and quantitatively by condition assessment numbers. The results demonstrate the ability of NDE technologies to capture and quantify progression of deterioration. Strong agreement between different NDE technology results improves the confidence level of the condition assessment of the deck. The complementary use of multiple NDE technologies identified corrosion as the primary cause of damage in both decks.

Keywords: Performance monitoring, Condition assessment, Concrete bridge deck, Nondestructive evaluation, Corrosion, Delamination

1 Introduction

Aging and deterioration of bridges in the United States (U.S.) mandates strategies for bridge maintenance, rehabilitation, and repair. Bridge decks deteriorate faster than all other bridge components because of the routine application of deicing salts, repeated freeze-thaw cycles, direct application of traffic loading, and other damaging effects. Concrete bridge deck performance was identified as one of the key bridge performance issues in the FHWA's Long Term Bridge Performance (LTBP) Program (*1*). To improve the understanding of the mechanisms and timing of bridge deck deterioration because of the effects of age, material types, traffic loading, and climatic conditions, high quality performance data should be collected over an extended period of time, and condition assessment should be conducted periodically.

Given the large, diverse population of bridges throughout the U.S., one of the most significant challenges to the LTBP Program was selecting representative bridges. The sampling challenge was mitigated by the selecting “reference bridges” and representative clusters of bridges in the same vicinity of the “reference bridges” that have similar characteristics (age, type, climate, and maintenance practices). This selection helps compare the performance of concrete bridge decks of a similar type geographical area but, for instance, carry different traffic loads to ascertain the influence of traffic load on deck performance.

The common practices of State transportation departments for condition assessment and monitoring concrete bridge decks have been visual inspection, sounding methods, and destructive methods. While visual inspection and sounding methods have their merits, they are limited when used for the early detection and characterization of defects. Similarly, while destructive testing usually provides reliable assessment of the structure, the time and effort needed make this type of test impractical. As a result, the need emerged for nondestructive evaluation (NDE) technologies that can qualitatively and quantitatively assess the condition of concrete decks. The qualitative nature of NDE data helps capture deterioration progression, and its quantitative nature assists in developing more reliable deterioration, predictive, and life cycle cost models. Since concrete decks are affected by various deterioration processes, multiple NDE technologies should be used for condition assessment. The commonly used NDE technologies to assess and monitor the condition of bridge decks include impact echo (IE), ground penetrating radar (GPR), half-cell potential (HCP), ultrasonic surface waves (USW), and electrical resistivity (ER). In addition to providing comprehensive information about the deck condition, combining the results of different NDE technologies increases the confidence level of detection.

This paper demonstrates the ability of the mentioned NDE technologies to monitor deck performance and capture deterioration progression with time.

2 Description of the Bridge

This study was performed on a bridge carrying southbound U.S. Route 15 over Interstate 66 (I-66) in Haymarket, Virginia. The bridge (structure number 14178) is 276 ft. (84.1 m) long and 42 ft. (12.8 m) wide and was built in 1979. The bridge is a two-span, six-girder steel built-up superstructure (Figure 1a) with a bare, cast-in-place 8 in. (200 mm) thick reinforced concrete deck constructed with removable forms. The southbound carries two lanes of traffic.

The deck condition (including corrosion, deterioration, delamination, and concrete degradation condition maps) has been monitored four times since 2009, most recently in May 2015 (in years 2009, 2011, 2014 and 2015). The NDE results from the periodic surveys will be used to better understand deck performance and monitor deck deterioration progression with time.

All the NDE measurements were made on a 2 ft. by 2 ft. (0.6 m by 0.6 m) grid, except for the GPR surveys which were conducted in the longitudinal direction of the bridge with survey lines 2 ft. (0.6 m) apart. It took two 5-hour (total 10 hours) lane closures to survey the deck. In other words, considering that the surveyed deck area was about 11000 ft^2 (1020 m^2), the surveys were conducted at production rates of about 1100 ft^2 (102 m^2) per hour. A section of the deck surface with clearly visible patches and spalling is shown in Figure 1b.

(a)

(b)

Fig. 1. (a) Side view of the bridge and (b) the deck surface of the southbound U.S. Route 15 bridge in Haymarket, VA.

3 NDE Technologies Description and Results

3.1 Electrical Resistivity (ER)

Rebar corrosion leads to concrete deterioration, delamination, contamination, and loss of rebar section. If the damages are not repaired in a timely manner, it will cause large cracks and areas of delamination, ultimately leading to spalling of concrete. Chloride ions typically penetrate from the surface into a bridge deck, resulting in a higher chloride concentration and creating a more corrosive environment. A corrosive environment and its correlated corrosion rate can be evaluated by the ER method. The electrical resistivity of concrete decreases as the moisture and chloride concentration increases (2). It has been observed that a resistivity of less than 5 kOhm cm supports very rapid rebar corrosion (3). A four-point Wenner probe is used for resistivity measurements (Figure 2).

The assessment of the corrosion progression is illustrated in Figures 3. The ER maps in Figure 3 describe concrete resistivity in kOhm-cm. The threshold for corrosive environment was identified to be 30 kOhm-cm based on correlations with other NDE methods. It can be clearly observed qualitatively that ER captured corrosion progression during the 6 year period. Expansion of corrosion affected areas and increase of severity of corrosive environment in 2015 occurs in the same areas identified as corrosive in 2009.

Fig. 2. Electrical resistivity survey.

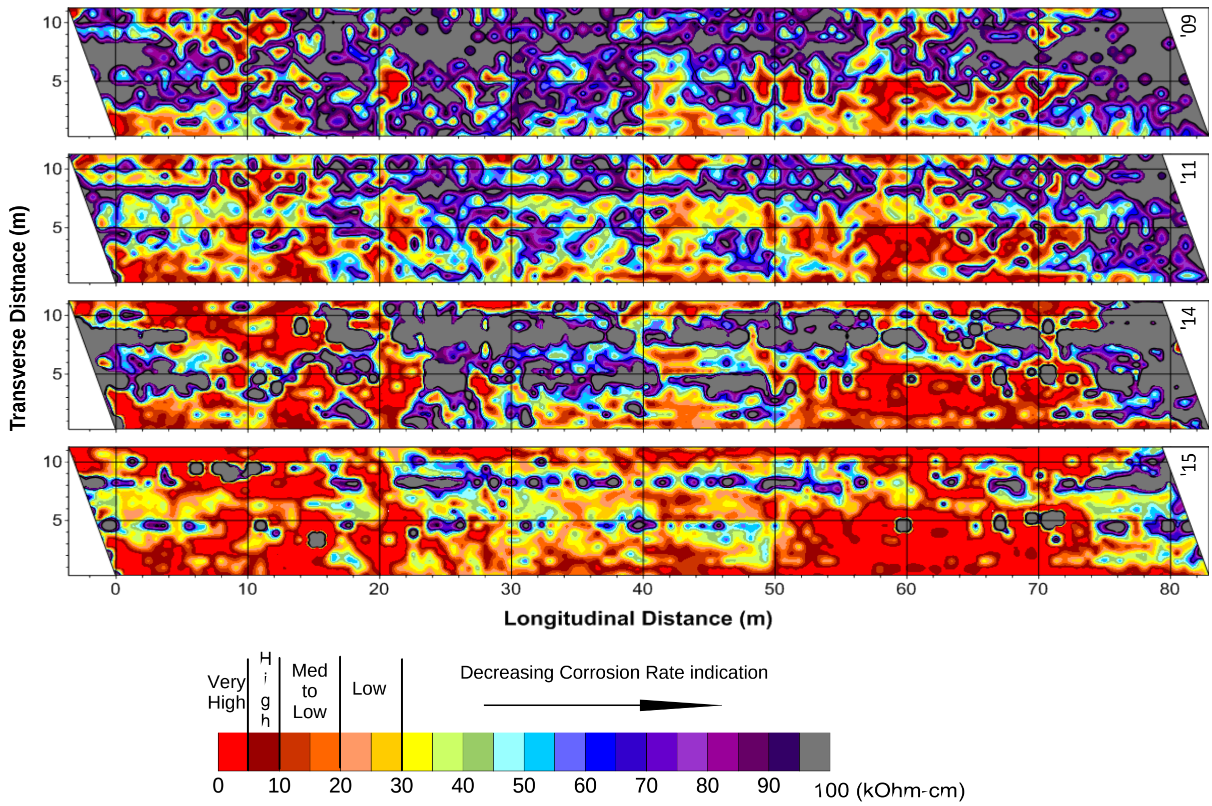


Fig. 3. Corrosion assessment maps from ER surveys conducted on the bridge deck from 2009 (top) to 2015 (bottom).

Since the data obtained from NDE technologies are quantitative, they can be used to assess the condition of decks quantitatively and compute a condition assessment number. The condition assessment number combines the effect of the extent and severity of deterioration obtained from the NDE surveys. The NDE measurements and the calculated condition assessment numbers, which vary from 100 for the best condition to 0 for the worst, could be entered to the bridge management systems to assist bridge owners in data-driven decision making regarding maintenance, rehabilitation, or replacement of the deck. For example, the most transportation agencies already have criteria for different intervention levels based on the assessment of deck area affected by delamination or corrosion, where corrosion assessment are obtained through chloride concentration or half-cell potential measurements. The condition assessment number of each type of defect is calculated as a weighted average of percentages of the deck area, with different severity levels of that specific defect. The area described as sound is assigned a weight factor of 100. The area with fair to poor grade is assigned a factor of 50, and the area in the state of severe condition a factor of 0.

The ER-based corrosion condition assessment numbers along with the percentages of the decks areas with various levels of anticipated corrosion rates are compared in Table 1 for the 2009 and 2015 ER surveys. The four corrosion states are defined by four electrical resistivity ranges: less than 10 kOhm-cm as high, 10 to 20 kOhm-cm as moderate, 20 to 30 kOhm-cm as low, and above 30 kOhm-cm as very low. The corrosion condition assessment number for the deck dropped from 86 to 41 in 6 years. The overall decrease in condition assessment numbers is well-reflected in the increase of deck area in different severity levels of corrosion. The majority of the deck area had very low corrosion rates in 2009, while the majority of the same had high to moderate corrosion rates in 2015.

Table 1. Corrosion Assessment from ER: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Very Low (%)**	**Low (%)**	**Moderate (%)**	**High (%)**
2009	86.3	72	15	10	3
2015	40.9	22	17	17	44

3.2 Ground Penetrating Radar (GPR)

A qualitative assessment of the deck condition can be made using a GPR survey through measuring the attenuation of electromagnetic waves on the top rebar level *(4)*. The amplitude of the reflection will be highest when the deck is in a good condition. The presence of moisture, chloride ions, iron oxide, cracks, and air-filled delaminations alter dielectric properties and increase the attenuation of electromagnetic waves. Thus, zones of highly-attenuated signal in GPR attenuation maps indicate locations of likely concrete deterioration, delamination, and/or corrosive environment. A GPR survey conducted using a 1.5 GHz ground coupled antenna is shown in Figure 4.

GPR condition maps obtained from GPR surveys of the deck in 2009 to 2015 are shown and compared qualitatively in Figure 5. The GPR threshold levels of deterioration are specific to the bridge conditions and equipment used and were obtained from correlations with other NDE technologies. The correlations between the attenuation levels and condition grades are indicated in the figure. A serious condition is described for both bridges with attenuation level of below -20 dB. Similar to ER, GPR qualitatively captured deterioration progression in the deck during the 6 year period. The progression and level of deterioration correlate well with the corrosion progression captured during the ER surveys. The similarity can be attributed to the fact that both measurements are primarily affected by the same elements affecting the electrical conductivity and dielectric value of concrete: moisture, chlorides, salts, etc.

Fig. 4. GPR survey using a 1.5 GHz ground coupled antenna.

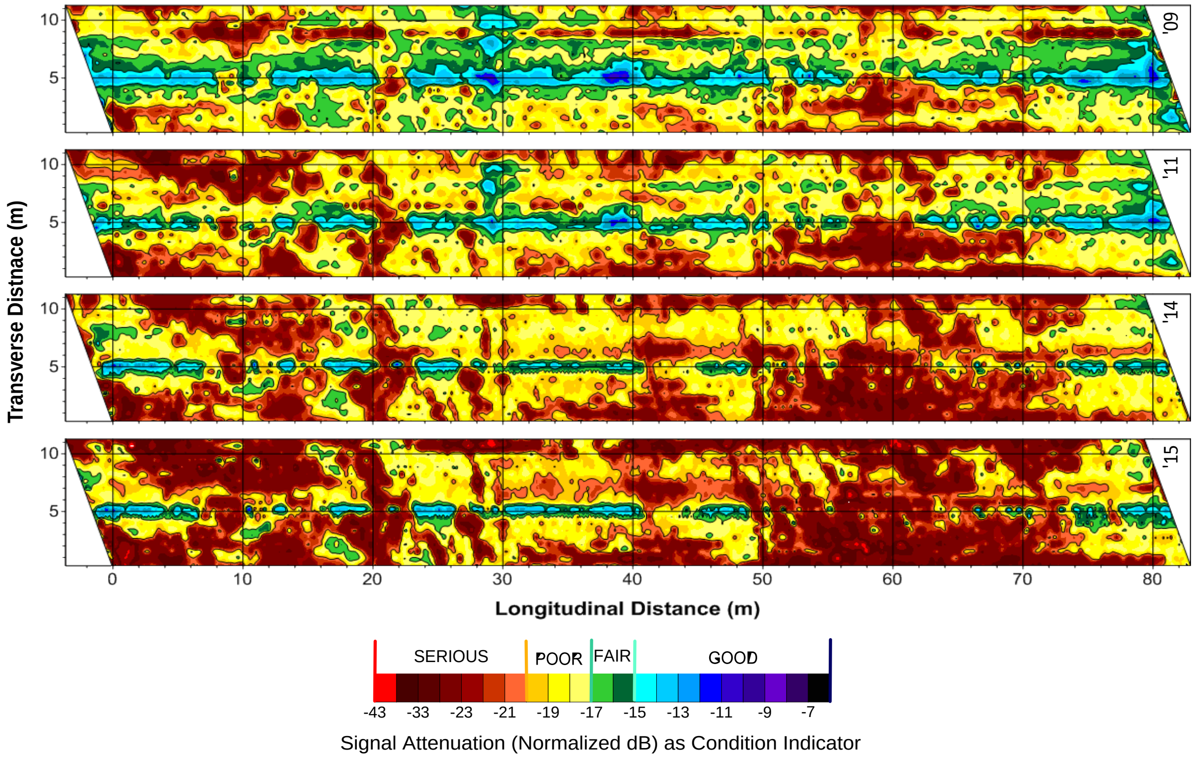


Figure 5. Deterioration condition maps from GPR survey on the bridge deck in 2009 (top) and 2015 (bottom).

To quantify deterioration progression, the condition assessment numbers along with the percentages of the decks areas at various severity levels of deterioration are compared in Table 2 for the 2009 and 2015 GPR surveys. The GPR condition assessment number for the deck dropped from 48 in 2009 to 22 in 2015. A large percentage of the deck area was in a poor condition in 2009, while the most of the deck area is currently in a serious condition.

Table 2. GPR Assessment: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Good (%)**	**Fair (%)**	**Poor (%)**	**Serious (%)**
2009	**48.1**	**14**	**24**	**41**	**21**
2015	**22.4**	**5**	**5**	**35**	**55**

3.3 Impact Echo (IE)

Concrete delamination is most often a result of rebar corrosion, though other types of concrete deterioration or repeated overloading, or a combination of those can also lead to delamination. IE has been successfully implemented in detecting and characterizing delamination in bridge decks *(5)*. IE measurements can be made by using a single IE probe, consisting of an impactor and a nearby receiver (figure 6a) or by using multiple IE probes (figure 6b), each consisting of an impactor and a sensor.

Fig. 6. Two types of IE testing systems: (a) an IE cane and (b) a stepper.

The primary objective of IE testing is to locate reflectors (bottom of the deck or a delamination) in the deck. The extent and position of reflectors can be estimated by analyzing the frequency response of the waves reflected from the reflector. The results from IE surveys on the deck are shown in figure 7. The delamination grades from IE surveys are defined based on the measured dominant response frequencies. The condition is described as good or sound when the depth of the reflector based on the measured response frequency matches the thickness of the deck. In the case of a delaminated deck, reflections of the compression wave occur at shallower depths, causing a shift in the response spectrum towards higher frequencies. Depending on the extent and continuity of the delamination, the partitioning of energy of waves being reflected from the bottom of the deck and delamination may vary. Initial delamination (fair condition) is described as occasional separations between the two deck zones. Thus it will have two distinct peaks corresponding to reflections from the bottom of the deck and the delamination. Progressed delamination (poor condition) is characterized by a single peak at a frequency corresponding to the depth of the delamination. Finally, in cases of wide or shallow delaminations, the dominant response of the deck to an impact is characterized by a low frequency response of flexural mode oscillations of the upper delaminated portion of the deck. This condition is graded as a serious condition and is always in the audible frequency range.

Figure 7 depicts the condition of the deck obtained from the IE surveys conducted from 2009 to 2015. The maps illustrate progression of delamination during the 6 year period.

Similar to the ER and GPR results, the condition assessment numbers from the 2009 and 2015 IE surveys, along with the percentages of the decks areas at various severity levels of delamination, are compared quantitatively in Table 3. The delamination condition assessment number for the deck dropped from 69 in 2009 to 40 in 2015. While a large percentage of the deck area was sound in 2009, the majority of the deck was extensively delaminated in 2015.

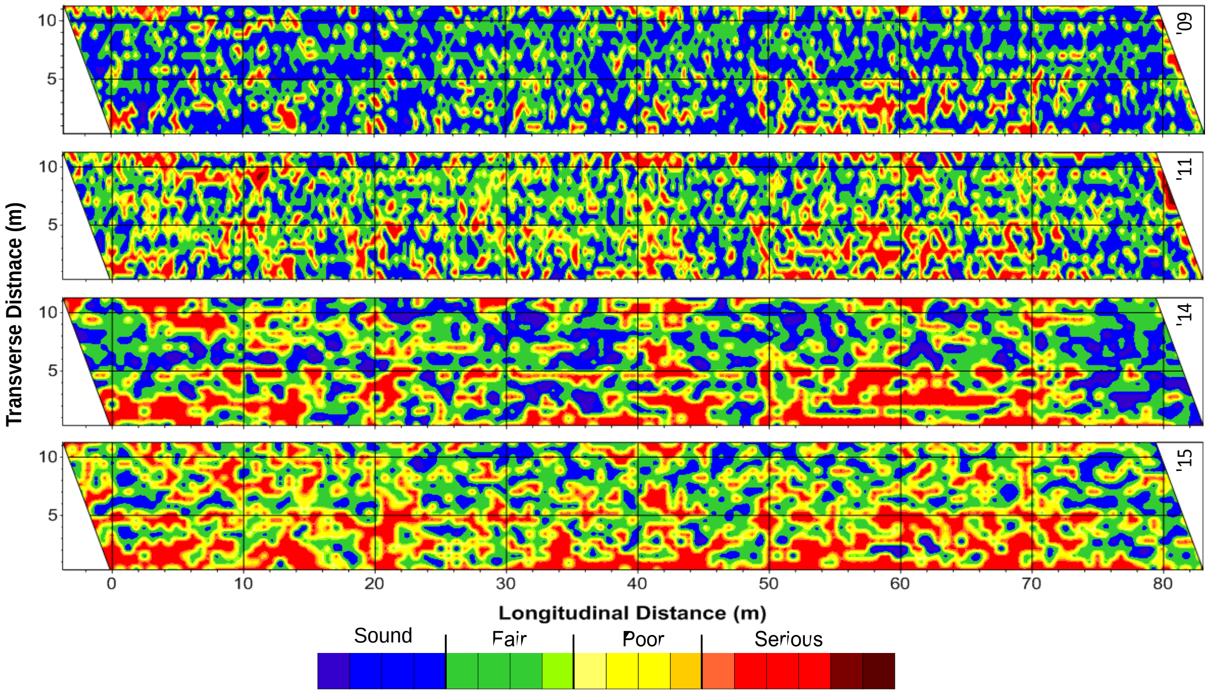


Fig. 7. Delamination maps from IE surveys conducted on the bridge deck from 2009 (top) to 2015 (bottom).

Table 3. Delamination Assessment from IE: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Sound** (%)	**Fair** (%)	**Poor** (%)	**Serious** (%)
2009	69.5	54	26	4	15
2015	39.9	21	31	7	41

3.4 Ultrasonic Surface Waves (USW)

Besides corrosion, a number of other deterioration processes in decks occur as a result of repeated freeze and thaw, alkali-silica reaction, mechanical stressing, overloading, etc. These may lead to expansive stresses (cracks) within the concrete and either reduced mechanical properties or altered dielectric properties. The USW method is effective for assessing concrete degradation and detecting and measuring changes in mechanical properties. Surface waves are stress waves traveling along the surface of the deck, with their body extending to the depth of approximately one wavelength (6). Therefore, as long as the USW testing is limited to the wavelengths comparable to the deck thickness, the surface wave velocity will be controlled by concrete properties (elastic modulus). Devices like the portable seismic property analyzer (PSPA), shown in Figure 8, can be used in the evaluation of concrete modulus by the USW method. A variation in concrete modulus in the deck does not necessarily mean deterioration. Such variations can often be introduced at the time of construction due to material variation and placement procedures. Therefore, only a periodic measurement of changes in the concrete modulus would lead to identification of deterioration processes.

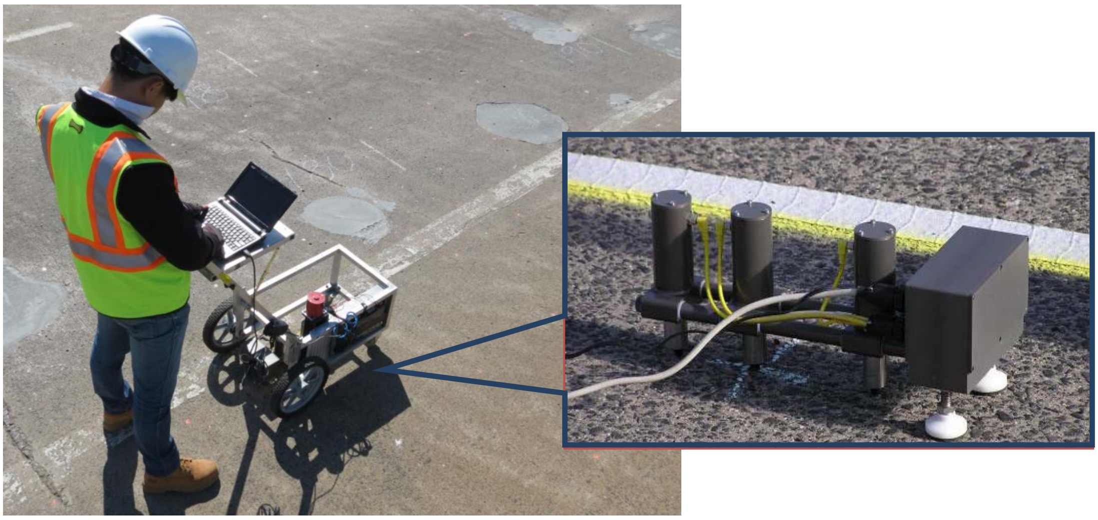
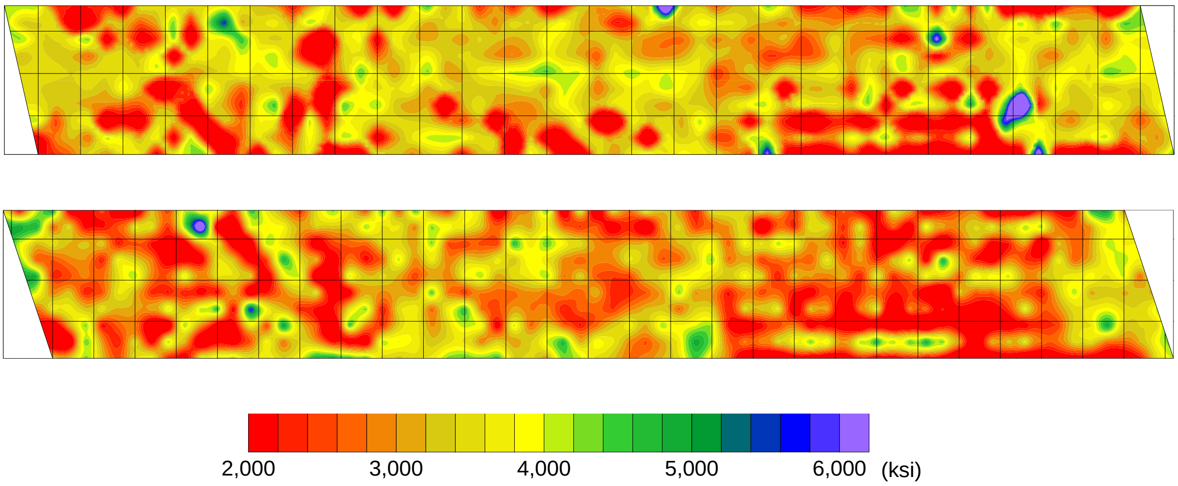


Fig. 8. Surface wave testing using a PSPA.

Concrete quality maps of the deck are shown in Figure 9 for the USW surveys conducted in 2009 and 2015. Areas of very low concrete modulus obtained from the USW testing are, in general, at locations of delamination. Similar to IE, concrete quality of the deck, in terms of the concrete modulus, decreased in 2015, compared to the one from the USW survey conducted in 2009.

The results of the 2009 and 2015 USW surveys are also presented quantitatively in Table 4 in terms of the percentages of the decks areas with various concrete elastic moduli. The quality of concrete in the deck did not change much from 2009 to 2015. A large percentage of the deck area had a modulus of less than 3,500 ksi (24 GPa).



1 ksi=6.89 MPa

Fig. 9. Concrete modulus maps from USW surveys conducted on the bridge deck in 2009 (top) and 2015 (bottom).

Table 4. Concrete Quality Assessment from USW: Average Modulus and Standard Deviation.

	Distribution				
Bridge-Year	**< 3,500 ksi (%)**	**3,500 – 4,500 ksi (%)**	**> 4,500 ksi (%)**	**Mean *E* (ksi)**	**STDEV (ksi)**
2009	66	29	5	3,286	705
2015	68	27	5	3,008	988

Note: 1 ksi=6.89 MPa

3.5 Conclusions

The condition of the deck of the southbound U.S. Route 15 bridge in Haymarket, Virginia was monitored over a period of 6 years to improve knowledge about the bridge deck performance and to better understand progression rates of different deterioration and defect types over an extended period of time.

Results of the surveys over a 6 year period show that NDE technologies have the ability to monitor deterioration progression with time, whether qualitatively through increase of deteriorated areas or quantitatively through changes in condition assessment numbers. Expanding deterioration and increasing severity of deterioration in 2015 occurs in the same areas identified as deteriorated in 2009 in all NDE condition maps. The condition assessment numbers facilitate development of more objective and realistic deterioration and prediction models for the bridge deck.

The complementary use of multiple NDE technologies identifies corrosion as the primary cause of damage in the deck. The NDE results provide strong correlation between the ER and GPR condition maps, explained by similar electrical properties affecting the two. Additional correlations are established regarding delamination detection between GPR and IE, and USW and IE, and concrete degradation between USW and GPR. These correlations also point to corrosion as the primary cause of deterioration for these decks.

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Disclaimer Statement

The contents of this report reflect the views of the authors, who are responsible for the facts and the accuracy of the information presented herein. The U.S. Government assumes no liability for the contents or use thereof.

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Resistivity as indicator of durability: survey in existing bridge

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Abstract. Electrical Resistivity of concrete is an indicator of several properties of the material and of the risk of reinforcement corrosion. It can be monitored in specimens of new structures, in cores but also on the structure itself is measured the so called "surface resistivity" (four points method). In present communication is presented the basis for its use for service life prediction and the case of a bridge is in which it has been measured for final indicator of homogeneity and concrete quality are presented. The relation of resistivity with diffusivity is based in the known Einstein's relation, However as chlorides and carbon dioxide penetrate in concrete by a reactive diffusion mechanism, it is necessary to account for a "retarder factor" named r_b for the calculation of service life, it is also necessary to define an environmental factor, F_{env} and an aging factor "q", which are defined in the text. The bridge was built by FCC CONSTRUCTION on the Danube River, between the towns of Vidin (Bulgaria) and Calafat (Romania). The owner decided a post-built survey in order to verify quality and homogeneity of the concrete elements fixing the minimum value of the resistivity in 1000 Ω.m. Most of the bridge segment was inspected and more than 900 measurements were performed, which revealed that the concrete quality was higher than that specified in 99.25% of the measurements.

Keywords: Resistivity, Durability, Concrete, Rebar, Reinforced Concrete Structure

1 Introduction

Codes and standards contain in general provisions for concrete durability related to the proportion of raw concrete materials, the limit of dangerous substances (such as chlorides or sulphates) limitations to crack width and the minimum cover thickness in function of exposure classes. However, there is an increasing demand to incorporate into the current standards more advanced concepts related to concrete durability, because of the need to better foresee and prevent distresses, in particular the corrosion of the reinforcement. Several proposals exist based in modelling the mechanisms of attack (Tuutti, 1982; Page, 1981) or in the so called "performance" concepts (Andrade, 1997) or in the use of "durability indicators" (Baroghel-Bouny, 2002). Nevertheless, their effective incorporation into the standards seems to be slow and a worldwide controversy exists on which is the best approach, because of the lack of experience with these new proposals.

Regarding existing structures the situation is similar, as the same models than for new structures are applied. On-site concrete quality testing can be done by coring or by using ultrasound or Schmidt impact testers once it is calibrated. These non-destructive methods are related to the mechanical strength but they are not correlated with the concrete durability. The durability and the corrosion processes are associated with the concrete pore distribution and connectivity. In this sense, Andrade et al. (Andrade, 2004; Andrade, 1993) proposed a service life model based on the measurement of electrical resistivity, which is a NDT. The electrical resistivity of concrete is an indication of the volume of pores and also the degree of water saturation and therefore has a strong relationship with the concrete durability and in particular to the corrosion of reinforcements (Andrade, 2000b). Present paper presents the summary of the model based in the electrical resistivity that tries to be comprehensive by responding to the demand related to the introduction of performance parameters or durability indicators and, at the same time, is applicable for predicting service life. The case of application to a bridge after its construction for verification of concrete quality and homogeneity is described.

1.1 Reinforcement Service Life Modelling

The service life of reinforcements, t_l, is usually modelled by assuming two periods: the time to initiation of corrosion t_i and its propagation, t_p Thus,

$$t_l = t_i + t_p \tag{1}$$

The calculation of the duration of t_l is usually undertaken by considering that the aggressive penetrates through concrete cover by diffusion and therefore, Ficks law's is used to calculate a diffusion coefficient able to predict the concentration of the aggressive at a certain depth and several periods of time. Providing that the aggressive threshold (pH-drop front in the case of carbonation or a certain chloride amount) is defined, the end of t_i indicates the initiation of t_p. The initiation of corrosion establishes then a limit state. The propagation period, t_p, is calculated by assuming a constant or averaged corrosion rate, V_{corr}. In a similar manner than for t_i the "limit state" or maximum corrosion has to be defined first in order to account for the length of this period.

The model proposed here based in the measuring of electrical resistivity makes use of this parameter for determining both t_i and t_p periods (Andrade, 2004). This is possible because of the comprehensive character of the resistivity regarding concrete microstructure. Thus, the electrical resistivity of water saturated concrete is an indirect measurement of the concrete pore connectivity. The electrical potential difference or the current applied by means of two external electrodes to a concrete specimen is carried through concrete pore network by the electrical carriers (ions). As the porosity increases, the resistivity is reduced due the higher volumetric fraction of pores. On the other hand, while resistivity is related to porosity and connectivity, in non-water saturated concrete, it is as well an indication of its degree of saturation.

The electrical resistivity is the ratio between the potential applied by means of two electrodes and the current circulating in the material standardized by a geometrical factor, which depends on the position of the electrodes (Ohm's law):

$$R = \frac{V}{I} = \rho \cdot K_{geom} \tag{2}$$

The ability of resistivity to quantify the diffusivity is based on one of the Einstein laws which relate the movement of electrical charges to the conductivity of the medium (Andrade, 2000b):

$$D_e = \frac{k}{\rho_{ef}} = k_{Cl}\ \sigma \tag{3}$$

Where: D_e = effective diffusion coefficient, k = a factor, which depends on the external ionic concentration, ρ_{ef} = "effective" resistivity (in this case of concrete saturated with water) and σ = conductivity (inverse of resistivity)

A value of k_{Cl} of $20x10^{-5}$ can be used for external chloride concentrations of 0.5 to 1 M (Andrade, 2000b). This expression only accounts for the transport of chloride ions and chloride binding has to be taken into account separately. This is proposed by means of introducing a new factor, r_{Cl} (reaction or binding factor). This reaction factor is a "retarder" of the penetration of chlorides (Andrade, 2014). The above equation maintains its mathematical structure but can now be given as follows (where D_{ap} is an "apparent" diffusion coefficient in saturated conditions, ρ_{ef} is the Effective Resistivity and ρ_{ap} is the Apparent resistivity):

$$D_{ap} = \frac{k_{Cl}}{\rho_{ef} \cdot r_{Cl}} = \frac{k_{Cl}}{\rho_{ap}} \tag{4}$$

Equation (4) can also be applied to the case of carbonation providing another constant k_{CO2} is considered for the atmospheric exposure. Relating r_{CO2} to the amount of alkaline material able to bind CO_2, it can be written as:

$$D_{CO2} = \frac{k_{CO2}}{\rho_{ef} \cdot r_{CO2}} \tag{5}$$

Other parameters that need to be incorporated in the model are: a) The ageing factor ***q*** (Andrade, 2004) and b) The environmental factor k (Andrade, 2004), which will be described in the following sections. The final expression of the model is:

$$t_l = \frac{x^2 . \rho_{ef} \left(\frac{t}{t_0} \right)^q}{F_{Cl,CO_2}} \cdot r_b + \frac{P_{corr} \cdot \left(\rho_{ef} \cdot \left(\frac{t}{t_0} \right)^q \cdot W_s \right)}{K_{corr} \cdot 0.00116} \quad (6)$$

Where: t_l= life time, t_0= 28 days, t= time until aging is accounted, ρ_{ef} = resistivity at 28 days in saturated conditions, x= cover depth, $F_{Cl,CO2}$= environmental factor, P_{corr} = steel cross section reached at the time t_p,, W_s = environmental factor of the corrosion rate (it can be of 10±2 for carbonation and 30±5 for chlorides),K_{corr} = constant with a value of 26 µA/cm^2·kΩ·cm= to 26 mV/cm relating the resistivity and the corrosion rate I_{corr}

In summary, the electrical resistivity provides indications on the pore connectivity and therefore, on the concrete resistance to penetration of liquid or gas substances, and so resistivity is a parameter which accounts for the main key properties related to reinforcement durability.

With respect to the measurement mode, there are several methods to measure the resistivity in concrete structures. The direct (bulk) method is used primarily at laboratory and a sample with regular geometry in which two electrodes are placed to carry out the measure. The specimen is described by the UNE 83988-1 standard (PrUNE – 83988-1). For a cylindrical geometry, the resistivity, ρ, is related to the resistance between the two electrodes, R, the section S, and the distance between electrodes, L, as shown in the following equation.

$$\rho = R \frac{S}{l} \quad (7)$$

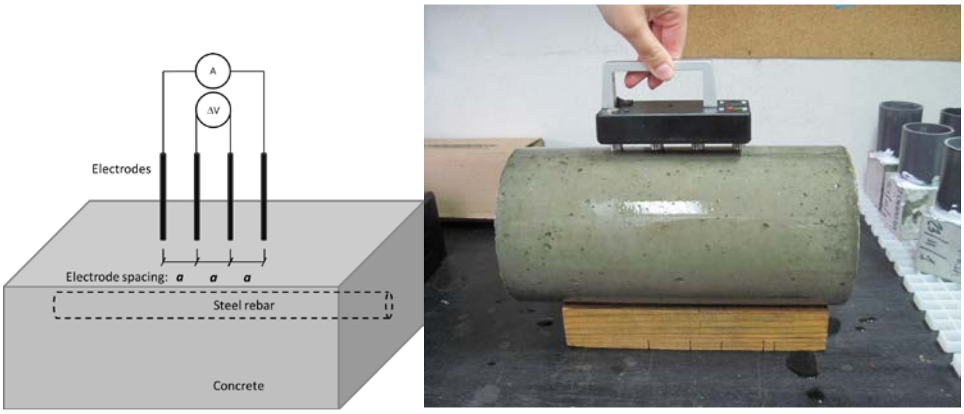

Fig. 1. Wenner methodology applied to reinforced concrete slab

Finally, one of the most used methods for measuring the resistivity is the Wenner method or the four point method also defined in the UNE 83988-2 standard (Pr-UNE 83988 - 2). This method comprises a four electrodes separated an equidistant distance “a” and arranged in a line over the concrete surface (see Figure 1). A current pulse is applied on one of the external electrodes, while the other external one is grounded. The potential drop is measured between the central electrodes. This resistance can be measured as the ratio of the potential drop and the injected current. The resistivity in this case depends on the separation between electrodes, as indicated in the following equation:

$$\rho = 2\pi \, a \, R \quad (7)$$

2 Measurement in a Bridge

The bridge is located in Vidin (Bulgaria) which passes of Danube River connects Vidin with Calafat in Romania. It has road and train connections. It has been designed and built by Fomento de Construcciones y Contratas (FCC) of Spain. It was started in 2007 and has been finished in 2013. An aspect of the bridge is given in figure 2 with the section of one of the spans.

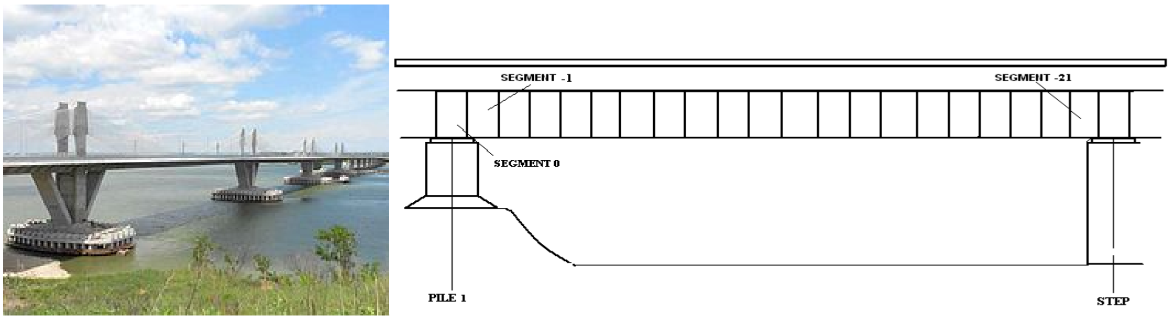

Fig. 2. Section of one span in which the segments were identified.

The purpose of the measurements was to show how homogeneous is the concrete quality and providing the cover depth could have been lower than designed whether the values of resistivity are above or not of 1000 Ω·m. This was the threshold established by the owner as indication of quality.

Additionally, the cover depth was measured by means of a cover-meter with the second objective for the resistivity measurements of avoiding the places were the rebar is located.

2.1 On-site Resistivity Measurement Methodology

The measurements were made by means of a basket (see figure 3) to reach the bottom of the deck (Figure 3). Approximately 50% of the segments were measured randomly. The measurements were made by marking a grid of approximately 1 m x 1 m in the segment, locating the rebar. The concrete is watered at least 60 minutes before measuring. The segments were saturating with a pump and a tank placed on the bridge. Thus, it comes to measuring the resistivity of concrete close to saturation conditions (Figure 3 left). While the cover of the reinforcement was measured with a cover-meter and in some cases a drill hole was made to reach the rebar and measuring the cover directly (Figure 3 right).

Fig. 2. Left: On-site measurement of resistivity in the segments after watering. Right: Checking the concrete cover.

3 Results and Discussion

At the laboratory was first developed a model to verify the possible error introduced by the presence of bars with low cover depth. As a consequence of these results, it was taken the decision to use the sensor electrode spacing of 1 cm. Following equation shows the fitting expression from of the rebar factor and concrete cover:

$$f_b = -4.350\ 10^{-6}\ r^5 + 4.422\ 10^{-4}\ r^4 - 1.752\ 10^{-2}\ r^3 + 0.3385\ r^2 - 3.206\ r + 13.13 \qquad (8)$$

where f_b is dimensionless and r is the concrete cover in mm.

3.1 On-site Resistivity Measurements.

According to the methodology described, the resistivity was measured with a sensor electrode spacing of 1 cm while the coating was measured. Figure 4 shows all resistivity and concrete cover measurements. The rebar factor is calculated according to equation 7 with the concrete cover. It has set a threshold on the concrete cover of 30 mm. Therefore, if the concrete cover exceeds this threshold value, then it has not been taken into account in this study since the durability is guaranteed. There have been 931 measures and only 5 of them (0.54%) have a lower resistivity value than 1000 Ω·m (100 KΩ·cm).

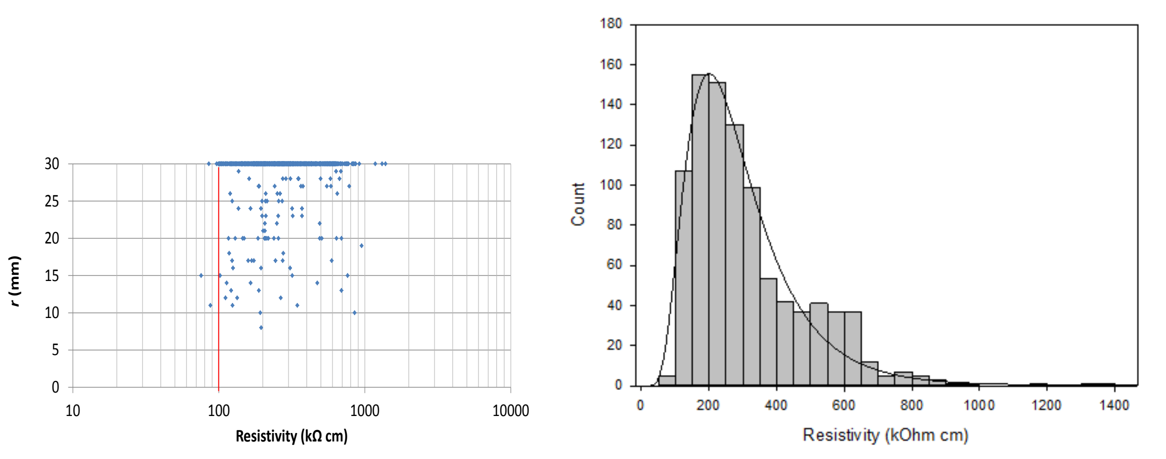

Fig. 4. On-site measurements of resistivity and concrete cover in the bridge segments.

Data were fitted to a log-normal distribution obtained an average 2597 Ω·m and standard deviation 5.11 Ω·m (Figure 4 right). According to this setting, the probability that the resistivity is less than 1000Ω·m is of 3.1%. This value is higher than that obtained directly from the experimental data (0.54%) that shows the difference between the theory statistical function and data, and the errors induced in extreme values.

Regarding the calculation of the lifetime through expression 6, the propagation period is neglected. Assuming the most conservative options of not having aging (aging factor of 0.01) and not retarder factor (r_b= 1) and a F_{CO2} = 3000 (carbonation is the corrosion risk in atmospheric conditions) the results are given in table 1:

$$t_l = \frac{x^2 \cdot \rho_{ef}}{F_{CO2}} \tag{9}$$

Table 1. Service life calculated through equation 9 for three cover depths.

COVER DEPTH	SERVICE LIFE
1 cm	33 years
2 cm	133 years
3 cm	300 years

Although when a concrete resistivity is $\rho_{ef} \geq 1000$ Ω·m, the porosity is so low that aggressive substances cannot penetrate through the pores, for the sake of the service life calculation a trial is made in the case of 10 mm cover depth which gives mathematically values of short service life. Thus, in this case, with an aging factor of around 0.3 (which is relatively low) during 10 years, the service life would be increased to above the 100 years required.

4 Conclusions

Mapping of resistivity and cover depth have been made in a bridge after construction for the sake of verifying its quality and homogeneity. The measurements were made in 50% of the segments with 931 measurement points made in around three days.

The values found confirmed that the risk to find a zone with resistivity values smaller than the threshold fixed by the owners of 1000 Ω·m, is smaller than 3%.

The resistivity, together with the cover depth has been a method to verify compliance with design requirements.

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Reliability of existing bridges determined with physical models – chloride induced corrosion

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Abstract:

Management of bridges includes activities which aim to optimize the use in a manner that maximizes the benefits while satisfying proscribed requirements over a predetermined period of time. Forecasting long-term performance of bridges with deterioration models is a crucial component in any management strategy. To cope with the performance forecasting, a model in use should describe the process of degradation and allow prediction of condition of structure in time, taking into consideration environmental surrounding, nature of use and maintenance actions. Deterioration models can basically be divided into mathematical (statistical), empirical and physical models. Statistical models are formed by analysing data that describe condition of a greater number of bridges, empirical models are based on experience, while physical models are based on knowledge and modelling of damage-causing processes. The main aim of this manuscript is to present the physical model of chloride ingress in evaluation of performance of existing reinforced concrete bridges. Special attention was given to accessibility of material and environmental parameters which govern chloride ingress formulation. In the end, analysis was performed in order to present the achieved reliability of reinforced concrete bridges for combination of different exposure classes and material characteristics.

Keywords: *bridge performance, physical models, chloride ingress, reliability index*

1 Introduction

The research presented in this manuscript was done as a part of a LeCIE – “Life-Cycle Assessment for Railway Structures” project, currently being realized at the University of Natural Resources and Life Sciences in Vienna under auspices and for the needs of ÖBB-Infrastruktur – Austrian Federal Railways. The objective of the project is to develop a comprehensive concept for an anticipatory life cycle management system for engineering structures in railway systems. This thorough approach is to link inspection results relating to damage symptoms and damage processes with probability-based degradation predictions and forecasting methods and with monitoring and assessment methods. Presented part of research is connected with the probability-based degradation predictions.

The decision of modeling deterioration by using physical models was due the reason that mathematical (statistical) deterioration models were excluded under the assumption of input data based on visual inspections rating being prone to subjectivity of engineers performing inspections. Deterioration processes implemented in LeCIE concept are primarily chloride and carbonation induced corrosion, freeze-thaw, alkali-silica reaction and fatigue. This contribution will focus on chloride ingress as an initiation phase of chloride induced corrosion.

To best describe a physical deterioration process, significant attention and effort needs to be pointed in characterization of parameters which influence particular deterioration process. The quality and accuracy of input parameters is of great significance for the realistic and useful prediction.

Since this study was undertaken in Austria, it is important to mention that the analysis was made only for the exposure classes related to chlorides from sources other than sea water, in other words only for exposure classes XD.

2 Chloride ingress

A full probabilistic design approach for the modelling of chloride induced corrosion in uncracked concrete presented in this manuscript, has been developed within the research project DuraCrete and slightly revised in the research project DARTS, each project was funded by the European Union. It is based on the limit state equation Eq. (1) in which the critical chloride concentration is compared to the actual chloride concentration at the depth of the reinforcing steel. The model is applicable both to marine environment and for de-icing salts on roads/bridges.

$$g(c,t) = C_{c} \quad - C(c,t) \tag{1}$$

The model is based on Fick's 2nd law of diffusion, takin into account that most observations indicate that transport of chlorides in concrete is diffusion controlled. Fick's 2nd law for diffusion was first proposed for application in chloride exposed structures by M. Collepardi in 1970 (Collepardi , Marcialis, & Turriziani, 1972).

The content of chlorides at corresponding depth *x* (usually for *x* the value of the depth of reinforcing steel *c* is considered) is given by Eq. (2):

$$C_x(x,t) = C_0 + \left(C_{S,\Delta x} - C_0\right) \cdot \left[1 - e \quad \frac{x - \Delta x}{2 \cdot \sqrt{k_e \cdot D_{R} \quad {}_{,0} \cdot \left(\frac{t_0}{t}\right)^{\alpha} \cdot t}}\right] \tag{2}$$

where:

C_0 initial chloride content;
$C_{S,\Delta x}$ chloride content at depth Δx;
x depth with a corresponding content of chlorides ($x = c$ – depth of reinforcement);
Δx depth of convection zone;
k_e environmental variable (considers temperature);
$D_{RCM,0}$ apparent diffusion coefficient (rapid chloride migration test);
t_0 reference point of time;
α ageing exponent;

$$k_e = e \quad \left[b_e\left(\frac{1}{T_r} - \frac{1}{T_r}\right)\right] \tag{3}$$

where:

b_e temperature coefficient (proportional to activation energy) [K]
T_{ref} reference temperature [K]
T_{real} temperature of the structural element or the ambient air [K]

For the quantification of reliability indices the parameters presented in the **Table 1** were used. Fixed parameters were obtained from the literature and/or chosen for specific conditions in Austria. Varied parameters are those which are hard to access and therefore different combinations of them were made.

Table 1. Fixed and varied parameters used for reliability analysis

Fixed parameters						
Parameter	**Unit**	**Distribution**	**Mean**	**Standard Deviation**	**a**	**b**
t_0	years	Constant	0,0767	-		
t	years	Constant	50	-		
T_{ref}	K	Constant	293	-		
T_{real}	K	Normal	281	5		
b_e	K	Normal	4800	700		
C_{crit}	wt.-%/c	Beta	0,60	0,15	0,2	2,0

Varied parameters			**XD1**	**XD2**	**XD3**	**XD1**	**XD2**	**XD3**		
Parameter	**Unit**	**Distribution**	**Mean**			**Standard Deviation**			**a**	**b**
α	-	Beta	0,65	0,30	0,30	0,12			0	1
$C_{S,\Delta x}$	wt.-%/c	Normal	0,5-1,5	2-4	2-4	μ * 0,75				
c	mm	Normal	40-60			10				
Δ*x*	mm	Constant	0	0	10	0	0	5	0	50
$D_{RCM,0}$	m²/s	Normal	8,9-25,0			μ * 0,20				

3 Accessibility of parameters of existing bridges

When performing an evaluation of an existing bridge, even by carrying detailed study of documents, reviews and other evidences it is hard to assess and completely understand the environmental and material characteristics of the bridge. For older bridges in most cases design documents have not been preserved, and when they were they contain scarce data which cannot be used for durability related analysis. One of the future ways to deal with the problem of scarce data is introduction of birth certificate document (BCD). BCD is an extract of as-built documentation which contains information such as: cover thickness to the reinforcement, diffusion coefficient for the concrete cover and so on (fib Model Code for Concrete Structures 2010, 2010).

Hard accessible parameters important for analysis of chloride ingress and thus analysis of chloride induced corrosion are mainly cement type, w/c ratio and chloride content at the surface. The type of cement and w/c ratio are the two parameters which strongly affect the pore structure of concrete and hence, its potential chloride migration coefficient (Benchmarking of deemed-to-satisfy provisions in standards: Durability of reinforced concrete structures exposed to chlorides; fib Bulletin 76, 2015).

The amount of cement, a minimum which provides sufficient guaranteed compaction as well as type, shape and size of aggregate are less important.

The use of cement types in past can be accessed from literature or roughly estimated in cooperation with cement and concrete producers and providers. In the best case this estimation provides notion of extent of use of cement types in particular geographical regions, which still does not indicate which particular cement was used for which bridge.

Originally used w/c ratio for construction of specific bridge can hardly be known. In best case, w/c ratio can be restricted between limiting values. Generally it is considered that in the past, the value of w/c lower than 0,4 was not used due to technical reasons. For the upper limits of w/c ratio the survey of historical guidelines and norms has to be performed. Usually, restriction of upper limits of w/c ratio is connected with specific exposure conditions (mainly in newer norms) and quality of concrete needed due to structural demands.

As indicated in (von Greve-Dierfeld & Gehlen, 2016) the chloride content at exposed concrete surface $C_{s,0}$ as well as the substitute surface content $C_{S,\Delta x}$ are variables that depend on material properties and on geometrical and environmental conditions. An adequate quantification of the potential chloride impact turns out to be very complex, as for bridges that are subjected to chloride impact due to de-icing salt, the variables describing the amount of de-icing salt applied are hard to quantify because of seasonal loading. Under real exposure conditions the chloride surface content varies randomly due to the variations in the chloride concentrations of ambient solution, frequency of application of de-icing salts, temporal and spatial variations in the humidity conditions of the concrete and so forth.

To cover the possible cases of chloride content at the depth of convection zone, the analysis was performed for maximal amount for exposure class XD1 ($C_{S,\Delta x}$ = 1,5), and for minimal, average and maximal amount for exposure classes XD2 and XD3 ($C_{S,\Delta x}$ = 2; $C_{S,\Delta x}$ = 3 and $C_{S,\Delta x}$ = 4).

4 Achieved reliability

In the case of chloride induced corrosion, it is assumed that the depassivation of the reinforcement occurs as soon as a critical, corrosion inducing chloride content C_{crit} has reached the depth of the reinforcement. Or reformulated, if the content of chlorides at the depth of reinforcement (action) $C(c,t)$ is higher than corrosion-inducing chloride content (resistance) C_{crit}.

Actions and resistance are uncertain quantities, not deterministic ones. They are therefore introduced as random variables and contrasted in the limit state function $g(c,t)$, as presented in Eq. (1). The difference between action and resistance will then also be a random variable. Since the action and resistance are not normal distributed random variables, it is not possible to perform an analytical calculation of probability of failure p_f. For this purpose the Monte Carlo method with 10^6 simulations was performed. In this case, the reliability index β was stated instead of the probability of failure p_f.

For obtaining the indices of reliability β the parameters presented in **Table 1** were used, and the results are presented in the **Table 2**.

Table 2. Obtained indices of reliability β

CEM I								
w/c [%]	**c_{nom} [mm]**	**XD1**	**XD2**			**XD3**		
		$C_{S,\Delta x}$ = 1,5	**$C_{S,\Delta x}$ = 2**	**$C_{S,\Delta x}$ = 3**	**$C_{S,\Delta x}$ = 4**	**$C_{S,\Delta x}$ = 2**	**$C_{S,\Delta x}$ = 3**	**$C_{S,\Delta x}$ = 4**
0,40	**50**	3,1	2,8	-0,3	-0,4	1,8	-0,5	-0,6
	45	2,8	2,4	-0,4	-0,5	1,3	-0,5	-0,7
	40	2,4	1,9	-0,4	-0,6	0,9	-0,6	-0,7
	35	2,0	1,5	-0,5	-0,7	0,5	-0,7	-0,8
	30	1,5	1,1	-0,6	-0,8	0,2	-0,8	-0,9
0,45	**50**	3,0	2,6	-0,3	-0,5	1,6	-0,5	-0,6
	45	2,6	2,2	-0,4	-0,5	1,2	-0,6	-0,7
	40	2,2	1,8	-0,5	-0,6	0,8	-0,7	-0,8
	35	1,8	1,3	-0,6	-0,7	0,4	-0,8	-0,8
	30	1,4	0,9	-0,7	-0,8	0,1	-0,8	-0,9
0,50	**50**	2,3	1,8	-0,5	-0,6	1,1	-0,6	-0,8
	45	1,9	1,5	-0,6	-0,7	0,7	-0,7	-0,8
	40	1,6	1,1	-0,6	-0,8	0,4	-0,8	-0,9
	35	1,2	0,8	-0,7	-0,8	0,2	-0,8	-0,9
	30	0,9	0,5	-0,8	-0,9	-0,1	-0,9	-1,0
0,55	**50**	2,0	1,5	-0,6	-0,7	0,8	-0,7	-0,8
	45	1,6	1,2	-0,6	-0,8	0,6	-0,8	-0,9
	40	1,3	0,9	-0,7	-0,8	0,3	-0,8	-0,9
	35	1,0	0,6	-0,8	-0,9	0,1	-0,9	-1,0
	30	0,7	0,3	-0,8	-0,9	-0,2	-0,9	-1,0
0,60	**50**	1,6	1,2	-0,6	-0,8	0,6	-0,8	-0,9
	45	1,3	0,9	-0,7	-0,8	0,4	-0,8	-0,9
	40	1,0	0,7	-0,8	-0,9	0,1	-0,9	-0,9
	35	0,8	0,4	-0,8	-0,9	-0,1	-0,9	-1,0
	30	0,5	0,2	-0,9	-1,0	-0,3	-0,9	-1,0

**Grey assigned cells are the cells with index of reliability lower than targeted*

In current specifications the target reliabilities are between 0,5 ≤ β ≤ 2,3. Most of the current standards and specifications distinguish reliability with respect to the consequence of failure (consequence class CC) and measures for increasing safety (Model Code for Service Life Design of Concrete Structures, *fib* Bulletin 34, 2006).

The consequence of chloride ingress and subsequent depassivation of the reinforcing steel may be assessed by, for example, the corrosion rate after depassivation. For exposure classes XD1 and XD2 (i.e. predominantly dry or wet conditions) the consequence of depassivation is not expected to be significant because corrosion rate is limited either due to the lack of moisture or the lack of oxygen. However, in exposure class XD3 higher corrosion rates are expected because it is likely that sufficient moisture and oxygen are present (von Greve-Dierfeld & Gehlen, 2016).

The following values of target reliability β at the age of 50 years are proposed:

- $\beta \geq 1{,}5$ for bridge elements exposed to exposure class XD3
- $\beta \geq 0{,}5$ for bridge elements exposed to exposure class XD1 and XD2

5 Conclusions

In the presented manuscript the physical model of chloride ingress was used for the evaluation of reliability of existing reinforced concrete bridges. Attention was pointed to problems of accessibility of material and environmental parameters which govern chloride ingress formulation. An analysis was performed in order to present the achieved reliability of reinforced concrete bridges for combination of different exposure classes and material characteristics. Achieved reliability indices were compared with the targeted for the bridges at the age of 50 years for the cement type CEM I. The reliability indices higher than targeted were obtained for almost all of the combinations of exposure class XD1 and for exposure class XD2, for the minimal $C_{S,\Delta x}$. On contrary, the reliability indices lower than targeted were obtained for exposure class XD3 for almost all the combinations of w/c ratio and depth of reinforcement c_{nom} and for exposure class XD2 for the average and maximal $C_{S,\Delta x}$.

From the presented results it can be concluded that for the concrete structures made from concrete with cement type CEM 1 special attention should be given for the exposure classes XD2 and XD3.

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Quality control of masonry bridges based on empirical influence lines of displacements

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Abstract. Quality control of masonry bridges carried out through measurements of arch barrel deformation generated by their typical service loads is analysed. Attention is paid to displacements of characteristic point of the structure i.e. vertical deflection of the arch crown section. In the study measurements of deflection carried out on two masonry arch bridges in Poland are used. On the basis of the direct results empirical influence functions of displacements are being created while considering also various speeds of loading vehicles, the empirical influence lines are developed. Careful comparison of the experimental and numerical outcomes shows potential of field tests carried out under live loads to be a useful tool of masonry bridge numerical models calibration and control of the structures' condition.

Keywords: bridge, live loads, influence line, FEM, quality control

1 Introduction

Masonry arch bridges are complex structures characterised by many parameters being sometimes difficult to determine having however an important influence on their mechanical performance. These technical parameters cover both the material properties of the structural components as well as their geometrical characteristics including those hidden by the backfill and inaccessible for the direct measurements. Determination of them may be done during diverse laboratory tests as well as by means of site NDT or SDT methods. Therefore a reasonable quality control of the masonry bridges becomes a demanding and time-consuming task especially in case of no technical data on the structure available.

A simpler solution to the problem may be a careful study on the structure deformation under live loads. Especially in case of the railway bridges the that kind of analysis can be effective taking into account regularity of the exploitation load scenarios however the road structures may undergo analogical procedure as well. The main idea of the approach is related to measurements of the structure displacements on site. An important advantage is a possibility to carry out the tests during regular exploitation of bridges – without any disturbance to the traffic (Kamiński & Bień 2015). Even if the measurements are limited to a single point of the structure, like the midspan arch deflection presented within this study, the obtained results can provide comprehensive information on the structural response to many independent loading cases. Thus, in this way that kind of tests may be an efficient tool of a masonry bridge model calibration verifying it in a global way. The proposed analysis is presented in two case studies of arch bridges with a similar structure. Effects of various railway vehicle crossing them with different speed are recorded.

2 Experimental analysis of masonry bridge displacements

2.1 Analysed structures

In the analysis two similar single-span brick railway bridges are considered (Fig. 1) with structure, technical condition and age representative for the bridges in this part of Europe. Both of them are located along Polish railway line no. 281: in Oleśnica and Milicz. The bridge in Oleśnica have semi-circular arch barrel with intrados radius R = 4,97 m and thickness h = 0,78 m (assumed theoretical span length L_t = 10,72 m). The bridge in Milicz is differing with intrados radius R = 6,0 m and thickness h = 0,80 m (thus L_t = 12,80 m). The structures have the same width equal to B = 8,55 m and both are dating back about 1875 (Kamiński 2008b).

Fig. 1. Analysed bridges during the tests: a) in Oleśnica, b) in Milicz

2.2 Applied live loads

Among all types of transport vehicles a locomotive has the most regular, invariable in time and easy to determine loading parameters including values of axle loads and spaces between the axles. Wagons of trains can be more diverse regarding these parameters, therefore only effects of locomotive's action crossing the bridge are considered within the presented study. The applied locomotive types and their technical parameters are presented in Tab. 1 and described in Fig. 2 for a representative vehicle.

Table 1. Characteristics of locomotives and their passages across the bridges.

Bridge	Passage	Locomotive type	Axle load P [kN]	Bogie type	Space between bogies d [m]	Spaces between axles [m]			Speed v [m/s]
						a	b	c	
Oleśnica	V11	ET22	200	Co'Co'	10,30	1,75	2,720	6,80	10,8
	V10	ET22	200	Co'Co'	10,30	1,75	2,720	6,80	10,0
	V13	EU07	196,2	Bo'Bo'	8,55*	3,05*	2,317	5,50	12,9
Milicz	V16	E31	203	Co'Co'	10,95	2,40	1,525	6,15	15,6
	V20	Dragon	202,2	Co'Co'	10,50	1,95	2,965	6,60	19,8

* 4-axle locomotive

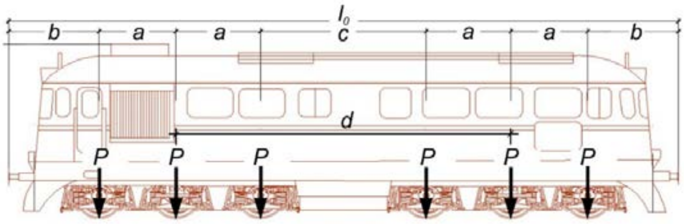

Fig. 2. Dimensions of a representative locomotive.

2.3 Testing procedure

The applied testing procedure is based on measurements of the arch barrel deflection during passages of the locomotives across the bridge (Machelski 2014). Results of deflection measurements used within the study were collected by means of various types of gauges including LVDT gauges, laser distance sensor and microradar equipment (Helmerich et al. 2012). The controlled points are located on intrados of the arch barrel in the midspan (crown cross-section). An example of the recorded deflection $w(x)$ as a function of the front (reference) bogie axis position x against the arch crown section for the bridge in Oleśnica during passage of ET22 locomotive is shown in Fig. 3. The two extremes in the figure indicated when the front and the rear locomotive bogie axes correspondingly are over the arch crown section. Effects of individual three axles of each bogie is invisible due to dispersion of the loads within the track and the bridge structure. When $x = 0$ the reference bogie is located centrally over the arch midspan and the other bogie is in the distance d from the midspan what corresponds to the asymmetrical position (A). For $x = d/2$ both bogies are equally distant from the midspan and the locomotive takes the symmetrical position (S). On the basis of the time difference corresponding to the extremes of $w(x)$ and the distance between bogies the average speed of the locomotive crossing a bridge is evaluated (see Tab. 1).

3 Empirical influence functions of displacements

The diagrams $w(x)$ (given in Fig. 4 and Fig. 5 for the bridge in Oleśnica and Milicz correspondingly) may be further used to create empirical influence functions of the arch crown deflection $\xi(x)$. For this purpose constant axle load values P given in Tab. 1 for each case are assumed. Accordingly, a general relationship between the arch crown deflection $w(x)$ and ordinates $\xi(x)$ of the influence function is expressed by a formula:

$$w(x) = P\sum_{i=1}^{n}\xi(x+x_i) \tag{1}$$

where: x – location of the locomotive reference axle against the arch midspan section, x_i – location of the consecutive locomotive axles i against the reference axle, n – number of locomotive axles.

To find the values of the influence function $\xi(x)$ a progressive calculation procedure is applied starting from the point $x = x_0$, for which the measured deflection is equal to $w(x_0)$, while for all previous points laying at least in distance a away from x_0, it is equal to $w(x_0 - a) = 0$. Thus, at the beginning of the analysis the initial position of the first axle load is considered as follows:

$$w(x_o) = P\cdot\xi(x_0 + a)) \tag{2}$$

From the formula (2) the function value $\xi(x_0+a)$ can be calculated taking into account $\xi(x_0) = 0$. The second function ordinate is being calculated for a point in distance a from the previous one, according to the formula:

$$w(x_o + a) = P\left[\xi(x_o + a) + \xi(x_o + 2a)\right] \tag{3}$$

Further procedure carried out with subsequent positions of the locomotive defined by $x = x_0 + i{\cdot}a$ allows to find the next ordinates of the influence function $\xi(x)$.

Fig. 3 presents the shape of the deflection influence function $\xi(x)$ calculated according to the described procedure for the structure in Oleśnica on the basis of ET22 locomotive crossing the bridge alone. For the compatibility of the units with the diagrams $w(x)$ the ordinates ξ are multiplied by axle load P, treated here as a constant factor. In case of a single locomotive crossing a bridge the confirmation of the influence function $\xi(x)$ correctness should be an agreement between diagram $w_\xi(x)$ developed backward from $\xi(x)$ with the diagram of the directly measured deflections $w(x)$ – as it is presented also in Fig. 3 – according to the formula:

$$w(x) \cong w_\xi(x) = P\sum_{i=1}^{n}\xi(x+x_i) \tag{4}$$

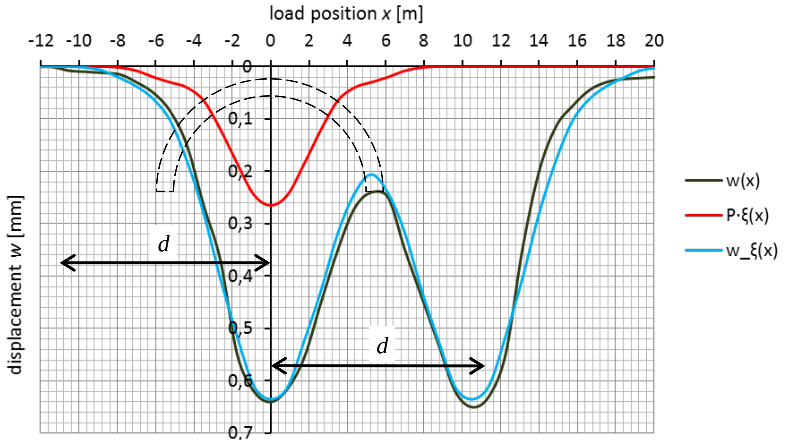

Fig. 3. Comparison of the arch crown deflections of the bridge in Oleśnica (generated by ET22 locomotive) directly measured during the tests $w(x)$ and calculated $w_\xi(x)$ from the influence function $\xi(x)$ (presented rescaled by force P).

The lack of a prefect agreement between the diagrams $w(x)$ and $w_\xi(x)$ can arise from many reasons including: imprecision of measurements, different real axle loads, variable speed of the locomotive or nonlinear behaviour of the real structure (Machelski 2014).

Analogical diagrams are presented in Fig. 4 for the bridge in Milicz also on the basis of ET22 locomotive passage. In this case additional influence of wagons following the locomotive is visible in the diagram $w(x)$ for $x > d/2$.

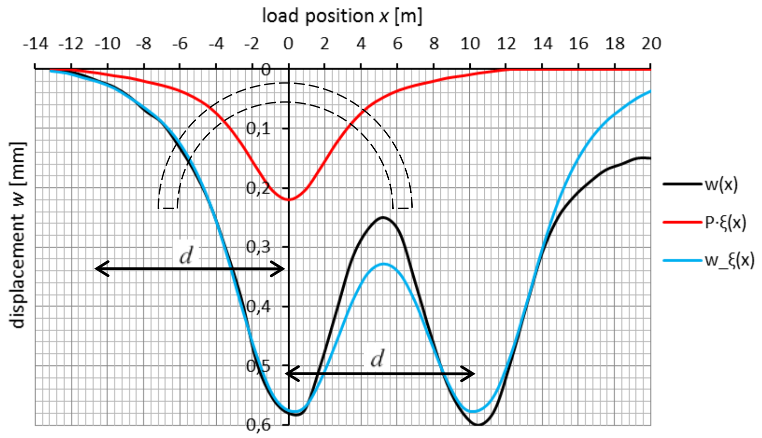

Fig. 4. Comparison of the arch crown deflections of the bridge in Milicz (generated by ET22 locomotive) directly measured during the tests ($w(x)$) and calculated ($w_\xi(x)$) from the influence function $\xi(x)$ (presented rescaled by force P).

4 Influence lines of the arch crown displacements

Influence functions of deflection $\xi(x)$ presented in the previous chapter are calculated on the basis of the arch deflections generated by selected single runs of the vehicles. In Fig. 5 influence functions of deflection $\xi(x)$ determined by means of all locomotives travelling with various speeds are given together for the bridge in Milicz.

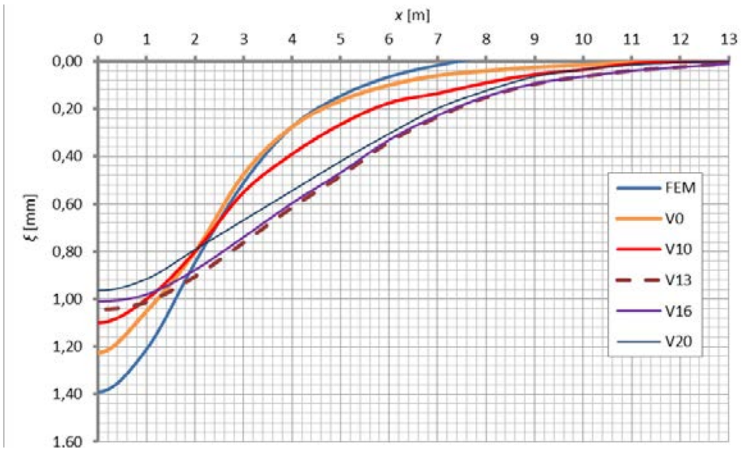

Fig. 5. Influence functions ξ(x) of the arch in Milicz obtained on the basis of measured deflections (for various locomotive speeds V10-V20) and extrapolated (V0) – compared with calculation (FEM).

Diagrams V10-V20 (presented in Fig. 5) corresponding to speeds given in Tab. 1, have different shapes what is not related to various geometry of the locomotives. The crucial feature of the diagrams is their dependence on the speed of the running locomotives. According to the results, the extreme ordinate ξ_{max} equal to $\xi(x=0)$ is getting higher with the decrease of the speed v. When the speed of the vehicle is $v = 0$ the received influence function of deflection $\xi(x)$ corresponds to the influence line of deflection $\eta(x)$ describing static effect of a single force. The difference between $\eta(x)$ and $\xi(x)$ is not caused by the effect of the dynamic vibrations (what was found negligible for the bridge) but rather is related to large inertia of masonry bridges responding with some delay to the loads.

Anyway, separate extrapolation of ordinates of the influence functions of deflection $\xi(x)$ for various speeds but for the same location x gives the influence function of deflection $\xi_0(x)$ corresponding to zero speed which is also presented as a virtual diagram (V0) in Fig. 5. Such extrapolation should eliminate all unknown effects manifesting themselves in speed dependent shapes of the function $\xi(x)$ and finally give the empirical influence line $\eta(x)$.

5 Finite Element analysis

5.1 Modelling technique

Two-dimensional FE models are applied in analysis of both considered bridges representing the effective width of the structures equal to half of the total width. They are composed of a masonry arch barrel, masonry backing, soil backfill and pavement layer (see Fig. 6). The extent of the models covers the area of the soil about 20 m away from the arch to both sides to consider the most distant positions of the live loads.

The masonry arch is modelled with application of so called mezomodelling technique (Kamiński 2008a), related to direct representation of selected radial masonry joints in the model and using average (homogenized) masonry properties for the remaining area of the arch.

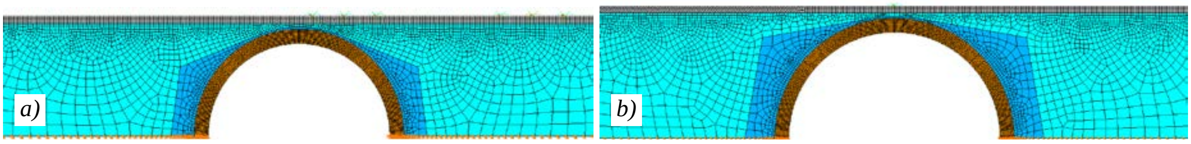

Fig. 6. *FE models of the analysed bridges (central part visible): a) in Oleśnica and b) in Milicz.*

Defined properties of all materials are determined by means of field and laboratory testing as well as on the way of numerical calibration based on loading test results; farther details on it can be found in (Kamiński 2008b). The live loads represent action of the locomotive axles; each axle is applied to the top of the pavement layer as a pressure uniformly distributed over a width equal to 80 cm.

Within the carried out analysis the deflection of the arch barrel intrados in the midspan is controlled. Various loading scenarios are considered including: action of a single axle located in positions along the whole model differing by 1 m – to create a numerical influence line of deflection as well as action of selected locomotives in positions corresponding to those recorded during measurements.

5.2 Results of analysis

The calculated influence line of deflection for the bridge in Milicz is compared in Fig. 6 with the one corresponding to zero speed developed by means of extrapolation procedure (from the measurement results for various speeds) described in the previous chapter. The calculated global response of both bridges triggered by selected positions of the applied locomotives is presented in Fig. 7. Precise control of the values of displacements of the arch mid-span section of the bridge in Milicz triggered by all locomotives – each located in two specific positions A and S – is presented in Tab. 2. These numerical results w_c are compared to the directly measured w_v (at a given speed) and to extrapolated w_0 deflections. The extrapolated deflections are recalculated according to the formula:

$$w_0(x) = P\sum_{i=1}^{n} \xi_0(x + x_i) \tag{5}$$

where: $\xi_0(x)$ – extrapolated influence function of deflection corresponding to zero speed (presented in Fig. 6).

a)
b)

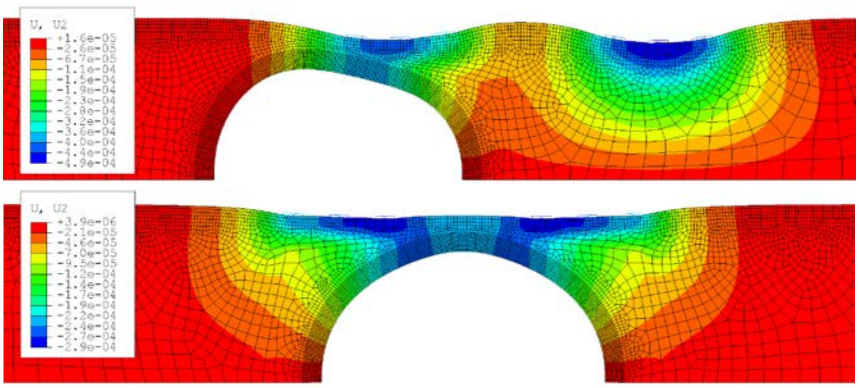

Fig. 7. Deformation calculated in FE models: a) of the bridge in Oleśnica under ET22 locomotive, b) of the bridge in Milicz under symmetric position of DRAGON locomotive..

Only the comparison of the calculated values with the extrapolated measured deflections (corresponding to v=0) shows good compatibility of the results while the results of direct measurements of the train passages significantly differ (especially for higher speeds) from the FE based calculations. The average discrepancy expressed on the percentage basis between w_0 and w_c considering all loading cases is very low being equal to $\overline{\omega}$ = -4.6 %.

Table 2. Comparison of the measured and calculated deflections of the bridge in Milicz.

Passage	Locomotive position	Directly measured deflection w_v [mm]	Extrapolated measured deflection w_0 [mm]	Calculated deflection w_c [mm]	Discrepancy $\omega = (w_c - w_0)/w_0$ [%]
V10	A	0,508	0,595	0,562	-5,5
	S	0,245	0,236	0,233	-1,3
V13	A	0,375	0,385	0,369	-4,2
	S	0,256	0,274	0,261	-4,7
V16	A	0,504	0,522	0,487	-6,7
	S	0,453	0,248	0,235	-5,2
V20	A	0,509	0,573	0,536	-6,5
	S	0,410	0,240	0,233	-2,9
					$\overline{\omega}$ = -4,6%

6 Conclusions

The presented procedure of testing and analysis of masonry arch bridge deflections under live loads may be an effective method of a comprehensive calibration of bridge numerical models including verification of the assumed material properties, invisible geometry or, in case of a 2D model, its effective width. Simultaneously it may be useful in the quality control procedures especially to monitor changes in time of the structural behaviour through e.g. control of structural stiffness (Machelski 2015) of the bridges.

The opportunity to get sufficient results from measurements carried out during regular exploitation of a bridge without any disturbance to the traffic is very attractive and in many situation makes the testing possible at all. The proposed approach is especially useful in analysis of railway bridges undergoing very regular and easily characterised loading vehicles represented by locomotives. However it can be also used in analysis of road bridges. The procedure can be based as well on other mechanical effects (including both vertical and horizontal displacements or strains) in any structural point other than the midspan section presented within this work.

Calibration process carried out on the presented bridge FE models according to the proposed procedure enabled formulation of conclusions about specific mechanical features of the bridges. First, large area of the soil in the approaching zones of the bridge (reaching at least *L* outside the arch springing) needs to be included in the model to eliminated impact of the side boundary conditions and to enable consideration of live loads affecting the arch even from large distance. Besides, an essential meaning of the surrounding backfill properties (found to be defined by very large modulus of elasticity exceeding 100 MPa) as well as the shape of the masonry backing (visible in Fig. 6) to the behaviour of the arch was discovered. Finally, an evident participation of the railway pavement in the bridge stiffness is visible which also significantly influence distribution of concentrated axle loads and therefore its consideration in the model is indispensable to provide compatibility of the numerical results with the values measured on the real structure.

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Overview and Preliminary Results of Long Term Bridge Performance (LTBP) Program

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Abstract. The FHWA's Long-Term Bridge Performance (LTBP) Program is a 20-year plus research effort to collect scientific performance field data from a representative sample of bridges nationwide that will help the bridge community better understand bridge deterioration and performance. The products from this program will be data-driven tools, including predictive and forecasting models, which will enhance the abilities of bridge owners to optimize their management of bridges. Concrete bridge deck performance was identified as one of the key bridge performance issues in the LTBP Program. The overall objective of the LTBP Program is to inspect, evaluate, and periodically monitor representative samples of bridges nationwide to collect, document, maintain, and manage high-quality quantitative performance data over an extended period of time by taking advantage of sensing and nondestructive evaluation (NDE) tools and technologies such as impact echo (IE), ground penetrating radar (GPR), half-cell potential (HCP), ultrasonic surface waves (USW), and electrical resistivity (ER). This paper presents the condition change of a bridge deck in Virginia over a 6 year period. The assessment covered corrosive environment and corrosion processes, concrete degradation, and deck delamination. Deterioration progression from periodic NDE surveys is illustrated qualitatively by condition maps and quantitatively by condition assessment numbers. The results demonstrate the ability of NDE technologies to capture and quantify progression of deterioration. Strong agreement between different NDE technology results improves the confidence level of the condition assessment of the deck. The complementary use of multiple NDE technologies identified corrosion as the primary cause of damage in both decks.

Keywords: Performance monitoring, Condition assessment, Concrete bridge deck, Nondestructive evaluation, Corrosion, Delamination

1 Introduction

Aging and deterioration of bridges in the United States (U.S.) mandates strategies for bridge maintenance, rehabilitation, and repair. Bridge decks deteriorate faster than all other bridge components because of the routine application of deicing salts, repeated freeze-thaw cycles, direct application of traffic loading, and other damaging effects. Concrete bridge deck performance was identified as one of the key bridge performance issues in the FHWA's Long Term Bridge Performance (LTBP) Program (*1*). To improve the understanding of the mechanisms and timing of bridge deck deterioration because of the effects of age, material types, traffic loading, and climatic conditions, high quality performance data should be collected over an extended period of time, and condition assessment should be conducted periodically.

Given the large, diverse population of bridges throughout the U.S., one of the most significant challenges to the LTBP Program was selecting representative bridges. The sampling challenge was mitigated by the selecting "reference bridges" and representative clusters of bridges in the same vicinity of the "reference bridges" that have similar characteristics (age, type, climate, and maintenance practices). This selection helps compare the performance of concrete bridge decks of a similar type geographical area but, for instance, carry different traffic loads to ascertain the influence of traffic load on deck performance.

The common practices of State transportation departments for condition assessment and monitoring concrete bridge decks have been visual inspection, sounding methods, and destructive methods. While visual inspection and sounding methods have their merits, they are limited when used for the early detection and characterization of defects. Similarly, while destructive testing usually provides reliable assessment of the structure, the time and effort needed make this type of test impractical. As a result, the need emerged for nondestructive evaluation (NDE) technologies that can qualitatively and quantitatively assess the condition of concrete decks. The qualitative nature of NDE data helps capture deterioration progression, and its quantitative nature assists in developing more reliable deterioration, predictive, and life cycle cost models. Since concrete decks are affected by various deterioration processes, multiple NDE technologies should be used for condition assessment. The commonly used NDE technologies to assess and monitor the condition of bridge decks include impact echo (IE), ground penetrating radar (GPR), half-cell potential (HCP), ultrasonic surface waves (USW), and electrical resistivity (ER). In addition to providing comprehensive information about the deck condition, combining the results of different NDE technologies increases the confidence level of detection.

This paper demonstrates the ability of the mentioned NDE technologies to monitor deck performance and capture deterioration progression with time.

2 Description of the Bridge

This study was performed on a bridge carrying southbound U.S. Route 15 over Interstate 66 (I-66) in Haymarket, Virginia. The bridge (structure number 14178) is 276 ft. (84.1 m) long and 42 ft. (12.8 m) wide and was built in 1979. The bridge is a two-span, six-girder steel built-up superstructure (Figure 1a) with a bare, cast-in-place 8 in. (200 mm) thick reinforced concrete deck constructed with removable forms. The southbound carries two lanes of traffic.

The deck condition (including corrosion, deterioration, delamination, and concrete degradation condition maps) has been monitored four times since 2009, most recently in May 2015 (in years 2009, 2011, 2014 and 2015). The NDE results from the periodic surveys will be used to better understand deck performance and monitor deck deterioration progression with time.

All the NDE measurements were made on a 2 ft. by 2 ft. (0.6 m by 0.6 m) grid, except for the GPR surveys which were conducted in the longitudinal direction of the bridge with survey lines 2 ft. (0.6 m) apart. It took two 5-hour (total 10 hours) lane closures to survey the deck. In other words, considering that the surveyed deck area was about 11000 ft^2 (1020 m^2), the surveys were conducted at production rates of about 1100 ft^2 (102 m^2) per hour. A section of the deck surface with clearly visible patches and spalling is shown in Figure 1b.

(a)

(b)

Fig. 1. (a) Side view of the bridge and (b) the deck surface of the southbound U.S. Route 15 bridge in Haymarket, VA.

3 NDE Technologies Description and Results

3.1 Electrical Resistivity (ER)

Rebar corrosion leads to concrete deterioration, delamination, contamination, and loss of rebar section. If the damages are not repaired in a timely manner, it will cause large cracks and areas of delamination, ultimately leading to spalling of concrete. Chloride ions typically penetrate from the surface into a bridge deck, resulting in a higher chloride concentration and creating a more corrosive environment. A corrosive environment and its correlated corrosion rate can be evaluated by the ER method. The electrical resistivity of concrete decreases as the moisture and chloride concentration increases (2). It has been observed that a resistivity of less than 5 kOhm cm supports very rapid rebar corrosion (3). A four-point Wenner probe is used for resistivity measurements (Figure 2).

The assessment of the corrosion progression is illustrated in Figures 3. The ER maps in Figure 3 describe concrete resistivity in kOhm-cm. The threshold for corrosive environment was identified to be 30 kOhm-cm based on correlations with other NDE methods. It can be clearly observed qualitatively that ER captured corrosion progression during the 6 year period. Expansion of corrosion affected areas and increase of severity of corrosive environment in 2015 occurs in the same areas identified as corrosive in 2009.

Fig. 2. Electrical resistivity survey.

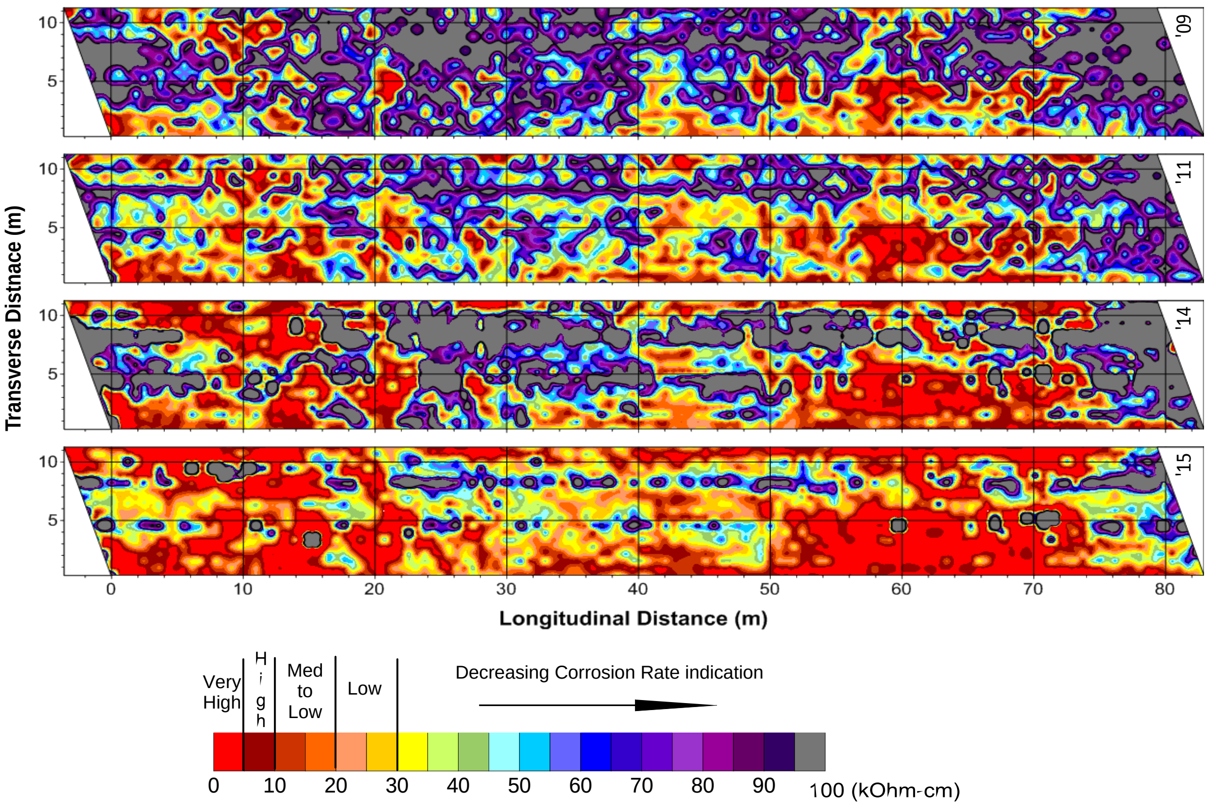


Fig. 3. Corrosion assessment maps from ER surveys conducted on the bridge deck from 2009 (top) to 2015 (bottom).

Since the data obtained from NDE technologies are quantitative, they can be used to assess the condition of decks quantitatively and compute a condition assessment number. The condition assessment number combines the effect of the extent and severity of deterioration obtained from the NDE surveys. The NDE measurements and the calculated condition assessment numbers, which vary from 100 for the best condition to 0 for the worst, could be entered to the bridge management systems to assist bridge owners in data-driven decision making regarding maintenance, rehabilitation, or replacement of the deck. For example, the most transportation agencies already have criteria for different intervention levels based on the assessment of deck area affected by delamination or corrosion, where corrosion assessment are obtained through chloride concentration or half-cell potential measurements. The condition assessment number of each type of defect is calculated as a weighted average of percentages of the deck area, with different severity levels of that specific defect. The area described as sound is assigned a weight factor of 100. The area with fair to poor grade is assigned a factor of 50, and the area in the state of severe condition a factor of 0.

The ER-based corrosion condition assessment numbers along with the percentages of the decks areas with various levels of anticipated corrosion rates are compared in Table 1 for the 2009 and 2015 ER surveys. The four corrosion states are defined by four electrical resistivity ranges: less than 10 kOhm-cm as high, 10 to 20 kOhm-cm as moderate, 20 to 30 kOhm-cm as low, and above 30 kOhm-cm as very low. The corrosion condition assessment number for the deck dropped from 86 to 41 in 6 years. The overall decrease in condition assessment numbers is well-reflected in the increase of deck area in different severity levels of corrosion. The majority of the deck area had very low corrosion rates in 2009, while the majority of the same had high to moderate corrosion rates in 2015.

Table 1. Corrosion Assessment from ER: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Very Low (%)**	**Low (%)**	**Moderate (%)**	**High (%)**
2009	86.3	72	15	10	3
2015	40.9	22	17	17	44

3.2 Ground Penetrating Radar (GPR)

A qualitative assessment of the deck condition can be made using a GPR survey through measuring the attenuation of electromagnetic waves on the top rebar level *(4)*. The amplitude of the reflection will be highest when the deck is in a good condition. The presence of moisture, chloride ions, iron oxide, cracks, and air-filled delaminations alter dielectric properties and increase the attenuation of electromagnetic waves. Thus, zones of highly-attenuated signal in GPR attenuation maps indicate locations of likely concrete deterioration, delamination, and/or corrosive environment. A GPR survey conducted using a 1.5 GHz ground coupled antenna is shown in Figure 4.

GPR condition maps obtained from GPR surveys of the deck in 2009 to 2015 are shown and compared qualitatively in Figure 5. The GPR threshold levels of deterioration are specific to the bridge conditions and equipment used and were obtained from correlations with other NDE technologies. The correlations between the attenuation levels and condition grades are indicated in the figure. A serious condition is described for both bridges with attenuation level of below -20 dB. Similar to ER, GPR qualitatively captured deterioration progression in the deck during the 6 year period. The progression and level of deterioration correlate well with the corrosion progression captured during the ER surveys. The similarity can be attributed to the fact that both measurements are primarily affected by the same elements affecting the electrical conductivity and dielectric value of concrete: moisture, chlorides, salts, etc.

Fig. 4. GPR survey using a 1.5 GHz ground coupled antenna.

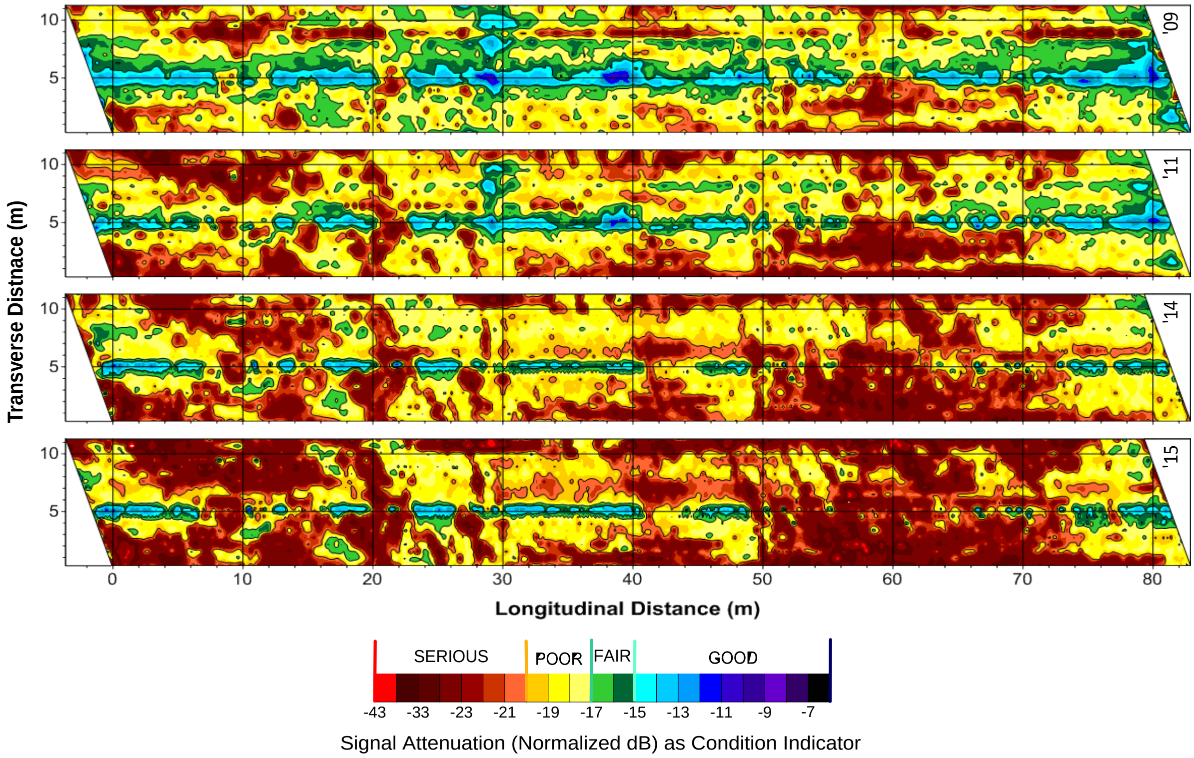


Figure 5. Deterioration condition maps from GPR survey on the bridge deck in 2009 (top) and 2015 (bottom).

To quantify deterioration progression, the condition assessment numbers along with the percentages of the decks areas at various severity levels of deterioration are compared in Table 2 for the 2009 and 2015 GPR surveys. The GPR condition assessment number for the deck dropped from 48 in 2009 to 22 in 2015. A large percentage of the deck area was in a poor condition in 2009, while the most of the deck area is currently in a serious condition.

Table 2. GPR Assessment: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Good (%)**	**Fair (%)**	**Poor (%)**	**Serious (%)**
2009	**48.1**	**14**	**24**	**41**	**21**
2015	**22.4**	**5**	**5**	**35**	**55**

3.3 Impact Echo (IE)

Concrete delamination is most often a result of rebar corrosion, though other types of concrete deterioration or repeated overloading, or a combination of those can also lead to delamination. IE has been successfully implemented in detecting and characterizing delamination in bridge decks *(5)*. IE measurements can be made by using a single IE probe, consisting of an impactor and a nearby receiver (figure 6a) or by using multiple IE probes (figure 6b), each consisting of an impactor and a sensor.

Fig. 6. Two types of IE testing systems: (a) an IE cane and (b) a stepper.

The primary objective of IE testing is to locate reflectors (bottom of the deck or a delamination) in the deck. The extent and position of reflectors can be estimated by analyzing the frequency response of the waves reflected from the reflector. The results from IE surveys on the deck are shown in figure 7. The delamination grades from IE surveys are defined based on the measured dominant response frequencies. The condition is described as good or sound when the depth of the reflector based on the measured response frequency matches the thickness of the deck. In the case of a delaminated deck, reflections of the compression wave occur at shallower depths, causing a shift in the response spectrum towards higher frequencies. Depending on the extent and continuity of the delamination, the partitioning of energy of waves being reflected from the bottom of the deck and delamination may vary. Initial delamination (fair condition) is described as occasional separations between the two deck zones. Thus it will have two distinct peaks corresponding to reflections from the bottom of the deck and the delamination. Progressed delamination (poor condition) is characterized by a single peak at a frequency corresponding to the depth of the delamination. Finally, in cases of wide or shallow delaminations, the dominant response of the deck to an impact is characterized by a low frequency response of flexural mode oscillations of the upper delaminated portion of the deck. This condition is graded as a serious condition and is always in the audible frequency range.

Figure 7 depicts the condition of the deck obtained from the IE surveys conducted from 2009 to 2015. The maps illustrate progression of delamination during the 6 year period.

Similar to the ER and GPR results, the condition assessment numbers from the 2009 and 2015 IE surveys, along with the percentages of the decks areas at various severity levels of delamination, are compared quantitatively in Table 3. The delamination condition assessment number for the deck dropped from 69 in 2009 to 40 in 2015. While a large percentage of the deck area was sound in 2009, the majority of the deck was extensively delaminated in 2015.

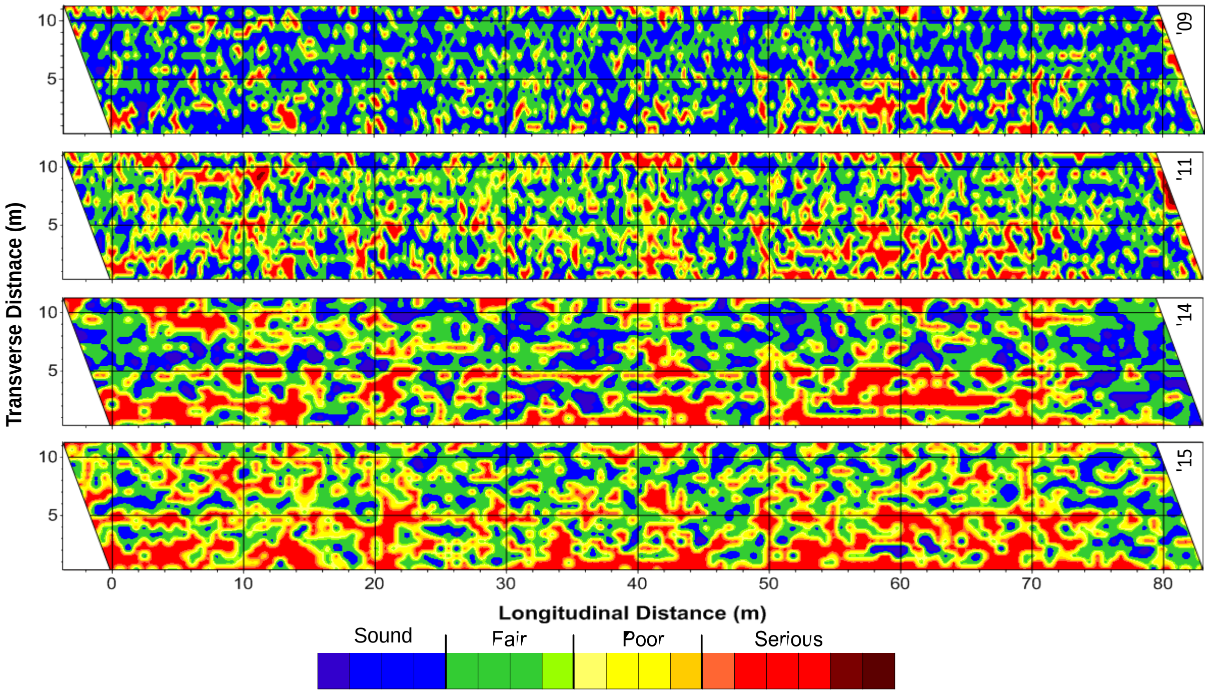


Fig. 7. Delamination maps from IE surveys conducted on the bridge deck from 2009 (top) to 2015 (bottom).

Table 3. Delamination Assessment from IE: Condition Assessment Number and Percentages of Deck Area.

		Distribution			
Bridge-Year	**Condition Assessment Number**	**Sound** (%)	**Fair** (%)	**Poor** (%)	**Serious** (%)
2009	69.5	54	26	4	15
2015	39.9	21	31	7	41

3.4 Ultrasonic Surface Waves (USW)

Besides corrosion, a number of other deterioration processes in decks occur as a result of repeated freeze and thaw, alkali-silica reaction, mechanical stressing, overloading, etc. These may lead to expansive stresses (cracks) within the concrete and either reduced mechanical properties or altered dielectric properties. The USW method is effective for assessing concrete degradation and detecting and measuring changes in mechanical properties. Surface waves are stress waves traveling along the surface of the deck, with their body extending to the depth of approximately one wavelength (6). Therefore, as long as the USW testing is limited to the wavelengths comparable to the deck thickness, the surface wave velocity will be controlled by concrete properties (elastic modulus). Devices like the portable seismic property analyzer (PSPA), shown in Figure 8, can be used in the evaluation of concrete modulus by the USW method. A variation in concrete modulus in the deck does not necessarily mean deterioration. Such variations can often be introduced at the time of construction due to material variation and placement procedures. Therefore, only a periodic measurement of changes in the concrete modulus would lead to identification of deterioration processes.

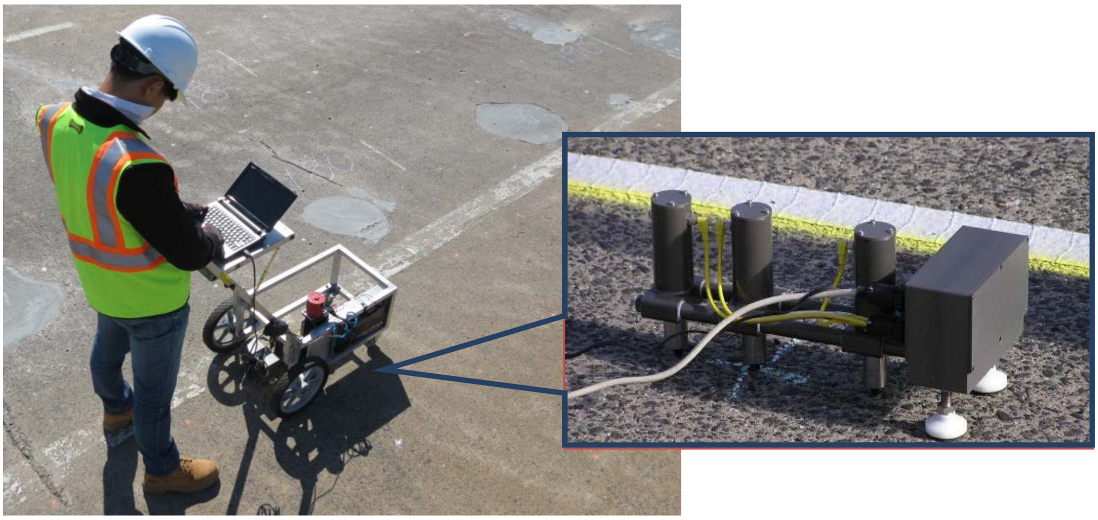
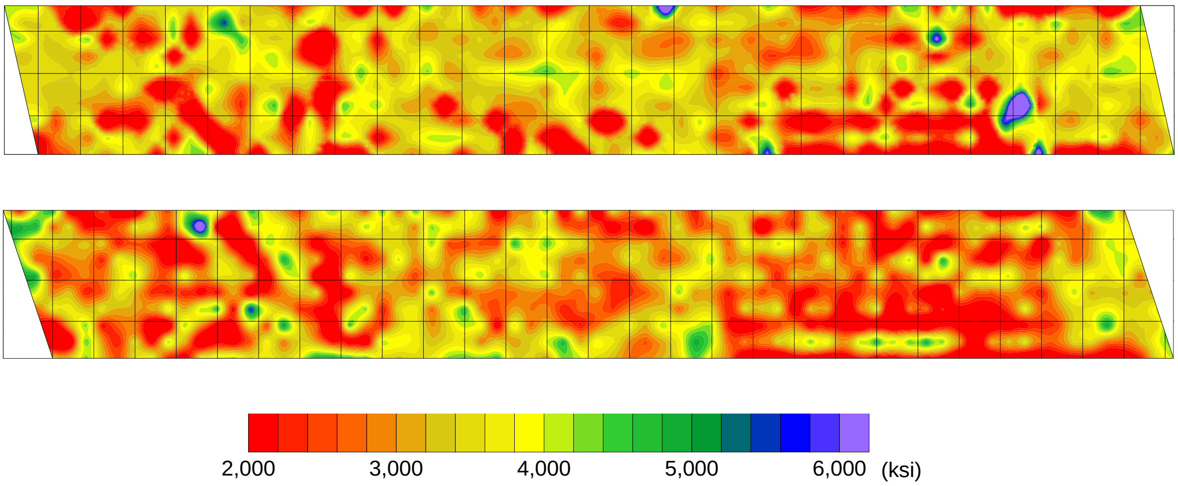


Fig. 8. Surface wave testing using a PSPA.

Concrete quality maps of the deck are shown in Figure 9 for the USW surveys conducted in 2009 and 2015. Areas of very low concrete modulus obtained from the USW testing are, in general, at locations of delamination. Similar to IE, concrete quality of the deck, in terms of the concrete modulus, decreased in 2015, compared to the one from the USW survey conducted in 2009.

The results of the 2009 and 2015 USW surveys are also presented quantitatively in Table 4 in terms of the percentages of the decks areas with various concrete elastic moduli. The quality of concrete in the deck did not change much from 2009 to 2015. A large percentage of the deck area had a modulus of less than 3,500 ksi (24 GPa).



1 ksi=6.89 MPa

Fig. 9. Concrete modulus maps from USW surveys conducted on the bridge deck in 2009 (top) and 2015 (bottom).

Table 4. Concrete Quality Assessment from USW: Average Modulus and Standard Deviation.

	Distribution				
Bridge-Year	**< 3,500 ksi (%)**	**3,500 – 4,500 ksi (%)**	**> 4,500 ksi (%)**	**Mean *E* (ksi)**	**STDEV (ksi)**
2009	66	29	5	3,286	705
2015	68	27	5	3,008	988

Note: 1 ksi=6.89 MPa

3.5 Conclusions

The condition of the deck of the southbound U.S. Route 15 bridge in Haymarket, Virginia was monitored over a period of 6 years to improve knowledge about the bridge deck performance and to better understand progression rates of different deterioration and defect types over an extended period of time.

Results of the surveys over a 6 year period show that NDE technologies have the ability to monitor deterioration progression with time, whether qualitatively through increase of deteriorated areas or quantitatively through changes in condition assessment numbers. Expanding deterioration and increasing severity of deterioration in 2015 occurs in the same areas identified as deteriorated in 2009 in all NDE condition maps. The condition assessment numbers facilitate development of more objective and realistic deterioration and prediction models for the bridge deck.

The complementary use of multiple NDE technologies identifies corrosion as the primary cause of damage in the deck. The NDE results provide strong correlation between the ER and GPR condition maps, explained by similar electrical properties affecting the two. Additional correlations are established regarding delamination detection between GPR and IE, and USW and IE, and concrete degradation between USW and GPR. These correlations also point to corrosion as the primary cause of deterioration for these decks.

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Disclaimer Statement

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Reflections on Quality Control Plans for Girder and Frame Bridges

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Abstract. With bridge Quality Control (QC) Plans Infrastructure Managers works every day to assure a desired quality with minimum traffic interruption balancing primarily technical and economic performance. These Quality Control Plans varies significantly among European countries, which urges the establishment of a European guideline. COST TU1406 Working Group 3 has the aim of establishing detailed explanation of the steps towards the establishment of a Quality Control plan. This paper reflect on Quality Control Plans for concrete girder and frame bridges with due reference to recent literature. Agreeing on and implementing a triggering criteria related to a degradations processes (e.g. corrosion) or a demand processes (e.g. traffic loading) calls for a deep understanding of the aforementioned processes as well as the measures needed in order to obtain quality data. Most data is gathered by visual inspection and Non-Destructive Techniques (NDT). In this paper, triggering criteria's and inspection interval and methods has been motivated.

Keywords: quality control plans, degradation processes, demand processes, inspection, maintenance, triggering criteria

1 Introduction

Roadway bridges are long living objects and even if they are of high initial quality, they may not meet quality requirements after some time due to degradation processes (e.g. corrosion) and/or change in demands (e.g. traffic loading).

This paper uses the same terminology as (Mainline, 2014), where:

Degradation is a general term covering the loss of performance of an asset, whether measured in terms of visual appearance or reduction of functionality (i.e. the ability to carry out its design requirements without restriction).

Deterioration is degradation caused by natural processes or through legitimate use of the asset. For example, this can take the form of rebar corrosion (whether carbonation or chloride induced) or spalling in reinforced concrete structures.

Damage is degradation caused by accidental events such as collisions, overloading or design and construction errors.

1.1 Quality Control Plans (QC Plans)

Quality control plans should define at which interval the quality controls are necessary and which condition the more detailed investigations or corrective actions are necessary. A general procedure for quality control has been proposed by Hajdin (Belgrade, 2016), ref. Figure 1.

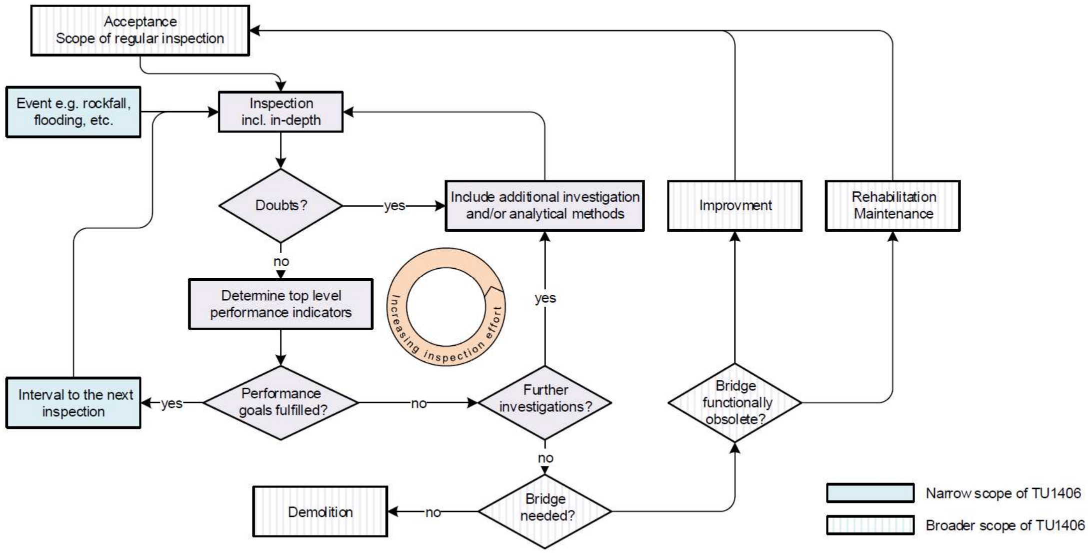

Fig. 1. General procedure for quality control by Hajdin (Belgrade, 2016).

Quality control plans should be based on a multilevel procedure, exemplified by the flow diagram developed by the Sustainable Bridges project, ref. Figure 2.

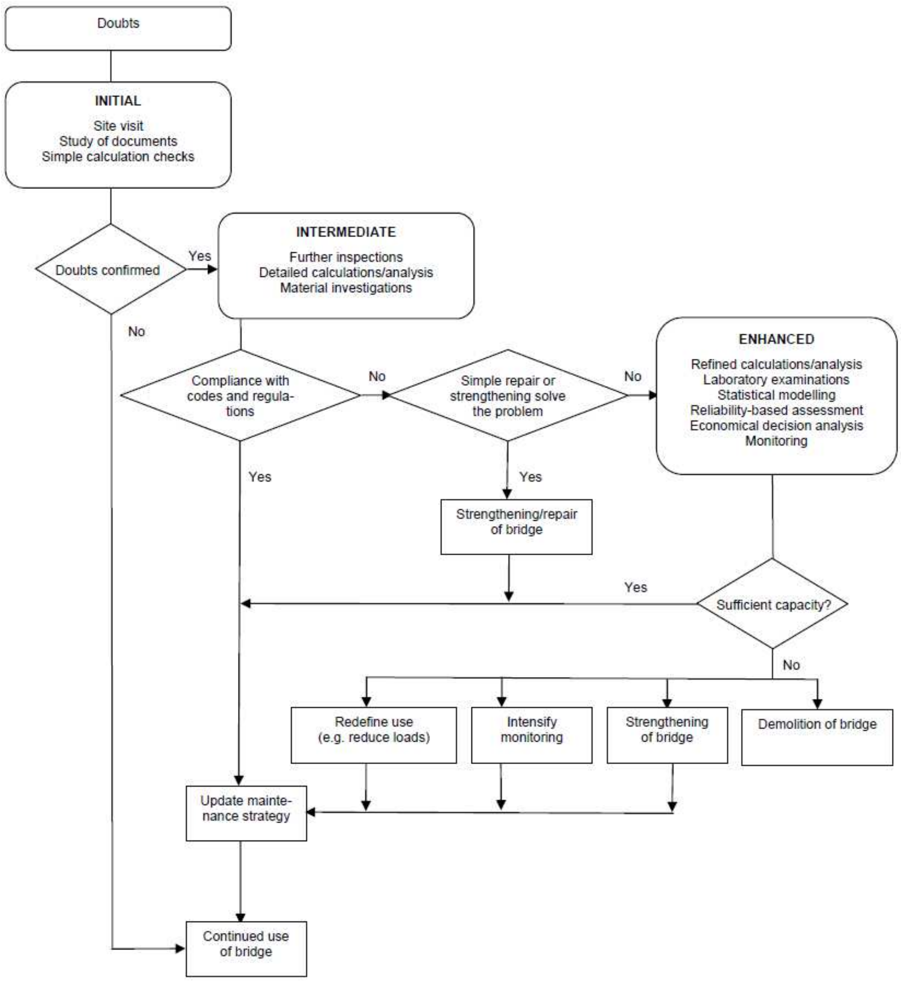

Fig. 2. Flow diagram for reassessment of exiting bridges, © Sustainable Bridges.

Assessment methods has also been recommended by (COST 345, 2002) as well as (Sustainable Bridges, 2007) for each level of assessment.

1.2 Performance Indicators (PI)

In relation to establishment of QC plans, a breakdown into low-level and top-level PI's is appropriate:

Low-level PI's:

- Findings: Cracks, spalling, colour change, leakage, efflorescence etc.
- In-situ measurements: Crack widths, half-cell measurements, accelerometer measurements, measurement of electrical resistance, geo radar map etc.
- Lab testing: Chemical analysis, mechanical testing on specimens etc.

Top-level PI's (ref. Figure 2) are based on low level PI's:

- Qualitative: Condition rating
- Quantitative: Probabilities of undesired scenarios

PI's are evaluated at the time of commissioning, inspections and interventions (maintenance, repair and replacement).

1.3 Performance Goals (PG)

Performance goals are understood as quality requirements. Performance goals could be satisfying a threshold or extremising (no threshold, i.e. maximising or minimising).

2 Scope of Work

Based on results from COST TU1406 WG1 and WG2 as well as existing approaches WG3 shall provide a methodology with detailed step-by-step explanations for establishment of QC plans.

Network issues (downstream consequences) as well as landmark bridges are not treated by WG3.

WG3 work in general comprise:

- Guideline on inspection intervals, methods and instruments
- Criteria for triggering more detailed investigations, safety and serviceability checks or maintenance interventions

QC plans will address the dynamics and uncertainty of the processes that may significantly comprise the bridge performance.

This paper only deals with Girder and Frame bridges. A vast majority of these bridges are made of concrete, i.e. metallic bridges are excluded from this document.

Methodology for handling uncertain information is not included in this paper. Furthermore, multi-criteria analysis etc. for prioritization is not included in this paper, refer to e.g. (Belgrade, 2016).

3 Triggering criteria related to degradation processes

In the following, degradation processes, inspection intervals and methods are summarised from existing literature related to concrete girder and frame bridges. This information constitute the background for a discussion of triggering criteria's.

3.1 Degradations Processes

The condition of a highway bridge can be detrimental affected by various factors. These may act singly or in combination to generate functional, load-carrying and long-term durability problems.

The defects that may appear on a bridge are the result of active degradation processes. The analysis of degradation processes and their relations with defects allows identification of defect causes. The information on degradation processes is very important for planning of preventive maintenance as well as for planning rehabilitation of a structure. By means of reliable information on the degradation processes, a maintenance strategy can be optimized in order to reduce the number of interventions (i.e. reduce Life Cycle Costs and traffic disturbance).

Defects result from:

- Design
- Materials
- Construction
- Loading
- Environmental conditions

In order to assess degradation processes, defects should be graded with respect to their nature, intensity and extent. Gradation should be done in a manner that fits the type of damage, the cause of damage, and the material forming the structural element. Degradation mechanisms in relation to defects for concrete bridges has been summarized by (Sustainable Bridges, 2007) and (COST 345, 2002).

3.2 Inspection Interval

Inspections are undertaken to:

- check the design assumptions underlying the quantification of some actions
- detect changes in use that could affect the safety or serviceability of a structure
- detect damage due, for example, to vehicle impact, ground movement and vandalism
- detect signs of structural distress due to overloading
- identify areas of material degradation (cause, extent and rate)
- provide a basis for determining structure-specific loads
- provide information of the cost-effectiveness of various remedial measures

As analyzed by (Sustainable Bridges, 2004) various inspection regimes have been implemented within Europe. The Sustainable bridges project suggested a common procedure, if no other procedure is required by national recommendations. Recommendations are also provided in Fib bulletin 22 (2003).

Furthermore, as summarized by A. Kedar at Cost TU 1406 meeting (Belgrade, 2016) inspection interval and type is infleunced by:

- Inspection quality
- Bridge condition
- Cost
- Bridge type
- Bridge location

3.3 Inspection Methods

With due reference to Figure 2 (multilevel assessment procedure) a phase wise division of inspection methods may be formulated, see Figure 3.

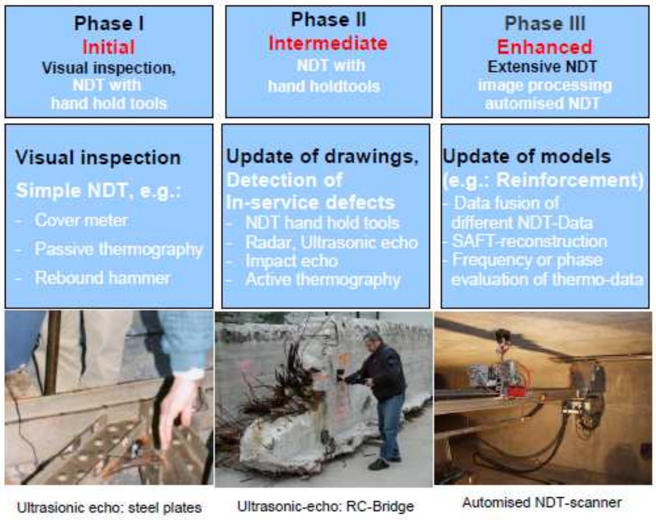

Fig. 3. Some examples for the choice of appropriate NDT-methods in a phase wise assessment © Sustainable Bridges.

Some test methods provide good overview others a good insight. The challenging in any investigation is finding the optimum combination, ref. Figure 4.

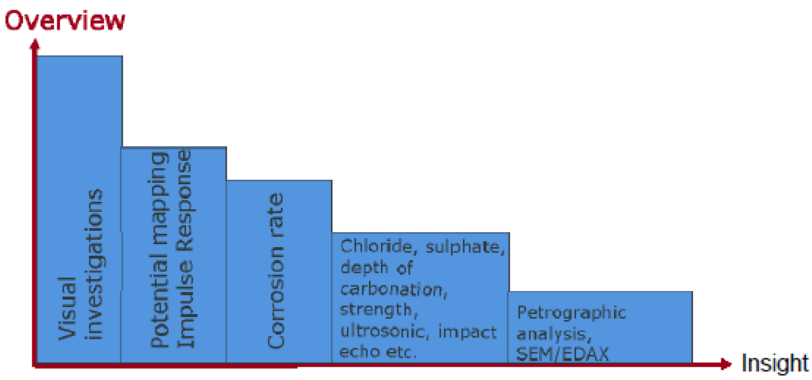

Fig. 4. Insight vs. overview © Sustainable Bridges.

For a summary of NDT and semi-destructive testing capabilities for concrete structures, ref. (COST 345, 2002).

For a more elaborate guidance on inspection and condition assessment of exiting bridges, ref. (Sustainable Bridges, 2007) and (Fib Bulletin 22, 2003).

Guidance on load tests and monitoring, ref. (COST 345, 2002) and (Fib Bulletin 22, 2003).

3.4 Triggering Criteria

A triggering criteria related to a degradation process defines if more detailed investigation/assessment or an intervention (maintenance, repair or replacement) is needed.

The condition of a highway bridge can be detrimental affected by various factors. These may act singly or in combination to generate functional, load-carrying and long-term durability problems. Because of this, there is no 1:1 relation between a vast amount of damage processes and a possible triggering criterion.

In many countries triggering criteria's are related to a top level performance indicator that may be qualitative (condition rating) or quantitative (probabilites of undesired scenarios). As a remark, condition rating in some countries does not account for in-service loads acting on the structure, i.e. it does not provide a direct measure of the level of safety and therefore of the priority of interventions.

ContectVet (ContectVet, 2001) has proposed a simplified and a detailed assessment procedure. These procedures are formulated for corrosion-affected concrete structures. Similar guidance is provided by ContectVet for frost and ASR affected concrete structures. For the two assessment procedures, ContectVet has formulated associated triggering criteria's (urgency of intervention). They may serve as inspiration for COST TU1406.

A detailed assessment ends in a management strategy. The management strategy should be technical and economic optimal given the actual constraints (budgets, politics etc.).

Fib bulletin 22 (Fib bulletin 22, 2003) also summarize reliability states, attributes and maintenance actions. The project REHABCON (REHABCON, 2004) has also summarized the decision process.

4 Triggering Criteria related to Demand Processes

4.1 Demand related Processes

Demands can be grouped as show below. Furthermore, it has been highlighted in ( ) whether demands are subject to maximisation or minimisation.

- **Operational**
  - Traffic volume (max)
  - Traffic loading
    - Gross Vehicle Weight (max)
    - Axle loads (max)
    - Width (max)
    - Height (max)
  - Maintainability (max)
  - Life Cycle Costs (min)
  - Visual appearance (max)
- **User**
  - Reliability (max)
  - Availability (max)
  - Safety (min. fatalities/injuries)
  - Affordable travel (min)
- **General (regulation by law or other measures)**
  - Human health (max)
  - Environmental protection
    - Climate change (min)
    - Noise (min)
    - Waste (min)
    - etc.

As shown above the concept of RAMS is included in the above list of demands. Also the Dutch concept of RAMSHEЄP is included in the above list of demands (Klatter at al, 2009).

The above demands may be contradictionary and are resolved in a political process and set by agencies in collaboration with professional organizations.

4.2 Inspection Interval and Methods

Below is listed previous demands, inspection interval and primary inspection methods.

Demand	Inspection interval	Primary inspection method
› **Operational**	› **Operational**	› **Operational**
› Traffic volume (max)	› Ad hoc	› Survey
› Traffic loading	› Traffic loading	› Traffic loading
› Gross Vehicle Weight (max)	› Ad hoc	› Inspection and assessment
› Axle loads (max)	› Ad hoc	› Inspection and assessment
› Width (max)	› Ad hoc (often once)	› Laser
› Height (max)	› Ad hoc (often once)	› Laser
› Maintainability (max)	› Same as degrad. process	› Visual inspection
› Life Cycle Costs (min)	› Ad hoc	› Desk study
› Visual appearance (max)	› Same as degrad. process	› Visual inspection
› **User**	› **User**	› **User**
› Reliability (max)	› Ad hoc	› Desk study
› Availability (max)	› Ad hoc	› Survey
› Safety (min. fatalities/injuries)	› Ad hoc	› Survey
› Affordable travel (min)	› Ad hoc	› Desk study
› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**
› Human health (max)	› Ad hoc	› Survey
› Environ. protection	› Environ. protection	› Environ. protection
› Climate change (min)	› Ad hoc	› Desk study
› Noise (min)	› Ad hoc	› Survey
› Waste (min)	› Ad hoc	› Survey
› Etc.	› -	› -

4.3 Triggering Criteria

A triggering criteria related to a demand process defines if more detailed investigation/assessment or an intervention (maintenance, repair or replacement) is needed.

These demands act singly or in combination. For some demands, there is a 1:1 relation between demand and triggering criteria.

Below are listed previous demands, primary triggering criteria and primary investigation/assessment or intervention. Triggering criteria's may be technical or economic, but even technical triggering criteria's are balanced by budget constraints.

Demand	Triggering criteria	Investigation/assessment or intervention
› **Operational**	› **Operational**	› **Operational**
› Traffic volume (max)	› User delay	› Capacity calc. / widening of bridge
› Traffic loading	› Traffic loading	› Traffic loading
› Gross Vehicle Weight (max)	› Bridge Class	› Reassessment or strengthening
› Axle loads (max)	› Axle load	› Reassessment or strengthening
› Width (max)	› Width	› Widening of bridge
› Height (max)	› Height	› Change of roadway profile (if possible)
› Maintainability (max)	› Budget constraints	› Refurbishment, upgrading
› Life Cycle Costs (min)	› Budget constraints	› Reassessment of operation and maintenance regime
› Visual appearance (max)	› Public perception	› Maintenance
› **User**	› **User**	› **User**
› Reliability (max)	› Codified Limit State/ Target Prob. of Failure	› Reassessment or strengthening
› Availability (max)	› Closure, restrictions	› Reassessment of operation and maintenance regime
› Safety (min. fatalities/injuries)	› Fatality/injuries	› Reassessment of operation, refurbishment
› Affordable travel (min)	› Cost	› Reassessment of operation
› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**	› **General (regulation by law or other measures)**
› Human health (max)	› …	› Survey
› Environ. protection	› Environ. protection	› Environ. protection
› Climate change (min)	› Global Warming Potential etc.	› Reassessment of operation and maintenance regime
› Noise (min)	› dBA	› Reassessment of operation and maintenance regime
› Waste (min)	› Tonnes	› Reassessment of operation and maintenance regime
› etc.	› -	› -

In future COST TU1406 work maybe a more appropriate presentation of the above would be a refined decision tree.

5 Summary

Bridge Quality Control (QC) Plans shall assure a desired quality with minimum traffic interruption balancing primarily technical and economic performance. QC Plans varies significantly among European countries, which urges the establishment of a European guideline. COST TU1406 Working Group 3 has the aim of establishing detailed explanation of the steps towards the establishment of a QC plan. Reflections on QC Plans for concrete girder and frame bridges with due reference to recent literature has been provided. Agreeing on and implementing a triggering criteria related to a degradations processes (e.g. corrosion) or a demand processes (e.g. traffic loading) calls for a deep understanding of the aforementioned processes as well as the measures needed in order to obtain quality data. Most data is gathered using visual inspection and Non-Destructive Techniques (NDT). Triggering criteria's and inspection interval and methods has been motivated in this paper with due reference to recent literature.

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Project performance appraisal frameworks as blueprints for bridge quality control

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Abstract. Various project performance appraisal frameworks (PPAFs) have been established in practice for engineering projects of both the public and private sector. Their aim is to measure the targeted and tangential attributes of project performance and conformance to specified quality standards. In this paper, the following PPAFs are summarily presented: CONQUAS and BDAS of Singapore, PASS and BAM of Hong Kong and SBTool of Portugal, Spain and Italy. Then, the current state of research regarding the key bridge performance indicators (KBPIs) is noted and considerations regarding the possible adaptation of a PPAF-inspired blueprint for a general quality appraising framework for roadway bridges take place. Concluding, the importance of lessons-learned and best practices in the establishment of a novel conceptual and computational framework is discussed.

Keywords: Project performance appraisal frameworks, quality standards, key bridge performance indicators, Quality Control plan, bridges

1 Introduction

PPAFs are the subjects of numerous research efforts and the corresponding development of cognitive, mathematical and software tools. Their field of implementation ranges from the holistic, full project lifecycle monitoring, to the evaluation of the satisfaction level of certain lifecycle notions (like constructability, buildability, sustainability, structural integrity, serviceability, operability and maintainability) and finally to the estimation of discretized constituents of the aforementioned notions (like the gross floor area, the percentage of formwork in relation to in situ components, the percentage of prefabricated elements in relation to the sum of structural parts, the project cash flow, the site productivity and others). Certain PPAFs have been developed by agents of the AEC industry or national authorities and have finally been established in practice, providing performance appraising guidelines for building, infrastructure and special project cases.

In this paper, specific PPAFs which are applied in practice are identified through a targeted literature review, and their conceptual and computational procedures are summarily delineated. Furthermore, after noting the current state of research regarding the key bridge performance indicators (KBPIs), considerations regarding the adaptation of a PPAF-inspired blueprint for a Quality Control plan for roadway bridges take place. Finally, both the importance of project performance appraisal lessons-learned and best practices in the establishment of a novel conceptual and computational framework are discussed.

2 Established PPAFs in practice

In this paper the implemented PPFAs will be compactly presented: the Construction Quality Assessment System (CONQUAS), of the Construction Industry Development Board (CIDB) in Singapore (Kam & Tang 1997, Low 2001); the Performance Assessment Scoring System (PASS) of the Hong Kong Housing Authority (HKHA) (Kam & Tang 1997); the Buildable Design Appraisal System (BDAS), of the Building and Construction Authority (BCA) in Singapore (Poh & Chen 1998, Low 2001, Lam et al. 2006, Ying & Pheng 2007, Lam & Wong 2009, Lam & Wong 2011, Nourbakhsh et al. 2012, Zolfagharian et al. 2012); the Buildability Assessment Model (BAM) (Wong et al. 2006, Lam & Wong 2009, Lam & Wong 2011, Nourbakhsh et al. 2012), utilized in Hong Kong; and the SBTool, developed by iiSBE (International Initiative for a Sustainable Built Environment), customized and utilized in Portugal, Spain and Italy (Larsson 2015, Bragança 2016).

2.1 Construction Quality Assessment System (CONQUAS)

CONQUAS was developed by CIDB in Singapore and compulsorily put in effect in 1989. It appraises the quality of public-sector building projects in terms of the structural frame, the assorted architectural works and the external works (Kam & Tang 1997). A scoring system, containing certain elements of the three aforementioned categories, which are awarded with quality points up to a corresponding per element threshold, is used by state evaluators to produce the basic CONQUAS building score. The adapted checklist of the scoring system is depicted in Fig. 1.

Table I. Items assessed regarding structural frame

Item	Points
Formwork	5
Reinforcement	10
Finished concrete	15
Concrete quality	8
Reinforcement quality	2

Table II. Items assessed regarding architectural works

Item	Points
Floors	8
Internal walls	8
Ceiling	4
Doors and windows	4
Rainwater down-pipes, plumbing, sanitary fittings	3
Installation of mechanical and electrical services	3
Components (permanent fixtures)	3
Roof	5
External walls	6
Material and functional tests	6

Table III. Items assessed regarding facilities of external works

Item	Points
Aprons and drains	2
Roadworks and carparks	2
Footpaths and turfing	2
Fencing and gates	2
Other areas, specific to the project	2

Fig. 1. Checklist of the CONQUAS scoring system (adapted from Kam & Tang 2007)

Positive correlations between structures with a high CONQUAS scores and the corresponding site productivity were found and validated (Low 2001). In order to provide an incentive for contractors to enhance and maintain their quality standards and improve their productivity, bonus and discount scoring thresholds were established, granting tendering advantages to contractors achieving or surpassing them (Kam & Tang 1997). The current thresholds in the continuously upgrading public-sector building schema are shown in Fig. 2.

Bonus/Discount Threshold Scores (1/4/2016 to 31/3/2017)

Building Category	Bonus Threshold Score for FY16	Discount Threshold Score for FY16
Residential	91.3	85.3
Commercial	92.9	86.9
Institution	89.2	83.2
Industrial/Others	85.9	79.9

Fig. 2. Contractor bonus and discount CONQUAS thresholds in the Fiscal Year of 2016 (adapted from Building and Construction Authority 2016)

Following the implementation of CONQUAS, the framework CE CONQUAS was developed for various engineering projects (drainage and sewerage works, marine structures etc.) and has been adopted since 1993. It featured analogous to the above depicted checklists.

2.2 Performance Assessment Scoring System (PASS)

Based on CONQUAS, PASS is the corresponding framework that has been adopted in Hong Kong since 1990. It features a similar to CONQUAS structure, with allotted points per specific elements concerning structural, architectural and external works, and also featuring a fourth miscellaneous category titled as “other obligations”.

However, unlike CONQUAS, which focuses on the evaluation of the quality standards of completed projects (thus providing a score classification of contractors), PASS also takes into account the contractor productivity and managerial performance (along with the conformance to the specified quality thresholds) in relation to projects currently under construction, featuring a system of allotted points for the management and organization of works, the resources flow, the coordination and control, the project documentation and the real-time schedule progress (Kam & Tang 1997). The discretized schema of interval assessment of PASS is featured in Fig. 3.

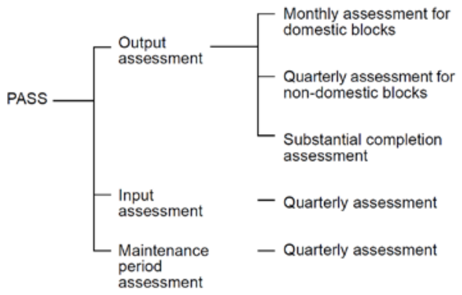

Fig. 3. PASS schema hierarchy (adapted from Kam & Tang 1997)

The depicted output assessment is related to the score related to construction itself, and the input to the productivity and managerial notions mentioned earlier.

2.3 Buildable Design Appraisal System (BDAS)

Buildability is “the extent to which the design of a building facilitates ease of construction, subject to the overall requirements for the completed building” (CIRIA 1983). Its implementation in the culture of the AEC industry was promoted in order to instigate a practical integration of design and construction, resulting in better deliverables, fewer discrepancies between the as-designed and as-built project states and more thorough satisfaction of the defined project objectives.

The BCA of Singapore, complementarily to CONQUAS put in effect by CIDB, developed and established BDAS, a scoring system measuring, classifying and awarding the buildability attributes of construction designs. High BDAS scores have positive correlations to high CONQUAS scores (Low 2001) and the other aforementioned notions that are essential for delivering projects of high quality, utility and conformance to specifications (Poh & Chen 1998, Low 2001, Lam et al. 2006, Lam & Wong 2009). BDAS promotes the buildability of designs through the 3S principle: standardization, simplification and single integrated elements (Zolfagharian et al. 2012). Standardization refers to the repetition of grids, component sizes and connection details; simplicity, to the utilization of construction systems and installation details of low complexity; and single integrated elements, to the combination of multiple components to form composite elements (Low 2001). Following the implementation of the 3S principal, BDAS features a scoring system regarding the project construction and reflecting the buildability level considered in the design. The system is depicted in Fig. 4.

CONQUAS and BDAS form a composite project quality and performance assessment framework in Singapore, primarily targeted to high-rise buildings, but also readily adapted for engineering projects in general.

2.4 Buildability Assessment Model (BAM)

Alike PASS being inspired by CONQUAS, BAM is inspired by BDAS in considering the buildability of project designs (Lam & Wong 2009). PASS and BAM form a combined assessment framework in Hong Kong.

The main difference between BDAM and BAM is that the latter extends the 3S principle into a set of nine buildability factors, namely the economic use of the contractors’ resources, the easy visualization and coordination of design requirements by the site staff, the development and adoption of alternative construction details, the overcoming of restrictive site conditions, the standardization and repetition, the freedom of choice between prefabricated and on site works, the simplification of construction details in case of non-repetitive elements, the mitigation of adverse weather impact by enabling flexible construction schedules and the consideration of site work sequencing in the designs (Lam & Wong 2011). The BAM framework is depicted in Fig. 5.

Buildability Score of Building	=	**Buildability Score of Structural System (including Roof System)** **+ Buildability Score of Wall System** **+ Buildability Score of Other Buildable Design Features** **+ Bonus Points**
BScore (In mathematical terms)	=	**50[∑(As x Ss)] + 40[∑(Lw x Sw)] + N + Bonus points**
where As	=	Asa / Ast
Lw	=	Lwa / Lwt
Aw	=	Awa / Awt
As	=	Percentage of total floor area using a particular structural design
Ast	=	Total floor area which includes roof (projected area) and basement areas
Asa	=	Floor area using the particular structural design
Lw	=	Percentage of total external & internal wall length using particular wall system
Lwt	=	Total wall length excluding external basement wall for earth retaining purpose
Lwa	=	External & internal wall length using particular wall system
Aw	=	Percentage of total external & internal wall areas using particular wall design
Awt	=	Total wall area, excluding perimeter wall of the basement. All internal walls in the basement are to be considered.
Awa	=	External & internal wall areas using particular wall design
Ss	=	Labour saving index for structural design
Sw	=	Labour saving index for external & internal wall design
N	=	Buildability Score for other buildable design features
Bonus points	=	Bonus points for single integrated components

Fig. 4. The BDAS scoring system (adapted from Lam et al. 2006)

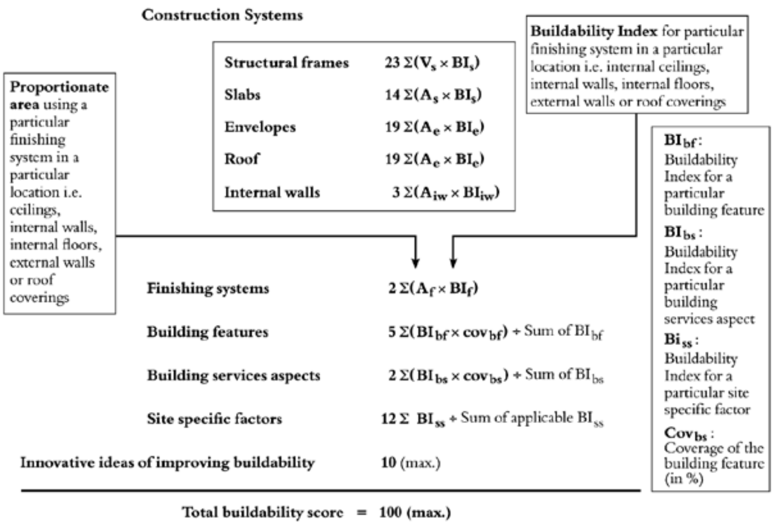

Fig. 5. The BAM framework (adapted from Lam & Wong 2009)

2.5 SBTool

The SBTool was firstly developed by iiSBE in 2007, overhauling the previous tool known as GBTool. It is a generic performance assessment framework for rating the sustainable performance of sites and building projects (Larsson 2015), and has since been customized and adapted for use in Portugal, Spain and Italy. The system can be used by: a) authorized organizations, such as municipalities or non-governmental organizations (NGOs) to establish rating systems suiting their own regions and building types, b) owners and managers of large building portfolios to specify their performance requirements to their staff and consultants, and c) educators of graduate engineering students (Larsson 2015). It takes into account many sustainability indicators discretized according to the social, environmental and economic sustainability dimensions (Bragança 2016). These indicators are then benchmarked through the principles of conventional and best practice (Bragança 2016).

SBTool features a top-down layout, with a core framework encompassing established, regional and generic sustainability standards, requirements, thresholds and specifications, which then spawns separate and targeted computational sheets referring to and producing the sustainability score of specific projects. The general methodology of the SBTool is depicted in Fig. 6.

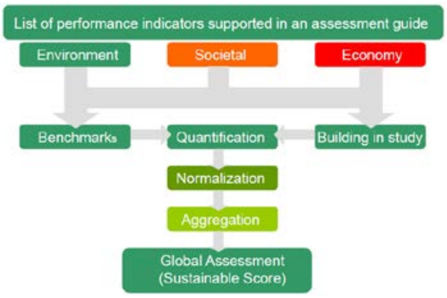

Fig. 6. The SBTool methodology (adapted from Bragança 2016)

In the resulted sustainability score, the performance values obtained for each parameter and indicator are normalized on a scale between 0 (reference/conventional value) and 1 (best performance). Then, the quantified values are converted in a graded scale, form A+ to E, thus producing the sustainability grade of the project. The normalization equation, along with the graded scale, is depicted in Fig. 7.

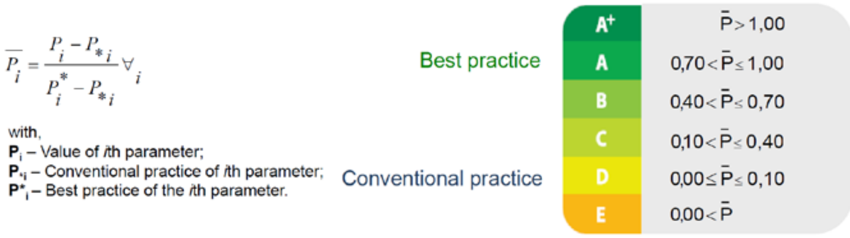

Fig. 7. The indicator value normalization equation and the graded scale of the SBTool (adapted from Bragança 2016)

3 Integration of KBPIs and PPAFs

In the context of the COST Action – TU1406, KBPIs are discretized according to five homogenized categories: (i) the defect corresponding to the KBPI, (ii) the relation of the KBPI to certain parameters (material properties, equipment and protection, geometry changes, bearing capacity, structural integrity and joints, original construction sequence and design, dynamic behavior, environmental exposure), (iii) the KBPI rating, (iv) the cost and importance related to the KBPI and (v) the loads corresponding to the KBPI.

All of the aforementioned PPAFs utilize databases in checklist format, with indicators that are discretized into categories (e.g. structural system, wall system and design properties in the case of BDAS) and then calculated according to the corresponding PPAF methodology. Since KBPIs are already discretized in a similar way, the applicability of the presented PPAFs can be logically deduced. By swapping the overhead system categories with the five homogenized KBPI ones, and then substituting the corresponding indicators with the KBPIs, all of the presented PPAFs could potentially be used as blueprints for bridge quality appraisal. The mathematical, computational, normalized and interface-related elements of each PPAF could also be readily adapted for KBPIs, since the generalized form and weight/point allocation scheme of the said elements is not greatly affected by the aforementioned indicator substitution.

Among the presented PPAFs, SBTool seems to be more suitable for a quality appraising blueprint, mainly due to the following: a) it is the only sustainability-oriented PPAF, thus offering a head start for the sustainability considerations related to the KBPIs, b) it is the only PPAF adapted and validated in practice in European countries, c) its mathematical schema ensures that as many KBPIs as desired can be used, since all elements are in the end normalized into a single scale. In this way, no substitution is required, and all KBPIs can be taken into account in addition to the already existent SBTool indicators (if such thing is deemed necessary), d) its versatility ensures an easier adaptation to infrastructure projects and e) it relies not only on expert input, like the rest of the PPAFs, but also in specific mathematical methodologies like multivariate and linear regression (Bragança 2016) and machine learning schemes like artificial neural networks (Bragança 2016). Thus, an extra layer of robustness is added to the framework.

4 Conclusions

PPAFs already used in practice can provide valuable data concerning best practices and lessons-learned in regard of project performance and quality appraising. Case studies and applicational examples of such frameworks, especially those easily adaptable for infrastructure projects and lifecycle performance (including the aspects of sustainability and quality), should be collected, scrutinized and serve as blueprints for the establishment of the performance goals and the production of a quality appraisal framework plan for bridges. Thus, past experience will be reclaimed and problematic or bottlenecking aspects will be located and dealt with more efficiently.

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Value-based method for condition assessment and management of bridges

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Abstract. This paper presents a value-based assessment method to assess the condition of a bridge based on visual inspection. The method has been developed following MIVES, a multi-criteria model for decision making that evaluates alternatives to solve a defined generic problem through an index value. This method increases the objectivity and consistency in the assessment of bridges by establishing general criteria to identify damages and automatically quantifying their relative importance. For this purpose, bridge components and types of damage have been classified, damage indicators have been defined, and relative weights and value functions have been assigned to each indicator. The value-based assessment method has been implemented in a tool that provides a global condition index of the bridge, reports specific warnings for indicators that exceed alarm thresholds, and recommends repair actions.

Keywords: bridge management, condition index, prioritization, value-based analysis, MIVES

1 Introduction

This paper presents a value-based method and tool to assess the condition of a bridge based on visual inspection. This assessment tool has been developed following the Integrated Value Model for Sustainability Assessment or MIVES, a multi-criteria methodology for decision making that evaluates each of the alternatives that can solve a defined generic problem, through an index value. MIVES has been developed by Tecnalia, the University of the Basque Country, the Technical University of Catalonia and the University of Coruña (MIVES I, II, III and IV).This methodology is included within the multi-attribute utility theory, because to get the index value of each alternative, a weighted sum of the valuations of the considered criteria is done assuming that there is certainty. That is, the preferences of the decider, with respect to the proposed indicators, are known. MIVES has already been used in number of applications for the sustainability assessment of construction elements and projects, such as the sustainable design of concrete structures of the Spanish Structural Concrete Code (Ministerio de Presidencia, 2008) and instruction of structural steel (Ministerio de Presidencia, 2011).

2 Description of the assessment methodology

Experience on bridge assessment based on visual inspection demonstrates that the importance attributed to the same damage by different inspectors, even if experts, is different. To minimize the subjectivity in the evaluation of damages, it is crucial to develop an inspection tool with an exhaustive damage catalog. This allows to establish a general criterion to identify defects, to assess their relative importance and to assess the general condition of a bridge.

The assessment tool developed here will require the inspector to indicate just the type, location and extension of the observed damage. The tool will automatically assign a weighted value of the damage depending on their location and material by intrinsically considering how severe the damage is, how it is expected to evolve and how it affects other elements of the bridge. For this purpose, the bridge components and types of damage have been classified, and damage indicators have been defined. Relative weights and value functions have been assigned to each indicator following the MIVES methodology.

2.1 Bridge component classification

The assessment methodology classifies the elements of a bridge in four different levels, as shown in Table 1. The highest level, Level 1, corresponds to the entire bridge system, which is broken down into five components in Level 2: foundation, substructure, superstructure, connecting elements and equipment. Levels 3 and 4 deepen this decomposition into elements and sub-elements, respectively. For example, the substructure is decomposed into abutments, pier and arches (Level 3), and subsequently into Abutment 1, Abutment 2, Pier 1, Pier 2, etc. in Level 4.

Table 1. Classification of bridge elements in levels

LEVEL 1	LEVEL 2	LEVEL 3	LEVEL 4	Damage
BRIDGE	FOUNDATION	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
	SUBSTRUCTURE	Abutments	Abutment 1 Abutment 2, etc.	
		Piers	Pier 1, Pier 2, etc.	
		Arch	Arch 1, Arch 2, etc.	
	SUPERSTRUCTURE	Deck	Span 1 Span 2, etc.…	
		Arch	Arch 1, Arch 2, etc.	
	CONNECTING ELEMENTS	Bearings	Bearings in Abutment 1, Bearings in Pier 1. etc.	
		Dilatation joints	Transverse, longitudinal	
		Cables	Cables in Pilon 1, Cables in Pilon 2, etc.	
	EQUIPMENT	Pavement	Direction A, Direction B.	
		Drainage system	Substructure, superstructure	
		Safety Elements	Railings, protection elements , etc.	
		Protection Elements	Substructure, superstructure	

2.2 Library of types of damages

A library of damage has been defined for different structural materials. The library contains 66 types of damage for concrete elements, 62 types of damage for steel elements and 35 types of damage for masonry elements. Damage for wooden structures will be added in the future. Damage caused by both poor resistance and poor durability are considered. In addition, the library contains over 100 types of damage related to connection elements (bearings, dilatation joints, cables, etc.) and equipment (pavements, barriers, drainage systems, etc.).

2.3 Damage indicators

Damage indicators have been defined based on the Spanish guidelines for main roadway bridge inspections (Ministerio de Fomento, 2012), which considers three different indicators for each damage type: *damage extension*, *damage severity* and *damage evolution*. A forth indicator has been proposed here, *damage implication,* to evaluate how easily a damage can affect other elements of a bridge or can trigger other damage mechanisms.

- *Damage severity*: evaluates the capacity reduction of an element to fulfill its specific function as a result of the damage. It is classified as null, very low, low, medium, high or very high.
- *Damage implication***:** evaluates the repercussion of the damage in an element into other types of damage or elements. It is classified as null, low, medium or high.
- *Damage evolution:* evaluates the probability that the damage will rapidly worsen. It is classified as null, slow, medium or fast.
- *Damage extension:* evaluates the surface or volume affected by the damage with respect to the total surface/volume of the element. It is classified as less than 10%, between 11% and 30%, between 31% and 50%, between 51% and 75% and more than 75%.

2.3.1 Indicators’ weights

The relative weights of each of the indicators (damage severity, evolution, involvement and extension) have been assigned using the Analytical Hierarchy Process (AHP) decision method (Saaty, 2006) shown in Figure 1.

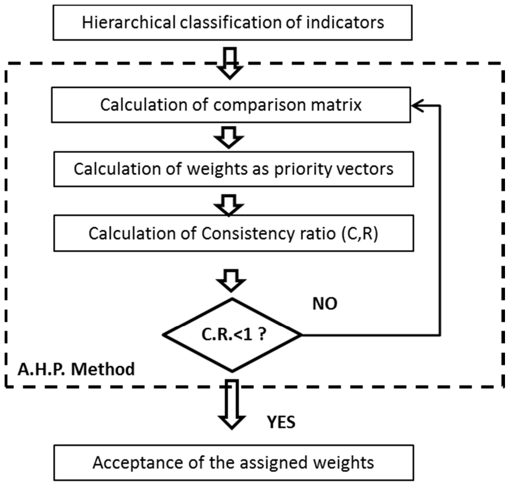

Fig. 1: Analytical Hierarchy Process for assigning weights

In this method, the relative weights are assigned based on the relative importance of an indicator with respect to the rest as defined by a number of expert decision-makers. For this purpose, each expert has developed a pairwise comparison between indicators using the criteria presented in Table 2 (Saaty, 2006).Based on these comparisons, a comparison matrix is built. The eigenvalue of the resulting matrix are the relative weights assigned to the indicators.

Table 2 Pairwise comparison to build decision matrix (Saaty, 2006)

Relative importance R_{ij} (i compared to j)	
Verbal scale	*Numerical scale*
Extremely more important	9
Much more important	7
More important	5
Slightly more important	3
Equally important	1
Slightly less important	1/3
Less important	1/5
Much less important	1/7
Extremely less important	1/9

To define the comparison matrix of the four damage indicators, pairwise comparisons have been conducted by ten engineers with broad experience in bridge inspection, damage assessment and structural design and rehabilitation. The consistency between the pairwise comparisons of each matrix received was checked with the method proposed by (Alarcón, 2011).The individual comparison matrices have been averaged and the weights have been calculated as the eigenvalues of the averaged comparison matrix.

The libraries of damages have been also evaluated by expert decision-makers and each damage is classified by values of different alternatives, considering the weights of severity, implication and evolution indicators (the extension depends on the on-site verification).

2.3.2 Defintion of the Value Functions

Defining the value function requires measuring preference, or the degree of satisfaction produced by a certain alternative option for a given indicator. Each indicator may be given in different units; therefore, it is necessary to standardize them into units of value or satisfaction, which is basically what the value function does. The method proposed rates satisfaction on a scale from 0 to 1, where 0 reflects minimum satisfaction (S_{min}) and 1 reflects maximum satisfaction (S_{max}).

To determine the satisfaction value for an indicator, the MIVES model (MIVES I, II, III and IV) outlines a procedure consisting of the four following stages:

1. *Definition of the tendency (increase or decrease) of the value function*. The value function can be increasing or decreasing depending on the nature of the indicator. An increasing function is used when an increase in the measurement variable results in an increase in the decision maker's satisfaction. In contrast, a decreasing value function shows that an increase in the measurement unit causes a decrease in satisfaction (see Fig.3).
2. *Definition of the points corresponding to the minimum (S_{min}, value 0) and maximum (S_{max}, value 1) satisfaction*. The points of minimum and maximum satisfaction define the limits of the value function on the x-axis: Smin (point of minimum satisfaction) and S_{max} (point of maximum satisfaction). These two points have a satisfaction value of 0 (S_{min}) and 1 (S_{max}) respectively. These limits correspond to the satisfaction values and not necessarily to the minimum and maximum values of the measurement variables, which may have (and will in general have) a wider range.
3. *Definition of the shape of the value function.* Four types of functions are suggested: concave, convex, linear and S-shaped. These four curves represent the most common relationships that can be found in practice. They include the modeling of the different behavior of people regarding their indifference, aversion or attraction to risk with respect to the decisions to be taken, in addition to the different strategies that can be defined in order to promote improvement. The latter is related to the range between the points of minimum and maximum satisfaction where the satisfaction increases more rapidly.
4. *Definition of the mathematical expression of the value function.* Equation (1) is proposed as a mathematical model for the value function.

$$V_{ind} = B \cdot \left[1 - e^{-K_i \cdot \left(\frac{|X - X_{mín}|}{C_i} \right)^{P_i}} \right] \quad [1]$$

$$B = \frac{1}{\left[1 - e^{-K \cdot \left(\frac{|S_{máx} - S_{mín}|}{C} \right)^{P}} \right]} \quad [2]$$

Where:

- V_{ind} is the value of the indicator being evaluated.
- B is a factor that allows the function to remain within the range from 0 to 1. It is assumed that the highest level of satisfaction has a value of 1. This factor is determined by Equation (2).
- S_{min} is the point of minimum satisfaction, with a value of 0.
- S_{max} is the point of maximum satisfaction, with a value of 1.
- X is the abscissa that generates a value equal to V_{ind}.
- P defines approximately the shape of the curve: concave, convex, linear or S-shaped. If $P < 1$ the curve is concave; if $P > 1$ the curve is convex or S-shaped; if $P = 1$ it is linear.
- C is a parameter that approximately defines the x-value of the point of inflexion for curves with $P > 1$.
- K is a parameter that approximately defines the y-value at the point C.

When the specific shape of the value function for an indicator is unclear, it may be defined by a working group. When this is the case, several value functions (discrete or continuous) may initially be defined according to the proposals given by each or some of the members of the group for the measurement variable (indicator). This means that rather than a single function, a combination of functions is obtained, as shown in Figure 3.

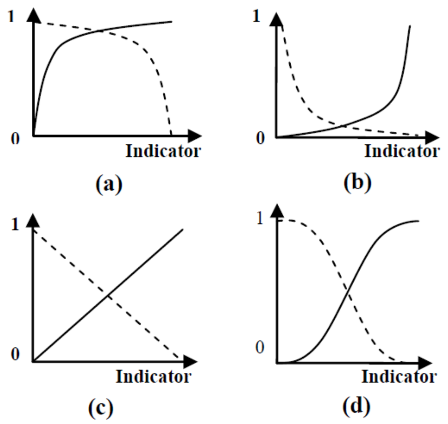

Fig. 3. Different types of value functions (Alarcon, 2011)

a – convex; b – concave; c –linear; d – S-shaped

After determinating the importance of the indicators (damage severity, damage extension, damage evolution, damage implication) and assigning value functions to each alternative, the relationship among them is set by the resulting some damage according to the bridge material and elements.

Therefore, damage observed during the inspection and their extension are introduced in the tool and the condition value of the element is returned.

3 Implementation of the assessment methodology

The methodology has been implemented in a tool to allow an automated assessment of the bridge condition. With this tool, the bridge inspector has to determine only the type of damage observed, their location and extension. The tool automatically assigns a weighted value to the damage based on the implemented methodology, and a global condition index is provided to the decision maker. This allows a more objective and consistent assessment of bridge condition.

The global condition index used here is that defined by the Spanish guidelines for main roadway bridge inspections (Ministerio de Fomento, 2012) (see Table 3). Weighted average from 1 to 6 is obtained via the linear function determinated by the data obtained from the inspection of the elements. The assessment tool also provides condition indexes for each element or group of elements, reports specific warnings for indicators that exceed alarm thresholds, and recommends repair actions.

Tabla 3. Global condition index

Índex	Definition
1	Apparently undamaged structure or minor defects with no consequences
2	The structure has defects that can evolve into structural damage, or might need to be repaired in the short or medium term
3	The structure has defects that indicate onset of structural damage
4	Defects in the structure indicate that there is an ongoing process of structural damage. This situation requires a more detailed inspection in the short term, or a repair action in the short or medium term.
5	The bridge has damages that cause a modification of the structural behavior. A special inspection or repair action is needed in the short term.
6	The damages are such that the structure is approaching its serviceability limit state. The bridge has to be closed or its use restricted. A special inspection and an urgent repair action are required.

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Quality Control Plan for Earth Retaining Walls – Conceptual Framework

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Abstract. Bridge Management Systems and Quality Control Plans (QCP) for Bridges are widely known, available and used worldwide. However, concerning earth retaining walls, few management systems are known and the literature related with this specific topic is scarce. Regarding an adequate approach throughout earth retaining wall's life-cycle, as well as an optimal management of the road assets as whole, this paper presents a proposal for a QCP for earth retaining walls in view of the road network under the control of the general concessionaire Infraestruturas de Portugal, S.A.

Keywords: Quality control plan, earth retaining wall, asset management, inspection

1 Introduction

Infraestruturas de Portugal (IP) is the state owned Portuguese general concessionaire for roadways and railways, managing over 5300 roadway bridges and more than 13 000 kilometers of motorways. Despite bridge and road pavements management systems are implemented for several years with significant results, the management of earth retaining structures remains with a reactive approach, instead of the preventive approach made possible with the implementation of a periodic assessment (Amado and Freire, 2014a).

In view of the development of a quality control plan and a management system for this type of assets, an inventory campaign was performed, aiming to a more objective development of a QCP.

Several concepts and definitions were necessary to establish the terms of a detailed and standardized inventory, therefore a Manual for the Inventory of Earth Retaining Structures was developed prior to the inventory campaigns (Estradas de Portugal, 2011). The analysis of the results of these campaigns allows for a more clear definition of the activities to be included in a future quality control plan.

This paper summarizes the proposal of a QCP for the earth retaining structures included in the IP's road network, attending to the results of the inventory campaigns, as well as the already implemented practices related with bridge management. In this particular topic it was considered that the similarity with established practices related with bridge management would bring important advantages and synergies.

The Quality Control Plan was assumed as a process with specific intervenient, interrelated activities, both internal and external to the process, specific inputs and standard outputs.

2 Assets definition

Inventory campaigns were based in broad criteria, having in mind that a more detailed approach could be postponed to a subsequent phase of the QCP, according to the results of this 1st phase. This first approach was based in two main criteria related with the function of the structures and their location.

The *functional* criterion was established as "to consider geotechnical structures with the aim of soil retaining". The *location* was considered as "included between lanes, roadsides, slopes, earthworks and expropriated land". A third criterion was included to consider eventual interaction with other assets, namely bridges, in order to avoid duplication of inventories in several databanks (Estradas de Portugal, 2011).

2.1 Types of structures included in the inventory

The inventory campaign was defined to include 6 different types of earth retaining structures: gravity walls, cantilevered walls, piling walls, anchored walls, soil-strengthened and soil nailing. Special cases of earth retaining walls that do not fit in these categories were considered as "others" or "mixed solution" (Estradas de Portugal, 2011).

The analysis of the data obtained in the inventory campaigns revealed a non-uniform distribution of the assets in these categories, with approximately 98% of the assets classified as gravity or cantilevered walls (Amado, 2015).

According to the methodology proposed by SÉTRA (2005), two mains groups of assets can be defined, according to the existence of buried elements such as piles or anchorage. Therefore, "simple structures", without buried structural elements, were defined as those that can be assessed in visual inspections, performed by technicians specifically trained to the task. On the other hand, "complex structures" were defined as those whose condition cannot be derived from a simple visual inspection, due to the existence of structural buried elements, needing an in-depth inspection performed by structural/ geotechnical engineers.

Having in mind this definition gravity and cantilevered walls were considered as "simple structures" and the other four of the six defined types were considered as "complex structures".

2.2 Criteria derived from the inventory campaigns

Despite inventory was performed independently from structure jurisdiction, height and distance to the road, these information was thoroughly recorded in each inventory.

Inventory campaigns were also designed in order to contribute to a prioritization of further actions such as inspections. An attempt was made to link this to the concept of "consequences of a collapse". Therefore, this contribution was given through the inclusion in each inventory of information related with the following topics, independently from the rest of the data and disregarding any redundancy (Estradas de Portugal, 2011):

- Existence of constructions, habitational, services or industry in the surrounding area;
- Installed monitoring equipment;
- Existence of anchorages.

In future phases of implementation of the QCP periodic actions such as inspection campaigns can be planned to start in the most critical structures, defined by crossing this criteria with other data such as maximum high, structure type or traffic.

2.3 Assets to include in the QCP

Considering that the development of a QCP would benefit if it considers the size of the set and the general specifications of the assets to be included, a first approach to the set of assets to include in the QCP was proposed (Amado, 2015):

- To include all structures under the company jurisdiction with 2 meters high or more;
- To include all structures defined as "complex structures" independently from jurisdiction and height;
- To include all "simple structures" falling out of the company jurisdiction, with height higher or equal to 2 meters, if the minimum distance to the road is less than the maximum structure height.

In future developments these set of structures can be refined, opting for an individual analysis of the inventory of structures falling in a minority situation.

3 Activities included in the QCP

It was intended that the processes included in QCP should be suitable to manage the previewed number of structures calculated according to the previous criteria, having in mind the company reality and current internal practices in bridge management.

According to Hearn (2003) visual inspections are suitable to the establishment of a condition assessment of earth retaining walls, if necessary combined with complementary investigations. Periodic inspections at regular intervals are needed since some defects may occur in hidden elements, only becoming evident when causing other defects in other parts of the structure. Therefore, regular monitoring may help the estimation of the remaining service life (Brutus and Tauber, 2009).

The QCP was developed considering periodic visual inspection as the core activity of the process, to be completed with occasional activities, internal or external to the defined process.

3.1 Periodic activities

Periodic activities were defined as inspections performed at specific intervals, according to an established practice and standard reporting of observed conditions and deterioration. A maximum interval of 5 years between consecutive inspections is proposed in the literature, even to privately owned retaining walls (Brutus and Tauber, 2009).

Two different types of periodic inspections are proposed in the QCP, seeking an affinity with the company Bridge Management System (BMS) (GOA®), namely Routine Inspections and Principal Inspections. The two types differ in periodicity and detail, which is related with the level of specialization of the technicians who perform each of them. Based on the methods developed by SÉTRA (2005, 2008), it was defined that Principal Inspections would be performed by specialized technicians (engineers) and Routine Inspections by technicians trained on the method. Therefore, Routine Inspections will focus on the visual observations, without inferring the causes or consequences of the observed defects, and being supported by specific bibliography such as a defect catalog.

Regarding periodicity, as a first approach to calculate the total number of human resources needed to implement the QCP, and bearing in mind the economic constraints, it was assumed that Principal Inspections would be performed each 6 years, or when proposed by a Routine Inspection due to specific findings or doubts in their interpretation. Routine Inspections were defined with a periodicity of two years.

Different inspection types also differ in possible subsequent actions, as detailed in Table 1:

Table 1. Inspection types, findings and possible subsequent actions (Amado, 2015)

Type of Inspection	Findings and possible subsequent actions
Routine	Damage/ defect identification
	Maintenance works assignment (defined as small works such as cleanings)
	Triggering of traffic safety alerts
	Ask for a Principal Inspection
Principal	Condition Rating (according to damage/ defect identification, possible causes and consequences)
	Maintenance works assignment
	Repair works assignment
	Monitoring assignment
	Risk mitigation measures
	Triggering of traffic safety alerts
	Definition of further investigations (periodic inspection, detailed inspection including tests and/or structural analysis)

Condition Ratings are assigned in Principal Inspections, derived from visual observations, and if appropriate from other activities such as tests or monitoring. The Condition Rating scale used in this proposal is the same used by the company BMS, from zero (optimal condition) to five (critical condition).

Follow-up Visits were also considered as a periodic activity, based on the BMS experience, and related with the necessity of an annual visit to the most critical structures, to guarantee the safety during the period in between the identification of an intervention need, development and designing of a solution and the beginning of the construction works (Amado and Freire, 2014b). In addition to these three types of inspection, structural monitoring was also considered as an independent periodic activity, despite only performed to those structures where this was considered since their design and construction or when it was assigned through a Principal or Detailed Inspection.

3.2 Occasional and external activities

Occasional activities are those that independently from their importance to the process are not performed in a periodic base, but only when their necessity is detected.

Asset inventory and data updating along service life are key activities to any asset management system. The importance of these activities are justified by the support they represent to other activities of the management process, displaying reliable information about key features of each asset, such as, and among others, jurisdiction, location, dimensions and historical data. New structures, change in jurisdiction, demolitions, repair and strengthening are triggers to a new inventory or update of an existing one.

The already mentioned differentiation between "complex and simple structures" means that a visual inspection on a complex structure, with non-visible elements, may only establish a pre-diagnosis that needs to be confirmed by other means (SÉTRA, 2005) (LCPC, 2003). Therefore it was considered that, when needed, a Principal Inspection may determine the execution of a Detailed Inspection that can include the use of special equipment to access certain areas of the structure, perform tests or even intrusive actions in order to observe buried elements.

Other complementary investigations, such as structural analysis, design, maintenance and repair works are also considered as occasional activities, external to the QCP. This means that although being triggered by the QCP process and that their outputs are inputs to it, they are performed outside the process.

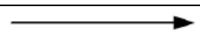
4 The Quality Control Plan Process

According to *ISO 9000:2015(en)* a process can be a "set of interrelated or interacting activities that use inputs to deliver an intended result". Furthermore, a business process can be defined as "the combination of a set of activities within an enterprise with a structure describing their logical order and dependence whose objective is to produce a desired result" (Aguilar-Savén, 2012).

Being a proposal to a systematic management of assets it was intended to develop this QCP according management best practices. Therefore it was developed according Business Process Management discipline (BPM), which stands for a systematic approach for capturing, designing, executing, documenting and improvement of processes, seeking alignment with company strategies, better results and resources optimization (Freund and Rücker, 2005).

Making use of the notation and principles presented in the standard *ISO/IEC 19510:2013(en)*, namely the Business Process Model and Notation, usually known by the acronym BPMN, the process was designed and represented as a non-executable process according to Figure 1. This means that the intervenients in the process are not represented neither all details nor exceptions, in favor of a more immediate perception of the key activities and their interdependencies. Basic symbols used in BPMN notation are presented in Table 2.

Table 2. Basic BPMN modeling elements and description according to ISO/IEC 19510:2013(en)

Category	Type	Symbol
Events	Start event (triggered by a received message)	
	Intermediate event related with received / sent message	
	End	
Activities	Task	
	Call activity (compound activity forming a sub-process)	
Gateways	Divergence and convergence of the flow	
	Allowing parallel paths	
Flows	Sequence flow	
	Default flow	
	Association	

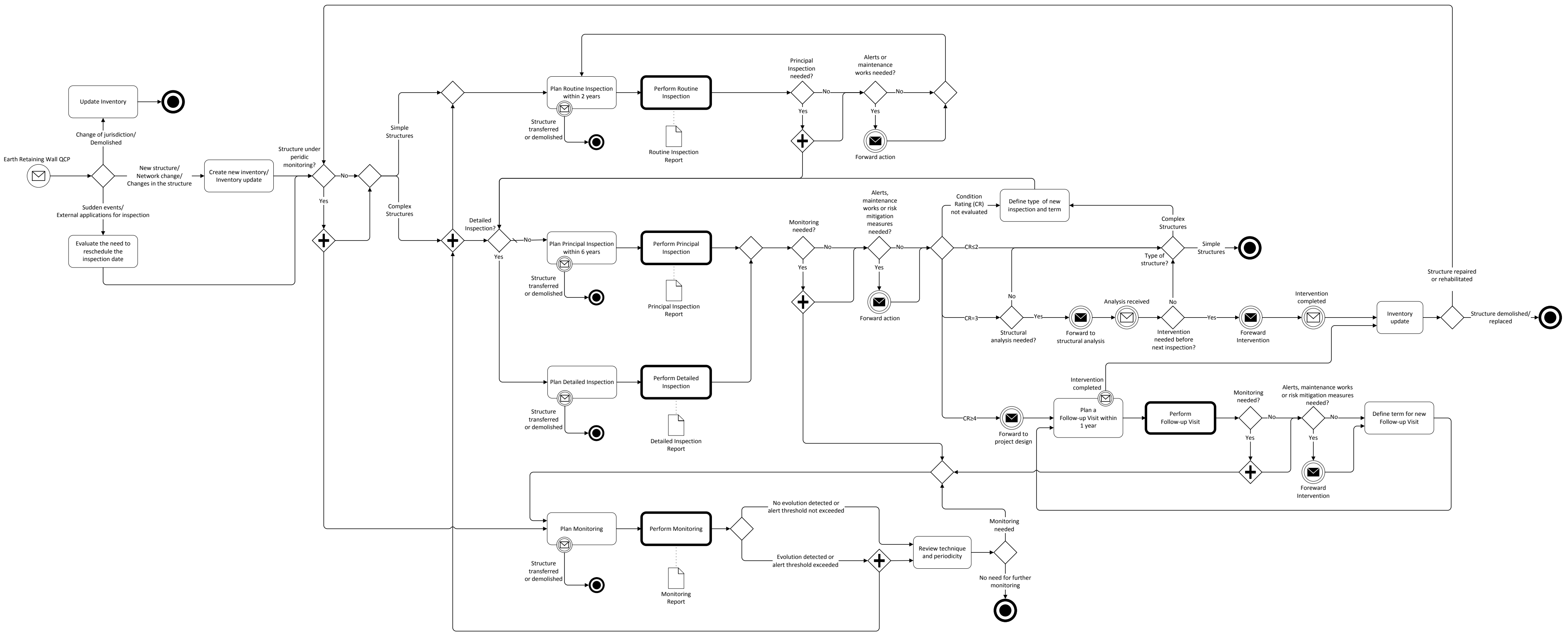


Fig. 1 Quality Control Plan for Earth Retaining Walls (Amado, 2015)

According to the representation proposed in Figure 1, inventory activities are the starting point for any other activity. First differentiation is related with existence or not of a periodic monitoring, although these activities are performed in a parallel path in relation to the periodic Routine or Principal inspections.

The second and most important differentiation is related with the categorization previously explained, according to Simple or Complex structures. In order to achieve a realistic number of human resources involved in this process, attending the company reality, it was defined that Simple Structures will only have Routine Inspections, unless or until the need of a Principal Inspection is detected in the course the Routine Inspections.

On the other branch of the path, Principal Inspections will be performed according to the proposed periodicity to all complex Structures included in the set.

Periodic inspections (Routine or Principal) lead to the possible subsequent actions defined in section 3.1. In Principal Inspections possible subsequent differ according to the Condition Rating, which is related with the need and urgency of repairs. Exception made to Monitoring and the conduction of Detailed Inspections since both may be needed for a more accurate assessment.

5 Conclusions

The main conclusion that might be drawn is that earth retaining structures are suitable to manage according to a quality control plan based on the bridge management systems experience, taking advantage from the alignment of practices relative to these different assets.

Secondly, the development of a QCP will take advantage if based on the results of inventory campaigns, allowing to suit the activities, periodicities and resources to the subjacent reality of the set of assets and the company itself.

Finally, the use of the management disciplines in the scope of structural engineering broadens the concepts, introducing new tools such as the BPMN and the idea of continuous improvement of the processes.

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Non-destructive investigation techniques in bridge inspection

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Abstract. Regular bridge maintenance is primarily based on visual inspections. However, when assessing critical or concealed damage, visual inspection is often inadequate as intensity of a damage cannot be reliably determined, especially its depth. In such cases, inspector has to impose additional investigations. Investigation techniques can be by nature either destructive or non-destructive. Results of destructive techniques are more reliable, but non-destructive techniques are easily repeatable without affecting the state of the inspected construction. Within the field of bridge management, where large group of bridges over long time periods is dealt with, the use of non-destructive investigation techniques is considerably more appropriate. More than one investigation technique is often available to assess the same type of damage. The article discusses the available methods of investigations and for each of them briefly indicates their advantages and disadvantages.

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

1 Introduction

Regular bridge inspections are in most cases based on visual inspection only. During the inspection of severely damaged bridges, the intensity of certain type of damage cannot be adequately assessed, as its depth cannot be determined by surface perception. A more detailed examination needs to be performed, wherein two basic options are available: destructive and non-destructive investigations. Destructive investigation techniques are commonly used in cases, where intention to accede to rehabilitation of a selected bridge is already confirmed, but lack of accurate data prevents project documentation completion. By removing damaged, protective and other layers of material, intensity of the damage and material properties can be determined in the majority of cases, while the reliability of the results is high. Implementation of destructive examination results in additional damage to the bridge and even quicker deterioration, but in short term does not constitute a substantial impairment. However, remediation needs to be performed within a reasonable time. If remedial actions are not already confirmed, the use of destructive investigation techniques is recommended only to a very limited extent.

In such cases, the use of non-destructive tests (NDT) is recommended. Their results are, in some cases, less reliable, however they do not cause damage to the construction under investigation and are repeatable. The reliability of the results can also be improved by using appropriate NDT methods, depending on the specific requirements of the investigation. In most cases, several methods can be used to measure same properties, therefore the selection of the measuring technique needs to be carefully thought over.

2 Non-destructive investigation techniques

The majority of the existing bridge maintenance systems in the past were based primarily on information obtained through visual inspections (Gattulli&Chiaramonte, 2005). Although documented past (Phares et al., 2004) and ongoing experience (Kušar, 2014) reveals that this type of inspection is often unreliable, it will remain the main aid collecting data due to its simplicity and cost effectiveness (Tenžera et al., 2012). As long as bridges are without significant damage, deformations or other irregularities, visual inspections are sufficient for determining slow continuous processes of degradation. Problems occur when it becomes necessary to accurately determine the risk of intense or concealed damage. Although senior inspectors can be reliable in their assessment due to their past experience, visual assessment is always subjective to a certain degree. Consequently, in cases of intense damage, it is necessary to implement NDT to confirm or revise the estimates obtained via visual inspection.

Visual assessment is necessary to determine the types of damage (cracks, compressive strength reduction, corrosion, porosity and similar) and prescribe a suitable investigation method. For the majority of damage and other deficiencies, several NDT methods are available, each having its advantages and disadvantages. The most commonly used methods are collected in Tables 1 to 7.

Table 1. NDT for measurement of: cracks, leaking, mechanical damage, scaling, segregation…

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Visual inspection	Fast, economical	Quality depends on inspectors competence	Eye sight
Image Pro Plus (IPP)	Easy to use, economical	Prolonged acquisition of results	Color comparison
Acoustic emission (AE)	Fast	Expensive	Transient elastic waves
Impact echo	Fast, reliable for thinner elements	Reliability decreases with thickness	Electromagnetic waves transmission
Infrared thermography	Easy to use	No information regarding a depth of a damage	Change in surface temperature
Impulse response	Fast, easy to use	Reliability depends on inspectors experience	Stress waves method
Radiography	Determines thickness, irregularities…	Reliability decreases with thickness	X-rays and gamma radiation
Petrography	Easy to use	No information regarding a depth of a damage	Change in surface temperature
Lamb wave Theory	Precise results	Demanding interpretation of results	Guided waves theory

Table 2. NDT for measurement of: compressive strength, surface, hardness, adhesion

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Rebound hammer	Easy to use, fast, economical	Moisture and uneven surface can affect results	Spring-loaded mass rebound
Ultrasonic pulse velocity (UPV)	Fast, economical, deep penetration	Moisture and reinforcement can affect results	Ultrasonic waves velocity
CAPO test	Precise results	Surface damage	Pull-out test
Probe penetration	Easy to use	Damage (narrow hole)	Penetration depth measurement
Micro-coring	Precise results	Demanding extractions of samples	Extracted core analysis
Pull-of test	Easy to use, fast	Surface damage	Pull-of force measurement

Table 3. NDT for measurement of: Chloride concentration

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Quantab test	Fast, precise results	Limited thickness	Silver dichromate and chloride ions reaction
Potentiometric titration	Precise results	Skilled specialist needed	Chemical titration method
Fast chloride test	Easy to use, fast	Results may vary	Pull-out test

Table 4. NDT for measurement of: corrosion

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Galvanostatic pulse method	Fast	Unstable readings	Electrical potential (anodic pulse)
Linear polarization resistance	Fast	Local damage needed	Electrical conductivity
Electrical potential measurement	Easy to use	Low precision	Electric potential
Time domain reflectometry (TDR)	Easy to use, shows location of irregularities	Inadequate detection of smaller defects	Electromagnetic waves
Ultrasonic waves	Indicates location and extent of the damage	Poor reliability	Ultrasonic waves
X-ray diffraction and atomic absorption	Easy to use, reliable	Radiation	X-ray absorption

Table 5. NDT for measurement of: carbonation (concrete pH)

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Phenolphthalein indicator test	Easy to use, fast, economical	Saturation affects results	pH measurements
Rainbow indicator	Easy to use, fast, economical	Drilling necessary	pH measurements

Table 6. NDT for measurement of: internal damage and defects, delamination

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Vibration based damage identification	Easy to use, economical	External factors may affect results	Construction dynamics
Seismic refraction method	Reliable	In depth damage detection only	Seismic waves
Ultrasonic longitudinal waves	Large area scanning	Not suitable as primary method	Ultrasonic waves
Ultrasonic continuous spread spectrum signal	High sensitivity	Demanding interpretation of results	Ultrasonic waves

Table 7. NDT for measurement of: porosity, water absorption

NDT Method	*ADVANTAGES*	*DISADVATAGES*	*Method BASED on*
Water permeability test	Reliable	Extensive preparation	Absorption measurement
Initial surface absorption test (ISAT)	Reliable laboratory results	Unreliable in-situ result at temperature change	Absorption measurement
Covercrete absorption test (CAT)	Reliable in-situ results	Sensitive to W/C ratio and moisture	Absorption measurement

The content of tables 1 to 7 shows high variety of NDT available. In practice, NDT tools already owned by the operator or company performing inspections are most commonly used for measurements regardless of their advantages and disadvantages. Results can therefore occasionally be less reliable, which can lead to imperfect remedial plans. Choosing the most suitable NDT may prove more expensive in short term, but higher inspection cost are in most cases quickly repaid both in the design phase and in the implementation phase of proposed remedial actions.

3 Conclusions

Easy to use and cost effective NDTs are most widely used because of their low implementation cost. In most cases they produce reliable results, and their disadvantages are reflected in demanding conditions only. In these cases, inadequate detection, reliability decrease, unstable readings and similar is exhibited. Highly sensitive and reliable methods are frequently demanding, cause surface damage or have health issues (radiation) and are consequently rarely used.

Choosing the right inspection method for a specific combination of material, element properties and type of damage is vital to assess particularly the intensity and extent of concealed damage. The development of visible damage can be regularly monitored while concealed damage is commonly detected only when the indirect signs of its presence is recorded. The detection of such signs should not be neglected as hidden damage is the most common cause of excessive deformation or even collapse of bridges. NDT should therefore be a necessity in regular bridge inspection and maintenance.

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Structural health monitoring based on static measurements with temperature compensation to detect stiffness reductions

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Abstract

Some results from a series of repeated static loading tests on a prestressed concrete beam are presented in this paper. The beam was subjected to stepwise artificial damage by cutting prestressed tendons and damage detection is performed through the monitoring of deflection line.

During a one-month period, it revealed that displacements measured along the structure were clearly influenced by temperature variation. For improving the regularity of data, a compensation algorithm is proposed which reflects the measured data according to a reference temperature. The results are presented with compensated measurements.

Keywords: bridge, localization, damage, displacement, temperature

1 Introduction

In civil engineering, static load tests have a long tradition and provide important information on deformation, displacement, rotation and strain. They have been since ever an appropriate alternative and an amendment to visual and dynamic inspections as deflection measurements are relative easy. Our paper presented in the last COST's Workshop (Nguyen et al., 2016) showed some detection results based on the static measurements including deflection line and its derivatives as slope and curvature. This paper still deals with static measurements by considering the effect of temperature variation. By referring to a reference temperature, the data are compensated that facilitates the comparison between measurements from different conditions of temperature.

2 Situation of the testing beam

The part was taken from a real prestressed concrete bridge with the length of 46 meters and the weight of 120 tons. The testing beam was positioned as on the real bridge before as simply supported beam, like shown in Figure 1. More details of the test setup can be found in (Nguyen et al., 2016).

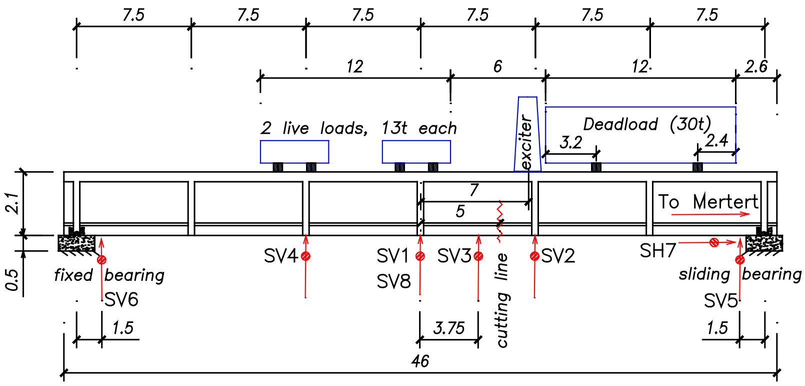


Figure 1 : Configuration of the beam and positions of sensors for static measurements

During the bridge's lifetime, the beams had not only to carry the own weight but also the traffic lane with asphalt layer, sideway and other additions. In order to simulate this additional dead load, a mass of approximately 30t was positioned on top of the structure. Although it was not distributed equally over the whole beam length like an asphalt layer, its induced stresses were checked and considered as an equivalent approximation. This mass stayed on the beam during the whole test period and is hereafter referred to as dead load.

Furthermore for repeated static testing purposes with always the same mass loading, two concrete blocks with a mass of 13t each were used and are subsequently denominated as live load. The bridge was subjected to similar charging due to high traffic loading during its life, i.e. the 26t stayed within the permitted service loading. The weights were positioned on precisely defined locations and removed again after minimum 24 hours. Displacements were recorded in several locations, as detailed in Figure 1, in the vertical (SV1-SV6, SV8) and the horizontal direction (SH7).

During the whole time, the deflection of the beam and the temperature condition were permanently registered. The temperature of the structure was measured within a hole of 10cm depth inside the concrete.

The concrete beam was prestressed by 19 steel tendons along the longitudinal direction of the beam. Increasing artificial damage was introduced in four successive steps by cutting tendons at the cutting line shown in Figure 1. The initial state is named #0. Four levels of damages, named #1, #2, #3 and #4 correspond to the cutting of 2, 4, 6 and 9 tendons. From the first damage scenario, horizontal cracks appeared near the cutting line due to shear from back-anchorage of the cut tendons while the first vertical cracks were engendered from damage #3 onwards.

3 Data analysis

The deflection of the beam was monitored by several sensors along the beam during one month. Below the most typical sensors are shown: SV1 in the middle of the beam and SV3 near the cracking line. The two principal different situations of loaded (L) and unloaded (UL) can clearly be separated: a loading was performed by putting the two weights of 13 tons each on the top of the beam, as indicated in Figure 1. Figure 2 presents also the variation of temperature T4 measured in the bottom of the bridge, inside the concrete; this figure reveals clearly the modulation of displacements due to the variation of temperature.

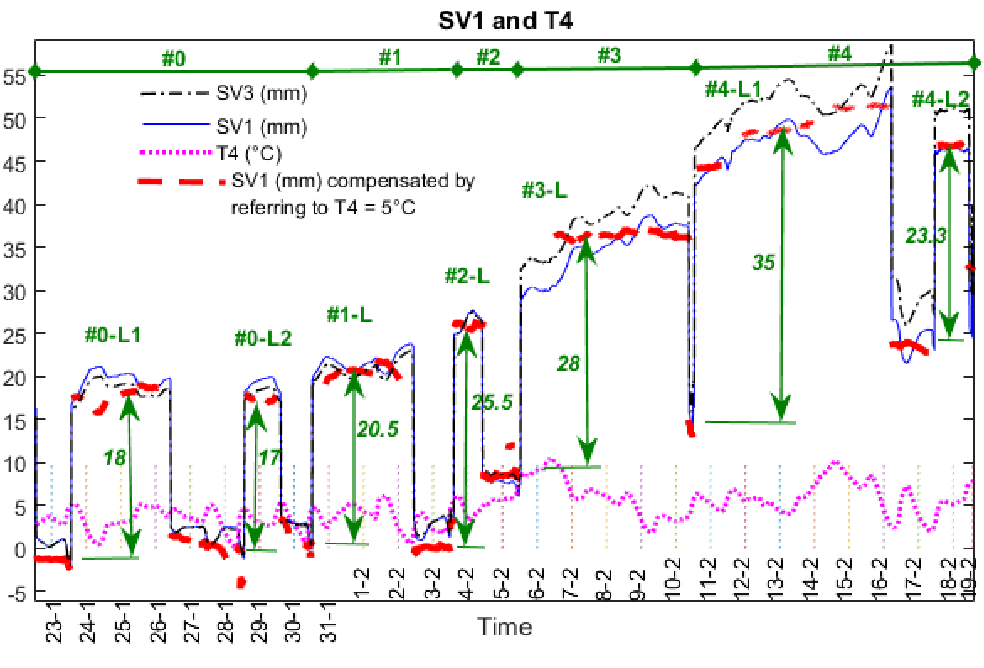


Figure 2 : Vertical displacements SV3 (dash-dot black); SV1 as measured (continuous blue) and compensated (thick red); temperature T4 (dot magenta)

The correlation between the displacement transducers SV1 (middle) and SV3 (near the cutting line) reflects also the evolution of cracks in the beam. Being at the middle of the beam span, SV1 showed at the beginning the largest displacements of the beam. However, from damage scenario #2, SV3 near the cutting line increased and overtook SV1.

Furthermore, in the interest of a temperature compensation for static measurements, a graph is built to present their correlation, as illustrated in Figure 3 for all loading states according to SV1. In this figure, magenta lines present the complete data, while dark blue ones select data in periods with little or no creep and horizontal movement in the sliding bearing, which we called “retained” data. A linear regression line was calculated for each “retained data” branch and added in green colour.

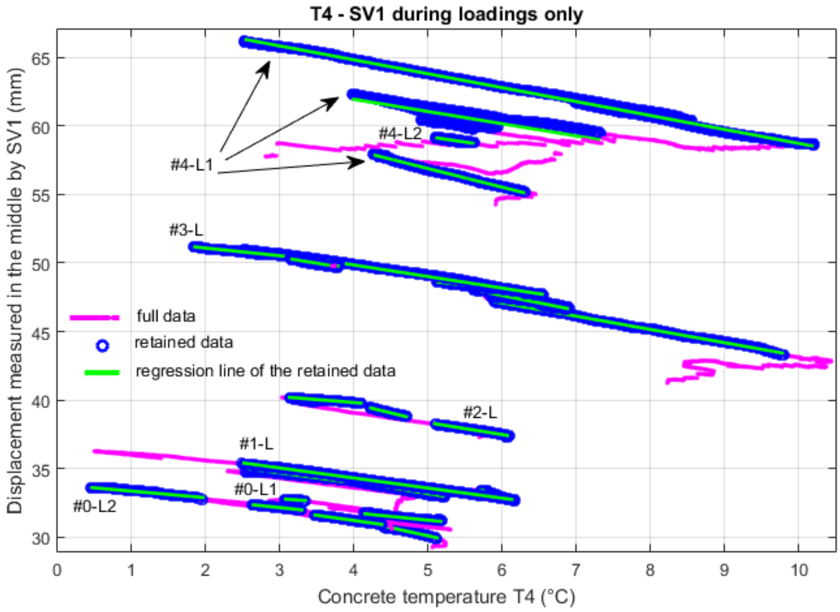
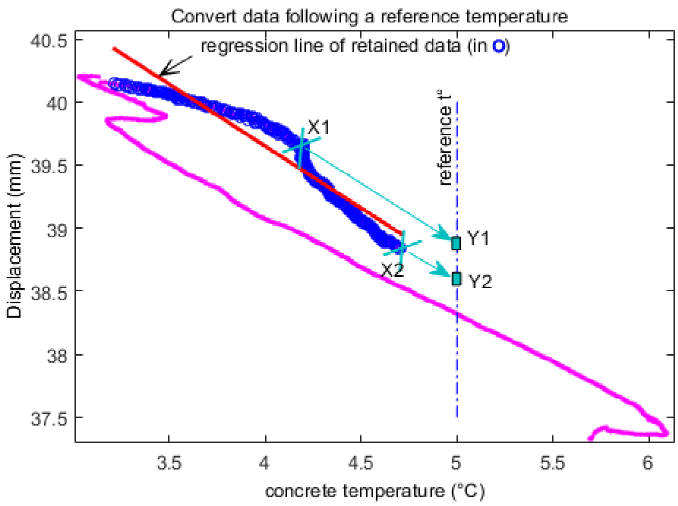
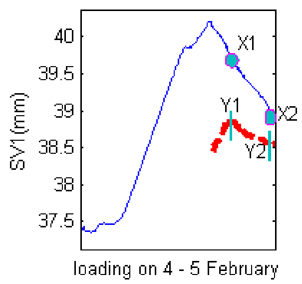


Figure 3: Relation between the temperature T4 and the vertical displacement SV1 in the middle

3.1 Temperature compensation



a) Data shown in the T4-SV1 plot



b) Data shown in the time-SV1 plot

Figure 4: Example of temperature compensation: 2 points X1 and X2 are projected following the slope of 1mm/°C to the reference temperature, resulting in new points Y1 and Y2

As the regression lines are quite parallel (Figure 3), a temperature compensation can be done by referring all measurements to a common temperature, for example T4 = 5°C. An example is given in Figure 4 to explain, how measured data X1, X2 are shifted to the chosen ‘reference temperature of 5°C’ and represented by Y1 and Y2. This procedure is repeated for all ‘retained’ measured data in SV1, i.e. data without horizontal movement and creep. These temperature-compensated values are shown in red dashed lines in Figure 2. They are by far less fluctuant, corresponding by far better to the applied step-loading and hence permitting a better damage detection.

Figure 2 presents also the difference between loaded and unloaded data of each damage state after the temperature-compensation in red. These data were then transferred into Figure 5. In damage states #0 and #4, two loadings (L1 and L2) were performed; whereas unfortunately in the other damage scenarios #1, #2, #3, only one loading was done. In #0 and #4, a clear reduction is observed between loading L1 and L2 due to plastic straining/cracking effect highlighted in Figure 6. This well-known phenomenon was described for instance in (Waltering, 2009): during the first loading (L1), a plastic, i.e. non-reversible deformation took place when yield stress of metal or the crack load of reinforced concrete is exceeded. Figure 6 shows the principal stress-strain behaviour in this case: during the first loading (L1), the deformation was at the beginning elastic (segment OA) and then evolved to plastic (segment AB) leading to a total strain of ε_{total}. With unloading (BC segment), the beam's behaviour followed a straight line parallel to OA until zero stress level, leaving a residual strain ε_r (OC) and so a residual deformation. Any subsequent 2nd, 3rd …etc. loading (L2, L3, …) up to the same maximum stress from C follows the line CB. Hence, the total strain in L1 $\boldsymbol{\varepsilon}_{total}$ is significantly higher than the elastic strain in L2 $\varepsilon_{elastic}^{CB}$. Figure 6 reveals clearly the non-linear behaviour including plastic, non-reversible deformation.

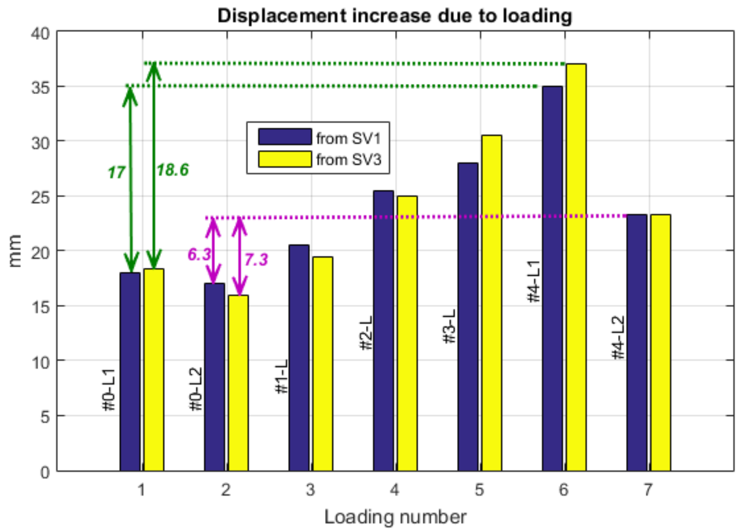


Figure 5: Step-height of compensated vertical deflection between loaded and unloaded state

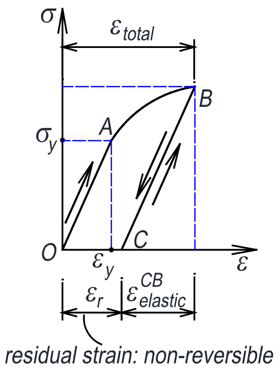


Figure 6: Stress-strain diagram for a loading-unloading procedure

From this perspective, all loadings have to be repeated at least twice where only the second loading is taken into consideration damage assessment in order to assure there is no unwanted plastification. Comparing the initial state #0 with the most serious damage #4, we detect 17 and 18.6 mm in the first loadings and only 6.3 and 7.3 mm in the second one. Nevertheless these increases of approximately 7 mm of deflection induced by a constant test-load indicate progressive damage or reduced stiffness. It shows that the temperature-compensated step height can be used as an index for damage detection.

3.2 Deflection lines

Damage can also be localized by the monitoring of deflection line of the beam in every damage state, as presented in our previous study (Nguyen et al., 2016). Each state was represented there by a "raw", i.e as measured deflection line reported in Figure 7. Now here temperature compensation was done and the displacements become more uniform continuous, as shown by red dashed lines in Figure 2. Hence the deflection lines can then also be compensated for temperature effects that is shown in Figure 8. Localization of damage is feasible with both lines as the maximum moves from the middle to the cutting line with increasing damage. Furthermore the shape changes from uniform continuous to an angled form with the summit at the cutting line. However, let us raise explicit differences between Figures 7 and 8, including two interesting phenomena. Compensated deflection lines are convex, i.e show small positive deflection for some first unloaded states (hereafter referred as feature 1) and secondly an important difference between the two unloaded states #4-L1 and #4-L2 in Figure 8 while in Figure 7 before compensation they are very close (feature 2).

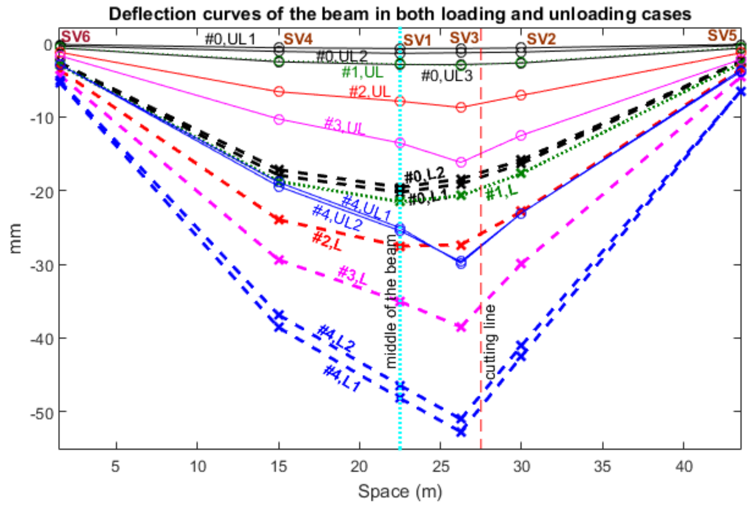


Figure 7: Deflection lines from the raw data

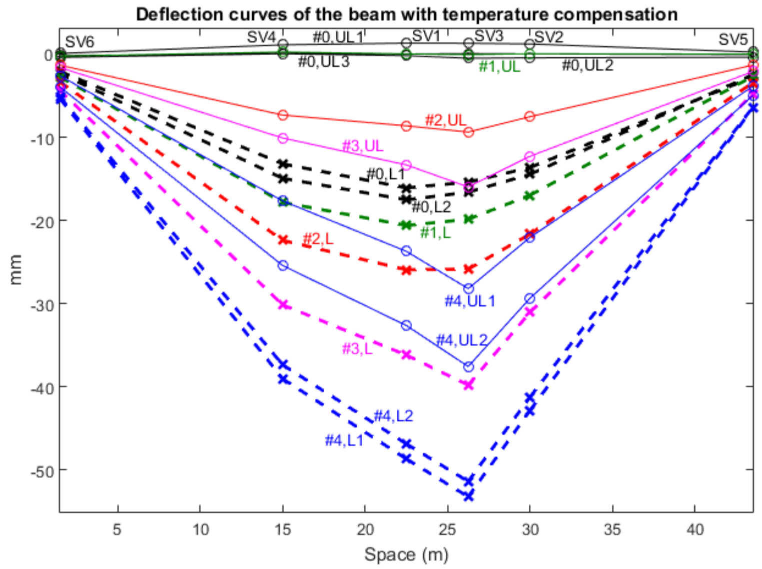


Figure 8: Temperature compensated deflection lines

The origin of these two features relies obviously to the compensation technique and can be explained in Figure 9 by several sets of data. Since the reference temperature is chosen at 5°C while during the first tests from #0 to #2, the temperature was often below this reference as can be seen in Figure 2. Hence the mapping of set C including weak displacements (near 0) can result in negative compensated displacements C*. That justified a convex shape of deflection line for some first unloaded states – Feature (1).

Furthermore, Feature (2) can be understood by two sets of data containing (A, D) and B in Figure 9. For example without temperature compensation, the two data B and D were used for the construction of deflection lines as in Figure 7 because B and D have equivalent ordinates. But after compensation it may result in two clearly different deflection lines #4-UL1 and #4-UL2 reflecting the increased sagging and the plastic deformation including cracking of concrete.

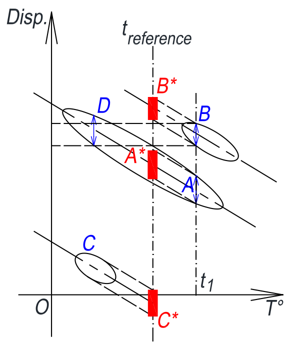


Figure 9: Illustration on the compensation of different ranges of temperature

4 Conclusion

Thanks to repeated static load testing with the same mass loading, the deflection line can be used for damage localization and even reflecting the level of damage. A prestressed concrete beam of a real bridge was jacked-up and artificially progressively damaged in 4 steps by cutting tendons until wide vertical cracks occurred.

As cracking of concrete and plastification of reinforcement-steel are non-reversible and non-linear phenomena, the residual strain leads to a sagging of the bridge, which should be used as another important damage indicator. This sagging under gravity is in principle monotonous, but may be hidden by temperature effects. The step-height in the deflection due to mass loading is traditionally also used as damage indicator, but less pronounced as often assumed. Nevertheless, attention should be paid that at least the structure is twice loaded then unloaded in order to separate plastic and elastic phenomena. Therefore, only the step-height from the second loading and the corresponding deflection curve should be considered.

A temperature compensation algorithm is proposed based on the slope of the deflection-temperature curve. This curve can be measured prior to damage detection in the healthy reference state and then used for subsequent temperature compensation. The proposed algorithm shows promising results in the discussed example based on the absolute temperature, but may also be used based on temperature differences between the top and bottom side of the bridge, depending on the required forces for axial expansion/contraction, which themselves depend on the used type of sliding bearing.

Photogrammetric and GPS measurement technology have improved significantly in the last years and may hence be used in future for quick and easy capturing of the deflection line under a test load and/or for detection of the sagging of the bridge under gravity referring to the supports, i.e. referring to an initially defined constant zero-line.

The repeated measurement of deflection lines with constant mass loading over years can also be used after temperature compensation for model-updating of finite element models, which in return can highlight stiffness reductions and hence damage.

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Sustainability score for roadway bridges

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Abstract. The integration of bridge management into a broader asset management approach asks for the introduction of indicators related to network performance. The use of generic, qualitative indicators may be sufficient for a standard inspection process aimed at planning regular maintenance tasks. However, decision making may be enhanced by introducing bridge specific, quantified indicators. This is further necessary for bridge managers facing aging infrastructure where additional information and indicators are necessary to facilitate decision making. This paper presents an approach to introduce bridge specific indicators in regular inspection processes comparing the as is with the as built situation. Furthermore, when also the as desired situation is assessed, a sustainability score may prove useful in selection potential end of service life candidates out of the total bridge stock.

Keywords: sustainability, performance indicators, end of service life, inspection, performance goals

1 Introduction

In the Netherlands, focus of road authorities has shifted from the construction of new infrastructure to the optimization of existing road networks. This has supported the introduction of asset management principles for bridge management where activities are focused on meeting required network performance. At the same time, an aging bridge stock results in a rising need for renovation or even replacement of bridges (Klanker et al. 2016). Within this context, life cycle considerations are increasingly important. For a public organization, these considerations should involve not just financial (life cycle cost) but also other performance aspects (life cycle management). This raises two questions:

1. Does the bridge still fulfill its intended purpose? The answer to this question is relevant for the planning of maintenance
2. Does the intended purpose meet today’s requirements? The answer to this question is relevant for the planning of bigger investments for example related to end of service life issues

2 Introducing network performance in bridge management

Rijkswaterstaat is the executive agency of the Netherlands’ Ministry of Infrastructure and the Environment. Rijkswaterstaat is responsible for the management of three infrastructural networks, among which the national highways network. Part of these networks are around 4.000 bridges. For Rijkswaterstaat, bridge management is part of a broader network management approach involving asset management principles (Bakker & Klatter 2012). Activities are aimed at meeting required network performance (van der Velde et al. 2013). It becomes therefore necessary to relate (planned) activities to network performance (Stipanovic Oslakovic & Klanker 2016). This poses some challenges as there is no exclusive relationship between network performance and bridge maintenance. maintenance activities for other asset types, but also external factors as diverse as traffic numbers, weather conditions and oil prices influence the network performance. At Rijkswaterstaat, bridge maintenance planning is based on the risk of not meeting network performance. Required performance has been specified using RAMSSHEEP criteria (table 1).

Table 1. RAMSSHEEP criteria

Criteria		**Sub-criteria**
Reliability	1.1.R	Satisfy reliability requirements for moving parts and equipment
	1.2.R	Meet structural requirements in relation to damages
	1.3.R	Meet structural requirements in relation to revised standards
	1.4.R	Meet structural requirements in relation to different use
	1.5.R	Meet structural requirements in relation to defects in design, execution or management
Availability	2.1.A	Meet object specific requirements with regard to the fulfilment of the object functions
	2.2.A	Prevention of calamities
Maintainability	3.1.M	Meet requirements relating to the maintainability of elements
Safety	4.1.Sa	Meet object specific requirements with regard to the safe performance of the object functions
	4.2.Sa	Prevention of calamities
Security	5.1.Se	Meet the requirements with regard to the prevention of vandalism
	5.2.Se	Meet the requirements relating to the protection of the object
Health	6.1.H	Meet health and safety decisions
Environment	7.1.E	Meet aesthetics requirements
	7.2.E	Meet environmental requirements
	7.3.E	Comply with requirements relating to use/ comfort
Economics	8.1.Ec	Water management in order
	8.2.Ec	Prevent widespread or irreparable damage
Politics	9.1.P	Meet requirements for reputation

The RAMSSHEEP criteria are specified in a generic and qualitative way. This allows for use at all bridges, which is of particular importance as, in practice, information regarding requirements is often not readily available. During inspection processes, risks for meeting required performance are registered, and classified at a range of 1 to 5, based on probability and effect. To aid the assessment by the bridge inspector, a description of probability and effect classes is used (table 2).

Table 2. Effect classes for Reliability criteria

	Reliability
Minimal	Impacts object reliability but does not have any impact on the reliability of the corridor
Minor	Impacts object reliability and does have a limited impact on the reliability of the corridor
Major	Impacts both object reliability and corridor reliability
Severe	Impacts object reliability and has a major impact on corridor reliability

Integrated in the approach are a number of semi-quantified assessment tools for bridge components that may have a significant impact on performance or costs. The CRIAM method for the super structure of the bridge was developed to assess the need for a quantified assessment of the structural reliability (de Boer & Booij 2012). The method values differences between the current state of the bridge and the bridge design. The following indicators are included:

- Damage and use: structural damages, structural safety risks and type of use
- Materials: concrete quality, reinforcement quality, reinforcement configuration
- Design: length, lane width, number of lanes, pavement thickness, additional loads, reinforcement type

The values for the different indicators result in an index value (the CRIAM index) which is categorized in three categories indicating the need for a further more detailed assessment:

- ‘green’ (score ≤ 54): no further investigations are necessary
- ‘orange (54 < score ≤ 100): further assessment could prove useful but is to be decided upon by a senior engineer
- ‘red’ (score > 100): further assessment is necessary

For some bridges, information on materials and design may be lacking. In these cases, a conservative assumption is allowed based on the design year of the structure. Additional investigations to determine this information will be part of more detailed assessment.

The RAMSSHEEP criteria and the associated semi-quantified tools are integrated in the regular inspection procedures. Bridges are subjected to a major inspection every six years. During inspections, risks are registered based on a cause-effect analysis using a Failure Mode Effect and Criticality Analysis (FMECA) format. This procedure enables the bridge manager with information on the risk profile of his bridge stock. Figure 1 shows the costs of maintenance tasks associated with the RAMSSHEEP requirements.

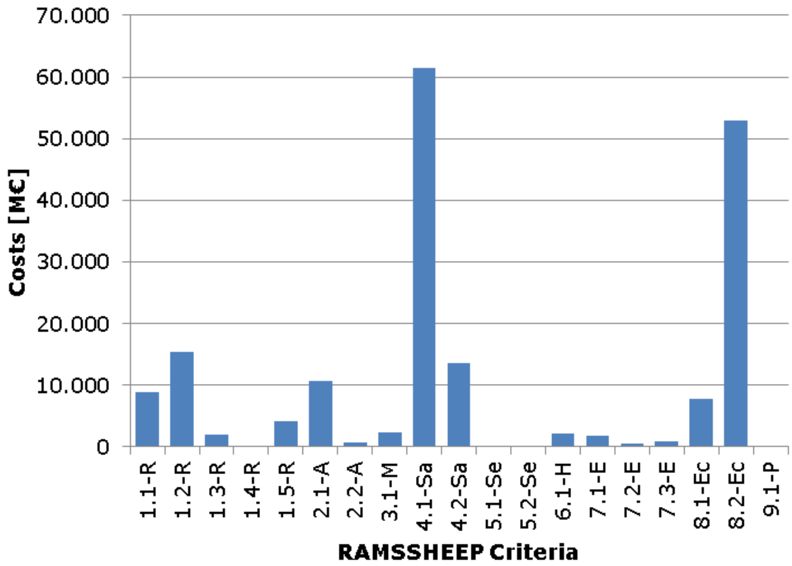

Fig. 1 Costs associated with RAMSSHEEP criteria

3 Involving bridge specific performance indicators

The generic RAMSSHEEP requirements and associated inspection procedures can be used out without explicitly taking into account bridge-specific requirements. While this is often a necessity because of lacking information, this is far from ideal especially for the planning of activities with a big impact in terms of performance or costs. While bridge design nowadays makes use of explicit design methods like Systems Engineering, explicitly stating required functionality and performance, this was not always the case. One proposed solution is to define performance indicators for each RAMSSHEEP criteria for different bridge types. For each indicator, values may be obtained for the as built and at present state.

The values thus specified would enable the bridge inspectors to make a more precise cause-effect analysis and assess risks for each performance aspect. Comparing present and as-built values provides information to determine risk levels during inspections (table 3). If values are unknown as-designed values may be assumed based on the design year of the structure, similar to the approach chosen for the CRIAM analysis.

Table 3. Object specific performance indicators based on RAMSSHEEP criteria

RAMSSHEEP criteria	Performance indicator	As built	As is	Risk level
1.2-R Meet structural requirements in relation to damages 1.3-R Meet structural requirements in relation to revised standards 1.4-R Meet structural requirements in relation to different use 1.5-R Meet structural requirements in relation to defects in design, execution or management	CRIAM index	-	Orange	4 – High
2.1-A Meet object specific requirements with regard to the fulfilment of the object functions	Peak hour traffic intensity [veh/hour]	4.000	4.000	1 – Negligible
	Vertical clearance [m]	4,2	4,2	1 – Negligible
4.1-Sa Meet object specific requirements with regard to the safe performance of the object functions	Lane width [m]	3,25	3,25	1 – Negligible

4 Use of RAMSSHEEP performance for end of service life analysis

In dealing with an aging bridge stock, it is important that the remaining service life of a bridge is taken into consideration for decisions on maintenance or renovation. End of service life decisions in practice rely on a combination of information on bridge condition, budget and bridge performance on different aspects. During the life of a bridge, requirements for performance aspects will likely change, quite often without explicit documentation. Based on the performance indicators related to the RAMSSHEEP criteria, a next step could be to also obtain a value for each indicators as currently desired (table 4).

Table 4. Object specific performance indicators based on RAMSSHEEP criteria - as desired

RAMSSHEEP criteria	Performance indicator	As built	As is	As desired
1.2-R Meet structural requirements in relation to damages 1.3-R Meet structural requirements in relation to revised standards 1.4-R Meet structural requirements in relation to different use 1.5-R Meet structural requirements in relation to defects in design, execution or management	CRIAM index	-	Orange	Green
2.1-A Meet object specific requirements with regard to the fulfilment of the object functions	Peak hour traffic intensity [veh/hour]	4.000	4.000	5.000
	Vertical clearance [m]	4,2	4,2	4,5
4.1-Sa Meet object specific requirements with regard to the safe performance of the object functions	Lane width [m]	3,25	3,25	3,5

A comparison of the values of the performance indicators for the as built, as is and as desired situations gives valuable information for decision making on investments to maintain the structure. A so called sustainability score could be implemented to express the gap between the as built or as is situation and the desired situation (table 5). The sustainability level expresses:

- Whether actual performance (for a specific criterion) meets current design requirements; and
- The effect any deviation from current design requirements has on network performance

Table 5. Sustainability level

Sustainability level	Description
1	Meets current design requirements
2	Current design requirements are not met, network performance is not compromised
3	Current design requirements are not met, network performance is compromised at object level
4	Current design requirements are not met, network performance is compromised at road link level
5	Current design requirements are not met, network performance is compromised at network level

Note that two structures with the same value for the same performance indicator may result in a different sustainability level when the effect on network performance is different. For example, viaduct A and B both have a vertical clearance of 4,2 meters, not meeting the current design requirement of 4,5 meters. However, viaduct A is the only low clearance object in a highway linking a port with its hinterland whereas viaduct B is part of a stretch of road with multiple low clearance objects. In this case, the sustainability level for viaduct A could be rated at ‘5’ (impact at network level) where the level for viaduct B would be a ‘3’ (impact on object level).

Data collection of the as built, as is and as required values for performance indicators can be a part of regular inspection procedures. In doing so, information that forms a part of end of service life considerations will be collected in a more structured way. It should be noted however that for the planning and decision making of bigger investments related to end of service life issues it will almost always be necessary to perform more exhaustive investigations. The proposed sustainability score therefore functions as a filter to be able to select a subset out of the total bridge population for further investigation. When as built, as is and as desired values for the performance indicators are collected during inspection processes, a sustainability score can be derived for every bridge.

The sustainability score monitors the difference between the as built and as desired situation, covering only a part of the total performance (figure 2). The sustainability score may be used alongside the 'End of Life indicator' (ELI), an indicator based on life cycle costs that compares the life cycle cost of maintenance with the replacement of the structure (Bakker & Knoops 2016). The End of Life indicator is a measurement of the deviation from the as built situation comparing planned maintenance with replacement of the bridge:

$$ELI = \frac{NPV_{ST} + NPV_{Main} + NPV_{RP;t=p}}{NPV_{RP;t=n} + NPV_{Main} + NPV_{RP;t=n+100}} \qquad (1)$$

Where NPV_{ST} = Net Present Value of Short Term maintenance needs; NPV_{Main} = Net Present Value of regular maintenance needs; $NPV_{RP;t=p}$ = Net Present Value of Replacement at Planned date; $NPV_{RP;t=n}$ = Net Present Value of Replacement at the earliest possible moment and $NPV_{RP;t=n+100}$ = Net Present Value of Replacement after 100 years of service life.

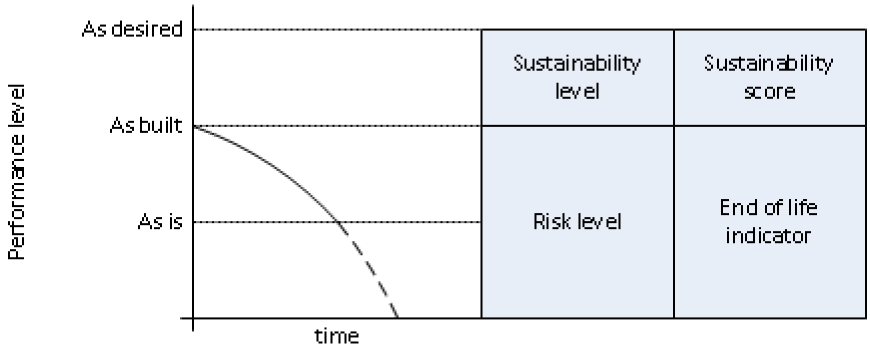

Fig. 2 Costs associated with RAMSSHEEP criteria

5 Conclusions

The integration of bridge management into a broader asset management approach asks for the introduction of indicators related to network performance. The use of generic, qualitative indicators may be sufficient for a standard inspection process aimed at planning regular maintenance tasks. However, decision making may be enhanced by introducing bridge specific, quantified indicators. This is further necessary for bridge managers facing aging infrastructure where additional information and indicators are necessary to facilitate decision making. This paper presented an approach to introduce bridge specific indicators in regular inspection processes comparing the as is with the as built situation. Furthermore, when also the as desired situation is assessed, a sustainability score may prove useful in selection potential end of service life candidates out of the total bridge stock.

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Environmental performance framework for bridge infrastructure maintenance

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Abstract. This paper presents a combined economic and environmental framework for evaluating the economic and environmental performance of alternative maintenance strategies for maximising the service life of existing bridge structures. It is intended to assist bridge managers in identifying the best overall combination of both economic and environmental criteria in their decision making. An approach to assess the potential environmental effect of delaying maintenance work into the future, through the introduction of time-weighting of environmental impacts, is proposed. It is intended to aid asset managers to determine the potential benefit or dis-benefit of such delaying options. The methodology is based on life cycle analysis and accounts for the cost and environmental impacts that arise during the working life of different alternative maintenance strategies. The economic performance of the alternatives is analysed by using Life Cycle Costing (LCC) whilst the environmental performance is assessed based on Life Cycle Assessment (LCA). The framework addresses the potential unequal working lives of alternative maintenance strategies by evaluating cost and environmental performance through an 'equivalent annual' term. The combined economic and environmental performance of the alternatives is evaluated using multi-criteria decision analysis (MCDA). A case study using three typical alternative maintenance strategies for a metallic railway bridge is presented to demonstrate the use of developed methodology.

Keywords: environmental impact, Life Cycle Costing (LCC), Life Cycle Assessment (LCA), maintenance strategies, metallic bridge

1 Introduction

Ageing infrastructure forms a significant part of transport networks in Europe and worldwide. As a result, an increasing number of assets will need to be fully replaced when deemed uneconomical or unsafe. However, in the majority of the cases, maintenance to prolong their service life is the preferred option both from an operational and environmental point of view.

Bridge infrastructure maintenance projects, like other public-funded projects, have to compete for limited capital resources available. The economic cost of different maintenance investment strategies has to be evaluated to ensure that the chosen option is the most cost effective from a life cycle perspective. Furthermore, the growing importance of environmental considerations in the highway and railway sectors, associated with resource use, waste and increasingly due to climate change, means that maintenance projects have to be quantitatively evaluated so that such impacts can be kept to a minimum. Although it is qualitatively appreciated that maintenance work during an asset's life can consume substantial quantities of resources and potentially have an adverse environmental impact, there is relatively little work aimed at quantifying these effects, especially in comparison to the corresponding requirements associated with an asset renewal. The few studies focus on impacts arising from individual work schemes, but do not consider the impacts from the whole strategy performance nor the structure life-cycle perspectives, usually in terms of the unequal life extensions provided by different strategies.

Therefore, asset managers need to be able to evaluate both the economic costs and the environmental consequences of diverse maintenance and rehabilitation options so that an informed and optimised choice can be made, taking into account both perspectives. Furthermore, a thorough understanding of any trade-off between economic and environmental implications, generated by the options available, is required to ensure cost-effectiveness whilst reducing the environmental consequence to a minimum. However, an integrated economic and environmental decision support model for the assessment of infrastructure maintenance is still lacking. This paper presents such decision support model for evaluating the combined economic and environmental performance of diverse maintenance strategies for maximising the service life of bridge infrastructure assets. It is intended to assist asset managers in identifying the optimum combination of both economic and environmental criteria, out of different maintenance options, and to support the sector in delivering sustainable solutions for extended asset service lives.

Time is one of the factors considered during any decision making process. The decision makers have to decide the best timing for investing in a maintenance scheme in order to incur the least costs while keeping the environmental impact to a minimum. Deferring maintenance can be an alternative measure for decision consideration. This is particularly useful for managing large potential emissions in the short term while allowing time for less energy intensive and low environmental emission innovations to be developed for bridge infrastructure products and construction technologies. Moreover, deferring asset maintenance work is common for the European infrastructure industry when maintenance budgets are inadequate to undertake all the identified maintenance workload and hence the works have to be prioritized.

This brings the question of whether impacts that occur at different times should be given the same weight in the decision evaluation process. Unfortunately, current research into the value of maintenance emissions over time is lacking. An approach to accounting for temporarily delayed emissions is also explored in the decision analysis part of the framework through the introduction of time-weighting of environmental impacts. This concept is relevant to situations where there is a potential benefit for temporarily mitigating large emissions such as those produced from asset replacement works. A case study considering different maintenance strategies for a typical metallic railway bridge is presented to illustrate the use of the framework.

2 Methodology Framework

The developed framework can be used for appraising the economic cost and environmental performance of maintenance strategies that have different working life extension horizons. It provides a consistent evaluation method for comparing the performance of the strategies based on unequal life extension periods. The model also incorporates an optional step for evaluating and comparing a temporary delayed strategy with a ‘maintain now’ base scenario by introducing a novel time-weighted function, which will be discussed in detail below.

Figure 1 presents the methodology framework in terms of a flowchart. The framework adopts the life cycle analysis approach that accounts for the costs and environmental consequences that occur throughout the entire service life of a maintenance strategy. A maintenance strategy is defined as a single or a group of planned interventions that form a maintenance plan. Therefore, the strategy system can be an aggregation of associated intervention systems. In the methodology, the analysis period for the economic and environmental performance assessment is defined as the period between the construction year of the first intervention and the end-of-life year of the last intervention. The maintenance strategy system covers all the main activities and processes involved during the service period of the intervention or strategy, i.e. from the construction and use to the end-of-life stage of the system. In the model analysis, life cycle costing (LCC) and life cycle assessment (LCA) techniques are used in parallel to estimate the economic cost and environmental impact of the alternative maintenances, respectively. A detailed description of the framework is given by Lee et al. (2014) and Lee (2016).

In order to achieve a consistent evaluation in the LCC and LCA analyses and in the integration between the cost and the environmental aspects, the system boundary of the appraised maintenance systems has to be equivalent. A typical system comprises three primary sub-systems for construction, use and end-of-life stages. The construction sub-system consists of the main activities and processes involved for producing construction materials and for constructing the maintenance physical system onto the existing asset. It also includes the removal and disposal of obsolete old materials or components on the existing structure before installation of the new system, such as old paint or obsolete waterproofing. The use stage sub-system covers the main activities of planned routine maintenance tasks that are necessary to maintain the effectiveness of the constructed system, such as periodic preventative maintenance and minor repairs. The end-of-life sub-system includes the main activities and processes involved in the disassembly, demolition and disposal of waste to landfill when the system becomes obsolete at the end of its service life.

With respect to the LCC assessment, the input parameters are the main direct costs of the construction, maintenance and end-of-life of a maintenance system; they can be categorised into materials and components, labour, equipment and plants and indirect costs. The assessment output is the life cycle cost for the agency. The LCA assessment main flows are materials, energy and water for production, construction, maintenance, demolition and disposal of the maintenance system, whilst the output flows are the emissions to air, water and land including waste. Both the economic and the environmental aspects deal with the direct costs and environmental consequences of the maintenance system, however indirect/external costs can also be considered but are not covered as part of this paper. Once the economic and environmental analysis are performed, the decision evaluation is carried out using a form of ‘scoring and weighting’ multi-criteria analysis (MCA) approach.

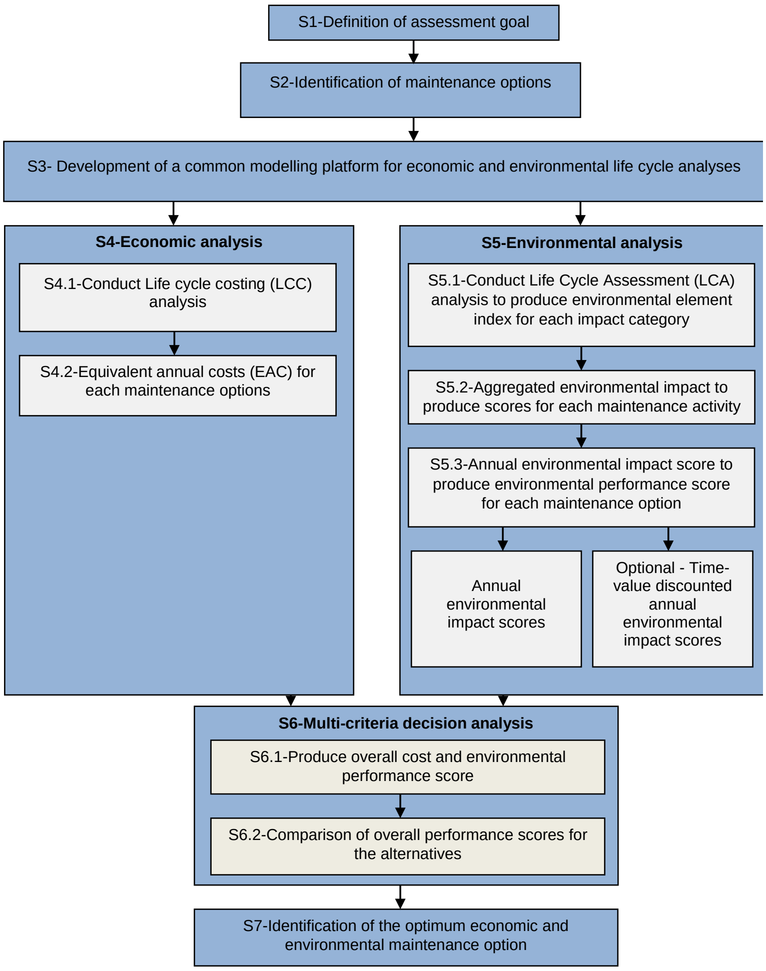

Fig. 1. Flowchart of the combined economic and environmental assessment methodology framework

2.1 Economic Performance Analysis

In order to evaluate the economic performance of the maintenance activities, the universally accepted Life Cycle Cost (LCC) approach is adopted. The annual life cycle cost of the competing maintenance strategies can be estimated using the Equivalent Annual Cost (EAC) method to address the cost comparison between maintenance strategies with dissimilar service lifespans. The EAC converts all the costs occurring over a period of time and presents the LCC cost as an equivalent uniform annual amount (Riggs, 1997). The EAC expression is presented in Equation 1 below for estimating the annual performance cost of a single maintenance system. The alternative with the lowest EAC cost is the most cost-effective.

$$\mathrm{EAC} = (\mathrm{NPV}_{\mathrm{activities\ ,year\ j}} + \mathrm{NPV}_{\mathrm{activities\ ,year\ k}} + \mathrm{NPV}_{\mathrm{activities\ ,year\ m}})\ \mathrm{x\ AF} \qquad (1)$$

where

$NPV = \left(\frac{\mathrm{Cn}}{(1+r)^{\mathrm{n}}}\right)$ is the Net Present Value of the total activity cost occurring at year $n = j, k, m\ldots$etc.;

$AF = \frac{r}{1-\frac{1}{(1+r)^t}}$ is the Annuity Factor, used to convert the total NPV into a uniform annual cost equivalent

t is the total extended service period of the utilised maintenance intervention or strategy system;

C is the total activity cost; and

r is the discount rate in percentage.

2.2 Environmental Performance Analysis

The environmental assessment involves typical LCA analyses according to the ISO 14040 (BSI, 2006) LCA methodology framework, which includes the following steps; goal and scope definition, inventory analysis, impact assessment and results interpretation. The inventory analysis stage of the LCA involves data collection and quantification of input flows (raw material, energy and water) and output emissions that are associated with each strategy's process system. The output emissions are then classified and characterized, during the impact assessment step, into six impact categories. The impact categories considered here are acidification, climate change, eutrophication, ozone depletion, photochemical ozone creation and resource depletion. In this step, the environmental impacts are converted into scores through normalization and weighting. A detailed description of the LCA analysis is provided in Lee (2016).

The aggregated environmental impact obtained from the LCA analysis is converted into an annual aggregated environmental impact (Ann. Agg, EI) for determining the environmental performance of maintenance alternatives with different life extension capabilities. Ann. Agg. EI represents the life cycle impact generated for an additional service year extended by a maintenance. Basically, the lower the Ann. Agg. EI index value, the better the environmental performance of an alternative system. In addition, a time-value weighting option is utilised to model time-preference, so that the temporary impact delay can be assessed as one of the potential alternatives. The temporary impact delay can be adopted as a measure to help the transport infrastructure industry in managing their environmental burdens such as delaying high polluting and resource consuming maintenance projects to slow down the environmental emission rate or 'buying time' for deploying cleaner technologies and higher eco-efficient designs.

Equation (2) below presents the general expression for the Ann. Agg. EI. It denotes the impact level of a maintenance alternative for prolonging the structure service life by an additional year.

$$\begin{aligned}\text{Ann. Agg. EI (in unit/ year)} &= (\textstyle\sum \text{Agg.EI}_{\text{prod,year i}} \times \text{Tw}_{\text{year i}}) + (\textstyle\sum \text{Agg.EI}_{\text{constr,year j}} \times \text{Tw}_{\text{year j}}) \\ &+ (\textstyle\sum \text{Agg.EI}_{\text{use,year k}} \times \text{Tw}_{\text{year k}}) + (\textstyle\sum \text{Agg.EI}_{\text{end,year m}} \times \text{Tw}_{\text{year m}}) \qquad (2)\end{aligned}$$

where

$\sum Agg.EI_{,xxx,\ year\ i}$ is the sum of aggregated environmental impacts for the 'material production', 'construction', 'use' and 'end-of-life' activity modules that occur on years 'i' to 'm', respectively

$Tw_{year\ x}$ is the time-value weighting factor for years 'i' to 'm'.

The time-value weighting factor (Tw) can be expressed as follows:

$$\text{Tw} = \text{Discount Factor} \times \text{Annuity Factor} = \frac{1}{(1+r)^n} \times \frac{r}{1-\frac{1}{(1+r)^T}} = \frac{r(1+r)^T}{(1+r)^n[(1+r)^T-1]} \qquad (3)$$

where r is the environmental discount rate in percentage,

n is the aggregated impact occurred due to activity at a specific year i, j ,or k,

T is the total residual service life duration of the structure; $T = t + t_d$, where t = extended years duration and t_d = delayed year duration

2.3 Combined Economic and Environmental Performance Analysis

The combined economic and environmental performance score is proposed for measuring the overall cost and environmental performance of the maintenance alternatives, as presented by Equation 4:

$$\text{CEE score} = (\text{W}_{\text{eco}} \text{ x EAC}) + (\text{W}_{\text{env}} \text{ x Ann. Agg.EI}) \qquad (4)$$

where *CEE Score* is the combined economic and environmental score

W_{eco} is the economic weighting factor

EAC is the Equivalent Annual Costs

W_{env} is the environmental weighting factor

Ann. Agg. EI is the Annual Aggregated Environmental Impact score

The derivation of the CEE score is based on the Simple Multi-Attribute Rating Technique using Swings (SMARTS) technique (Edwards and Barron, 1994). Scores and weighting for MCDA are recommended and practised by the UK public sector for transport investment appraisals (Communities and Local Government, 2009). The Economical Annual Equivalent Costs and the Environmental Annual Impact Scores are first converted into a single dimensional utility. The relative strength preference technique (Jin, 2007) is used to enable the costs and environmental scores to be measured from a same performance scale perspective, i.e. 0-100. The LCC costs and environmental scores of the alternatives are normalised into a 0-100 score performance, where 0 represents the most favourable and 100 the least favourable performance.

3 Case Study

The case study example is a typical railway metallic bridge located in the UK. The bridge deck was found to be in poor condition and requires intervention to restore the structure to a good condition level. Three maintenance strategy options are investigated, as shown in Table 1: (i) deck replacement; (ii) standard deck restoration; and (iii) minor deck restoration. As can be seen, each is associated with a different service life extension for the bridge as well as different costs and amounts of material used. Inspection activities every 6 years are also included within all strategies. These strategies are typical strategies obtained from Network Rail in the UK, the railway infrastructure operator. A comparative life cycle study is carried out by using the developed framework to evaluate and compare the economic and environmental performance of the alternative maintenance strategies and hence determine the best combined economic and environmental performance option. A discount rate of 3.5% is assumed for this study.

Table 1. Different maintenance strategies considered for case study.

Year	Maintenance strategy		
	(i) Deck replacement	(ii) Standard deck restoration	(iii) Minimal deck restoration
0	Deck reconstruction	Steel repair and paint treatment	Steel repair
5			Steel repair
10			Steel repair
15			Steel repair
21			Repaired system removal and disposal
27		Minor repair and patch repainting	
32		Repaired system removal and disposal	
75	Deck removal and disposal		

Table 2 presents the results from the economic and environmental performance analysis. The table shows the estimated equivalent annual costs (EAC) for the three maintenance strategies investigated in this case study. Option (i) deck replacement has clearly the highest EAC equal to £11,684. Small difference is observed between Options (ii) and (iii). Although the total net present costs of Option (ii) are approximately 30% higher than those of Option (iii), their EACs are comparable because of the longer working life (life cycle period) of Option (ii). The total aggregated impact for the entire working life of each strategy as well as their annual average aggregated impacts are presented. As mentioned earlier, the lower the annual impact value, the better the option's environmental performance is considered to be. Based on the results presented in Table 2, Option (ii) is the most environmentally efficient, followed by Options (iii) and (i). One of the main reasons that Option (ii) performs best is its considerably longer working life compared to Option (iii), even though its aggregated impact is slightly higher than that of Option (iii). Working life plays an influential role in determining the environmental performance. Another factor is the extent of the works in which Option (ii) uses significantly less resources than Option (i). Because Option (i) involves the production of a new steel deck, it requires approximately 400% higher amount of steel material to construct a 1m^2 deck section compared to the steel required to repair the same deck area. The last two columns of Table 2 also show the transformed economic and environmental scores used when the multi-criteria analysis is carried out at the last step of the developed framework. The results show that Option (i) performs worst in both economic and environmental criteria. Option (ii) performs second in economic score but best for environmental score. On the other hand, Option (iii) has the best economic score but is second in environmental criteria.

Figure 2 shows the effects of delaying the deck reconstruction (Option (i)) on the environmental performance score (S_{AE}). The results are shown for different assumptions for the environmental discount factor. The figure clearly shows that the S_{TWE} score decreases with deferment of the replacement work, consistently showing lower scores for longer delay timings for all discount rate cases.

Table 2. Different maintenance strategies considered for case study.

Strategy Option	Total NPV (£)	Working Life (years)	EAC (£)	Total Agg. impact	Ann. Av. Agg. impact	Economic score (S_{AC})	Environmental score (S_{AE})
(i)	308,540	75	11,684	7.3 x10^{-8}	9.7 x10^{-10}	100 points	100 points
(ii)	212,260	32	11,131	1.5 x 10^{-8}	4.6 x10^{-10}	4 points	0 points
(iii)	165,952	21	11,106	1.3 x10^{-8}	6.4 x10^{-10}	0 points	34 points

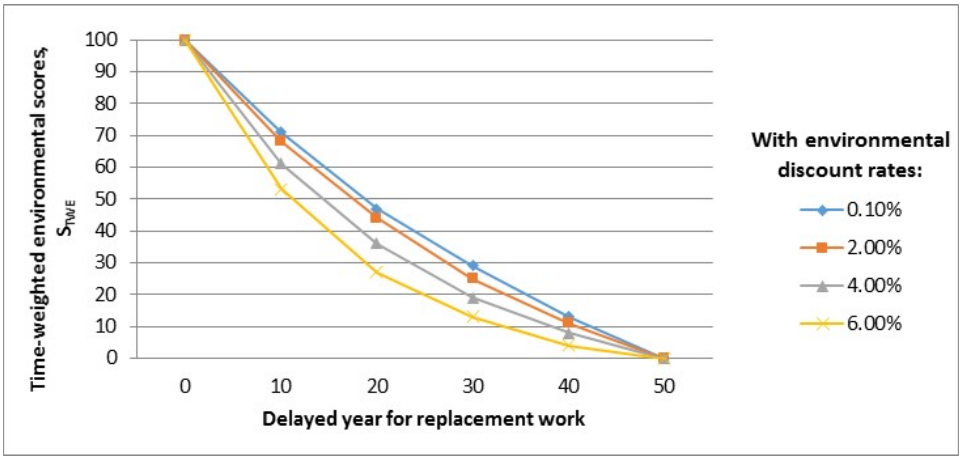

Fig. 2. Time-weighted environmental scores (STWE) for different replacement work delay timings

4 Concluding Remarks

This paper presented a decision support framework that is capable of assessing the economic and environmental performance as well as the combined economic and environmental performance of alternative bridge maintenance strategies with unequal working lives through proposing the novel adoption of the EAC method and an Annual Aggregated Environmental Impact. The framework methodology is capable of evaluating the effects of delaying maintenance activities with respect to the environment through proposing a novel time-weighting factor to account for delayed emissions. The applicability of the framework wad demonstrated through a metallic bridge case study for different maintenance strategies.

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Life-cycle cost optimisation on a set of bridges

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Abstract. This manuscript presents a life-cycle cost optimisation methodology for a set of roadway bridges, during a medium or a long period of time. The methodology analyses multiple scenarios and evaluates the performance and cost consequences of implementing different types of interventions, in different periods of time, for each bridge. The methodology takes into consideration the direct and indirect costs associated with different types of interventions, in addition with a degradation model to predict the deterioration along time. The optimisation is performed with a genetic algorithm to find the optimised intervention plan, over the desired analysis period. That process takes into consideration the life cycle cost minimization and the boundary constraints which could be fixed by the decision maker, either in terms of performance or cost. The methodology was tested on set of concrete Portuguese roadway bridges and the results of its application is presented. Attending to the nature of the problem, data's uncertainty is considered and Monte Carlo simulation Method is used to present the results in a probabilistic manner. To better support the decision, a multi-objective optimisation is also presented. The gathered results are useful to minimize the associated high life-cycle costs of bridges and also to prepare an eventual extra-budget request argumentation. The methodology is then an approach to support decisions related to the scheduling of the necessary interventions on a set of bridges over time, ensuring the required performance level and taking the available budget into consideration.

Keywords: bridges management, life cycle cost, optimisation, scheduling of interventions, probabilistic analysis, decision support.

1 Introduction

Some of the existing Bridges Management Systems are more oriented on technical decision issues than on economic issues related to costs and benefits, providing only a short-term analysis without consideration of several alternative scenarios, as stated in a report related to the USA (Markow & Hyman, 2009). Furthermore, the optimisation is usually done in two phases: identifying the type of intervention and establishing the time of implementation (Orcesi & Cremona, 2011; Sarja, 2004). The methodology presented in this manuscript is based on a life cycle cost minimization for a set of bridges and considers those two phases simultaneously.

2 Methodology

The methodology was developed to optimise the schedule of the interventions on a set of brides over time. It analyses multiple scenarios in performance and cost, to identify the one with the lower total cost that fulfils the desired safety level and all the other imposed restrictions. The methodology involves five main sections – input, degradation prediction, life cycle cost estimation, optimisation and output – and their interconnection is presented in Figure 1.

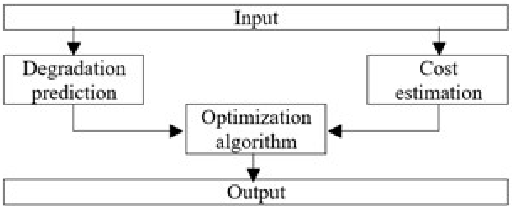

Fig. 1. Methodology's main modules

The information gathered for the set of bridges is systematized in the input section and used to categorize them in several terms. As illustrated in Figure 1, the input section feeds two predictive modules, one to predict degradation over time and the other for life cycle cost estimation. Processing the results of those two sections in the optimisation algorithm is then possible to get useful outputs and generate decision support reports. What happens in each of the three main sections is explained below.

2.1 Degradation model

The adopted performance indicator is the bridge condition state, considered with a five level scale where one corresponds to good and five corresponds to an alarming situation.

In years without intervention works, the condition state evolution over time is predicted with Markov matrices proposed by Roelfstra (2001), bridge by bridge, taking into account the correspondent kind of environment.

When a repair intervention is foreseen, it is considered an improvement in the bridge condition state. For that purpose, the probabilities of passage to the best levels of scale are specified as proposed in the matrices presented by Vesikari (2003).

When a replacement is considered, the condition state changes to one, the best level of the scale.

2.2 Life cycle cost estimation

The life-cycle cost for the set of bridges are calculated from the sum of the costs of each of the bridges, according to the following equation:

$$LCC_{set\ of\ bridges} = \sum_{b=1}^{nb} LCC_b = \sum_{b=1}^{nb} \left[\sum_{t=t0}^{tu} \left(\frac{DC_{b,t,a} + IC_{b,t,a}}{(1+DR)^{(t-t0)}} \right) + \frac{RC_p}{(1+DR)^{(tu-t0)}} \right] \tag{1}$$

where *LCC* - life-cycle cost; *b* – bridge number; *nb* – total number of bridges; *t* – year; *t0* – first year of the considered period of analysis; *tu* – last year of the considered period of analysis; *a* – type of intervention; *DC* – direct cost; *IC* – indirect cost; *RC* – residual cost; *DR* – discount rate. As shown in Eq (1), the determination of each cost parcels is made taking into account the characteristics of the bridge (*b*) the previewed repair work (*a*), as well as the condition state predicted with the degradation model for the time of its realization (*t*). Costs are calculated from the initial year (*t0*) to the last year considered (*tu*), with an update to the initial moment through an annual discount rate (*DR*).

The prediction of each one of the three different parcels - direct costs for the administrations (*DC*), indirect costs for users (*IC*) and residual costs (*RC*) – is presented next.

2.2.1 Direct costs

The direct costs parcel reflects the costs incurred by the bridges' administration and can be calculated according to the following equation:

$$DC_{b,t,a} = VE_{b,t}.VC_a.UC_{a,b}.A_b \tag{2}$$

where *DC* – direct cost*;* *b* – bridge number; *t* – year; *a* – kind of intervention; *VE* – bridge condition state at the time of the intervention; *VC* - multiplicative condition factor to correct the unitary cost of the maintenance and repairs for different condition state indexes; *UC* - intervention unitary cost by area for such type of bridge; *A* – bridge deck area.

The direct costs are then estimated taking into account the size of the bridge and the type of intervention unitary cost for the type of bridge. As the repair and maintenance unitary costs are established to the 4th condition state, for other condition states at the time of the intervention, those values are corrected by multiplicative condition factors (*VC*): 25% for 2nd, 75% for the 3rd and 150% for the 5th condition state.

2.2.2 Indirect costs

Repair and replacement interventions on bridges my also involves indirect costs for its users. The indirect costs parcel is found by using the following equation:

$$IC_{b,t,a} = VE_{b,t}.VC_{a}.(TC_{b,t,a} + CC_{b,t,a}) \tag{3}$$

where *IC* – indirect cost; ; *b* – bridge number; *t* – year; *a* – kind of intervention; *VE* – bridge condition state at the time of the intervention; *VC* - multiplicative condition factor to correct the unitary cost of the maintenance and repairs for different condition state indexes; *TC* – time costs related to the passengers delay; *CC* - extraordinary vehicle circulation costs.

The determination of *TCb,t,a* and *CCb,t,a* is made taking into account the duration of the traffic restrictions, as well as some parameters associated with the kind of the road, such as speed and average daily traffic.

2.2.3 Residual costs

The analysis of the life cycle cost is carried out for shorter periods than the lifetime of the bridges. A residual cost is considered to reflect the asset value of each bridge at the end of the studied period of time, and it is estimated according to the following equation:

$$RC_{b} = \frac{LT[AG(tu);CS{=}1] \quad - LT[AG(tu);CS(tu)]}{LT[AG(tu);CS{=}1]} .\, \mathrm{UC}_{\mathrm{a=2,b}} .\, \mathrm{A}_{b} \tag{4}$$

where *RC* - residual cost; *b* – bridge number; *tu* – last year of the considered period of analysis; LT – lifetime; *AG* – age of the bridge; *a=2* – replacement; *CS* – condition state; UC - intervention unitary cost by area for the kind of bridge; A – bridge deck area.

The life time (*LT*) is calculated as the time that the bridge in question takes to reach the worst level of condition state (the 5th). With that, the residual cost for each bridge is directly related to potential future costs associated with their replacement and inversely related to its residual asset value. That way, the residual cost parcel allows to ensure that the optimal plan of intervention is determined independently of the considered analysis period (Almeida 2014).

2.3 Optimisation process

The predicted degradation and the cost evaluation allows the comparison between different intervention scenarios in each bridge, either in terms of condition state or in terms of inspection, maintenance and repair costs. Then, in the optimisation process, the objective is to identify the best intervention scheduling on a set of bridges over the desired analysis period of time, in order to minimize their life cycle costs.

The optimisation is made with a genetic algorithm that can respect the restrictions that the decision maker wishes to provide, both in terms of performance and in terms of costs. Thus, the responsible can impose a certain performance level and, for instance, limit the investments to be made in some given years with budgetary constraints.

The bridges' administrations may also have in account different goals and reasons for their choices on multiple points of view, using a multi-objective optimisation based on Pareto charts, as illustrated in the following section.

3 Results for a set of bridges

The results presented on this manuscript were obtained for a 100 concrete roadway bridges. Those bridges are located in different earthquake zones of Portugal, with a total length between 50 and 1500m, several ages and condition states between one and four, with an average value of two.

Attending to the nature of the problem, some variables were considered by probabilistic density functions. Thus, it was possible to make a probabilistic analysis that allows estimating the variability that may be associated with the obtained results. The Monte Carlo simulation Method was used for probabilistic analysis, with 1000 simulations.

The results of the probabilistic analysis for the considered set of 100 bridges, for 20 years, with a 5% annual discount rate, are shown in Figure 2.

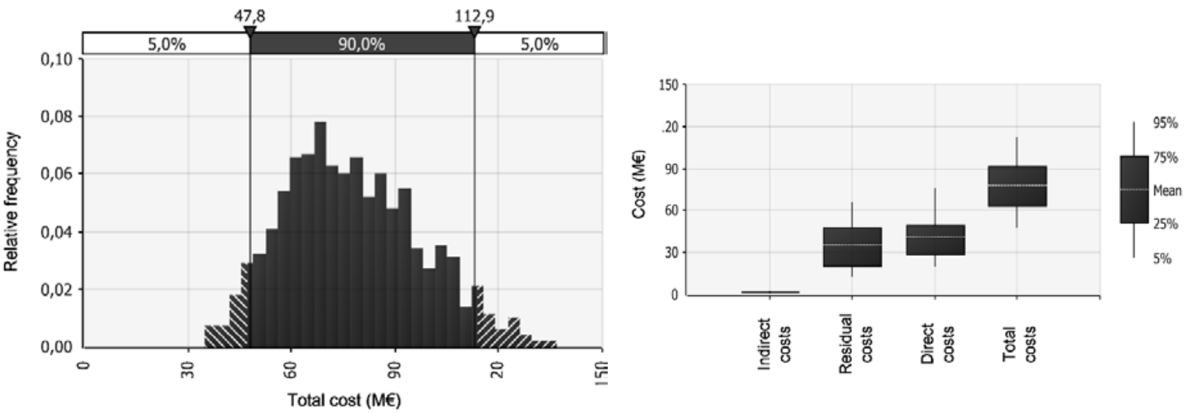

Fig. 2. Optimised total costs for the considered set of bridges for 20 years
a – total cost variability; b – cost parcels variability

The probabilistic distribution for total cost (Figure 2a) has a roughly triangular shape, as expected since the costs depend directly on variables that were defined with probabilistic density functions of this type. The average total costs is 78M€, but considering a range of variation between the mean minus the standard deviation and the mean plus the standard deviation, it may be said that the total cost will be between the 58M€ and 97M€. However, that variation will be reduced because the analysis could be redone periodically to actualize bridges' condition states and to redefine some parameters that were better known in the meantime.

Figure 2b shows that the considered users indirect costs, related to the traffic restrictions, are little expressive (around 1%) in total costs because only repair works were planned. Furthermore, in that figure it is also possible to conclude that the residual cost parcel is not negligible. As the goal of optimisation is to minimize the total costs, when more interventions are provided the direct and indirect costs increase, but the residual costs decreases. On the other hand, when fewer interventions are provided their implementation costs decrease, but the residual costs increase.

A multi-objective optimisation was also possible, based on non-dominated Pareto fronts – the set of solutions where no objective can be improved without sacrificing at least one other objective. The multi-objective optimisation allows the decision maker to analyse graphics with two main objective functions presented at the same time, for example the cost minimization and the performance maximization. Figure 3 shows an example gathered with the determination of the minimum total cost for different probabilities of being in the worst condition state.

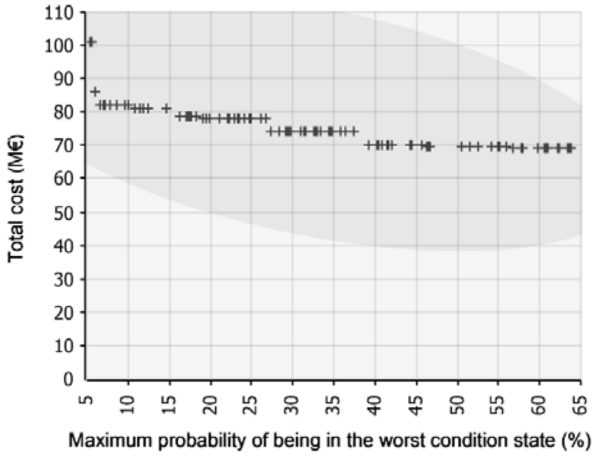

Fig. 3. Total cost variation with the maximum probability of being in the worst condition state - non-dominated Pareto fronts

The increase of the total cost for more demanding performance levels is natural, but that behaviour has several stages, as it is possible to notice in Figure 3. It is possible to see that in some cost levels it is possible to reduce the risk almost without increasing the investment – passing for example from 38% to 27% of the maximum probability of being in the worst condition state, or from 27% to 15%.

4 Conclusions

The manuscript presents a model to estimate bridges life cycle cost and also a way to use it, together with a degradation prediction model, for an optimised schedule of the different kinds of actions necessary in a set of bridges during a certain period of time.

The model proposed to estimate bridges life cycle costs may consider different kinds of interventions and the condition state predicted for the bridge at the time of its implementation. The life cycle estimation is made considering both direct and indirect costs. Moreover, it uses a residual cost value that showed to be important to ensure that the determined optimal plan of intervention is independent of the considered analysis period. The results gathered for a set of Portuguese concrete bridges show that the considered indirect costs, related to the traffic restrictions, are little expressive in total costs because only repair works were planned, but that can change for bridges in the worst conditions.

The presented management methodology can estimate the bridges' performance and costs over time considering different scenarios of intervention. Then, using an optimisation algorithm and the Monte Carlo method it is possible to make a probabilistic estimation of the future needs in terms of intervention and budget, minimizing the total life cycle costs.

The presented methodology application shows that it is appropriate for schedule different kinds of actions in a set of bridges in a medium or long time term, considering different kinds of restrictions, maximizing the performance level and minimizing the investment. Thus, the presented methodology can be easily used by bridges administration to support their decisions under multiple points of view.

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Bridge management practice & methodologies related to flooding hazards

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Abstract: The management of bridges exposed to natural hazards has been extensively elaborated in the last 30 years, but the current practice still cannot account for sudden events such are earthquakes and gravitational hazards. There is a need for adequate approaches to take into account these threats, thus ensure reliable levels of bridge performance and mobility of goods and people in a society. Here, as the most adequate top-level performance indicator - the quantitative measure of vulnerability of a bridge failure is suggested since it may point out the bridges in a network that need specific attention. In this paper, a review of the current bridge management practice & methodologies related to flooding hazards is performed. The aims were to point out state-of-the art approaches, discuss their drawbacks and give essential ideas for their update. This will in turn lead to elaboration of adequate guidelines in quality control plans for roadway bridges endangered by natural hazards in Europe. The homogenization of the performance indicators used in European countries is currently underway and its results will be used in the future research tasks of the Work Group 3 within the COST TU1406 action.

Keywords: bridge management, performance indicators, vulnerability, quality control plans, flooding hazard

1 Consideration of a threat of flooding in bridge management practice

The bridge failures due to sudden events such are earthquakes and gravitational hazards often occur regardless of bridge age, static system and construction materials. Here, the knowledge of factual risk is deemed necessary since the validation of applied management practices is only possible after the failures happen. This has become one of the primary research topics in bridge management, especially in the case of flooding events, which are the most common cause of inadequate bridge performance around the world. For this hazard, the current practice usually rests on qualitative approaches and regular inspections, which does not provide satisfactory results in scheduling adequate risk mitigating actions.

The review of the applied performance indicators may be found in documents related the Long-Term Bridge Performance Program (Hooks & Frangopol, 2013), (Brown et al, 2014). Here, the flooding event is recognized as an influencing factor for two categories of bridge performance: structural condition (durability and serviceability) and structural integrity (safety and stability in all failure modes). However, only a general guidance on application of indicators such are resilience, robustness, vulnerability & risk is given.

The most common cause of bridge failure in flooding events stems from local scour and in the National Bridge Inventory (NBI) there is a specific item number 113 which is related to scour critical bridges. The condition rating values of this item obtained from inspections in combination with ratings of items related to substructure and foundations, govern the plan of actions at endangered bridges. The idea to combine the value of the item 113 with other relevant NBI items in a procedure which uses weighting factors in order to introduce a new index - bridge sufficiency index is presented in (Sivakumar et al., 2003). In some US states the bridges are specifically ranked using qualitative assessments based on their hydraulic vulnerability and in turn scheduled for a specific plan of action (NYSDOT, 2003).

Currently, there are only few software for risk analysis of bridges which account flooding hazard, but do not account the resistance of bridges to specific hazard magnitudes/scenarios. The HAZUS-MH (HAZards U.S. Multi-Hazard) is risk assessment software which uses Geographic Information System (GIS). It distinguishes the effects of flooding hazards for two elements of transportation infrastructure: roadway sections, which can become submerged, and bridges, which may fail due to scour during a flooding event. In both cases, it uses coincident analysis between the geographic extents of the flooding and the location of infrastructure. Herein, the probability of failure of bridges exposed to local scour is based on the bridge's structural configuration, rating from the NBI and flood return period, while only the direct costs of failure are considered (FEMA, 2007). The Swiss federal roads authority (FEDRO) developed the software Road Risk for evaluation of aggregate risks of road/traffic interruption due to gravitational hazards. It uses GIS and allows calculation of the probability of road links' failures for different intensities of gravitational hazards - flooding, rockfall and avalanche.

The bridge performance is lately the prime research topic in Europe as well. The ongoing COST TU1406 action has a goal to establish quality control plans (QCP-s) for roadway bridges, thus in turn enhance preparedness in face of future sudden/slow events. Here it is necessary to elaborate guidelines on maintenance/inspection scope & frequency as well as define related equipment and necessary human resources. Within the Work Group 3 of the COST action, the Task no. 4 has the goal to investigate and take into account the dynamics and uncertainty of the non-interceptable (i.e. sudden) processes that can significantly affect the bridge performance. The desired output is definition of triggering criteria for more detailed inspections and maintenance interventions in respect to required quality levels. At the moment, the bridge performance indicators are gathered and are in a process of homogenization. Their review and review of related management practice in Europe related to the threat from flooding events is a future task of the WG3. In this paper, the current practice in bridge management related to flooding is discussed. As a basis for elaboration of the quality control plans, here suggested is vulnerability – simple, yet sufficiently comprehensive top-level performance indicator.

2 Vulnerability – the top level performance indicator for hazards

The development of Bridge Management Systems (BMS) is underway in many countries, and one of the main tasks is establishing of new methodologies which allow risk based approaches. In the survey performed by IABMAS (Mirzaei et al., 2014), the information and basic comparisons among 25 BMS from the total of 18 countries is given. The findings of this report show that only a few BMS account risk of bridge failure due to hazards. Generally, common for many countries, the risk based approach for hazards comprises likelihood/consequences matrixes (i.e. risk matrix). Some related examples on this matter are given in (Cambridge systematics, 2011). The existing approaches in literature and practice are mostly qualitative or semi-quantitative. In these, the term failure or failure mode is related to a certain level of damage, displacement or consequence but neither does account resistance of bridge/s in specific hazard scenarios.

It must be highlighted that in a case of natural hazards, the quantitative performance indicators are preferred, particularly those which account both hazard magnitudes and related resistance of a bridge. Here, as a top-level performance indicator, the measure of vulnerability is suggested. It represents the product of a conditional probability of bridge failure in a hazard event of a specific magnitude and total consequences of such event, i.e. it is reflected through monetary units (Birdsall, 2009), (Erath et al., 2011):

$$V_n^s = P_n^s \cdot (DC_n + IC_n) \qquad \text{(Eq.1)}$$

Where:

V_n^s = vulnerability of a bridge with respect to a hazard event of a specific magnitude *s* and a chosen failure mode ***n***

P_n^s = conditional probability of specific bridge failure in the chosen failure mode ***n***, with respect to a hazard event of a specific magnitude ***s***

DC_n = direct consequences with respect to the chosen bridge failure mode ***n***

IC_n = indirect traffic related failure consequences with respect to the chosen bridge failure mode ***n***

In difference to the measure of risk, the vulnerability is more convenient to grasp since it relates simply to the given hazard magnitude. This is deemed sufficient for a comprehensive screening of an entire bridge population and identification of bridges that need to be examined in more detail.

The measure of vulnerability is suggested in the management of bridges with unknown foundations (Stein et al, 2006). The assessment is based on the HYRISK Methodology (Pearson et al, 2002). The recent application of this risk-based approach in the assessment of scour critical bridges in North Carolina confirms the benefits of the quantitative approaches. The savings from evaluation of 3752 bridges in comparison to the conventional method were estimated to nearly 7.0 million US dollars as reported in (Mulla, 2014).

Recently, a novel methodology for quantitative vulnerability assessment has been presented in (Tanasic et al., 2013), (Tanasic, 2015). Here, the evaluation of the conditional probability of a bridge failure due to local scour comprehensively accounts flooding magnitudes, related local scour action and elaborates combined soil-bridge failure modes. For the application of this approach on a network level, the minimum data set is required and one of the main issues, besides the data availability, is the selection of the adequate equipment for structural health monitoring and the retrieval of hazard data - sensoring/monitoring systems, which may aid in the early recognition of threats from flooding. The mentioned approach has potential to be applied in the case of other natural hazards as well. It is going to be basis for the elaboration of adequate quality control plans with respect to non-interceptable processes.

3 Required quality levels and adequate actions and for bridges vulnerable to flooding events

It is envisaged in the COST TU1406 that the QCP-s will be governed by specific performance indicators which may trigger adequate maintenance interventions. Here, the quality requirements for various bridge types and different sudden events, which distinguish the type of action, must be clearly outlined.

The required quality levels in a case of a flooding event are in fact performance goals related to structure maintenance and safety. The following statements for quality levels may apply:

- Certain percent of bridges in a network must not fail (traffic interruption included) during a flooding event of a specific magnitude & return period
- The probability of a bridge failure related to a certain return period/magnitude of a flooding event, regardless of traffic volume, is limited to a specific value (e.g. 10^{-3})
- Certain value of a vulnerability of bridge/bridges exposed to a specific flooding magnitude is required

For the first definition, the threshold values may be obtained by comparing budget requirements with indirect costs of inadequate bridge performance obtained in traffic simulations for different flooding hazard scenarios. Here, the only issue is how to comprehensively define the connection between the inadequate performance and the related scenario. Thus, some would find the second statement for a quality requirement to be more suitable. However, it is suggested to ultimately use the measure of vulnerability, which encompass both of latter mentioned statements.

The approach which use vulnerability assessment in decision making process has been introduced in (Stein et al, 2006), and the minimum performance levels (MPL) according to HYRISK were discussed and updated. Here, the MPL-s are in fact values of probabilities of a bridge failure in a flooding event, but these have been based on the frequencies of observed failures and overtopping. The related thresholds are given in accordance with the road importance and scores of specific NBI items. Based on the result of the vulnerability assessment the following actions are suggested:

- If the MPL are not met – immediate foundation survey is necessary
- Automated scour monitoring is considered warranted if the lifetime risk of death is greater than the cost of automated scour monitoring
- Scour countermeasures are considered warranted if the lifetime risk of failure is greater than the estimated cost of scour countermeasures

Although there are multipliers which take into account the static system of a bridge and type of foundations, the approach does not account flooding magnitudes nor soil resistance, which is the main drawback of this approach.

The threshold for initiating the additional hydraulic analysis and countermeasure installment at bridge sites is mostly based on the application of formulas for local scour evaluation at piers/abutments

(Arneson et al., 2012), (FHWA, 2009). The failures are defined here for the cases where the evaluated local scour depth exceeds foundation depth. It is a well-known fact that these formulas tend to overestimate the actual scour depths at sites, thus the related maintenance measures are often considered as overly conservative.

In the light of the novel quantitative methodology for vulnerability assessment (section 2), the level & frequency of the inspections & interventions are governed by availability of the minimum set of data and background information, which has been discussed in (Tanasic & Hajdin, 2016). These information are related to a bridge site, river, foundation soil, bridge structure and traffic and comprise:

- Investigation of a flooding scenario plausibility – e.g. pier/abutment exposure (angle of water attack on the piers/abutments)
- Hydrologic survey – flood return periods and water height/volume monitoring
- Gathering information on reinforcement detailing at joints
- Assessment of bearings' states in respect to deterioration processes & damage
- Foundation survey (if unknown) and foundation depth estimation
- Investigation of soil geo-mechanic characteristics (erodibility, friction, cohesion and weight)
- Underwater inspections to determine local scour depth
- Scour monitoring equipment – threshold for installing countermeasures
- Traffic simulation and monitoring (update of a road importance; evaluation of possible indirect consequences)

Based on the available data in the databases and performed inspections, the possible maintenance interventions and mitigation procedures that may be undertaken at bridge sites are:

- Soil works on river bank, channel and embankment (alleviating the adverse impacts of a flooding scenario)
- Bearings replacement – bridge resistance to withstand extreme events is increased
- Joint or member strengthening - bridge resistance to withstand extreme events is increased
- Scour countermeasures - eliminating the threat of failures due local scour at a pier/abutment

The thresholds of vulnerability for every detailed inspection, maintenance actions and mitigation procedure are going to be part of a future work within WG 3.

4 Conclusion and future tasks of the WG3 of the COST TU1406

The mitigation of risks related to the failure of a bridge due to flooding events is rarely in the scope of bridge management (BM) practices around the world. The maintenance/repair/rehabilitation actions are usually undertaken following a major extreme event. The diversity among bridges and unpredictable hazard intensities/scenarios impede gaining knowledge on the actual levels of risk of a bridge failure due to hazards. Thus for the purposes of BM, the measure of vulnerability is suggested to be used. Here, it is essential to account the bridge resistance to failure in a flooding event, and a novel quantitative approach is put forward.

The results of WG1 of the COST TU1406 action will comprise a database of key performance indicators and it is expected to gain information on those related to the threat of sudden events. However, the characteristics of methodologies in which these indicators are used will unfortunately remain unknown due to the fact that this was not in the scope of the questionnaire disseminated to the COST countries. Thus, among other tasks, the efforts of the WG3 will be aimed at the preparation of a suitable questionnaire in English for countries that have reported utilization of either low-level or top level performance indicators related to sudden events (earthquake and gravitational hazards). This will moderate the future research efforts in establishment of related QCP-s for roadway bridges.

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Assessment of bridge performance by load testing after reconstruction

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Abstract. Extraordinarily heavy rains fell over Bosnia and Herzegovina during the third week of May 2014 causing massive flooding in the northern, eastern and central parts of the territories bordering Croatia and Serbia. This caused destruction of many bridges in these regions. Several bridges were reconstructed and load testing was performed in order to measure the structural response of a bridge under various loading conditions and to determine its structural integrity. This paper presents a load-test study that evaluated the response of steel-composite bridge which was rebuilt after it was destroyed in May floods in Bosnia and Herzegovina. Unusual behavior was observed during nondestructive testing which clearly indicated the existence of some defects on the structure. On the basis of results of load testing and visual inspection causes of defects were identified and recommendations given to the Client.

Keywords: bridge load testing, static, dynamic, damages, non destructive testing

1 Introduction

Extraordinarily heavy rains fell over Bosnia and Herzegovina during the third week of May 2014 causing massive flooding in the northern, eastern and central parts of the territories bordering Croatia and Serbia. These regions received more than 250 (in some areas up to 300) liters of rain per square meter, the highest amount measured in the country in the last 120 years. This rainfall caused sudden and extreme flooding of several rivers (Bosna, Drina, Krivaja, Una, Sava, Sana, Vrbas) and their tributaries as well as landslides. As a result many bridges were destroyed and they had to be rebuilt. In order for the structures to be put in use it was necessary to conduct static and dynamic load testing of bridges in order to check their behavior.

2 Bridge load testing

Nondestructive load testing is an effective approach to measure the structural response of a bridge under various loading conditions and to determine its structural integrity. This load-test program integrates an optical surveying system, a sensor dilatation measurement on steel and concrete parts of the structure and deflection analysis by the use of Inductive Displacement Transducer. The bridge is exposed to static and dynamic loading in order to evaluate the behavior of the bridge. The actual response of a bridge to loads is usually better than what the theory dictates (NCHRP-234, 1998). Factors that contribute to the load capacity difference include unintended composite action, load distribution effects, participation of parapets, railings, curbs, and utilities, material property differences, unintended continuity, participation of secondary members, effects of skew, portion of load carried by deck, and unintended arching action due to frozen bearings (NCHRP-234,1998).

Load testing in Bosnia and Herzegovina is defined by (BAS U.M1.046, 2005) and represents as an "effective means of evaluating the structural response of a bridge." The purpose of conducting load testing on existing bridges is to evaluate their structural response without causing damages. In this respect, as load testing is usually conducted in a non-destructive manner resulting that it may be defined as such.

Every testing procedure is in one segment the same and in the other specific for each bridge. For each bridge a clear program needs to be set with clear testing objectives and load configurations, selection and placement of instrumentation, analysis technique, evaluation and comparison of test results and analytical results.

2.1 Static testing

In testing a bridge various structural elements need to be examined. The strength of these elements is generally determined by placing strain or deflection-transducer gages at critical locations along the elements. The bridge was tested to static loading and during that time deflections at the critical sections were measured in order to obtain data on the actual behavior of the bridge structure. Measuring of the deflection was carried out on the asphalt layer in the middle of each span and at the locations of the supports by survey instruments. Additionally, under the upper structure the deflection of the steel girders was measured using Inductive Displacement Transducer. In order to determine the stresses in the steel and concrete elements of the superstructure strain gages were glued and dilatations were measured during the loading and unloading phase.

For static testing the bridge was incrementally loaded up to the full ultimate design live load in order to induce maximum effects. In that respect each span of the bridge was loaded with an adequate number of trucks and their position. At each load step the instruments were monitored and the results were compared to the analytical model before proceeding with the next load step. At the end the measured data (deformations, strains-calculated stresses) was compared with the analytical results and adequate conclusions and recommendations were given.

2.2 Dynamic testing

The purpose of the dynamic load test is to determine the controlling parameters of the dynamic behavior of the bridges. The main dynamic characteristics of the structure are the fundamental vibration frequency, the dynamic amplification factor and the logarithmic decrement. These properties are usually not analyzed in detail in the design phase of small and middle sized structures as was the case with this bridge. However, these quantities are relatively easy to obtain experimentally, and can give valuable information for the exploitation and maintenance of the bridge (Burdet and Corthay, 1995).

Acceleration transducers were used in the dynamic test to measure the acceleration of the bridge. The accelerometers were mounted at the asphalt layer and at the curb in the span which was tested. Dynamic load testing of the bridge was conducted by one truck passing over a plank of 5 cm thick. This plank is used to represent the effect of deterioration of the pavement and in this way it causes the excitation of the bridge. The truck was traveling with different speeds; 10 km/h, 20 km/h, 30 km/h, 40 km/h, and 60 km/h, and in different locations: in the middle of the road, in the downstream and upstream lane. By varying the speed of the truck on the bridge, the full range of traffic speeds was investigated.

The measurements were taken and recorded by Spider 8, a dynamic data acquisition system with integrated Fast-Fourier Transform (FFT) analyzer utilizing a Software Catman 5.0, allowing an immediate interpretation of the results during the test. At the end the obtained data were compared with the analytical results and adequate conclusions and recommendations were given. The bridge was modeled using a finite element program Tower (Tower, 2015).

3 Specific data of the bridge on the regional road R-467 Zavidovići-Olovo

One of the bridges that was destroyed during the floods was a bridge on the regional road R-467 Zavidovići-Olovo crossing over river Krivaja (Figure 1a). During the floods the superstructure of the bridge was completely destroyed while the substructure was damaged. In that respect the piers of the bridge were strengthened while the upper structure was completely rebuilt.

It is a steel-concrete composite bridge whose deck composes of a concrete slab thickness 22cm connected to a twin steel girder. All the dimensions and data are illustrated in Figure 1b. Total length of bridge is 60.75 m, which is composed of three simple girders having individual lengths of 11,0+36,75+13,0m as seen in Figure 1c.

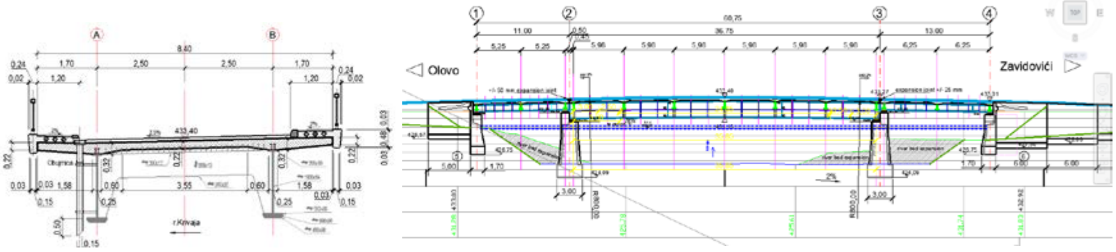

Fig. 1. Bridge over Krivaja River
a – bridge under investigation; b – characteristic cross section; c - longitudinal section

3.1 Visual inspection

In is important to state here that during the visual inspection several defects were observed which had a direct impact on the behavior of the bridge as will be demonstrated. In that respect only the mayor ones will be mentioned.

- Damage of expansion device (Figure 2a), as well as a "sag" of the expansion device in the amount of 2 cm (bridge in use only two months) (Figure 2b).

Fig. 2. Expansion joint
a - bad quality of expansion joint; b-"Sag" of the expansion joint

- Significant leakage of water at the location of the abutments (Figure 3a) as well as at the location of the piers (Figures 3b) in the area of expansion device is noted. This has caused corrosion of the bearings (bridge at that time was in use for only 2 months) as clearly illustrated in Figure 3c.

Fig. 3. Water leakage – cause to corrosion consequence
a- Water leakage(WL) abutment; b- WL mid-span expansion joint; c- Bearing corrosion

- Figure 4 shows direct contact of the steel girder with the back abutment wall, there is no spacing. This prevents proper operation of the expansion joint and adequate movement of the bridge in longitudinal direction. This is one of the causes of the expansion joint damage and this can lead to further degradation and its "outbreak".

Fig. 4. Direct contact between the steel girder and back abutment wall

3.2 Static testing

As the bridge is a two-girder steel bridge testing of the bridge was done with the centric (Figure 5a) and eccentric loading (Figure 5b) in order to cause maximum impact on the steel girders. It was interesting to note that during the symmetric loading in the first span (Figure 5) the deflections on the upstream and downstream steel girder were different, which should not be the case as the steel girders and the applied load were the same. Additionally, the values were higher in respect to the values obtained by calculations. This was a clear indication that something was "wrong". During the measuring it was observed that there was uplift in the location of the support N2 in the amount of 2 cm. This was clearly connected with the problem of the bearing and direct contact between the steel girder and back abutment wall as illustrated in Figure 4.

Fig. 5. Bridge static loading-second span
a- symmetric; b- asymmetric

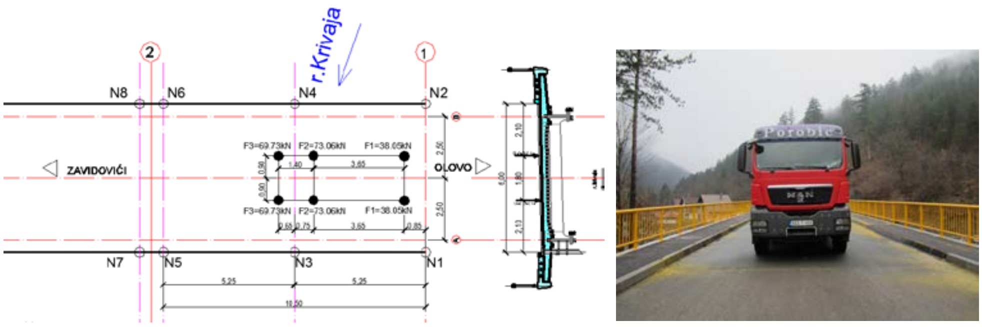

Fig. 6. Bridge static loading-first span
a- symmetric; b- on the site

Deflections in the middle of the first and third simple beam were a little bit higher than the calculated ones which can as well be connected with the condition of the bearings. The deflection (DF1) of the downstream girder during the measurement was -2.82 cm, while the calculated one was -1.50 cm. After reviewing all the data and connecting this with the visual inspection it was obvious that the defects in the bearings had a direct impact on the behavior of the bridge. In that respect recommendations were given to the client regarding this issue without providing a complete validation of the bridge.

The measured deflections in the middle of the second span for symmetric position of the trucks were -11.00 cm and -12.81 cm at the location of steel girders, while the calculated values were -13.95 cm and -13.98 cm respectively. The measured values were -12.49 cm and -13.03 cm, while calculated ones were -15.60 cm and -12.36 cm respectively for the truck placed downstream. For the trucks placed upstream the measured values were -14.25 cm and -11.79 cm, while calculated ones were -15.81 cm and -12.45 cm respectively. The problem which was noticed in the second span was as well at the location of the bearing N8 (see Figure 6) where unaccepted movement in the up direction (3.8 cm) was measured. The same problem with the bearings was observed.

Measured dilatation in the steel girders in the second span and concrete slab and from them calculated stresses were lower than the calculated ones. Dilatations in the steel tension part were in the range of 144.00 to 162.12 µm/m and in the range from -5.65 to -11.06 µm/m in the compressive zone. This consequently gave stresses from 30.24 to 34.05 MPa in tension and -1.19 to -2.32 MPa in compression. Stress values obtained in the calculation from 36.27 to 40.20 MPa in tension and -7.41 to -8.53 MPa in the compression. The measured stresses in the concrete were in the range from -1.29 to -1.16 MPa while the calculated values were -0.28 to -0.20 MPa. (Report, 2015) This clearly indicated that as mentioned in Chapter 2 that the actual response of a bridge to loads is usually better than what the theory dictates (NCHRP-234, 1998).

There were some residual stress after unloading but as they were insignificant small they may be neglected. After unloading the bridge returned into the position before loading, ie., the structure behaves elastically for a test loading.

3.3 Dynamic testing

Dynamic load testing of the bridge was conducted by a truck passing over a plank of 5cm thick. Here only results in the second span will be presented. Figure 7a shows passing of the truck over a plank of 5cm thick. The vertical excitation of the bridge when the truck passed having the speed of 30 km/h is shown in Figure 7b. Finally the calculated power spectra using the FFT is shown in Figure 7c.

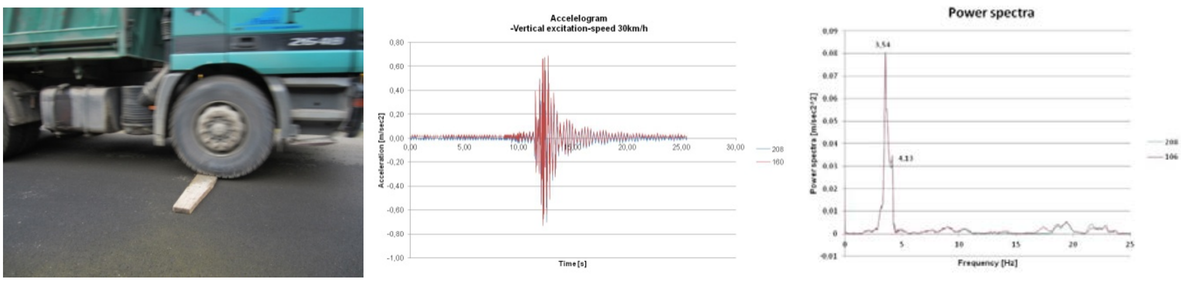

Fig. 7. Bridge dynamic load testing
a - Truck passing at different speeds; b – Accelelogram; c - Power spectra

Measured frequencies were 3.54 Hz and 4.13 Hz, while calculated ones were 3.83 Hz and 4.51 Hz. The obtained results were satisfactory.

The concept of a dynamic amplification factor (DAF) is used to describe the ratio between the maximum load effect when a bridge is loaded dynamically, and the maximum load effect when the same load is applied statically to the bridge. As stated in (Rule book, 1991) generalized DAF value was applied to the worst static load case for a given bridge. This is a conservative approach since DAF depends on the length of the bridge and ignores many significant bridge and truck dynamic characteristics. However, as this is currently enforced rule calculations were done obeying these rules. More complex calculations would be an advantage and this is planned to be done in the future.

Table 1. Dynamic amplification factor (DAF)

Phase	**Measured**		**Calculated**	
	Deflection DF3(downstream)	Deflection DF4 (upstream)	Deflection DF3 (downstream)	Deflection DF4 (upstream)
Zero	0.00	0.00	-	-
Static load-downstream	-4.00	-1.72	-4.87	-2.08
Static load-middle	-2.85	-2.80	-3.56	-3.38
Static load-upstream	-1.90	-4.20	-2.08	-4.87

Dynamic load-downstream	-4.16	-1.98	-	-
Dynamic load-middle	-3.26	-3.08	-	-
Dynamic load-upstream	-2.12	-4.27	-	-
DAF- downstream	**1.04**	**1.15**	-	-
DAF- middle	**1.14**	**1.10**	-	-
DAF- upstream	**1.12**	**1.02**		

From testing of the bridge under static and dynamic loading DAF was obtained using the well-known equation (1)

$$k_d = \frac{f_{dyn}}{f_{stat}} \tag{1}$$

Average value of the DAF amounts to 1.095, while the calculated value is 1.11, which is more than satisfactory.

4 Conclusion

This paper presents static and dynamic load testing of a reconstructed bridge which was destroyed during the major May 2014 floods is Bosnia and Herzegovina. Usual testing procedure were used, however due to bad construction of some elements clear discrepancies were notices in respect to the numerical calculations of the structure. During testing unexpected movement of the bearing points were noted opening issues regarding possible causes of such damage. Higher values of deflections in the middle of the first and third span in respect to the calculated values can as well be connected with the bearings and the direct contact between the steel girder and the abutment wall. After the unloading phase all deflections went back to the zero position indicating elastic behavior of the bridge. The dynamic behavior of the bridge was satisfactory in respect of frequency and dynamic amplification factor.

It was stated that the bridge “partially” satisfies the requirements of the enforced rules and standards behaves BAS U.M1.046. Bearing resistance of the bridge to take over the foreseen traffic loads is satisfied; however this cannot be stated for its durability. In this respect suggestions were given to the Client regarding expansion joints, bearings and direct contact between the steel girder and the abutment wall. All the defects on the structure need to be eliminated as the present state of the bridge will contribute to further and faster damaging of elements and globally will affect the durability of the bridge structure.

The Client conducted all the necessary corrections and improvements to the bridge so now it is in use.

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Interceptable Decaying Processes in Arch Bridges

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Abstract. Based on the discussion in the working group 3 of the COST TU 1406, held in Belgrade in March 2016, a categorization of decaying processes is proposed, in line with the previously agreed differentiation between interceptable and non-interceptable processes. A list of decaying processes in masonry arch is presented based on the findings of a literature survey.

Keywords: Masonry arch bridges, inspection, quality control plan, decaying processes, damages.

1 Introduction

In the 1st Working Group 3 meeting of the COST TU 1406, held in Belgrade, it was agreed that the focus of the group framework will be divided according to the most common bridge types and systems. Three main groups were defined, regarding girder bridges, arch bridges and frame bridges.

Decaying processes affecting bridges were classified, in a first instance, as interceptable or observable processes, and non-interceptable ones. Regarding interceptable processes, which will constitute the focus of this paper, a further differentiation was considered. Therefore, interceptable processes responsible for bridge decaying were divided according to those related with damaging processes or related with demand.

In this paper a list of interceptable processes affecting arch bridges is presented, according to specific literature on this topic. Also a categorization according to the classification agreed in the 1st WG 3 meeting of the COST TU 1406 is proposed.

2 The experience of the International Union of Railways - UIC

International Union of Railways (UIC) is a worldwide professional association representing the railway sector. The main goals are the promotion of technical cooperation between members, coordination of the sector's position relatively to the industry and coordination of the research needs on the related topics.

Concerning quality control plans for arch bridges, in this cases masonry arch bridges, two important documents were published by this association, namely:

- Recommendations for the inspection, assessment and maintenance of masonry arch bridges, Technical Report UIC Code 778-3, 2011;
- Catalogue of damages for masonry arch bridges, Technical Report UIC WP2/3, 2006.

These documents will be analyzed in the following chapters in the scope of the interceptable decaying processes.

2.1 Recommendations for the inspection, assessment and maintenance of masonry arch bridges (UIC, 2011)

This document pretends to provide a wide view on the topic of masonry arch bridges, presenting guidance on the activities since the inspection and assessment till maintenance and repair. Considering that a correct diagnosis of an arch structure is the first and vital step in the maintenance process, the document is divided in the following topics:

- *Outline of structural behavior of masonry arch bridges;*
- *Guidance on inspection, monitoring and condition appraisal;*
- *Description of methods for the assessment of load carrying capacity;*

- *Principles of maintenance planning and overview on repair and strengthening methods.*

Important ideas might be found for the establishment of a quality control plan for masonry arch bridges, for which a first step of a diagnosis is considered to be a detailed examination in the scope of a visual and tactile inspection, aiming to:

- *Identify the location and extent of any new damage since previous examination;*
- *Record changes to existing damages;*
- *Ascertaining if sufficient deterioration has occurred to warrant an assessment of load carrying capacity.*

According to the above, the inspection should seek the answer to the following questions:

- *Is the bridge condition adequate for the axle loads it is required to carry, or is it necessary to undertake an assessment?*
- *Is it necessary to carry out tests to obtain more information about the bridge?*
- *What are the causes of the damage to the bridge?*
- *Is it necessary to repair the bridge?*

As damage can provide information for the diagnosis of the causes a systematization is presented in the document, based on the mechanism of deterioration, being summarized in the following chapters.

2.2.1. Damages in foundations

The detection of foundation problems has to be linked with the analysis of the damages observed on the superstructure, caused by the rotation or differential movement of foundations. It is suggested that levels across the width of both abutments and piers, taken in consecutive inspections, can help the detection of progressive movement which can lead to compressive damage and failure. Sudden collapse because of a foundation movement is rare.

Damages in foundations were categorized in 2 main groups, *a)* and *b)*, for which damages and their causes are listed below:

a) Damage due to the degradation of structural elements:

- Loss of the scour protection (caused by water action);
- Local erosion of foundations (caused by water action);
- Irregular cracks on foundations (due to sulphate attack or alkali-silica reaction on mass concrete or mortar);
- Abrasion and rotting of timber piles (caused by water action and fungal attack);
- Corrosion of steel elements in foundations, such as metallic caissons (due to environmental exposure);

By themselves this damages are not serious since they do not affect the structural behavior of the bridge, although over time serious damage may follow. On the other hand foundation movements may be responsible for important changes in structural behavior, constituting the 2nd group:

b) Damage due to loss of foundation support:

- Instability and collapse (caused by cavities and undermining due to scour on the riverbed; mining subsidence; adjacent excavations; poor maintenance; inappropriate repairs or strengthening)

2.2.2. Damages in the superstructure

Damages in the superstructure were also categorized in 2 main groups, *c)* and *d)*, being listed below. Several causes might contribute to certain damage, therefore no unique relation is proposed:

c) Damage affecting structural resistance:

- Mechanical failure of masonry;
- Longitudinal cracks at the center of the arch barrel or under the spandrel wall at the edge of the arch barrel;
- Diagonal cracking of the arch barrel;

- Transversal cracking of the arch barrel (3 or 4 hinge mechanism, shear mechanism and multi arch mechanism);
- Loss or dislocation of material in the arch barrel (dropped stones)
- Vertical, horizontal or stepped cracks in the pier or abutment;
- Vertical cracks between the cutwater and pier;
- Bulging or sliding of spandrel walls;
- Rotation of wing walls;
- Stepped cracking of wing walls.

The second group is related with durability, with most common damages being identified bellow:

d) Damage affecting durability:

- Surface stains;
- Moss, lichen and other fungal growth,
- Efflorescence, crusts and superficial deposits,
- Differential surface or solar weathering,
- Honeycombing;
- Loss of material from joints or masonry including dropped stones;
- Dislodged masonry.

2.2 Catalogue of damages for masonry arch bridges (UIC, 2006)

The Catalogue of Damages is considered as a tool to the inspection, identifying and linking common damages, mechanism of deterioration, common causes, and effects of damage on the structural behavior of the bridge.

As conclusions the document lists the deterioration processes related to foundations or superstructure. The damages related with each processes are more or less the same previously listed in the document described above.

2.2.1. Deterioration process in foundations

Problems in bridges foundations were described as being related to one single process of deterioration:

a) Mechanical erosive action of water, which leads to the following problems:

- Loss of scour protection.
- Mechanical degradation of foundation.
- Chemical degradation by water containing sulphate or other salts.

2.2.2. Deterioration processes in the superstructure

Some of the most important damages in superstructure may arise from damaging processes in the foundation, constituting a group described as:

b) Superstructure problems arising from foundation problems, which lead to:

- Differential longitudinal settlement between springing and arch ring.
- Longitudinal rotation of pier or abutment
- Longitudinal settlement and longitudinal rotation of piers or abutment
- Transverse rotation along the longitudinal axis of a pier or abutment
- Differential transverse settlement of pier or abutment

- Relative movement between ends and center of piers

Other damaging processes in superstructure are described as:

c) *Problems caused by resistant behavior of skewed bridges*

d) *Problems caused by dynamic behavior*

e) *Problems caused by rotation of abutment and wing walls due to excessive earth pressure.*

f) *Problems arising from the use of the structure.*

g) *Problems caused by earth pressure on spandrels*

h) *Problems caused by differences in relative stiffness between elements*

i) *Problems caused by the construction process,* which can be related to:

   - Three-hinges mechanism due to the removal of centering following construction.
   - Differential response to rigid backfilling.
   - Waterproofing.
   - Different bonds between structural elements.
   - Change in the internal thickness of elements.

Every intervention of maintenance or repair can result in damage and affect the behavior of the structure, constituting a group described as:

j) *Problems arising from previous interventions:*

   - Use of incompatible materials.
   - Extensions or repairs which modified the management of water (changing the waterproofing and the drainage system of the bridge,) cause a prejudicial effect on the general behavior.
   - Changes which affected the permeability of the masonry
   - Intrados (shotcrete or concrete coating).

Durability was divided in two groups of damaging processes according to the existence or not of a change in the chemical composition of the material:

k) *Changes in the durability in which there is no change in the chemical composition of the material:*

   - Mechanical weathering by wind.
   - Mechanical weathering by water externally and internally. Damages from surface water. Water circulation internally as well as externally is one of the most important problems for these structures, especially on exposed elements.
   - Mechanical weathering arising from the freeze – thaw cycle.
   - Mechanical weathering due to thermal variations.
   - Mechanical weathering due to the crystallization of salt.
   - Mechanical weathering due to water expansion.
   - Damages caused by vegetation.

l) *Changes in durability due to changes to the chemical composition of the material.*

   - Erosion caused by salts.
   - Erosion caused by decomposition of carbonates.
   - Erosion caused by levigation of clay.

2.2.3. Categorization of interceptable processes

An attempt was made to categorize the deterioration processes listed in the document Catalogue of Damages for Masonry Arch Bridges (UIC, 2006), presented above, according to the classification agreed in the 1st WG 3 meeting of the COST TU 1406. Regarding interceptable processes the agreed classification divides them according to their correlation with damaging processes or with demand. The proposed categorization is presented in Table 1. The deterioration processes are those previously listed in chapters 2.2.1 and 2.2.3, from *a)* to *l).*

Table 1. Categorization of the interceptable processes identified by UIC (2006)

WG 3 Differentiation		**Deterioration processes in masonry arch bridges according to UIC (2006)**
Interceptable (observable) processes	Damaging processes	Problems arising from previous interventions (repairs or strengthening)
		Changes in the durability in which there is no change in the chemical composition of the material
		Changes in durability due to changes to the chemical composition of the material
	Demand	Mechanical erosive action of water
		Superstructure problems arising from foundation problems (foundation movement)
		Problems caused by resistant behavior of skewed bridges (skew effects)
		Problems caused by dynamic behavior
		Problems caused by rotation of abutment and wing walls due to excessive earth pressure
		Problems arising from the use of the structure
		Problems caused by excessive earth pressure on spandrels
		Problems caused by incompatible relative stiffness of adjacent bridge elements;
		Problems caused by the construction process

3 Guide for the Assessment of Masonry Bridges (IP)

Infraestruturas de Portugal developed the document Guide for the Assessment of Masonry Bridges (Aníbal Costa et al.) in order to support the inspection and assessment of masonry bridges, aiding the detection of defects and their correlation with possible origins and possible consequences. The document covers the following topics, some of them already presented in the 2nd Workshop Meeting of the COST TU 1406:

- Materials and structural systems;
- Defects, origins and condition assessment;
- Tests, monitoring and repair/ strengthening techniques.

Concerning damage detection the guide presents a systematized approach to common defects, related with structural behavior, durability and functionality. For each defect technical parameters and reference values were suggested in order to aid the definition of a condition rating for the component where they are observed.

Potential causes of the defects are also discussed in order to achieve an assertive diagnosis and consequent actions. General causes were described and categorized according to their effect on the structure, according to Table 2.

The analysis of the decaying processes, leading to damages on the structures, might be therefore analyzed according to the effects caused in the structure. Furthermore, the systematization of the effect or effects caused in the structure by certain damaging processes might be useful to the definition of triggering criteria to detailed investigations or maintenance activities.

Table 2. General causes of defects and their effect on the structure (Aníbal Costa et al.)

General Causes	Specific Causes	Effect on Structure
Accidental actions	Vehicles clash	Loss of structural equilibrium
Design/ conceptual problems	Movements of the supports	
Execution deficiencies	Bad rigging	
	Previous interventions	Loss of strength
Environmental actions	Material degradation	
	Backfill degradation	
Demand	Excessive loads	Increase in actions
	Excessive vibrations	

4 Conclusions

This paper seeks to contribute to the discussion on decaying processes affecting masonry arch bridges. According to a literature survey a categorization of decaying processes was proposed, in line with the previously agreed differentiation between non-interceptable and interceptable processes, being these last ones related with demand or damaging processes. Furthermore, also based on a literature survey, a correlation between causes of defects and their effect on the structure was presented, being proposed to consider triggering criteria leading to investigations or maintenance activities according to the effect caused on the structure by each particular defect or damaging process.

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