TU1406
COST ACTION

Quality specifications for roadway bridges,
standardization at a European level

REPORT OF THE INNOVATION SUBGROUP
COST ACTION TU 1406

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EXECUTIVE SUMMARY

National road owners are required to monitor the performance of infrastructures by means of an effective inspection and assessment framework as part of an overall asset management strategy. Performance Indicators are used to characterize the current and future structural conditions accounting for the goals specified by the codes, the owners or the operators.

In Europe, there is a large disparity regarding the way performance indicators are defined and quantified. In this context, COST action TU 1406 aims at providing quality specifications for roadway bridges standardized at a European level. One objective of the Action, pursued by the Innovation Subgroup, is the assessment of the current research and innovation advancement related to the definition of indicators and sensing technologies to assess the structural performance. The collaborative work in the Innovation Subgroup was focused on Innovative Indicators (Limongelli & Orcesi, 2017a) and Technological Innovations (Orcesi et al. 2019) that can improve accuracy and precision of quality controls of roadway bridges, thus achieving higher performances in terms of safety and availability. The ultimate goal was to provide owners with new tools to consider all the aspects: technical, social, economic, environmental, that influence the performance of the system and characterize the impact of a structure on the entire system.

A survey was performed among stakeholders to assess their needs, asking them to answer three questions related with inspections, testing and monitoring of bridges. Further, a survey was carried out in the COST TU1406 network to gather both Indicators and Technologies that are being currently investigated by researchers within and in connection with the network. The input from 36 countries in Europe was collected - through the country representative persons - and a rating scale, called Indicator Readiness Level (IRL), was introduced by the innovation Subgroup (Limongelli & Orcesi, 2017a) to assign a level of maturity to each indicator.

The aim of the rating according to the IRL is twofold:
- to check the eligibility of the considered performance indicators for quality checks;
- to underpin the indicators on which more research is needed to reach the level of full applicability for quality checks.

This may support the understanding of which improvements of best practices - in terms of performance indicators – are feasible.

Each country representative person performed a survey in his/her country gathering publications describing RPIs (Research Performance Indicators). Each indicator was rated, according to the IRL, by the researcher that proposed its inclusion in the database. Later, all the collected and rated RPIs were clustered in categories according to the Key Performance Indicators considered in the Action (Reliability, Serviceability, Economy, Cost) and also other innovative KPIs (Social aspects, Resilience) that emerged from the database of RPIs. The classification of RPIs based on their maturity level proposed herein takes basis on the scale of Technology Readiness Level (TRL) that was developed in the 70s by the NASA (National Aeronautics and Space Administration) to assess the stage of development (maturity) of new technologies and to compare different technologies in terms of the maturity level from idea to application (EARTO, 2014).

The COST Action TU1406 database of RPIs and the IRL rating of the RPIs is provided in excel format (Table 1) as a deliverable and constitute a base for further integrations and updates.

In Table 1 are presented Innovative Performance Indicators (with IRL rating and corresponding References) divided in the eight main groups related to Reliability, Serviceability, Life-Cycle Analyses, Climate Change, Seismic Performance, Scour, Social and Economic factors and Resilience.

In parallel to the analysis on research performance indicators, the study of innovative technological developments, with a focus on non-destructive testing tools and structural health monitoring solutions, is carried out with some academic and industrial partners to identify novel condition monitoring and sensing technologies for the assessment of structural serviceability and safety of existing structures. Advanced, integrated, cost-effective and reliable instrumentation solutions, techniques and concepts that can be used to compute innovative performance indicators have been considered.

The COST Action TU1406 database of Innovations on technologies is provided in excel format (Table 2) as a deliverable and constitute a base for further integrations and updates.

In Table 2 are presented Innovations on Technologies (with corresponding References) divided in the two main groups related to non-destructive testing (NDT) and structural health monitoring (SHM).

The aim of this study was to start a think tank on innovative technologies as an answer to the current needs of bridge owners.

It is noted that (i) in its current form, the presented innovations on indicators and technologies cannot be considered as exhaustive and should more serve as a starting point for future discussions and research activities, (ii) the maturity level rating of these indicators and technologies has to be considered as a dynamic processes that need to be updated constantly based on new, or previously unknown, research outcomes.

The report is organized in four sections. In Section A the work done by the Innovation Subgroup and its connections with the COST TU1406 project are described underlying extent and limitations of the outcomes. In Section B the Research Performance Indicators collected through the survey are classified, described and rated according to the IRL. Finally, Section C collects and describes the Innovative Technologies collected through the survey performed inside the TU1406 network.

Outcomes of the two surveys are summarized in Table 1 and Table 2 that have the scope to describe the current state of Research on Performance Indicators and Monitoring Technologies based on the knowledge related to the COST TU1406 network. Due to the nature of the RPIs and of the Technologies described- that are the outcomes of research investigations - and due to way the databases were gathered -based on the connections within and with the COST TU1406 and on the some expert opinion within the Action – they have to be considered dynamic processes that need to be updated constantly based on new research outcomes.

Table 1. Innovative Performance Indicators

Name of Indicator				Readliness Level (IRL)	References
B.1 Research performance indicators related to reliability	B.1.1 Durability RPIS	B.1.1.1 Chloride content at the reinforcing steel	Chloride content	9	(European Committee for Standardization, 2007); (Ozbolt, et al., 2010); (Castellote & Andrade, 2002); (Castellote & Andrade, 2006); (Kuster Maric, 2013); (Kuster Maric, et al., 2017); (Vidovic, 2018)
			Chloride content related to remaining service life	7	(British Standards Institution, 2010); (ASTM, 2004); (Vennesland, et al., 2013); (Markeset, 2009); (Tuutti, 1982); (Andrade, et al., 2013); (fib, 2006); (Vidovic, 2018); (Gode & Paeglitis, 2014)
			Chloride content related to 2D and 3D numerical prediction model for the remaining service life	7	(Meijers, et al., 2005); (Thomas & Bentz, 2001); (Ozbolt, et al., 2010); (Andrade & Tavares, 2012)
		B.1.1.2 Carbonation depth	Carbonation depth	9	(European Committee for Standardization, 2006); (Papadakis, et al., 1991)
			Carbonation depth related to remaining service life	7	-
			Corrosion Rate	9	(Kuster Maric, 2013); (ASTM Subcommittee G01.14, 2007)
		B.1.1.3 Corrosion Rate	Corrosion rate using 3D CHTM numerical model	7	(Andrade, 2014); (Andrade, 2017); (Ozbolt, et al., 2011)
			Time to first cracking appearance related to corrosion	9	(Andrade, et al., 1993); (Molina, et al., 1993); (Ozbolt, et al., 2012)
		B.1.1.4 Indicators of exposure to corrosion	B.1.1.4.1 Material indicators of exposure	8	(fib, 2006)
			B.1.1.4.2 Enviromental indicators of exposure	8	(Parrott, 1996); (Jensen, 2003)
			B.1.1.4.3 Enviromental and material indicators of exposure	8	(Izquierdo, et al., 2004); (fib, 2006); (fib, 2015); (NT Build 443:1995, 1995)
			B.1.1.4.4 RPIS related to long-term durability of repair/ protective measure	7	(Andrade & Martinez, 2009); (Vidovic, 2018); (Johansson, et al., 2008); (Matthews, 2007); (EN 1504-1:2005, 2005); (EN 1504-2:2005, 2005); (EN 1504-3:2006, 2005); (EN 1504-4:2005, 2005); (EN 1504-5:2013, kein Datum); (EN 1504-6:2006, 2006); (EN 1504-7:2006, 2006); (EN 1504-8:2005, 2005); (EN 1504-9:2005, 2008); (EN 1504-10:2004, 2004)
	B.1.2 RPIS related to general condition	B.1.2.1 Condition-based prediction using inspection rating system		8	(Mašović, et al., 2015); (Van Erp & Orcesi, 2018); (Papakonstantinou & Shinozuka, 2014); (Papakonstantinou & Shinozuka, 2014); (Schöbi & Chatzi, 2016); (Memarzadeh & Pozzi, 2016); (Kobayashi, et al., 2012); (Orcesi & Cremona, 2011); (Van Erp & Orcesi, 2018);
		B.1.2.2 Simplified index of structural damage		7	(Andrade & Martinez, 2009)
		B.1.2.3 Repair index method (RIM)		7	(Izquierdo & Andrade, 2005); (Andrade & Martinez, 2009)
	B.1.3 RPIS related to general performance	B.1.3.1 Reliability index		8	(JCSS, 2001); (Skokandic, et al., 2016); (JCSS, 2000); (Faber, 2015); (Faber, et al., 2007)
		B.1.3.2 Robustness index		8	(Faber, 2008); (Baker, et al., 2008); (Sorensen, et al., 2010); (Faber, 2015); (Faber, et al., 2017); (Frangopol & Curley, 1987); (Fu & Frangopol, 1990); (ISO, 2007); (Cavaco, et al., 2016); (Cavaco, et al., 2017); (Cavaco, et al., 2015); (Cavaco, et al., 2013)

B.2 RPI`S related to serviceability	B.2.1 RPIS related to loss of stiffness	B.2.1.1 Modal parameters		8	(Peeters & De Roeck, 1999); (Harmanci, et al., 2016); (Peeters & De Roeck, 2001); (Reynders, et al., 2014); (Spiridonakos, et al., 2013); (Bodeux & Golinval, 2001); (Polak, et al., 2005); (Peeker & Talvik, 2010); (Peeker & Talvik, 2012); (Peeker & Talvik, 2013); (Peeker, 2014); (Peeker, 2015); (Ntotsios, et al., 2009); (Teughels & De Roeck, 2005); (Maeck, et al., 2000); (Carden & Fanning, 2004); (Faraonis, et al., 2014); (Cerri & Vestroni, 2003); (Mahowald, et al., 2012); (Reynders, et al., 2014); (Spiridonakos, et al., 2013); (Mahowald, et al., 2012)
		B.2.1.2 Modal curvature		7	(Dilena, et al., 2015); (Shokrani, et al., 2016); (Radzienski & Krawczuk, 2009); (Wahab Abdel, 1999); (Anastasopoulos, et al., 2018)
		B.2.1.3 Modal flexibility matrix		7	(Polak, et al., 2005); (Schommer, et al., 2017)
		B.2.1.4 Interpolation damage index		7	(Domaneschi, et al., 2013); (Dilena, et al., 2015); (Limongelli, et al., 2017)
		B.2.1.5 Static novelty index		2	(Shokrani, et al., 2016)
		B.2.1.6 Novelty indicator		8	(Santos, et al., 2016); (Santos, et al., 2015)
		B.2.1.7 (Auto) regression error		8	(Spiridonakos, et al., 2013); (Harmanci, et al., 2016); (Sakaris, et al., 2015); (Ou, et al., 2016)
		B.2.1.8 Deflection		8	(Nguyen et al., 2016)
	B.2.2 Absolute position of elements under environmental load	B.2.2.1 Fixed point vs thermo mechanic load		-	-
		B.2.2.2 Static area deformation volume		-	-
B.3 RPI`S related to life-cycle analyses				7	(ISO, 14040), (ISO, 14044); (Frangopol, et al., 1997); (Vidovic, 2018); (ÖFG, 2017); (Woodward, et al., 2001); (SBRI, 2013); (SBRI+, 2017)
B.4 Consideration of the effects of climate change	B.4.1 Changes in the pattern of extreme events	B.4.1.1 Change in intensity		-	(Retief, et al., 2014); (Dikanski, et al., 2018)
		B.4.1.2 Change in frequency		-	(Retief, et al., 2014); (BLU, 2008)
	B.4.2 Change of vulnerability			-	(Bastidas-Arteaga, et al., 2010); (Wang, et al., 2010a); (Wang, et al., 2010b); (Wang, et al., 2010c); (Wang, et al., 2010d); (Bastidas-Arteaga, et al., 2013); (Retief, et al., 2014); (Preziosi & Micic, 2014)
B.5 RPI`S related to seismic performance	B.5.1 Indicators computed from design information	B.5.1.1 Displacement-based indicators	indicators related to seismic displacements, plastic rotations or shears forces	9	(Kolias, 2008); (Paulay & Priestley, 1992)
			indicators related to target displacement and the displacement profile (DDBD design)	2	(Kappos, 2015); (Kowalsky, et al., 1995); (Calvi & Kingsley, 1995); (Kowalsky, 2002); (Dwairi & Kowalsky, 2006)
			indicators related to drift ratio limits	7	(Liu, et al., 2012)
			indicators related to residual displacement (drift) of bridge piers	9	(Dazio, 2004); (Ardakani & Mohammad, 2013); (Japan Road Association, 1996)
		B.5.1.2 Fragility functions		7	(Moschonas, et al., 2009); (Ramathan, 2012); (Tecchio, et al., 2016); (Khosravikia, et al., 2018)
		B.5.1.3 Hybrid indicators		9	(Ostrom, 2016)
	B.5.2 Indicators computed from monitoring information	B.5.2.1 Visual inspections		9	(Veletzos, et al., 2008)
		B.5.2.2 Non-destructive evaluation (NDE)		7	(Benavent-Climent, et al., 2012); (Rens, et al., 1997); (Farrar & Worden, 2006); (Yehia , et al., 2007); (Peterson, 2013)
		B.5.2.3 Structural health monitoring (SHM)	B.5.2.3.1 Modal-based damage analysis	7	(Banks, et al., 1996); (Farrar & Doebling, 1999); (Alampalli, et al., 1997); (Doebling, et al., 1996); (Spiridonakos, et al., 2016); (Limongelli, et al., 2016); (Shokrani, et al., 2016); (Tatsis, et al., 2018); (Aktan, et al., 1997); (Schommer, et al., 2017); (Sampaio, et al., 1999); (Busca & Limongelli, 2015); (Moyo & Brownjohn, 2001); (Noh, et al., 2011); (Hwang & Lignos, 2018); (Yashodhya, 2016)
			B.5.2.3.2 Time-domain based methods	7	(Lin & Betti, 2004); (Lus, et al., 1999); (Loh & Lee, 1997)
			B.5.2.3.3 Machine-learning based methods	7	(Farrar & Worden, 2013); (Laory, et al., 2013); (Nguyen, et al., 2014)

B.6 RPI`S related to scour	B.6.1 Indicators computed from design data	B.6.1.1 Empirical local scour depth	local scour depth evaluated with evaluation formulas	9	(Park, et al., 2017); (FHWA, 1995)
			probabilistic evaluation of local scour depth	8	(Johnson & Dock, 1998); (Huizinga & Rydlund, 2004); (Govindasamy, et al., 2013); (Tubaldia, et al., 2017)
			risk-based approach	5	(Tanasic, et al., 2013); (Tanasić & Hajdin, 2018)
	B.6.2 Indicators computed from observations	B.6.2.1 Scour depth		5-7	(Hunt, 2009); (Prendergast & Gavin, 2014); (Sohn, et al., 2004); (Lin, et al., 2006); (Fisher, et al., 2013); (Zarafshan, et al., 2011); (Anderson, et al., 2007); (Prendergast & Gavin, 2014); (Fisher, et al., 2013); (Nassif, et al., 2002)
		B.6.2.2 Static deformation parameters		3	(Prendergast, et al., 2016); (Prendergast, et al., 2017)
		B.6.2.3 Natural frequency of vibration		2-3 to 7	(Xiong, et al., 2018); (Prendergast, et al., 2013); (Chen, et al., 2014); (Gavin, et al., 2018)
B.7 Social and economic RPI`S	B.7.1 Societal and economic research performance indicators in bridge management			1	(Skaric Palic & Stipanovic, 2018); (Azapagic & Perdan, 2000); (Safi, 2013)
	B.7.2 Other parameters			-	-
B.8 RPI`S related to resilience				1	(Faber, 2015); (Faber, et al., 2007); (Faber & Qin, 2016)

Table 2. Innovations on technologies

Innovations on technologies				References
C.1 Innovation in non destructive testing (NDT)	C.1.1 Non-destructive testing for reinforced concrete	C.1.1.1 Relations between indicators and measurements – gradients	C.1.1.1.1 Capacity probes	(Dérobert, et al., 2008); (Fares, et al., 2016); (Villain, et al., 2018)
			C.1.1.1.2 Use of electromagnetic waves in the radiofrequency range	(Ihamouten, et al., 2012); (Villain, et al., 2015); (Xiao, et al., 2017)
			C.1.1.1.3 Surface wave measurements	(Villain, et al., 2009); (Abraham, et al., 2012); (Song, et al., 2003); (Goueygou, et al., 2008)
			C.1.1.1.4 Combination of NDT data	(Sbartaï, et al., 2012); (Sbartaï, et al., 2006); (Ohdaira & Masuzawa, 2000); (Popovics, 2005); (Kheder, 1999); (Soshiroda, et al., 2006); (Qasrawi, 2000); (RILEM Draft Recommendation, Essais Non Destructifs Combines, DU Beton, 1993); (Kohl, et al., 2005); (Kohl & Streicher, 2006); (Zaid, et al., 2004); (Hugenschmidt & Kalogeropoulos, 2009); (Villain, et al., 2012)
			C.1.1.1.5 Electrical resistivity tomography	(Polder, 2001); (Andrade, et al., 2007); (Torrent & Fernández Luco, 2007); (Bonnet & Balayssac, 2018); (Loke & Barker, 1996); (Du Plooy, et al., 2013); (Du Plooy, et al., 2015); (Lecieux, et al., 2015); (Fares, et al., 2018)
			C.1.1.1.6 Impact echo	(Sansalone & Carino, 1988); (Sansalone & Streett, 1997); (Gibson & Popovics, 2005); (Villain, et al., 2011); (Villain, et al., 2012)
		C.1.1.2 Durability of reinforced concrete	C.1.1.2.1 Ultrasonic pulse echo	(Corbett, et al., 2018);
			C.1.1.2.2 High definition image-based technologies using infrared thermography	(ASTM, 2014); (Hiasa et al. 2018)
			C.1.1.2.3 Coda wave interferometry (CWI)	(Abraham, et al., 2018); (Planès & Larose, 2013); (Wang & Niederleithinger, 2018)
			C.1.1.2.4 Radar technique	(Lai, et al., 2018); (Klysz & Balayssac, 2007); (Dérobert & Villain, 2016)
			C.1.1.2.5 Development of robotic systems for NDE of concrete structures	(Gucunski, et al., 2015);

	C.1.2 NDT for bridge cables, ropes and prestressed concrete elements	C.1.2.1 Gamma- or X-ray radiography for detection of voids in tendon ducts		(Abraham & Côte, 2002); (Dérobert, et al., 2002)
		C.1.2.2 Impact echo technique as an alternative to Gamma- or X-ray radiography?		(Abraham & Côte, 2002); (Dérobert, et al., 2002); (Abraham, et al., 2000); (Abraham, et al., 2009)
		C.1.2.3 The capacity probe for detecting voids and white paste in external HDPE ducts		(Bore, et al., 2009)
		C.1.2.4 The US tomography for detecting voids in prestressing tendons		(Terzioglu, et al., 2018);
		C.1.2.5 Electromagnetic techniques to detect corrosion of cables		(Zahn & Bitterli, 1995)
		C.1.2.6 Magnetic flux leakage (MFL) for broken wires detection		(Xu, et al., 2012); (Kim, et al., 2014)
		C.1.2.7 Ultrasonic and acoustic techniques for inaccessible areas		(Kharrat & Gaillet, 2015); (Sluszka, et al., 2006); (Kurz, et al., 2013); (Zejli, et al., 2006); (Laguerre, et al., 2004); (Laguerre & Treyssède, 2011); (Laguerre, et al., 2018)
	C.1.3 Non-destructive crack detection methods of welded steel structures	C.1.3.1 Conventional methods	C.1.3.1.1 Bleeding (Standard NF EN 571-1)	(MIKTI, 2010)
			C.1.3.1.2 Magnetic particles crack detection (Standards NF EN 1290 AND 1291, A 09-590)	(MIKTI, 2010)
			C.1.3.1.3 Eddy current	(García-Martín, et al., 2011); (Janousek, et al., 2008); (Hashizume, et al., 1992); (Droubi, et al., 2017)
			C.1.3.1.4 Ultrasonic (Standards NF EN 1712 - 1714, Standard NF EN 583 - PARTS 1 TO 6, THE IS US 319-21 Document published in June 1995)	(MIKTI, 2010)
			C.1.3.1.5 Radiography (Standards NF P 22-471, EN 1435, EN 444)	(MIKTI, 2010)
		C.1.3.2 Unconventional methods	C.1.3.2.1 TOFD (Time of flight diffraction)	(Yeh, et al., 2018); (Silk & Lidington, 1975); (Ido, et al., 2004); (Chi & Gang, 2013); (Baskaran, et al., 2006); (Subbaratnam, et al., 2011)
			C.1.3.2.2 – Phased Array	(Deutsch & Kierspel, 2012)
			C.1.3.2.3 – ACFM (Alternating current field measurement)	(LeTessier, et al., 2002)
			C.1.3.2.4 – EMAT (Electro-magnetic-acoustic transducers)	(Isla & Cegla, 2017); (Jian, et al., 2006); (Shujuan, et al., 2010); (Noorian & Sadr, 2010); (Aliouane, et al., 2000)
			C.1.3.2.5 Towards EMAT Phased Array?	(Isla & Cegla, 2017)
			C.1.3.2.6 Acoustic emission	(Kosnik, et al., 2011); (Ranganayakulu, et al., 2014); (Droubi, et al., 2017)
			C.1.3.2.7 – Infrared thermography	(Hung, et al., 2009); (Ummenhofer & Medgenberg, 2009); (Huss, 1994)
			C.1.3.2.8 – Guided waves	(Sargent, 2006); (Fan & Lowe, 2012); (Pahlavan & Blacquière, 2016)
			C.1.3.2.9 – Magnetic barkhausen noise (MBN) technique	(Vourna, et al., 2015); (Yelbay, et al., 2010); (Ju, et al., 2003)
			C.1.3.2.10 – Shearography	(Hung, et al., 2009)
			C.1.3.2.11 – Use of low-frequency magnetoresistive sensors	(Tsukada, et al., 2006); (Kosmas, et al., 2005); (Jenks, et al., 1997); (Tsukada, et al., 2010); (Tsukada, et al., 2011); (Tsukada, et al., 2013)

	C 1.4 Non-destructive testing for masonry	C.1.4.1 Radar technique	(Lai, et al., 2018); (Dérobert, et al., 2019)
		C.1.4.2 Step-frequency radar	(Lambot & Andre, 2014); (Guan, et al., 2017)
		C.1.4.3 Ultrasonics: tomography and impulse echo applications	-
		C.1.4.4 Photogrammetry	(Napolitano & Glisic, 2019); (Mandelli, et al., 2017); (Malihi, et al., 2018), (Sapirstein, 2014); (Patias & Santana Quintero, 2009); (Percy, et al., 2015); (Shrestha, et al., 2017)
		C.1.4.5 Use of distributed strain and acoustic emission sensors	(Verstrynge, et al., 2018)
		C.1.4.6 Thermography	(Meola, 2007); (Khan, et al., 2014); (Khan, et al., 2015a); (Khan, et al., 2015b)
	C.1.5. NDT for bridge tower bolts		
C.2 Innovation in structural health monitoring (SHM)	C.2.1 High resolution photography		C.2.1 High resolution photography
	C.2.2 Laser scanning for bridge inspection		(Truong-Hong Linh & Laefer. 2014); (Riveiro et al. 2013); (Laefer 2013); (Truong-Hong and Laefer, 2019); (Walsh et al. 2013); (Lichti et al. 2002); (Zogg & Ingensand 2008); (Lovas et al. 2008); (Riveiro et al. 2013); (Liu et al. 2012); (Truong-Hong & Laefer, 2015a); (Truong-Hong and Lindenbergh, 2019); (Truong-Hong and Laefer., 2015); (Truong-Hong et al., 2016)
	C.2.3 Distributed optic fiber sensors (DOFS)		(Barrias, Casas, and Villalba 2016); (Barrias et al. 2018a); (Samiec 2012); (Grave et al 2015); (Villalba and Casas 2012); (Rodríguez et al.2015); (Barrias et al. 2018b,c); (Barrias et al. 2019); (Barrias et al. 2016)
	C.2.4 Concrete-embedded optic fiber		(De Mengin, et al., 2017)
	C.2.5 GNSS solutions	C.2.5.2 Use of network of GNSS stations using RTK processing to monitor bridge dynamic responses	(Yu, et al., 2016)
		C.2.5.3 Use of network of low-cost single frequency GNSS stations	(Yu, et al., 2016)
	C.2.6 Monitoring dynamic displacement using ground-based radar interferometry (GB-SAR) 86		(Zhang, et al., 2018); (Monserrat, et al., 2014)
	C.2.7 Video camera-based vibration measurements		(Mas, et al., 2014); (Zhang, et al., 2016); (Chen, et al., 2016)
	C.2.8 Digitized measurement of the cracking index		(Fasseu & Michel, 1997); (Moliard, et al., 2016)
	C.2.9 Relative deformation monitoring with Persistent Scatterer Interferometric space-born Synthetic Aperature Radar (PSInSAR)		(Ferretti, et al., 2001); (Crosetto, et al., 2016); (Ferretti, et al., 2005)
	C.2.10 Use of Unmanned Aerial Systems (drones)	C.2.10.1 The European project AEROBI	-
		C.2.10.2 Use of UAVs for visual inspection of building (cracks, genereal state) with photography	(Khaloo, et al., 2018)
	C.2.11 Automated processing dedicated to analyze large datasets	C.2.11.1 STRUM crack detection	(Prasanna, et al., 2016)
		C.2.11.2 Optical Digital Image Correlation for strain and displacement monitoring	(Nonis, et al., 2013)
	C.2.12 Wire break detection by acoustic monitoring		-
	C.2.13 Scour monitoring systems	C.2.13.1 Monitoring of dynamic behavior of pile	(Prendergast, et al., 2013); (Zarafshan, et al., 2012)
		C.2.13.2 Developement of a scouring sensor	(Zarafshan, et al., 2012); (Boujia, et al., 2018)
	C.2.14 Weigh in motion systems (WIM)	C.2.14.1 Updating remaining fatigue service life with WIM information	(ASTM, 2009); (Schmidt & Jacob, 2010); (Al-Qadi, et al., 2016)
		C.2.14.2 The use of fibre optical strands for WIM applications	(Tinawi, et al., 2017)

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A. INTRODUCTION

A.1. MOTIVATIONS

The transport system represents a fundamental factor for the economic and social development, as it allows the quick, safe and easy exchange of passengers and freight. For the most part, this mobility is sustained by the network of roads and highways providing high level of service and flexibility (Commission, 2013), (ASCE, 2013), (Hooks & Frangopol, 2013). Road bridges are key elements in terms of safety and functionality for the whole infrastructure. The ageing and deterioration of these components exacerbated by increasing traffic intensities ad loads on one hand and from extreme events brought by climate change from the other hand, make them the bottlenecks of the transportation infrastructure.

The decision to replace or repair, when and how to repair each individual structure, is a common and difficult management issue for asset managers. For structural assessment, various types of sensor information are used to generate data related to the health condition and load carrying capacity. Data can be collected on all types of structures in different ways, but the challenge is to translate it into information, which is necessary for decision-making.

Many bridge managers across Western countries are experiencing similar issues when dealing with preservation of transportation infrastructure networks: (i) they must manage ageing bridges generally exposed to operating conditions that are significantly different from those at the design stage. More specifically, such changes are due to traffic growth (in both intensity and frequency); (ii) most of these bridge managers also face significant budgetary cuts. As a result, results are not anymore adequate - nor in terms of technical staff and in economic terms - for managing and maintaining bridges in an efficient way. In some countries, state decentralization processes have sometimes transferred infrastructure management to non-technical authorities unable of addressing such technical responsibilities; (iii) besides, expectations of citizens and decision makers are continuously rising. They are requesting more transparency and more safety, less socio-economic impact and more consideration of environmental issues; (iv) operating and management responsibilities are becoming increasingly complex. Bridge managers, who essentially have technical backgrounds, have now to learn how to integrate more broadly all the expectations and issues of their surrounding environments, yet most of which do not originally fall under civil engineering area of expertise; (v) the profession is evolving and available tools for bridge management are no longer sufficient but, at the same time, many recent events across Europe demonstrate how important and significant the role of bridge managers can be.

To maintain a high quality of service, there is a significant need for tools that allow road administrations to better manage their infrastructure stock. Specifically, these tools have to enable condition assessment of the infrastructure asset and then help make decisions policies (PIARC Technical Committee C4.1 Management of Road Infrastructure Assets, 2008), (Orcesi, et al., 2016). The inspection/monitoring data have then to be converted into information and knowledge to be translated into actionable data for better appreciating the bridge performance and deterioration. It is then essential to monitor the bridge component behaviour and detect any drop in the performance level before major failure occurs. Also, projecting the asset network condition over the planning horizon serves to identify future funding needs. Reliable "performance indicators" (synthetic and applicable) are therefore needed for the decision-making. The objective of the bridge Managers is to deliver clear information to end-users concerning performance, deterioration and future performance of the selected assets.

The work in the innovation subgroup was started by assessing the needs of stakeholders. Three questions were sent to some stakeholders (see Table 3, Table 4, Table 5), to have their opinion, as on the needs they estimate as the most important when dealing with inspections, testing and monitoring of bridges.

Table 3. Answers to question Q1

Q1. Which types of bridges (materials and design) are the most critical ones ?	
Cofiroute, France (1)	- Prestressed structures - Steel culverts - Bridges with pile foundations in river
Bridge Maintenance Department of Iran, Iran (2)	It is a fact that scour and flood (Bridge Hydraulics) can be regarded as two main causes of bridge failures in the world. On the basis of these two main causes, no major differences exist for steel or concrete materials for superstructures except that reinforced concrete bridges are heavier which make them more vulnerable. Multi span bridges located over seasonal rivers or water ways which have single foots (no pile foundations) can be regarded as the first priority. In this case, medium or short span RC bridges more frequent. Considering the other causes of bridge failures (wind, Earthquake, Overload, and ...), the type of materials and designs are likely more effective.
Rijkswaterstaat Major Projects and Maintenance, The Netherlands (3)	Steel bridges suffering from fatigue and concrete bridges suffering from corrosion, Prestressed steel reinforced concrete girders, Orthotropic steel decks and main steel girders with a span > 30 m, Cables of cable-stayed or suspension bridges.
Direction of Asset Management, Infraestruturas de Portugal (4)	Steel culverts, construction types with hidden elements (such as cables anchorage, earth reinforced abutments), old steel bridges (subject to fatigue), Gerber systems, bridges and culverts subject to scour, cable-stayed bridges, external post-tensioning systems, brick masonry arches
Croatian Roads Ltd., Croatia(5)	Suspended and cable-stayed bridges, reinforced and prestressed continuous beam bridges, prestressed shallow arch bridges, older masonry (brick, stone) arch bridges.

Gebze İzmir İşletme ve Bakım (GİİB), Turkey [6]	Steel suspension bridges are the most critical ones since their structure is very flexible and subject to various forces such as earthquake, wind, vibration due to traffic. Steel is a strong bridge material but sensitive to corrosion and overheat
Directorate for State Roads of Montenegro, Montenrgro [7]	The most critical are reinforced-concrete bridges of arch and beam systems and their combination
ANAS Gruppo FS Italiane, Italy [8]	The experience resulting from the analysis of several cases along the roads of the Lombardy region indicates that the most critical ones are the prestressed concrete bridge
The Department of Transport, Tourism and Sport, Ireland [9]	80 % of the bridges on the Regional and local roads are masonry arch. The load capacity of the legacy bridges can be of concern
ASFINAG, Austria [10]	Bridges made of RC with wide cantilever plates, prefabricated constructions

Table 4. Answers to question Q2

Q2. Which failures (structural or equipment failures) are the most feared?	
Cofiroute, France [1]	- Scouring - Failure of a prestressing cable - Collapse of a steel culvert due to corrosion
Bridge Maintenance Department of Iran, Iran [2]	Structural failures (foots or foundations or retaining walls or reinforced concrete columns) are more prevalent in the case of scour or flood. Bridge unseating is more prevalent in the case of earthquake. Equipment failures are likely more prevalent in long span or cable supported bridges.
Rijkswaterstaat Major Projects and Maintenance [3]	- Corrosion and fatigue of steel bridges, especially in poorly or invisible spots. - Corrosion of steel enforcement of concrete bridges (again when not – yet visible)
Direction of Asset Management, Infraestruturas de Portugal [4]	Fatigue, tensioned cables, undermining and settlement of foundations, corrosion in tensioned cables, failure of hidden elements.
Croatian Roads Ltd., Croatia[5]	- Abrasion by swirling of water (under level of water), - Sudden excessive movements, - Hidden steel corrosion (pre-strained cables, inaccessible places of elements) that results in the cancellation of support, - Cancellation of support of elements due to frequent stress changes (strong winds, dynamic impact due to passing vehicles etc.).
Gebze İzmir İşletme ve Bakım (GİİB), Turkey[6]	- Strength loss in main cable : Overheating of main cable due to liquid pool fire on road lanes, fire in passing ships, explosion due to sobatage, collision by an aircraft, corrosion of wires - Strength loss in anchorage tendons: corrosion of tendons - Hanger rupture: Collosion of vehicles, collision by an aircraft, fire of tanker, fire in ships under bridge, overload - Damage in tower foundations: due to ship impact; earthquake - Damage in steel deck, steel tower : fatique cracks, cracks due to overload, corrosion, collision by an aircraft, explosion.
Directorate for State Roads of Montenegro, Montenrgro[7]	- The most feared comes from failures in the road plate, dilatation joints, scouring and waterproofing.
ANAS Gruppo FS Italiane, Italy[8]	- The most feared failures of the prestressed concrete bridge are the cable damages deriving from inaccuracy during the construction phases (mostly about inadequate cable injections).
The Department of Transport, Tourism and Sport, Ireland[9]	- Scour would be the primary concern, as it can remain unseen after visual inspections. Reinforcement corrosion within the concrete cover of concrete bridges and support piers
ASFINAG, Austria[10]	Equipment: - Expansion joints (Finger-construction) - Structure: - Defect in Gerber joint Local collapse in thin slab deck or T-beam

Table 5. Answers to question Q2

Q3. Which priorities would you like to set up concerning research topics when related to inspection,testing and monitoring of bridges?	
Cofiroute, France [1]	- Reliable non destructive testing of prestressed cables to detect corrosion progress. Highly efficient non destructive testing of waterproofing layers; - Predictive models for scouring, corrosion in reinforced concrete and prestressed concrete structures, development of low speed degradation processes(ESR and AGR).
Bridge Maintenance Department of Iran, Iran [2]	Inspection,testing and monitoring of bridges for scour and flood failures.
Rijkswaterstaat Major Projects and Maintenance [3]	- Remote sensoring as early warning system for failure of bridges (f.i. measurement of deformations by satellite radar imageing); - Detection techniques for fatique and reinforcement corrosion before these degradation can be descerned by visual inspection; - The same for critical locations where visual inspection is not possible or difficult; - Affordable monitoring techniques which can be used on large bridges (300 m or longer) for permanent monitoring of the bridge condition (e.q. crack detection, corrosion, or chloride penetration); - Improvement of calculation methods to extrapolate local monitoring of e.g. cracks to other areas of the bridge and to the design strength; - Traffic management systems (e.g. smart mobility) to protect bridges against overload as a result of too many (heavy) trucks on the same time on the bridge.
Direction of Asset Management, Infraestruturas de Portugal [4]	Expedite methods to complement visual inspections, dedicated to the topics in questions Q1 and Q2. Dynamic monitoring of vibration modes. Monitoring dedicated to degradation mechanisms in vulnerable zones.
Croatian Roads Ltd., Croatia[5]	- Development of engineers (inspectors) tasks for bridges: how to see, record and identify processes which are essential for assessing the situation, and indications or manifestations that lead to damage - Development of inspection methods on structures that contain sophisticated engineering solutions such on suspended and cable-stayed bridges - Recommendations to bridge designers about design of special devices for inspection (special platforms, elevators, etc) - Development of a tool for monitoring state of bridges elements and bridges generally (methodology, manuals, patterns, catalogs of defects etc.)
Gebze İzmir İşletme ve Bakım (GİİB), Turkey[6]	- Inspection of cables by a camera mounted on mobile equipment and remotely controlled by inspection team. - Inspection of tower and deck welds easily
Directorate for State Roads of Montenegro, Montenrgro[7]	Research priorities should relate to regular monitoring of the current state of bridges and removal of their deficiencies on time.
ANAS Gruppo FS Italiane, Italy[8]	Very interesting could be the real time bridge surveillance through advanced control devices that transmit data of displacements and loads under traffic
The Department of Transport, Tourism and Sport, Ireland[9]	We are in the process setting up a national data base of Regional and local road bridges. As part of the data base we will ask the surveyors to give the bridge an overall visual estimated rating. I expect from this data base we will in future be able to answer this question in more refined detail
ASFINAG, Austria[10]	- Reliable and accurate chloride profiles - Energy self-sufficient Monitoring for measuring horizontal deflection and deflections due to temperature

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Distribution of the most critical types of bridges and the most feared failures according to bridge owners who participated to this survey are summarized in Figure 1 and Figure 2. Crossing the innovation levels with the needs of stakeholders might help make some propositions of research strategies in the short and medium term.

The work of COST action TU1406 partly addresses this problematic (Matos, et al., 2016) with an approach based on the sharing of experiences at an international level in order to foster synergies to meet up these new challenges.

One effective answer to both managers' and citizens' expectations, who aspire to more reliable and safe infrastructure networks, dwells in Innovation that, in COST Action TU1406 is tackled by the Innovation Subgroup.

THE MOST CRITICAL TYPES OF BRIDGES

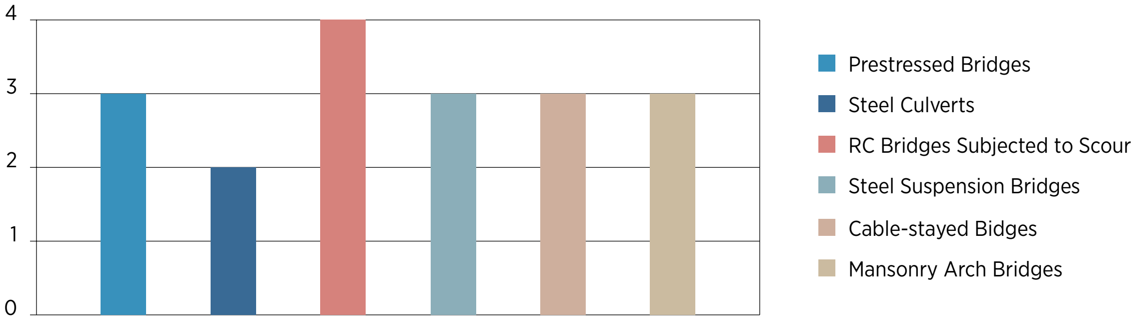

Figure 1. The most critical types of bridges based on bridge owners' feedback

THE MOST FEARED FAILURES

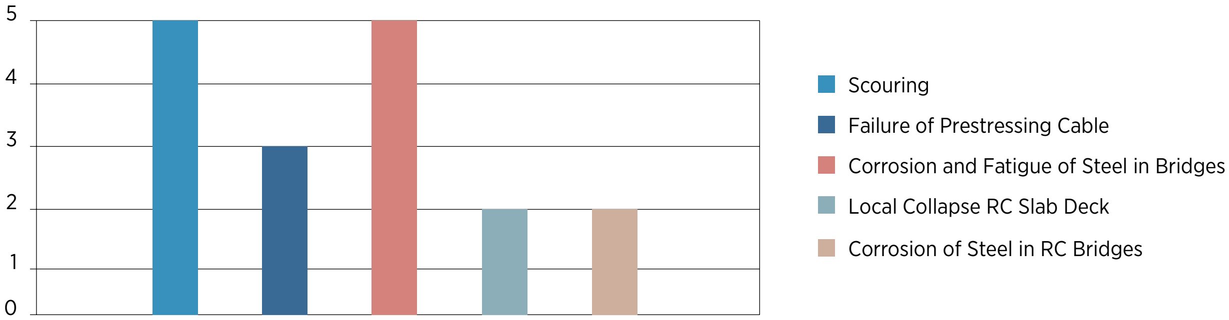

Figure 2. The most feared failures based on bridge owners' feedback

A.2. SCOPE OF THE INNOVATION SUBGROUP REPORT

The work in the Innovation Subgroup was focused on innovative indicators (Limongelli & Orcesi, 2017a) and Technological Innovations that can improve accuracy and precision of quality controls of roadway bridges, thus achieving higher performances in terms of safety and availability.

The goal was to provide owners with new tools to consider all the aspects: technical, social, economic, environmental, that influence the performance of the system and characterize the impact of a structure on the entire system.

The work of COST action TU1406 aims at addressing this problematic (Matos, et al., 2016) with an approach based on the sharing of experiences at an international level in order to foster synergies to meet up these new challenges. This is one of the reasons why the COST ACTION TU1406 management board invited a dedicated committee of advisers with bridge management expertise. This team consists of public and private bridges managers, consultants, experts from research institutes and engineering offices. Its role is to ensure the practical and operational dimension of COST TU1406 work.

A.2.1. RPIS: COLLECTION OF DATA AND RATING OF THE INDICATORS

Several Performance Indicators are still the object of scientific research and possess different level of maturity in terms of their application. They will be denoted herein as Research Performance Indicators (RPIs). Some RPIs have been applied only to simulated numerical cases, other to experimental cases on scaled models, other, more mature, to prototype real structures up to the most mature employed routinely for quality checks. This last level is the one of the Operational Performance Indicators.

A survey was carried out in the COST TU1406 network to gather RPIs from researchers within and in connection with the network. The input from 36 countries in Europe was collected - through the country representative persons - and a rating scale, called Indicator Readiness Level (IRL), was introduced by the innovation Subgroup (Limongelli & Orcesi, 2017a) to assign a level of maturity to each indicator.

The aim of the rating according to the IRL is twofold:
- to check the eligibility of the considered performance indicators for quality checks;
- to underpin the indicators on which more research is needed to reach the level of full applicability for quality checks. This may support the understanding of which improvements of best practices - in terms of performance indicators – are feasible.

Each country representative person performed a survey in his/her country gathering publications describing RPIs. Each indicator was rated, according to the IRL, by the researcher that proposed its inclusion in the database.

Later, all the collected and rated RPIs were clustered in categories according to the Key Performance Indicators considered in the Action (Reliability, Serviceability, Economy, Cost) and also other innovative KPIs (Social aspects, Resilience) that emerged from the database of RPIs.

Further to the outcome of this survey, a group of experts inside the Action, volunteered to review and integrate the list of RPIs and to verify, based on their expertise, the indicators rating according to the IRL. This report contains the results of this survey and review process.

It is important to note that the rating reported in Chapter B has the scope to describe the current state of Research on Performance Indicators **based on the knowledge related to the COST TU1406 network**. Due to the nature of the RPIs - that are the outcomes of research investigations - and due to way the database was gathered -based on the connections within and with the COST TU1406 and on the some expert opinion within the Action – the database and, as a consequence, the rating of the RPIs, have to be considered dynamic processes that **need to be updated constantly based on new, or previously unknown, research outcomes.**

Therefore the scope of the work herein reported of COST TU1406 is twofold:
1) Present the current RPIs database collected within the Action;
2) Provide a tool (the IRL) to rate the RPIs

It is worth to mention that, even if step 1 needs to be constantly updated, the maturity level assigned herein by the **IRL constitute a lower bound of the rating** of the indicator: the requirement to prove through published material the use of the indicator at a certain level, ensures that the Indicator reaches '**at least**' that maturity level. Further knowledge may of course increase the rate.

The COST Action TU1406 database and the IRL rating of the RPIs will be provided in excel format as a deliverable and will constitute a base for further integrations and updates (to check with the core group).

A.2.2. INNOVATIVE PERFORMANCE TECHNOLOGIES

In parallel of the analysis on performance indicators, a study was initiated with some academic and industrial partners to identify some novel condition monitoring and sensing technologies for the assessment of structural serviceability and safety of existing structures. Advanced, integrated, cost-effective and reliable instrumentation solutions, techniques and concepts that can be used to compute innovative performance indicators have been considered and a TRL has been provided each time it was assessed by contributors.

The aim of this study was to start a think tank on innovative technologies as an answer to the current needs of bridge owners. It is noted that (i) in its current form, the innovations on technologies presented in Chapter C cannot be considered as exhaustive and this work should more serve as a starting point for future discussions and research activities, (ii) the maturity level rating of these technologies (qualitatively or quantitatively using TRL) has to be considered as a dynamic processes that need to be updated constantly based on new, or previously unknown, research outcomes.

A.3. FROM PRESCRIPTIVE TO PERFORMANCE-BASED BRIDGE MAINTENANCE

A maintenance concept can be defined as a set of economically balanced activities on the level of individual components aiming to maintain the structure in the condition allowing it to fulfil its functions during a fixed period of time with a sufficient level of reliability, availability, serviceability, durability (Highways Agency (HA), Transportation Research Lab (TRL), Services d'Etudes Techniques des Routes et Autoroutes (SETRA), Laboratorie Central des Ponts et Chaussees (LCPC), 2005).

Two main categories of maintenance strategies can be applied to bridge components. These are corrective and preventive. A corrective maintenance implies that no action is undertaken until damage is noticed. In practice this kind of maintenance is interesting if the consequences of failure are not important and if to avoid the failure of the component is expensive.

Within the framework of preventive maintenance actions are regularly carried out. This kind of maintenance can lead to high costs but reduces the disturbance risks. A preventive maintenance can be systematic or conditional. When a systematic maintenance is applied, repair or replacement planning is to be optimized in order to minimize the global costs of the maintenance during the conventional lifetime of the structure.

A conditional maintenance involves the component to be inspected within a given inspection planning. Following inspection results repairs can or cannot occur. A conditional maintenance is pertinent if failure costs (Highways Agency (HA), Transportation Research Lab (TRL), Services d'Etudes Techniques des Routes et Autoroutes (SETRA), Laboratorie Central des Ponts et Chaussees (LCPC), 2005).

To move from corrective maintenance to preventive maintenance, it is essential to be able to monitor the bridge component behaviour and detect any drop in the performance level before major failure occurs. It is therefore essential to have data and reliable indicators on performance and condition, and to be able to predict the evolution of this performance for the decision-making.

Performance models are used to predict future scenarios for the asset network. Projecting the asset network condition over the planning horizon serves to identify future funding needs. Appropriate selection of performance models is essential to effective asset management. The objective is to deliver clear information to end-users concerning performance, deterioration and future performance of the selected assets.

A.4. FROM OBSERVATIONS TO KEY PERFORMANCE INDICATORS

Surveys carried out on the bridge through visual inspections, non-destructive testing, structural health monitoring, provide **Observations** in the form of perceptions of human senses (e.g. existence of spalling, scour, etc.) or measures (e.g. chloride /carbonation depth acceleration, displacement, loads, corrosion rate).

Observations themselves are normally not directly used for quality control i.e. assessment of the fitness of purpose and decision making with regard to future interventions. They are used to derive **Performance Indicators** (PIs) related to a current or future situation that needs to be avoided and that can be used to measure the 'fitness for purpose' of the system.

In some cases, the PIs coincide with the observations (for example chloride content, crack length, sag,…) in other cases they are computed from observations using heuristic rules (for example condition score computed from observations collected during visual inspections) or a model (as is the case of the probability of failure computed from a numerical model calibrated using modal parameters derived from measured accelerations).

The Performance Indicators can be used for diagnostic purposes - if they are employed with their current values - or for prognostic purposes - if their future value is forecasted using their current values and a **forecast model**.

The PIs at some time instance (current or future) are used to determine to which extent **Performance Goals** are achieved. In this manner the fitness of purpose of a bridge over time can be estimated. Depending on the considered problem and system (e.g. only the bridge or the bridge and the environment, or the bridge, the environment and the society,…) some performance goals are deemed more important than others when performing the Quality Control of a bridge. Some of these are Reliability, Availability, Security, Economy and Environment, other are still at research level such as those related to Sustainability or Resilience.

The **Key Performance Indicators** (KPIs) are Performance Indicators relevant to the Performance Goals that are considered as the most relevant and important for the problem at hand. The KPIs are used – as single values or combined using some criteria - for the quality control.

In the following are presented Innovative Performance Indicators related to Durability (reinforcement corrosion), Reliability, Cost, Climate Change, Seismic Performance, Scour, Social and Economic factors, Resilience. The list is not exhaustive but reflects the results of the survey carried out in the COST Action TU1406 among 36 European countries.

A.5. MATURITY LEVEL OF RESEARCH PERFORMANCE INDICATORS AND TECHNOLOGIES

The classification of research-based Performance Indicators based on their **maturity level** proposed herein takes basis on the scale of Technology Readiness Level (TRL) that was originally developed to classify new technologies in terms of their maturity level. A brief description of the TRL is reported to introduce the concepts that have been used in the definition of the Indicator Readiness Level (IRL).

A.5.1. THE TECHNOLOGY READINESS LEVEL (TRL)

The Technology Readiness Level (TRL) is a scale that was developed in the 70s by the NASA (National Aeronautics and Space Administration) to assess the stage of development (maturity) of new technologies and to compare different technologies in terms of the maturity level from idea to application (EARTO, 2014). Around 2005 its use became widespread in the international space development community and in several other fields, with some adaptations to suit the different needs.

The original drive for the TRL development was «communication and planning» aimed to synchronize the development of the individual technologies needed for the development of one high-tech technological systems. Development of individual technologies should be carefully planned and synchronized, to avoid losses in terms of both time and investments. The need to align the maturity levels of different individual components brought to the need of a common scale that can be used as a tool for decision making about the investments on the individual technologies.

In the last years, the use of TRL spread to other fields and the scale was adapted by changing the number of levels and/or grouping levels (Table 6).

For example, at EU level the TRL is used for decision making on research and development investments that is to classify research projects in order to assess their eligibility to access funding. In this case, there is a further goal (beside communication and planning) which is the assessment of the eligibility.

The definition of the TRL levels has been updated accordingly asking to research project with mid/high TRL to provide a business plan for future investments. In Table 6 are reported the nine levels currently defined for the TRL according to the H2020. The term «relevant» referred to the environment indicates a simulated environment representative of the «operational» environment that is the one where the technology will operate in real world.

This last use of the TRL suggested that a similar scale could be used to assess the eligibility of Performance Indicators for quality checks and related decision-making on real asset. This scale has been herein defined as IRL and will be detailed in the next section.

Table 6. Technology Readiness Level (TRL)

	Definition according to H2020 work programme 2014-2015. General annexes (European Commission, 2014)	**Description according to EU network for space** (COSMOS Space | NCP Network, n.d.)
TRL1	Basic principles observed	Principles postulated and observed but no experimental proof available.
TRL2	Technology **concept formulated**	Concept and application have been formulated. Examples are limited to analytic studies.
TRL3	Experimental **proof of concept**	First laboratory tests completed to validate analytical predictions of separate elements of the technology
TRL4	Technology **validated in lab**	Basic technological components are integrated and a small-scale prototype is tested in a laboratory environment ("ugly" prototype).
TRL5	Technology **validated** in relevant environment	A large-scale prototype is tested in relevant environment.
TRL6	Technology **demonstrated** in relevant environment	Prototype system tested in relevant environment to demonstrate operations under critical environmental conditions.
TRL7	System **prototype demonstration in operational environment**	Prototype tested in operational environment.
TRL8	System **complete and qualified**	Product in its final configuration. Manufacturing issues solved. Evaluation of the system to determine if it meets design specifications.
TRL9	**Actual system** proven in **operational environment**	Full commercial application, technology available for consumers.

A.5.2. THE INDICATOR READINESS LEVEL (IRL)

The IRL, proposed by the Innovation Subgroup of COST Action TU1406, defines the level of maturity of an indicator based on the condition in which its successful utilization has been already proved through published scientific results or successful real-world applications (Limongelli, et al., 2018).

This IRL scale takes basis on the scale of Technology Readiness Level (TRL) that was proposed in the 70s by the NASA (National Aeronautics and Space Administration) (EARTO, 2014) (European Commission, 2014).

The TRL scale in Table 6 has been adapted to take into account the differences between a technology and an Indicator:

a) Performance Indicators are also used for ranking purposes beyond quality check;
b) the computation of Performance Indicators requires experimental tests for which testing protocols have to be defined;
c) Performance Indicators are not object so a «prototype» cannot be defined

In order to take into account the previous requirements, the definition of the maturity levels of the IRL (Table 7) has been slightly changed with respect to those relevant to the TRL (Table 6). The interested reader can refer to references (Limongelli & Orcesi, 2017a) (Limongelli, et al., 2018) for more details about the IRL.

The lowest level, IRL=1, corresponds to the simple formulation of the concept. For example, modal frequency decreases with stiffness therefore its variations can be used as indications of stiffness losses.

Table 7. Indicator Readiness Level (IRL)

IRL1	Basic principles observed	The **principles underlying** the parameter are known
IRL2	Indicator concept formulated	The indicator is applied in **analytical studies**
IRL3	Experimental proof of concept	**Analytical and experimental studies** (indoor) performed on a **laboratory scale on specimens** to validate analytical predictions.
IRL4	Indicator validated in laboratory	**Experimental studies** are performed in **laboratory** on a **reduced scale model** of the structure/structural element to produce a database for estimation of the indicator.
IRL5	Indicator validated in laboratory in simulated environment	**Experimental studies** performed in **controlled laboratory (or outdoor)** on **reduced scaled model** of the structure/structural element reproducing real environmental conditions to produce a database for estimation of the indicator.

IRL6	Indicator demonstrated in relevant environment	**Experimental studies** performed **in controlled laboratory (or outdoor)** on a **full-scale model** of the structure/structural element reproducing real environmental conditions to produce a database for estimation of the indicator.
IRL7	Indicator demonstrated in operational environment	**Experimental studies** performed on a **real structure/structural element** and/or application of the indicator for ranking purposes and related decision-making. Applicability issues still exist
IRL8	System complete and qualified	Indicator is standardized and can be used for **quality control check** purposes and related decision-making. **Applicability issues are solved.**
IRL9	Actual system proven in operational environment	The **indicator is systematically applied** for the **quality check** of a structure/asset and related decision making

The lowest level, IRL=1, corresponds to the simple formulation of the concept. For example, modal frequency decreases with stiffness therefore its variations can be used as indications of stiffness losses.

Indicators that are systematically used for quality check reach the highest level IRL=9.

The intermediate levels correspond to indicators used in simulated or controlled conditions (e.g. laboratory tests of on scaled models) that do not account for all the real conditions (e.g. influence of environmental or operational conditions on the indicators, problems related to high computational requirements, etc).

A given IRL considers that all levels above are also satisfied. For example, IRL=7 means that the indicator has already been successfully utilized at levels 1 to 7.

The IRL scale is meant to serve as a supporting tool for a twofold aim:

1) To check the eligibility of a Performance Indicator for quality checks and related decision making based on its maturity.
2) To select research needs on Performance Indicators, that is to underpin the Indicators on which more research is still needed in order to bring them to the level of full applicability for quality checks.

In chapters 3 to 9 are reported the Performance Indicators that are currently being investigated by researchers.

The achievement of a certain level is documented by publication of results in technical reports or scientific papers.

B. INNOVATION ON INDICATORS

The Indicator Readiness Level (IRL) scale introduced Chapter A is considered thereafter to rate the maturity level of several performance indicators which are currently or which have recently been the object of research effort. These Indicators can be qualitative or quantitative based, data-driven or obtained with physically-based models.

They can be collected during principal inspections, through a visual examination, a non-destructive test or a temporary or permanent monitoring system. Moreover, they can describe the current condition (diagnostic performance indicators) and/or forecast future conditions (prognostic performance indicators).

B.1. RESEARCH PERFORMANCE INDICATORS RELATED TO RELIABILITY

B.1.1. DURABILITY RPIS

Contributors: Anja Vidovic, Alfred Strauss and Carmen Andrade

When considering reinforcement corrosion in concrete, service life of concrete structures can be divided in initiation and propagation phase. During the first phase, corrosion may be induced by chloride ingress or carbonation process. If the structure is exposed to aggressive environment, such are those structures that are located close to the seawater or exposed to de-icing salts, chlorides penetrate into concrete and start inducing corrosion. Alternatively, during the carbonation process, carbon dioxide penetrates the surface of the concrete and reacts with alkaline components in the cement paste, for the result of reduction of pH value.

The initiation phase is therefore characterized by these two processes, until the depassivation limit state is reached. When surface film of ferric oxide on the reinforcement is broken or depassivated, propagation phase starts. Steel corrosion itself is an electrochemical process that includes dissolution of iron and formation of corrosion products, such as rust. In order to capture the corrosion progress, corrosion rate is investigated, that is in addition referring to corrosion current density and electrical potential.

Altogether, important performance parameters for corrosion process are undoubtedly chloride content at the reinforcing steel (Section B.1.1.1), carbonation depth (Section B.1.1.2) and corrosion rate (Section B.1.1.3). They are measurable parameters that have well defined testing procedures and critical limits. As the change in these parameters, from the initial healthy state to the current one, indicates the level of deterioration, they are considered as diagnostic performance indicators at the element level, and when the prediction models are applied to same parameters, prognostic performance indicators are subsequently defined.

B.1.1.1. CHLORIDE CONTENT AT THE REINFORCING STEEL

Chloride content is the total amount of chloride ion in concrete, including bound in the solid phases and free chlorides in the pore solution, where the corresponding performance indicator is the difference between the total amount of chloride in concrete at the current and reference state. It is possible to estimate the change of the chloride content in concrete by various developed mathematical models, such as those from reference (fib, 2006).

Determination of chloride content in hardened concrete is a well-established procedure, prescribed in EN 14629 (European Committee for Standardization, 2007). The chloride threshold is not a fixed value because it depends on variables such as temperature, type and amount of cement, w/c ratio, etc. However, in the Codes or Guidelines some fixed values are specified from a conservative position.

The most frequent threshold, i.e. critical chloride concentration, used is equal to 0.4% by weight of cement (see also maximum chloride contents provided in EN 206 (European Committee for Standardization, 2005)), while the threshold for concentration of only free chlorides is following ACI of 0,015% in relation to cement content while some authors suggest of 7 kg per m3 of pore solution (Ozbolt, et al., 2010).

RILEM has delivered Recommendations for the determination of total and water soluble chlorides (Castellote & Andrade, 2002). With these developed testing procedures and defined performance goals, there are no issues in applying the indicator in quality checks and related decision-making. Experimental studies have been performed in laboratory, scaled models and on real bridge/elements of a bridge (Castellote & Andrade, 2006) (Kuster Maric, 2013), (Kuster Maric, et al., 2017), (Vidovic, 2018). Hence, the indicator is rated IRL=9.

During the bridge inspections, investigation of chloride concentration is indeed regularly performed test, reaching therefore the highest level of maturity. However, there are two manners of obtaining such values: by drilling cores and performing a chloride profile (British Standards Institution, 2010) (ASTM, 2004) and by taken small pieces of concrete in some locations or at the rebar level (Vennesland, et al., 2013) (Markeset, 2009). The assessment of the chloride-induced corrosion is essential to prediction of the service life of the structure and its limit state, as the increase of chloride concentration near rebar may lead to corrosion, decrease of the bridge load carrying capacity and consequently to decrease of its service life.

Chloride concentration can then be applied as a measurable input parameter to the prognostic performance indicator: the prediction model for remaining service life. The relationship between the chloride content, concrete parameters and service life can be obtained according to references (Tuutti, 1982) (Andrade, et al., 2013) (fib, 2006), (Vidovic, 2018) and (Gode & Paeglitis, 2014). Consultant companies have already used results for updating bridge service life estimation, but this indicator is not yet systematically applied in quality control checks and related decision-making due to the applicability issues, reaching therefore **IRL=7**.

Moreover and in spite of the numerous uncertainties around the diffusion model used, researchers have developed numerical models for prediction of depassivation time of reinforcement by modelling chloride ingress through concrete (Meijers, et al., 2005) and (Thomas & Bentz, 2001). More complex are 2D and 3D ones. Again, chloride content is used as an input for the prognostic indicator: 2D and 3D numerical prediction model for the remaining service life calculation has been developed where the chloride concentration is calculated using 3D chemo-hygro-thermo mechanical numerical model (3D CHTM model). The model is adapted to simulate the transport of chlorides in concrete, in both real and laboratory environment (Ozbolt, et al., 2010) (Andrade & Tavares, 2012).

Since it is possible to determine the chloride content and calibrate the prediction model in an existing structure, this indicator is also found at the maturity level IRL=7. When the complexity issues such as the computational time of the model would be solved, 3D CHTM numerical model could be used in QC checks and related decision-making, due to the good agreement between the measured and numerical results.

B.1.1.2. CARBONATION DEPTH

Carbonation depth is the layer of the concrete that is carbonated at its surface. The carbonation involves a decrease of pH in the pore solution which leads into the steel depassivation, i.e. carbonation-induced corrosion. Therefore, it corresponds to the diagnostic performance indicator, whose corresponding performance goal is not reaching the rebar front (concrete cover).

Determination of carbonation depth is commonly carried out by the phenolphthalein method (CEN/TS, 2007) (European Committee for Standardization, 2006) and the procedure can be performed on the specimens in the lab to those extracted from the real structure. Alternatively, due to the slow evolution of carbonation depth in normal environment, an accelerated carbonation chamber may be used in laboratories (prCEN/TS, 2018) (Papadakis, et al., 1991). As the indicator is already systematically used in QC checks and related decision-making it is found already at the operational level (**IRL=9**).

In the same manner as for the chloride content, if the models for the prediction of remaining service life are applied to carbonation process, the carbonation depth is then function of measure, and the prognostic model is rated **IRL=7**.

B.1.1.3. CORROSION RATE

Corrosion rate is another important parameter within the assessment of the prediction of remaining service life of the structure because it is directly linked to the rate of structural deterioration being the reinforcing bars section a key parameter in the structural load-bearing capacity. The accumulated corrosion rate during time gives the loss of steel section.

This loss in the bearing-sections of the steel and concrete (by the cracks generated by the oxide formation) also impacts the steel/concrete bond. Corrosion rate as indicator is referring to corrosion current density (loss of grams per steel section), and influences the electric potential on the rebar surface and is controlled by the humidity level of the concrete (by the concrete resistivity).

Then, the three corrosion parameters that can be measured related to steel deterioration are: electric potential, concrete resistivity and reinforcing corrosion rate/accumulated corrosion. The only quantitative one is the corrosion rate while the potential and resistivity are only qualitative. It is possible to perform experimental studies on a real bridge/component, in order to estimate this indicator by directly measuring the loss in bar diameter due to corrosion. Real case studies are reported in (Kuster Maric, 2013), using half-cell potentials. Determination of the macro-cell electric potential can be carried out according to reference (ASTM Subcommittee G01.14, 2007). The indicator is rated **IRL=9**.

The values of the corrosion rates in real structures will depend on the concrete quality, moisture level and temperature. There are some proposals for their prediction (Andrade, 2014) (Andrade, 2017). Care should be taken with the effect of the temperature along the seasons and then it is necessary to use an "averaged annual value".

Furthermore, although not calibrated, it is possible to predict corrosion rate using previously mentioned **3D CHTM numerical model**, after the depassivation of reinforcement in concrete had occurred (Ozbolt, et al., 2011). Since the corrosion current density and electrical potential could be used in decision-making, by performing the experimental studies on a real bridge/component it reaches level 7. Applicability of the numerical prediction model still exist, constraining therefore the indicator at **IRL=7**.

Based on testing, it appears the possibility to predict the time to first cracking appearance (Andrade, et al., 1993) which can also be modelled (Molina, et al., 1993) (Ozbolt, et al., 2012). The crack width reached in a corrosion depends very much on the reinforcement detailing and then it cannot be generalized although it can be used as an Indicator. That is cracks below 0.3mm are very initial and then this indicator that is very commonly used when assessing the condition can be classified as **IRL=9**.

B.1.1.4. INDICATORS OF EXPOSURE TO CORROSION

Chloride content at the reinforcing steel and carbonation depth may be used as the input for the prediction model of remaining service life of chloride- and carbonation-induced corrosion-endangered structures, as it has previously been introduced in Sections 3.1.1 and 3.1.2. Nonetheless, the proposed prediction model requires the input of many parameters influenced by material properties and environmental conditions, and not commonly accessible or determined during the prediction analysis.

Possible laboratory and field-testing could improve the characterization of the input data and subsequently the prediction of the remaining service life. The following section explores options for the investigation of these parameters during the bridge inspections and their maturity levels. All these are indicators of exposure to corrosion depending on material or environmental conditions that can trigger or increase the deterioration processes.

Carbonation of concrete in time t can be estimated according to reference (fib, 2006) using the following relationship:

$$c_{nom} = x_c(t) = \sqrt{2 \cdot \left[\frac{1-\left(\frac{RH_{real}}{100}\right)^{f_e}}{1-\left(\frac{RH_{ref}}{100}\right)^{f_e}}\right]^{g_e} \cdot \left(\frac{t_c}{7}\right)^{b_c} \cdot \left(k_t \cdot R_{ACC,0}^{-1} + \varepsilon_t\right) \cdot C_s \cdot \sqrt{t}} \cdot \left(\frac{t_0}{t}\right)^{\frac{(p_{SR} \cdot ToW)^{bw}}{2}} \quad (1)$$

where $x_c(t)$ represents carbonation depth at the time t [mm] and other parameters are given in Table 8.

Table 8. Parameters related to the depassivation caused by carbonation process

Parameter	Symbol	Unit	Type of parameter
Concrete cover	c_{nom}	mm	material
Environmental function			
Relative humidity of the carbonated layer	RH_{real}	%	environmental
Reference relative humidity	RH_{ref}	%	constant
Exponents	f_e, g_e	-	constants
Inverse effective carbonation resistance of concrete			
Inverse effective accelerated resistance	$R_{ACC,0}^{-1}$	(mm^2/years)/ (kg/m^3)	material
Regression parameter	k_t	-	related to ACC test
Error term	ε_t	(mm^2/years)/ (kg/m^3)	related to ACC test
Period of curing	t_c	days	environmental/material
Exponent of regression	b_c	-	related to procedure
Carbon dioxide influence			
CO_2-concentration	C_s	kg/m^3	environmental
Weather function			
Probability of the driving rain	p_{sr}	-	environmental
Time of wetness	ToW	-	environmental
Exponent of regression	b_w	-	constant
Time of reference	t_0	days	constant

Chloride penetration to concrete can be estimated according to the following Equation (fib, 2006):

$$C_{crit.} = C(x,t) = C_0 - (C_{s,\Delta x} - C_0) \cdot \left[1 - erf\frac{c_{nom} - \Delta x}{2 \cdot \sqrt{D_{app}(t) \cdot t}}\right] \quad (2)$$

where $C(x,t)$ represents the content of chlorides at a depth x and at time t; and erf stands for statistical (Gaussian) error function. The diffusion coefficient $D_{app}(t)$ may be determined as follows:

$$D_{app}(t) = k_e \cdot D_{RCM,0} \cdot k_t \cdot \left(\frac{t_0}{t}\right)^{\alpha} \quad (3)$$

In the Table 9 is provided a list of the material and environmental parameters that influence Chloride penetration and diffusion coefficient. together with their categorization.

Table 9. Chloride contamination-related model parameters

Parameter	Symbol	Unit	Type of parameter
Chloride penetration related parameters			
Total critical chloride content	Ccrit	Wt.-%/c	environmental/material
Initial chloride content of the concrete	C0	Wt.-%/c	material
Chloride content at a depth Δx	Cs, Δx	Wt.-%/c	environmental/material
Diffusion coefficient			
Environmental transfer variable	ke	-	environmental
Apparent chloride diffusion coefficient	Dapp	m2/s	environmental/material
Time of reference	t0	days	constant
Ageing exponent	α	-	regression variable

In the following the bolded parameters in the tables are classified according to the IRL scale. The remaining parameters are constants/ factors/coefficients related - for instance - to the main testing procedures within the developed analytical model.

B.1.1.4.1. MATERIAL INDICATORS OF EXPOSURE

Concrete cover is a material characteristic that can be used to classify the risk of carbonation. It is regularly measured during the field investigations, in order to verify the remaining service life.

Another material parameter – the **inverse carbonation effective resistance of concrete** - may be determined in natural conditions by normal carbonation test, or alternatively by accelerated carbonation tests (ACC-test method) (fib, 2006). Its maturity level is therefore **IRL=8**.

B.1.1.4.2. ENVIRONMENTAL INDICATORS OF EXPOSURE

The **relative humidity of the carbonation layer** is one environmental indicator. It is difficult to measure the humidity of the concrete bulk and in particular of the carbonated layer but, since carbonation process takes place in the outer parts of the concrete, providing there is not direct action of the rain, it is allowed to use the mean daily values of the relative humidity of the ambient air of the structure. In the zones exposed to rain the relative humidity is a function of the raining regime. In the case of sheltered from rain zones it is possible that data of the nearest weather station can be used as an input, or alternatively in-situ measurements in concrete may also be performed (Parrott, 1996) (Jensen, 2003). This exposure indicator can be used as input of forecast models to predict remaining service life. At the time being this indicator is not yet routinely used for quality checks of bridges therefore it is assigned **IRL=8**.

Other three important environmental indicators can be quantified from the data provided by the nearest weather station: **carbon dioxide concentration** (the actual CO_2 content in the atmosphere), **probability of the driving rain** (average distribution of the wind direction during the rain events) and **time of wetness** (average number of rainy days per year). These indicators as well reach **IRL= 8**.

The **environmental transfer variable** accounts for influence of the temperature on the diffusion coefficient. This variable depends on the temperature of the structural element and it can be estimated using data retrieved from a weather station nearby. Currently is not yes used for QC therefore it is assigned **IRL=8**.

B.1.1.4.3. ENVIRONMENTAL AND MATERIAL INDICATORS OF EXPOSURE

Total critical chloride content and the **content at the depth Δx** are considered as both environmental and material parameters, as the environment provides the chloride ions, and material properties (type of binder, concrete composition) affect them (Izquierdo, et al., 2004). For instance, a proper critical value cannot be generalized, as the thresholds scatter over a wide range of values. It should be estimated while observing in which cases the propagation period has been reached and corrosion has already started. Moreover, **initial chloride content of the concrete** is contained not only in the surface; rather it comes from chloride-contaminated aggregates, cements and water used for the production. Chloride content at a depth Δx could additionally be verified according to the procedure provided in reference (fib, 2006). These three parameters reach **IRL=8**, as they are not yet commonly used for the QC checks and related decision-making.

The rate at which the chloride ions penetrate intro concrete is governed not only by the concentration of chloride ions from the environment but also by diffusivity of concrete. This however presents a simplification, as also other transport mechanisms should be considered, but it has been acknowledged that for larger concrete depths, diffusion becomes the most effective and important mechanism related to initiation of corrosion (fib, 2015). Correspondingly, a proper knowledge about **the diffusion coefficient** would also significantly influence the prediction of the deterioration. The parameter can be determined, while assessing the existing structures, from field data or alternatively with short-term laboratory diffusion tests in accordance with (NT Build 443:1995, 1995), and with these developed testing procedures in both laboratory and real environmental conditions. Therefore, the indicator could be used in quality checks but currently is not therefore it is assigned **IRL=8**.

B.1.1.4.4. RPIS RELATED TO LONG-TERM DURABILITY OF REPAIR/PROTECTIVE MEASURE

Another important parameter that may be used to improve the prediction of remaining service life, is the long-term durability of performed repair or applied protective measure. The producers of the repair and intervention measures are often conducting this research, but it is hard to predict the durability of every specific product, considering that the type of the structure, and material and environmental conditions highly influence the performance of the intervention measure. Regardless, this indicator could improve the knowledge of the present condition of the repaired structure and within the prediction model suggest the future intervention measures (Andrade & Martinez, 2009) (Vidovic, 2018).

Repair and protection durability is defined as the period at which end the intervention measure has failed, and may be verified in accordance with European Standard EN 1504. It consists of 10 main standards ((EN 1504-1:2005, 2005) (EN 1504-2:2005, 2005) (EN 1504-3:2006, 2005) (EN 1504-4:2005, 2005) (EN 1504-5:2013, kein Datum) (EN 1504-6:2006, 2006) (EN 1504-7:2006, 2006) (EN 1504-8:2005, 2005) (EN 1504-9:2005, 2008) (EN 1504-10:2004, 2004)). Part 2-7 are the basis for CE marking of the different products used for protection and repair, whereas the Part 8 regulates the quality control of the products and Part 9 describes the principles for the use of the products. Part 10 gives a general guideline for site application and quality control of the works.

In addition, the Standard provides addition 61 standard for applicable test methods. In accordance with these standards, it is possible to investigate the performance of the protection and repair measure from a bridge/component (outdoor) (Johansson, et al., 2008). However, some issues related to its evaluation still exist since examinations are usually conducted only few weeks or months after the treatment (Matthews, 2007). For these reasons it is assigned **IRL=7**.

B.1.2. RPIS RELATED TO GENERAL CONDITION

Contributor: Carmen Andrade, André Orcesi

This section describes Performance Indicators that describe the general conditions of the structure and are computed based on observations collected during visual inspections. These indicators could be used for decision making at element and/or network level for the purposes of prioritization of the assets in the network.

B.1.2.1. CONDITION-BASED PREDICTION USING INSPECTION RATING SYSTEM

In most of the countries the inspection condition score is computed for both individual components and for the entire bridge based on observations collected during visual inspections. Factors unique to highway infrastructures such as relatively small number of discrete condition states and long service life make discrete time-homogenous Markov chains an attractive model for condition development (Mašović, et al., 2015). Indeed, the inspection score is usually based on a 3 to 5 points rating scale.

Several methods have been recently considered, based on Markov assumption stating that the condition of a facility at one inspection only depends on the condition at the previous inspection (Van Erp & Orcesi, 2018). With such an assumption, the present score is the only one which is taken into account to determine the future condition of the facility.

Those methods include Markov decision process (MDP) for which the distribution of a waiting time until a certain event does not depend on how much time has elapsed already (memorylessness), semi-Markov decision process (semi-MDP) that includes the concept of the time spent in a given state, namely sojourn time, to define the transition among states, and partially observable MDP (POMDP) when inspection techniques and observations do not reveal the true state of the system with certainty (Papakonstantinou & Shinozuka, 2014) (Papakonstantinou & Shinozuka, 2014) (Schöbi & Chatzi, 2016) (Memarzadeh & Pozzi, 2016). One should also mention the hidden Markov models that allow the unobserved condition state to be captured, eliminating the noise and bias associated with inspection/monitoring data (Kobayashi, et al., 2012).

At an applied research level, element/bridge condition based on inspection rating was already predicted using a Markov chain degradation model fitted to a collection of condition data (Orcesi & Cremona, 2011). The input for the prediction model was the current inspection score estimated based on (qualitative) data from visual inspections. More recently, (Van Erp & Orcesi, 2018) combined nested sampling with a Markov-based estimation of the condition rating of infrastructure elements to put some confidence bounds on Markov transition matrices, and ultimately on corresponding maintenance costs.

Some operators have already included some condition-based indicator in their Bridge Management Systems (BMS) to assess the need for additional funding for maintenance. However, since the indicator is not yes routinely used for Quality Checks it is rated IRL=8.

B.1.2.2. SIMPLIFIED INDEX OF STRUCTURAL DAMAGE

The simplified index of structural damage (SISD) is based on a classification model that takes into account several aspects of the problem (environmental conditions, corrosion process characteristics, and detailed structural characteristics) (Andrade & Martinez, 2009). This model can also establish whether a detailed assessment is necessary.

Two main factors are used in the calculation of the SISD index: the sensitivity of the structural load-carrying capacity to an active corrosion process, and the processes and damage apparent in the structure. Both factors are calculated using the information available from visual and in situ tests. The SCI (simplified corrosion index) attempts to characterize the environmental aggressiveness and the actual corrosion damage to the structure (EA and CDI). The SI value (structural index) provides an indicator of the structure's sensitivity to corrosion. This indicator has not yet been used for a real structure therefore it is rated **IRL=7**.

B.1.2.3. REPAIR INDEX METHOD (RIM)

When a structure reaches such a degree of deterioration that an intervention and repair decision must be made, it is then necessary to analyse the various repair options available and try to recommend the most suitable one for the particular structure (Izquierdo & Andrade, 2005) (Andrade & Martinez, 2009). The decision must be based on economic and technical aspects. The proposed methodology is based on the definition of requirements (R) of the repair material or repaired structure, which are qualified by performance indicators (PI) with values on a scale from 4 (very bad) to 1 (very good). Requirements may be Safety, Serviceability, Environmental impact, durability, economy. A high score indicates poor performance. The final assessment is made through the calculation of a repair index (RI), which is obtained by averaging all R-values with each multiplied by its relative importance. This indicator has not yet been used for a real structure therefore it is rated **IRL=7**.

B.1.3. RPIS RELATED TO GENERAL PERFORMANCE

Contributor : Michael Havbro Faber

B.1.3.1. RELIABILITY INDEX

The reliability index β is measure of structural safety, representing e.g. the annual probability of failure for a given failure mode. Failure modes might relate directly to structural failure of a member, parts of a structure or an entire structure. However, failure modes might also be formulated for any other state of the structural performance associated with consequences; such as related to reduced serviceability and damages requiring repairs and/or maintenance. Deterioration processes generally lead to a decrease in the reliability index over time.

Modern design codes with safety formats based on load and resistance factors such as the Eurocodes are usually calibrated based on probabilistic modelling of the structural performances and requirements to structural performances specified in terms of target reliability indexes, in dependency of consequences of failures and the relative costs of safety measures, see e.g. ISO 2394:2015 and the JCSS Probabilistic Model Code (JCSS, 2001). In accordance with ISO 2394:2015 and JCSS Guideline for Reliability Based Assessment of Structures, 2001, design and assessment of existing structures can also be based directly on reliability analysis.

There is a vast literature with numerous examples of practical applications of reliability analysis for both the design of new bridges and for the assessment and service life extension of existing structures. Examples addressing reliability based bridge assessments at both individual and network level are provided in (Skokandic, et al., 2016).

Based on its long history, its extensive practical use and the fact that it is applied directly in structural design codes, this indicator is assigned **IRL= 8**. (JCSS, 2000) (Faber, 2015) (Faber, et al., 2007)

B.1.3.2 - ROBUSTNESS INDEX

In literature there is a wide range of definitions applied to robustness and not yet a consensual definition has been reached.

During the last decades there has been a significant effort to develop methods to assess robustness. The basic and most general approach is to use a risk analysis where both probabilities and consequences are taken into account. First formulations of this are provided in (Faber, 2008) (Baker, et al., 2008) (Sorensen, et al., 2010).

Approaches to define a robustness index can be divided in the following levels with decreasing complexity:

1) A risk-based robustness index (Baker, et al., 2008) based on a complete risk analysis where the consequences are divided in direct and indirect risks. The approach divides consequences into direct consequences associated with local component damage (that might be considered proportional to the initiating damage) and indirect consequences associated with subsequent system failure (that might be considered disproportional to the initiating damage). An index is formulated by comparing the risk associated with direct and indirect consequences. In (Faber, 2015) and (Faber, et al., 2017), the formulation of the robustness index from (Baker, et al., 2008) and (Faber, 2008) is generalized and extended to ensure that the modelling of the robustness performance of structures does not mix up direct and indirect consequences originating from different scenarios of component and system failures.
2) A probabilistic robustness index based on probabilities of failure of the structural system for an undamaged structure and a damaged structure (Frangopol & Curley, 1987) and (Fu & Frangopol, 1990).
3) A deterministic robustness index based on structural measures, e.g. pushover load bearing capacity of an undamaged structure and a damaged structure (ISO, 2007).

In reference (Cavaco, et al., 2016) robustness is defined as: "a structural property which measures the degree of structural performance remaining after damage occurrence. The proposed robustness measure was developed to assist structural assets management in decision making related to the schedule of maintenance: more robust structures present a better tolerance to damage and therefore require less maintenance, or at least sustain longer period between maintenance actions (Cavaco, et al., 2017).

Robustness can be defined considering several damage phenomena. In references (Cavaco, et al., 2013) (Cavaco, et al., 2015) (Cavaco, et al., 2016) (Cavaco, et al., 2017) the robustness index is related to corrosion in RC structures, and it is assessed considering the structural performance in terms of load carrying capacity and damage as loss of effective reinforcement area. In some cases, its evaluation is performed using a probabilistic approach, see references (Cavaco, et al., 2016) (Cavaco, et al., 2017)) in others, due to the increased complexity associated to a probabilistic analysis, deterministic robustness indicators are used (Cavaco, et al., 2013) (Cavaco, et al., 2015).

Since robustness is at level of standardization (ISO,2007), it is herein rated as **IRL=8**.

B.2. RPI'S RELATED TO SERVICEABILITY

B.2.1. RPIS RELATED TO LOSS OF STIFFNESS

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Structural Health Monitoring is often performed through vibration based or static load tests. Both types of test allow to compute performance indicators related to the global stiffness of the bridge. During a static test, the structural response is measured in terms of displacements, whereas during vibration-based tests accelerations (less frequently: velocities, displacements or strains) are recorded continuously at one or more locations of the structure. Vibration-based tests rely on the fact that a loss of stiffness in a structural system leads to changes in its dynamic properties.

Therefore, performance indicators related to loss of stiffness are computed as functions of the modal parameters (natural frequency, mode shape, modal strain and damping). Vibration-based tests under operational conditions (i.e. ambient excitation) are much faster, easier and less expensive compared to static testing therefore much more suited for the on-site experimental assessment of possible variations of the global structural stiffness.

During vibration-based tests, responses are collected in terms of measured accelerations in several locations of the structure. From recorded responses, several indicators related to the structural condition (health) can be computed and used -directly or through the computation of proper key performance indicators - to assess the performance of the structure. Several methods are used to extract from measured responses the so-called 'damage sensitive features' that is performance indicators that allow to distinguish between undamaged and damaged states of the structure.

These methods fall into two categories: model based and data driven methods (Limongelli, et al., 2016). Large part of these methods and corresponding performance indicators, are meant to identify if, where and how much the current structural condition differs from the reference one which usually corresponds to the beginning of monitoring.

A traditional hierarchy of methods of damage identification was proposed by Rytter and corresponds to different levels of damage identification:

- **detection**: identification of the existence of damage;
- **localization**: identification of the location of damage;
- **assessment**: identification of the severity of damage.

These methods fall into two main categories: model-based and data-driven.

Model-based methods are based upon the development of a - physics-based or law-based - model of the structure. The model is calibrated using experimental data to allow an accurate representation of the structural response. The experimental data often consist of modal parameters, extracted from measured response time histories using modal analysis techniques. Relationships (analytical or implicit by the use of a numerical model) are used to define the dynamic properties or the model as functions of the model parameters and the measured and computed data are confronted in a cost function. The optimal values of the parameters are those that minimize the cost function.

Data driven methods use models based on experimental response data recorded on the structure.

Damage-sensitive features are extracted from data and their changes used to identify damage in the structures. They do not require a finite element model and may be applied with a limited number of available signals. Approaches in the frequency domain use Fourier analysis as the primary signal-processing tool. Approaches in the time domain use statistical tools to describe measured random signals and analysing their observed behaviour. More details about model based and data driven methods can be found in reference (Limongelli, et al., 2016).

In the following sections are described and rated, according to the IRL, several Research Performance Indicators that can be derived from vibration-based tests. The first group (from modal parameters to the Interpolation Errors) detect concentrated losses of stiffness through localized increases of curvature. The indicators of the second groups are not defined in terms of physical parameters but rather in terms of residual error between the response predicted by a regression model and the measured response. The regression model is calibrated using a set of training data that represent the reference condition. Any deviation from that is attributed to some damage.

B.2.1.1. MODAL PARAMETERS

Modal parameters can be determined from measured structural response data using experimental modal analyses. Depending on the availability of the measured excitation (for example seismic input) input-output methods or output-only methods (Peeters & De Roeck, 1999), (Harmanci, et al., 2016), can be applied. Operational and environmental conditions must be carefully accounted for in the identification of modal parameters since they may induce variations of their values that do not correspond to a change in the actual structural condition (Peeters & De Roeck, 2001), (Reynders, et al., 2014), (Spiridonakos, et al., 2013) and (Bodeux & Golinval, 2001).

Variations of modal frequency can be used to detect damage and make decisions about the hierarchy of interventions needed at a network level. However, more often, modal frequency is employed for the calibration of finite element models (Polak, et al., 2005) (Peeker & Talvik, 2010) (Peeker & Talvik, 2010) (Peeker & Talvik, 2010) (Peeker & Talvik, 2012) (Peeker & Talvik, 2012) (Peeker & Talvik, 2012) (Peeker & Talvik, 2013) (Peeker, 2014) (Peeker, 2015), (Ntotsios, et al., 2009). After calibration the models are used to perform quality checks of road bridges (Teughels & De Roeck, 2005). In this case the calibrated numerical model is used to compute performance indicators like reliability index or risk at the element level.

Modal frequency has been extensively studied by researchers as a performance indicator related to losses of stiffness (Maeck, et al., 2000), (Carden & Fanning, 2004), and several applications from laboratory specimens to real bridges have been published (Faraonis, et al., 2014), (Cerri & Vestroni, 2003), (Mahowald, et al., 2012), Influence of environmental conditions have been thoroughly investigated as well (Peeters & De Roeck, 2001), (Reynders, et al., 2014), (Spiridonakos, et al., 2013), (Mahowald, et al., 2012).

Based on the published documents, it can be concluded that for modal frequency applicability issues have been solved since that this parameter can be reliably identified using experimental modal analyses and procedures exist to account for the influence of environmental and operational conditions.

However, at the time being, this indicator is not yet systematically used for quality control check purposes therefore, according to the IRL scale, the performance indicator **modal frequency** i**s rated IRL=8.**

B.2.1.2. MODAL CURVATURE

Modal curvatures can be computed from modal shapes (retrieved from accelerations as mentioned in the previous sections) or from strains, if they are measured. Use of modal curvature to localize a loss of stiffness is reported in several numerical (Dilena, et al., 2015), (Shokrani, et al., 2016) and applications in laboratory experimental (Radzienski & Krawczuk, 2009) and on real bridges (Wahab Abdel, 1999). Some issues still exist for the systematic use of this indicator for quality control checks and related decision-making.

The main problem is related to the influence of noise in recorded data and to the need of a double integration to obtain modal curvatures form modal shapes (if accelerations are measured) that make this indicator more and more unreliable for damage localization at the increase of noise level (Dilena, et al., 2015). An interesting evolution is the direct measurement of modal strains, so modal curvatures, using optical fibers (Anastasopoulos, et al., 2018). For all previous reasons, the modal curvature is rated **IRL=7**.

B.2.1.3. MODAL FLEXIBILITY MATRIX

Another performance indicator that can be retrieved from the vibration-based monitoring and allows to localize concentrated losses of stiffness is the Modal Flexibility Matrix (Polak, et al., 2005). Published papers report computation of this indicator from measurements obtained on a real structure (Schommer, et al., 2017). Different from stiffness, a local flexibility change is not directly related to damage at the same place. Inversion of the flexibility matrix could resolve this. However since inversion of flexibility matrix deducted from experimental vibration measurements is often difficult, implementation issues still exist therefore this indicator is rated **IRL=7**.

B.2.1.4. INTERPOLATION DAMAGE INDEX

This performance indicator (Domaneschi, et al., 2013) can be computed from operational shapes and allows to detect losses of stiffness without requiring the computation of curvature, thus reducing the related shortcomings mentioned in the previous section. Applications of this indicators to real structures have been published (Dilena, et al., 2015) (Limongelli, et al., 2017), however also for this indicator applicability issues, related to noisy signals, still exist. For this reason, this indicator is rated **IRL=7**.

B.2.1.5. STATIC NOVELTY INDEX

This performance indicator is computed as residual error value using the Mahalanobis norm (Shokrani, et al., 2016). It increases significantly under a local stiffness reduction and is proposed only for damage detection that is the identification of the existence of damage (level 1). Use of this indicator on laboratory specimens or real bridges has not been found in literature therefore it is rated **IRL=2**.

B.2.1.6. NOVELTY INDICATOR

Novelty Indicator (NI) consists of the average distance between clusters automatically found from data. This indicator varies with changes observed in data acquired on site (Santos, et al., 2016), (Santos, et al., 2016). Since an adaptive confidence boundary based solely on the distribution of data is defined and controlled over time in a fully automatic manner, difference between NI obtained from modal quantities and measured data is considered DPI at the element level that can be used to prioritize intervention plans of various elements in the network. The indicator has been already used in QC and related decision-making (Santos, et al., 2015) for a real bridge but is not systematically used, therefore it is rated **IRL=8**.

B.2.1.7. (AUTO)REGRESSION ERROR

This indicator is used for damage detection and measures the error between a regression model and the response of the structure. The estimation error in a condition different form the reference one, on which the regression model is built, serves as an indicator of damage/irregularity in the system (Spiridonakos, et al., 2013), (Harmanci, et al., 2016). The assumption is that a change in the estimated error indicates damage or irregularity in the collected data.

Applications to both laboratory specimens (Sakaris, et al., 2015), full scale structures (Ou, et al., 2016) and real assets (Harmanci, et al., 2016), (Bogoevska, et al., 2016) have been published. A software - DamReg - has been developed to perform statistical regression analysis on recorded data. Among others parameters and available options it estimates the auto regression error (DamReg Software, Swiss Federal Office of Energy). However, the indicator is not yet systematically used in the decision-making therefore it is rated **IRL=8**.

B.2.1.8. DEFLECTION

All the previous indicators of stiffness loss are based on dynamic characteristics and can be computed from data collected during dynamic tests., Static load tests have a long tradition in civil engineering and provide data on deformation, displacement, rotation and strain. Monitoring of the deflection curves and computing from them their first and second derivative – slope and curvature respectively – is helpful for localization of damage through comparison of these features in the initial and in a damage state. A reduction of stiffness causes an increase of deflection therefore deflection under known loading conditions can be used as damage indicators. Furthermore, a localized loss of stiffness induces a corresponding increase of curvature that is another effective damage indicator. In reference (Nguyen et al. 2016) deflection has been used as an indicator for real bridge structures. At the time being this indicator is not systematically applied in quality control checks and related decision-making therefore it is rated **IRL=8**.

B.2.2. ABSOLUTE POSITION OF ELEMENTS UNDER ENVIRONMENTAL LOAD

Contributors : Hervé Lançon, Nicolas Manzini

B.2.2.1. FIXED POINT VS THERMO MECHANIC LOAD

Thermal effects constitute the most important load on slender structures over long periods of time. On long continuous deck of bridge, the almost-linear dilatation properties of materials mean that at least one point of the element, depending notably on attachment points, is fixed despite temperature cycles. Displacement of this virtual point can highlight structural or material properties changes but also on the limit conditions like bearings, piles, settlement, cable stay, prestressing… Because of those considerations, this long-term indicator is a good monitoring tool for the global health of a structure.

Position of the virtual point can be obtained by computation of the numerical model based on thermal gradients and fed with measured temperatures. Linear thermal expansion of the deck can be measured with GNSS technology or with consideration of fixed abutments with linear transducers on the expansion joints. This indicator has been used for 12 years on a large cable stay viaduct with steel deck.

B.2.2.2. STATIC AREA DEFORMATION VOLUME

Both dynamic and static response of a structure can be observed in time based on the displacement of one or more points of a structure. Traditional local positioning techniques may however miss part of the slowest displacements/drifts, notably due to average temperature variations over the years or global subsidence of a structure. Absolute positioning, which consists in formulating positions of points as coordinates in global fixed reference frame larger than the studied structure can be achieved through various combinations of techniques. Absolute positioning allows to observe both dynamic (depending on acquisition rate), and static deformation of elements of a structure.

By using position time series of a defined point, it is possible to compute a 3-dimensional volume which encompasses all absolute occupied positions over a defined period of time (daily, weekly, monthly): Static Area Deformation Volume. Characteristics of SADV, such as its shape, the location of its center, its dimensions and orientation are related to the global response of the structure under load. Variation of those characteristics in similar conditions/charges can be used to highlight structural change. The maturity level is **IRL=2**.

B.3. RPI'S RELATED TO LIFE-CYCLE ANALYSES

Contributor : Alfred Strauss, André Orcesi

Sustainability requires lifecycle thinking. In the context of sustainable construction, the design of a bridge goes beyond the traditional requirements of safety and initial costs. It comprises all lifecycle stages of the bridge, from raw material production to the bridge's demolition. This implies the prediction of the structural behavior of the bridge over its lifespan, the estimation of bridge maintenance and repair, etc. Moreover, non-traditional aspects of the environment, economy, and society shall be considered together with traditional ones and currently, most engineers are rarely prepared for these new requirements. The lifecycle environmental analysis (LCA) has currently the most well established standardized framework (ISO 14040, 14044) although there is still no generalized acceptable methodology in the scientific community.

In a decreasing order of development follows the lifecycle economic analysis. The ISO 15686-5 methodology defines the lifecycle costing as a technique which enables systematic economic evaluation of the lifecycle costs over the period of analysis. Life-cycle cost (LCC) is the total expected cost of one bridge and consists of construction costs, operating costs and demolition costs. It is one of the main tools of decision-making procedures within Bridge Management Systems where decisions are commonly based on the minimization of LCC while the performance is kept at satisfying levels (Frangopol, et al., 1997). LCC can be used at both element level and for ranking purposes of the bridges in the network. It is possible to obtain the LCC at the current state and projected in the future, by using the present value calculations and related discount rates (Vidovic, 2018).

It is possible to use LCC computations in the related decision-making (ÖFG, 2017) (Woodward, et al., 2001). However, the issues may exist when applying the computation to existing structures, when the construction and historical costs cannot be verified. Also, life-cycle analyses are usually time-consuming and thus costly and the lack of data is a problem often encountered. In addition, the benefits brought by a sustainability perspective are often perceived only in the long-term, which makes its effective implementation difficult to promote. Moreover, life-cycle methodologies have been developed for the analysis of simple products. The application of such approaches to more complex systems, like a bridge, entails specific problems that need to be addressed in order to make them feasible. For all these reasons, life-cycle indicators (LCA and LCC) are herein rated as **IRL=6**.

Recently, the European research project "Sustainable Steel-Composite Bridges in Built Environment" (SBRI, 2013) (SBRI+, 2017) considered the assessment of steel-composite road bridges by means of a holistic approach combining Lifecycle Assessment (LCA), Lifecycle Costs (LCC) and Lifecycle Performance (LCP) analyses (SBRI, 2013).

The aim was to give a better understanding of the benefits gained by the application of life-cycle approaches and to provide some background necessary for the discussions in the decision-making process when dealing with authorities like road ministries and concessionaires. Considering this, extensive data regarding Lifecycle Cost Analysis (LCC), Lifecycle Environmental Assessment (LCA) and Lifecycle Performance (LCP) for all the lifecycle stages of bridges were collected in the SBRI research project. The database forms then a basis for further detailed investigations on LCC, LCA, and LCP.

B.4. CONSIDERATION OF THE EFFECTS OF CLIMATE CHANGE

Contributors : André Orcesi, Boulent Imam

According to IPCC climate change predictions, countries around the world will likely face dramatic climatic changes soon. More floods, drier summers and wetter winters are expected, along with the rise of sea levels and increase of wind speeds. Associated effects on civil engineering structures (among which bridges) by the change in temperature, precipitation, sea level etc. are already recognized (see Table 10 and following references (CEDR, 2012) (PIARC, 2012) (PIARC, 2013) (Wang, et al., 2010a) (Wang, et al., 2010b) (Wang, et al., 2010c) (Wang, et al., 2010d) (RIMARROC, 2009) (Agency, 2008).

Table 10. Challenges linked to climate change effects and their impact on infrastructure elements.

Climate driver	Impacts
Flooding and erosion	Scouring around bridge foundations, impact on drainage systems, and erosion protection (Slopes and retaining walls alongside bridge piers and abutments).
Landslides and avalanches	Extreme flooding intensifies landslide risks.
Droughts	Deterioration of concrete; steel structures, and embankments.
Extreme heat	Damage to expansion joints, degradation of the bridge deck material (asphalt Asphalt pavement can experience softening and traffic-related rutting, as well as the migration of liquid asphalt to the bridge deck surface from older or poorly constructed pavements)
Sea Level Rise	Increased risk of chloride induced deterioration mechanism at areas above the designed splash zone.
Hurricanes	Bridges may encounter stronger and more powerful storm surges and waves causing direct physical damage.
Snowfall	Winter maintenance and operation (May also cause structural collapse in some types of structures).

Climate change will likely exacerbate existing roadway issues and further deteriorate the condition of bridges in both developed and developing countries (Orcesi, et al., 2016b) (Orcesi, et al., 2016c). When talking of indicator though, it is still a research issue to understand, qualify and quantify how natural hazards and the changing climate will likely impact infrastructure assets and services as it strongly depends on current and future climate variability, location, asset design life, function and condition. So far, there is no well-defined and agreed performance indicator that isolates the effects of climate change for bridges. Rather, one can mention thereafter (see the following sections) some key considerations on how climate change may produce changes of the exposure in terms of intensity/frequency of extreme events or changes of vulnerability due to physical and chemical actions affecting structural durability.

Besides, one should mention that the definition of the system is crucial when dealing with the effects of climate change. For example, in terms of assessing changes of vulnerability, the service that the structure provides over a wider transportation network is required to assess the true levels of consequences brought by extreme events or long-term changes in climatic conditions. For example, if a bridge structure is damaged by an extreme event, it can cause disruption to the transportation network, economic losses as well as environmental impact.

Decrease in performance may not necessarily be associated with the asset itself but also with management activities associated with its operation. For example, increase of the duration of heat waves or the magnitude of maximum temperatures during summer can influence the availability of workforce for carrying out bridge maintenance activities. On the other hand, a decrease on cold spells during winter may have a beneficial effect in terms of less need to use de-icing salts on bridge decks (Railway Safety and Standards Board, 2011).

B.4.1. CHANGES IN THE PATTERN OF EXREME EVENTS

B.4.1.1. CHANGE IN INTENSITY

Climate change might have the following penalising influence on the design values of the extreme environmental actions (floods, sea level rise, extreme wind events):
- increase in the mean value,
- increase in the coefficient of variation (CoV) due to uncertain development of its effects (aleatory uncertainty) and to limited knowledge and modelling of those effects (epistemic uncertainty).

The first case can be dealt with by a notional adjustment of the design value of the corresponding environmental action. In the second case one can either increase the design value of the action or keep the design value unchanged and accept a lower reliability level. The strategy to be followed is a matter of optimisation. In some cases, the effect of climate change may be beneficial; an example of this would be the reduction of number of cold days or of snow actions, which may reduce the need of de-icing salts used for combating snow/frost.

Furthermore, mean values of some actions may decrease, for example a reduction in the mean value of annual precipitation at a location. This may not necessarily translate in a more favourable condition since if the variability (CoV) increases at the same time, the upper tail of the distribution may still increase, leading to more extreme magnitudes of actions (Retief, et al., 2014).

For example, climate change is expected to increase the magnitude of river floods in many locations. Increase of river discharge will lead to increase in water velocities which may result in increase in contraction and local scour around bridge foundations, leading to increased likelihood of stability problems in bridge piers/abutments (Dikanski, et al., 2018).

B.4.1.2. CHANGE IN FREQUENCY

Climate change may have an influence on the return period of extreme events (floods, extreme storm events, drought). For example, return periods of floods may decrease, resulting in the same event having a higher likelihood in any given year (Retief, et al., 2014). In Germany for example flood protection systems like dikes, walls or dams are designed based on the 100-year flood. This characteristic value has been increased arbitrarily by 15% (climate change factor) in order to considerer the higher probability of large floods in the future (see for Bavaria for example (BLU, 2008)).

B.4.2. CHANGE OF VULNERABILITY

Climate change may produce changes on the resistance side. Following are some examples;

Example 1. The kinematics of the chloride ingress and corrosion propagation mechanisms is highly influenced by the surrounding environmental conditions including climate change that could accelerate or decelerate these processes depending on specific exposure and environmental conditions (Bastidas-Arteaga, et al., 2010) (Wang, et al., 2010a) (Wang, et al., 2010b) (Wang, et al., 2010c) (Wang, et al., 2010d).

Example 2. Carbonation affects the performance, serviceability and safety of reinforced concrete (RC) structures when they are placed in environments with important CO2 concentrations. Since the kinetics of carbonation depends on parameters that could be affected by climate change (temperature, atmospheric CO2 pressure and relative humidity (RH)), this study aims at quantifying the effect of climate change on the durability of RC structures subjected to carbonation risks (Bastidas-Arteaga, et al., 2013).

Example 3. The rate of corrosion of steel structures is affected by environmental parameters (temperature, relative humidity, time of wetness) as well as atmospheric pollution concentrations (CO2 and SO2). These two variables may also affect each other, for example higher atmospheric pollution may lead to higher temperatures whereas increase in temperatures may lead to increase in atmospheric pollution concentrations, especially at large cities due to the urban heat island effect (Retief, et al., 2014).

Example 4. Change in the embankment's strength due to rainfall infiltration which may cause failure due to slope instability (structural failure) (Preziosi & Micic, 2014).

B.5. RPI'S RELATED TO SEISMIC PERFORMANCE

Contributors : Andrej Anzlin, Eleni Chatzi, Juan Murcia-Delso

Seismic performance indicators (SPIs) can be divided in indicators that predict the seismic performance of a bridge prior to occurrence of a seismic event, and indicators for evaluating the actual performance and condition of a bridge after an event has occurred. The former are used in performance-based seismic design and assessment, as well as for risk analysis.

The latter are fundamental for emergency response, and support of repair and recovery decisions. Seismic PIs may be deduced in two schemes that are often interacting, namely;
- on the basis of design information (models),
- on the basis of monitoring information (data), or based on the fusion of models and data

Design information may be retrieved from project documentation and analytical calculation methods, while monitoring information is aggregated from visual inspections, non-destructive or destructive measurements, continuous monitoring data (typically vibration-based).

B.5.1. INDICATORS COMPUTED FROM DESIGN INFORMATION

Seismic bridge assessment typically involves modelling of structural performance for inferring metrics linked to capacity (traditional approach) or vulnerability (performance-based approach). Design codes, typically prescribe a mixed approach with performance goals defined in terms of limit states, and require fulfilment of standards related to the capacity–demand ratio and to member detailing, such as compliance with capacity design principles (Prendergast , et al., 2018) (Kattell & Eriksson, 1998) (Park & Paulay, 1975).

B.5.1.1. DISPLACEMENT-BASED INDICATORS

In terms of capacity, in the context of the Eurocode, ***seismic displacements*** (from linear analysis), ***plastic rotations*** (for ductile members), or ***shears forces*** at the members/joints (for non-ductile members) are used as indicators of performance (Kolias, 2008). These indicators were experimentally validated in laboratory (Paulay & Priestley, 1992) and are nowadays systematically used in the seismic design of bridges. They therefore feature, according to IRL scale, the more mature readiness level 9 (**IRL = 9**).

Further displacement-based indicators linking to design are most commonly associated with performance-based design (PBD) and displacement-based design (DBD) in particular. However, as underlined by (Kappos, 2015), implementation of DBD concepts to bridges has been more limited than corresponding adoption in building-design, albeit initial studies on the so-called 'direct' displacement-based design (DDBD) of bridge piers (Kowalsky, et al., 1995) or entire bridges (Calvi & Kingsley, 1995) has already appeared in the mid-1990s. The method is applicable to bridges that can be reasonably approximated by an equivalent single-degree-of-freedom (SDOF) system for calculating seismic demand.

In such an approach, the ***target displacement and the displacement profile*** could serve as SPIs, together with variables linked to performance of bearings, such as the maximum acceptable shear strain ratio. Based on the direct-DBD procedure proposed by (Kowalsky, 2002) (Dwairi & Kowalsky, 2006), calculation of target displacement accounts for inelastic effects, without however carrying out an inelastic analysis. The latter is achieved via an effective mode shape approach where higher mode effects are taken into account by determining the mode shapes of an equivalent elastic model of the bridge based on the column and abutment secant stiffness values at maximum response. The interested reader is referred to the work of (Kappos, 2015) for further details on variants of the DBD analysis methods and calculation of associated metrics used in the analytical sense (**IRL = 2**).

However, relating to the notion of displacement, indicators relating ***to drift ratio limits*** are also developed on the basis of laboratory tests, as explained in (Liu, et al., 2012). That work exploits statistical data of 127 seismic performance tests of RC bridge columns, with circular section subjected to flexural failure, and defines the drift ratio limit as a classifier of performance into five designated performance levels (**IRL = 7**).

Perhaps the most common seismic PI that is used both in pre-event and post-event assessments is the ***residual displacement of bridge piers*** (Dazio, 2004). Some recent studies have shown, that the control of residual drift ratio has negligible effect on the overall cost of the bridge (Ardakani & Mohammad, 2013). This indicator can be physically measured after a seismic event or it can be predicted during the design or assessment process by means of non-linear time-history dynamic analysis, from which the anticipated drift value is calculated.

The level of attained residual displacement is highly correlated with structural damage. Excessive residual drifts may be related to shortcomings in the seismic design and detailing of bridge components, but the damage manifested will eventually depend on the intensity and characteristics of the ground motion. Hence, thresholds values and performance goals have to be established in relation to the level of seismic hazard. In addition, the importance and specific circumstances of the infrastructure have to be considered.

For example, the permissible/threshold value of the residual displacement of bridge piers after an earthquake is in the case of high-speed railway lines more rigorous than in the case of structures located in remotely accessed areas, due to the need to ensure operation of the railway system even after the seismic event. Therefore, the design of residual drift is already implemented in the seismic design specifications for highway bridges (IRL9) in Japan (Japan Road Association, 1996).

B.5.1.2. FRAGILITY FUNCTIONS

Fragility functions, which define the probability of exceeding a specific damage state for a given earthquake intensity (using metrics such as peak ground acceleration or spectral acceleration), have been developed to evaluate the probable future seismic performance of bridges (e.g., (Moschonas, et al., 2009), (Ramathan, 2012), (Tecchio, et al., 2016), (Khosravikia, et al., 2018). To develop fragility curves, different demand parameters associated to a bridge component and/or damage mode are defined (column deformation, abutment seat displacement, shear key displacement, etc.).

Fragility functions are commonly developed analytically by conducting nonlinear (static or dynamic) analyses for demands of increasing intensity, and then performing a statistical analysis of the exceedance of damage thresholds for each of the demand parameters. The performance indicators resulting from these studies are the conditional probabilities of experiencing a specific level of damage for a given earthquake intensity. The demand parameters used in these fragilities can also be regarded as performance indicators, and their thresholds as performance objectives.

The main limitation of seismic fragility functions is that they are specific for a given bridge structure or prototype configuration as well as the seismic region for which they were developed. These SPIs may only be available for those owners that have conducted studies to develop their own set of fragilities. Hence, fragility functions are not yet systematically used for quality control check purposes. Based on the IRL scale, this performance indicator is rated **IRL=7**.

B.5.1.3. HYBRID INDICATORS

Bridge score indicators are also used to assess bridges from a seismic resistance standpoint. For example, the California Department of Transportation (Caltrans) has developed and refined over time a screening algorithm that provides a score, which takes into account the structural vulnerability, the seismic hazard, and the bridge importance. This score is used to identify bridges at risk and prioritize retrofit actions. The latest developments on this method incorporate new hazards other than shaking, such as bridges on liquefiable soils and bridges located over active faults, as well as new vulnerabilities, such as bridges with early retrofits that have not been fully effective and bridges with short seats and stiff restrainers (Ostrom, 2016). Since this method is systematically used for decision making, this performance indicator is rated **IRL=9**.

B.5.2. INDICATORS COMPUTED FROM MONITORING INFORMATION

B.5.2.1. VISUAL INSPECTIONS

Simple indicators are also available for post-earthquake damage evaluation based on visual inspection. For example, (Veletzos, et al., 2008) developed a visual catalogue that documents damage from laboratory experiments and from historic earthquake. This catalogue defines five levels of damage for different bridge components (e.g., columns, foundations, abutments) and bridge sub-assemblies.

Each level of damage has associated a performance level (from fully operational to collapse) and description of intervention needs (from no repair to replacement). Although empirical, and invariably subjected to the bias of the expert, this indicator is systematically applied for the quality check of a structure/asset and related decision making (**IRL = 9**).

B.5.2.2. NON-DESTRUCTIVE EVALUATION (NDE)

In order to detect damage, which is not easily picked up via visual inspection, such as cracking, local stiffness reduction, or loss of joint capacity more refined methods may be adopted. When non-destructive evaluation is desired, ultrasound, acoustic emission and radar-based techniques may be adopted. (Benavent-Climent, et al., 2012) use AE for damage detection under seismic loads and demonstrate how a correlation may be established between the energy dissipated by the concrete through plastic deformations and the AE energy associated with concrete cracking and friction. However, the method is typically limited in that it should be actively measuring when damage takes place.

In general, NDE techniques, such as ultrasonic, thermal, pulse-echo, thermography, and x-ray based methods require specialized, and often costly equipment, which leads to periodic implementation when damage is already suspected (Rens, et al., 1997) (Farrar & Worden, 2006). Thermal and magnetic methods prove more suitable for steel structures, while ultrasonic methods have been found to be unreliable for concrete structures (Yehia , et al., 2007) (Peterson, 2013). These methods have been demonstrated in experimental studies performed on a real structure/structural element but are not yet systematically applied for ranking purposes and related decision-making (**IRL = 7**).

B.5.2.3. STRUCTURAL HEALTH MONITORING (SHM)

Structural Health Monitoring (SHM) typically relies on response measurements, most commonly vibration response acquired through sensor networks (e.g. acceleration, displacement, strain sensors) in order to localize and quantify response changes generated by local degradation of conventional structural elements as well as anti-seismic devices (Benzoni, et al., 2013) (Çelebi, 2012). Seismic monitoring (Yi, et al., 2012) in particular aims to (1) assess the seismic performance of the bridge, (2) check design parameters, including the comparison of dynamic characteristics with actual response, and (3) better future design of similar bridges (Çelebi, 2006).

The benefits stemming from monitoring may be quantified via the concept of Value of Information. Reference (Omenzetter, 2017) elaborates on a pre-posterior Bayesian analysis framework, which may be adopted to quantify the value of long-term monitoring data for seismic emergency management. Since these indicators are derived from measurement from actual (operating) structures, they relate to an **IRL = 7**. However, these methods are highly differentiated in terms of their maturity, complexity robustness and validation on a broad class of systems.

Monitoring information can be frequently used to compare against reference design parameters for ensuring adequate capacity under seismic actions (Prendergast , et al., 2018). Perhaps, the most common instance of such an application is Finite Element Model Updating, where measurements are used to inform models of structural systems, for inferring possible damage or change with respect to the healthy status of the structure. Modal updating is often conducted in a Bayesian context (Simoen, et al., 2015).

In this context of monitoring, quantitative indicators based on simple field measurements have been historically used by Caltrans to characterize the bridge response after an earthquake, such as markers at expansion joints to track bridge movements or tension indicators on cable restrainers to indicate yielding of the cables (Yashinsky, 2018) (**IRL = 8**).

On the other hand, the aforementioned fragility functions (Ozer & Soyoz, 2015) can be used for both pre-event and post-event decision making. An example of the latter is the Shakecast system developed by Caltrans and USGS. This system retrieves measured shaking (vibration response) data within minutes after an earthquake has occurred and uses the fragility functions to provide a list and maps of the most likely impacted bridges. (**IRL = 7**, as applicability issues are not fully resolved and the assessment is not straightforward).

Relying on post-earthquake residual displacements as SPIs, (Yazgan & Dazio, 2011) developed and validated against experimental evidence (**IRL = 4**) a hybrid methodology to improve the estimate of maximum deformations occurred during an earthquake, taking into account a model of the structure, as well as observable damage and measurable residual displacements.

B.5.2.3.1. MODAL-BASED DAMAGE ANALYSIS

The comparison of modal characteristics of a structure (frequencies, damping rations, mode shapes) prior to and after a seismic event, can serve as an indicator of damage. However, a change on the natural frequencies of a bridge is a global metric, which is not always sensitive to local damage (Banks, et al., 1996) (Farrar & Doebling, 1999) (Alampalli, et al., 1997). A thorough literature review on this matter is found in (Doebling, et al., 1996). Furthermore, a proper assessment of damage when monitoring modal properties requires normalization against the effects of environmental and operational parameters for ensuring robust detection, as elaborated upon in the works of (Spiridonakos, et al., 2016) and (Limongelli, et al., 2016).

However, when damage does affect global properties it is possible to use ***natural frequencies***, ***mode shapes***, ***mode shape curvatures*** (Shokrani, et al., 2016), or ***modal strain energy*** (Tatsis, et al., 2018) as indicators of damage. Further alternatives rely on use of Flexibility Matrix based Methods (Aktan, et al., 1997); (Schommer, et al., 2017) or http://orbilu.uni.lu/handle/10993/30290 Operational Deflection Shape based Approaches (Sampaio, et al., 1999); (Busca & Limongelli, 2015).

Beyond classical modal-based methods, which rely on analysis in the frequency domain, wavelet analysis offers a time-frequency analysis tool (Moyo & Brownjohn, 2001). In this context, (Noh, et al., 2011) proposed engineering demand parameter indicators for non model-based seismic vulnerability assessment of steel frame structures, while (Hwang & Lignos, 2018) demonstrated that wavelet-based damage-sensitive features can facilitate seismic vulnerability assessment. (Yashodhya, 2016) offer a state-of-the-art review on application of the continuous wavelet transform in SHM.

Such methods have been demonstrated on monitoring data from real structures but are not yet systematically applied for ranking purposes and related decision-making (**IRL = 7**).

B.5.2.3.2. TIME-DOMAIN BASED METHODS

Another class of methods, exploits availability of a model of the structure, and fuses this model with data in order to predict damage in real-time. Bayesian filters (Lin & Betti, 2004) fall in this domain of methods, where real-time identification of the degrading system parameters is sought. This is a joint state and parameter estimation problem, which may be solved by use of Bayesian filters, such as the Extended Kalman Filter, the Unscented Kalman Filter (UKF), sequential Monte Carlo (or Particle Filter) methods (Chatzis, et al., 2015) or state-space observers such as the Eigensystem realization Algorithm and the Observer/Kalman Filter Identification (OKID) approach (Lus, et al., 1999).

These methods are restricted when used with systems of high-dimensionality, i.e., multiple degrees of freedom, an issue which is tackled via use of sub-structuring approaches and reduced order modelling. The strength of these methods lies in the potential to use them in real time in the context of early warning or rapid response.

Alternatively, purely data-driven methods may be adopted relying on the autoregressive class of models, which are then linked to the modal characteristics of the system, as in the work of (Loh & Lee, 1997).

Methods of this class have been demonstrated on monitoring data from real structures but are not yet systematically applied for ranking purposes and related decision-making (**IRL = 7**).

B.5.2.3.3. MACHINE-LEARNING BASED METHODS

Such methods do not straightforwardly operate on dynamic characteristics of the system, such a modal properties but instead rely on use of Machine learning (ML)-based approaches (Farrar & Worden, 2013). In this context, (Laory, et al., 2013) introduced a model-free data-interpretation method, which couples moving principal component analysis with four regression alternatives, namely robust regression analysis, multiple linear analysis, support vector regression, and the random forest method, for the purpose of damage detection under availability of continuous monitoring data.

The method is implemented on a number of case studies including the Ricciolo viaduct, a bridge over the Swiss motorway A2, where data during construction serve as data of "anomalous behaviour". (Nguyen, et al., 2014) implemented a principal component analysis-based damage detection technique for feature extraction, coupled with the concept of subspace angle for detection of irregularities, demonstrated on the Champangshiehl bridge; a two-span concrete box girder bridge located in Luxembourg.

ML-based methods have been demonstrated on monitoring data from real structures but are not yet systematically applied for ranking purposes and related decision-making (**IRL = 7**). It is important to note that for a number of these indicators, a proper standardization process is missing limiting their direct applicability.

However, research is currently underway for appropriate statistical, treatment, outlier analysis and normalization of these indicators over varing operational conditions. The latter aims to ensure robust and standardized indicators stemming from SHM data.

B.6. RPI'S RELATED TO SCOUR

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B.6.1. INDICATORS COMPUTED FROM DESIGN DATA

B.6.1.1. EMPIRICAL LOCAL SCOUR DEPTH

The empirical local scour depth represents an expected value of the maximum local scour depth (m) at a bridge pier/abutment affected by flooding. It is evaluated using local scour evaluation formulas, which have been derived on the basis of extensive hydraulic laboratory testing (flume tests) and available field data. Because of the empirical nature of these formulations and scaling problems moving from laboratory derived methods (See (Park, et al., 2017)) uncertainty around predictions is significant.

The maximum predicted value of local scour depth is primarily used in the design of new bridges, but in the last 20 years this is also applied in the case of existing bridges i.e. in bridge management. The value of evaluated scour depth is compared to the soil cover depth at the affected bridge foundation to rank bridges with respect to the threat of failure due to local scour. It is suggested to be applied when observations from visual/onsite inspections point to a possible problem with scour (e.g. related condition score is sufficiently low).

In the U.S., bridges are rated for scour criticality (National Bridge Inventory Item 113) (FHWA, 1995). This rating and successive decision making implies that bridge is determined to be scour critical for site conditions either directly (visual inspection) or indirectly (calculation). Thus, the potential local scour depth evaluated with adequate formulas is used systematically in quality checks (**IRL= 9**).

However, due to large number of parameters involved in formulas and various levels of parameter uncertainty and interdependency, probabilistic evaluation is deemed more adequate (Johnson & Dock, 1998) and is suggested to be applied when viable. The latter is not systematically performed. Improvement on predictions of local scour depth is an ongoing research topic as well as its application in bridge management e.g. (Huizinga & Rydlund, 2004), (Govindasamy, et al., 2013) (Tubaldia, et al., 2017)(**IRL= 8**).

Recently, probabilistic calculation of local scour depth is coupled with resistance of bridges to local scour in estimation of vulnerability of bridges to flooding (Tanasic, et al., 2013) and (Tanasić & Hajdin, 2018), which is suggested as a viable risk-based approach for a network level. The related approach is yet to be fully implemented and validated on road networks, thus **IRL=5**.

The computation of the aforementioned indicators requires the knowledge of:
- geometrical dimensions of the bridge (height, width, length), shape of piers (oval, rectangular), type of foundations (deep, shallow), alignment with respect to the water flow (angular degrees);
- foundation soil: erodibility (not necessary for some formulas) and median soil particle diameter;
- river channel at the bridge micro location: channel slope, manning coefficient, cross section geometry of the channel;
- Hazard (flooding): height of water in the river channel at the affected bridge substructure or flow [m3], duration of high flow conditions (not necessary in for some formulas). These two parameters can be extracted from a flood hydrograph.

There are several state-of-the art formulas suggested to compute the local scour depth [Arneson et al., 2012]. Here as an example the Hydraulic Engineering Circular No. 18 (HEC-18) pier scour equation (i.e CSU method) is given as the predominant method for design estimation of pier scour in the U.S and recommended by American Association of State Highway and Transportation Officials (AASHTO).

$$\frac{y_s}{y_1} = 2.0 K_1 K_2 K_3 K_4 \left(\frac{a^*}{y_1}\right)^{0.65} F_r^{\;0.43} \tag{4}$$

where:

y_s = local scour depth [m]

y_1 = flow depth directly upstream of the pier i.e. unscoured water depth [m]

a^* = equivalent pier diameter width [m]

$F_r = F_r = \frac{V_1}{\sqrt{g * y_1}} =$ Froude number directly upstream of the pier

V_1 = Mean velocity of flow directly upstream of the pier [m/s]

g = acceleration of gravity [9.81 m/s2]

The K1, K2, K3, K4 are correction factors respectively for pier nose shape, angle of attack flow, bed condition and armoring by bed material size. For their definition and values reader is directed to (Arneson, et al., 2012). It is suggested both for clear water scour and live bed scour conditions as well as for non-cohesive and cohesive soils.

B.6.2. INDICATORS COMPUTED FROM OBSERVATIONS

Prior to reaching a collapse the state the occurrence of scour causes a loss in foundation stiffness for bridges- A number of workers have therefore developed vibration-based damage identification approaches previously described be utilised to detect its presence (Prendergast, et al., 2016) (Foti & Sabia, 2010) (Xiong, et al., 2018) (Xiong, et al., 2018) (Prendergast, et al., 2017) (Prendergast, et al., 2013) (Elsaid & Seracino, 2014) (Ju, 2013). However, as scour is a separate phenomenon to the bridge in terms of damage – i.e. it occurs in the soil around the bridge as opposed to corrosion-related damage or cracking, there is a variety of RPIs capable of monitoring its presence/evolution.

B.6.2.1. SCOUR DEPTH

The most obvious scour RPI is the depth of scour itself. Scour depth can be measured directly using scour depth measuring instrumentation such as Magnetic Sliding Collars (Hunt, 2009) (Prendergast & Gavin, 2014), embedded rods with Fibre-Bragg Grating sensors (Sohn, et al., 2004) (Lin, et al., 2006), or vibration-based sensors on embedded rods (Fisher, et al., 2013) (Zarafshan, et al., 2011) and these can inform on the depth of a scour hole in the direct vicinity of where the instrument is placed. Scour depth can also be inferred from data such as Ground Penetrating Radar (Anderson, et al., 2007).

Placement of these systems near bridge piers or abutments where scour risk and subsequent consequence is high enables safe monitoring of its depth. Other similar systems including sound wave or radar methods (Prendergast & Gavin, 2014) (Fisher, et al., 2013) (Nassif, et al., 2002) also inform on the depth of scour affecting a foundation. A key drawback in using the depth of scour as an indicator of performance is the tenuous/uncertain link between these parameters and how it influences the bridge. Due to the highly subjective nature of scour depth and the obvious question 'how much is too much?' this is potentially not the most reliable approach for dealing with scour.

Moreover, in many countries in Europe the aging (and sometimes legacy nature) of existing infrastructure (such as the railway network in Ireland) contains many bridges with unknown foundation geometries/conditions. This means it is very difficult to say if, for example, 1 m scour is safe or drastically unsafe for a given bridge without further exploration and data required. The metric to compute scour depth depends on the method adopted. For pulse/radar instruments, the metric is changes in dielectric permittivity as the water-sediment boundary is detected (and subsequently scour depth located). For driven rod systems, the metric is changes in magnetic field due to changing elevation of sensor closing switches (for magnetic sliding collars, etc.). That said many studies have successfully implemented systems of this nature (see linked references) so Scour Depth measurement has an **IRL of 5-7**, depending on the instrumentation adopted.

B.6.2.2. STATIC DEFORMATION PARAMETERS

A more useful way to measure bridge performance under scour is to study changes in super-structural movements, which can be used to infer scour presence or even inform on scour extent. The result of scour action reducing the stiffness and capacity of foundations is to change the global properties of the system for example, inducing pier settlement, tilting of pier elements, pile group settlement, pile group tilting, differential settlement leading to strain gradients at the bridge deck level, deck movement due to changes in support conditions, pile buckling due to loss of lateral support and even deck collapse due to being unseated. To measure these occurrences usually only requires a small number of sensors.

Any inclination or rotations can be measured using inclinometers placed on the affected elements, which can inform on the continuous changes occurring during service or can be used before and after major flood events for rapid condition assessment. Strain gauges located on the deck can inform on differential strains occurring along the deck. Visual inspections or camera-based monitoring can be used for the global changes such as deck unseating or other instability issues arising.

Accelerations can be used to inform on changes in stiffness at foundation elements or of changes in the pile group capacity, due to scour. The metric for Static Deformation can be either strain, direct voltage changes for inclination/rotation or otherwise. These are normally calibrated back to the indicator in the data-logging systems employed. For strain, the metric is a certain strain value indicating potential differential settlement and thereby scour inference from this data. Similarly for rotation, the metric is rotation and excessive rotation is the indicator for scour. In terms of studies implementing these types of approach, this is at **IRL = 3** (Prendergast, et al., 2016) (Prendergast, et al., 2017), in that (as far as the author is aware) few full-scale studies using these static deformation type parameters has been applied.

B.6.2.3. NATURAL FREQUENCY OF VIBRATION

Recognising that scour changes stiffness and that this is inherently linked to dynamic properties, using Changes in Natural Frequency is a useful RPI for scour monitoring. What becomes apparent is that the RPI needs to be specific to the element in question or to the effect that is sought to be monitored.

To date, the most successfully applied RPI for scour occurrence utilising the global bridge response has been monitoring changes in frequency of vibration at various locations of the bridge due to changes in support stiffness resulting from scour. Several studies have investigated this at concept, laboratory and full scale application, therefore the IRL ranges from **IRL = 2-3** up to **IRL 7** (Xiong, et al., 2018) (Xiong, et al., 2018) (Prendergast, et al., 2013) (Chen, et al., 2014). Again, probabilistic methods are deemed preferable and the potential effects of geotechnical uncertainty on the accuracy of such methods is discussed in (Gavin, et al., 2018).

B.7. SOCIAL AND ECONOMIC RPI'S

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In the analysis of bridge impact on the society there is a lack of defined performance indicators for different aspects of society and methods for monetizing those impacts. In this chapter performance of a structure regarding its influence on the society at large is analysed. This influence can be divided generally into societal and economic performance. Social aspect in this formulation presents society in a form of local community, nation or region and the influence of a certain structure, e.g. a bridge, is translated into performance indicators. Economic aspect of the infrastructure impact, in our case a bridge on the society, is analysed through indicators which are normally used as indicators of societal economy performance, as e.g. GDP, employment rate, but need to be adjusted for a certain region and/or object and type of analysis.

(Azapagic & Perdan, 2000) define the importance of addressing issues concerning human rights, cultural values, equity and disparity within the current population and between current and future generation. They suggest two generic types of indicators, ethics and welfare indicators, in their framework for sustainable industry. Some of these indicators related to ethics can be adjusted for the use in the bridge management process.

The first set of ethical indicators refers to preservation of cultural values. Preservation of cultural values means the continuation of the way of life of people, and protection of their values, beliefs, arts, modes of perception, habits of thought and activity, in their natural and cultural conditions. Some of the structures, such as bridges or special buildings might be seen as monuments and icons which the citizens may relate with the cultural heritage of a certain area.

This atmosphere and the will to identify certain area and its values with an icon may motivate for bold and spectacular solutions. Some alternatives have exceeded all cost estimates, but they have been chosen as aesthetically the best. Certainly, there is a hidden value behind the external shape of structures, such as bridges in some special locations, see Figure 3 and Figure 4. The inclusion of this value in the evaluation process leads to eliminating the worst aspects (e.g. price) of bridge design and encourages the choice of other solutions. This value should be computed for the different feasible proposals in fair-bases and converted to a measurable value to be able to include it in the LCC model (Safi, 2013).

In the (Azapagic & Perdan, 2000) framework with a simple set of ethical indicators, which measure a discrepancy between operating principles, is proposed for measuring company's performance. They include the following ethical principles which can be translated into indicators and used for comparative assessment of different design solutions or maintenance activities:

- avoidance of improper inducements in business dealings (regarding construction, maintenance and operation of a bridge or a stock of bridges);
- payment of fair prices to local suppliers;
- avoidance of collaboration with corrupt political regimes.

The third set of indicators addresses the issue of intergenerational equity since one of very important societal issues is that needs of future generations must not be neglected. The framework suggests following ethical norms to serve as a basis for establishment of social indicators

- Does the activity (design, maintenance) leave the environment in a condition that we cannot expect to be accepted by the next generation?
- Does the activity (design, maintenance) create any problems for which solutions are not known to us today?

In the next chapters the philosophy of this framework has been applied in order to define bridge performance indicators related to society and economy (Skaric Palic & Stipanovic, 2018).

Figure 3. Samuel Beckett Bridge, Dublin, Ireland designed by Santiago Calatrava (https://en.wikipedia.org/wiki/Samuel_Beckett_Bridge)

Figure 4. Margaret Hunt Hill Bridge (the bride total cost was $182 million) designed by Santiago Calatrava (https://en.wikipedia.org/wiki/Margaret_Hunt_Hill_Bridge)

B.7.1. SOCIETAL AND ECONOMIC RESEARCH PERFORMANCE INDICATORS IN BRIDGE MANAGEMENT

Societal and economic indicators listed in Table 12 and Table 13 can be determined in order to evaluate the influence of a certain activity on a society in a wider range. For example, a bridge in a poor condition will have a negative influence on economic development of the whole surrounding area; aesthetically appealing structure could increase some economic branches such as tourism and have positive influence on the society and economy.

These PIs are not easily determined and quantified but defining them could be accomplished more successfully by combining analysis with experts from other fields, e.g. sociology, economy, demography or insurance.

B.7.2. OTHER PARAMETERS

There are different societal and economy aspects that are influenced by traffic infrastructure development as well as management. The problem is that performance indicators connected with this aspect are mainly qualitative and it is often difficult to quantify them and incorporate in the analysis with other well-known PIs such as e.g. condition index. One of PIs proposed in this paragraph is economy index unemployment rate. Philosophical aspect of connection between development of traffic infrastructure and influence on society through economic growth and development of new jobs is a well-known fact.

In this paragraph we are establishing connection between construction of highway to Zadar County which finished in 2003 and decrease of unemployment rate to show the actual numbers proving interaction between the two. Table 11 is presenting the data about average annual and summer daily traffic on highway section Zadar 1 and unemployment rate in Zadar County through the years. The correlation between the increase of traffic intensity, especially in the summer period, and decrease of unemployment rates is clearly visible and can be referred to the tourism.

Table 11. Traffic quantity data and unemployment rate through the years in Zadar County

Highway section	Year	Average annual daily traffic (AADT)	Average summer daily traffic (ASDT)	Unemployment rate in Zadar county (%)
Zadar 1	1998	-	-	29,8
	2003	5019	9872	28,8
	2005	6803	18537	22,4
	2010	9775	25897	21,1
	2016	11757	31153	16,0

B.7.3. SUMMARY

There is increasing need for environmental and societal issues to be addressed in a more precise and holistic way in all business practices including management of transport infrastructure especially because of its vast influence on the society at large.

Traffic itself and transport infrastructure including bridges influence society and economy in many different ways. Most of these interconnections are well known but mainly on a qualitative basis and without quantitative performance indicators which would enable detailed measurements of certain impacts. Strategic decisions regarding transport networks and bridges should be directed towards societal and economic development based on solid facts, reliable data and quantifiable measures.

Diverse maintenance and rehabilitation options through life span of a bridge that have different working life extension horizons should be assessed using suggested combined economic, environmental and societal analysis PIs. In the case study presented here, the establishment of connections between tourism as an important economic branch, traffic intensity, unemployment rate and infrastructure management is presented.

Novel performance indicators regarding society and economy presented suggest an approach towards sustainable strategic decision making and should be further developed in a multidisciplinary manner. For this kind of analysis professionals from different areas of expertise such as economists, sociologists and demographists should be included in the research to achieve reliable and concrete results.

Table 12. Economic Research Performance Indicators

Economic performance indicator	**Type of indicator**	**Definition of indicator**	**IRL**
Profitability	Quantitative / Qualitative	In general terms profitability is the ability of a business to earn a profit. It can be transferred into an analysis of a structure gaining profit to a company, influencing traffic flows or other economic parameters.	1
Income	Quantitative / Qualitative	Income is money that an individual or business receives on a regular basis. It is gained by working and/or making investments.	1
Human development	Qualitative	The first UNDP Human Development Report published in 1990 stated that: “The basic objective of development is to create an enabling environment for people to enjoy long, healthy and creative lives.”It also defined human development as “a process of enlarging people’s choices”, “and strengthen human capabilities” in a way which enables them to lead longer, healthier and fuller lives.	1
Average income	Quantitative / Qualitative	Per capita income (PCI) or average income measures the average income earned per person in a given area (city, region, country, etc.) in a specified year. It is calculated by dividing the area’s total income by its total population.	1
Floor area per capita	Quantitative / Qualitative	European Commission defines the floor area per dweller is one of the key indicators of dwelling comfort It is the result of the size of dwelling (m^2 floor area) and the number of persons living in the dwelling.	1
Housing cost	Quantitative / Qualitative	The total amount that a homeowner spends on mortgage payments, property insurance, homeowner’s association fees, property taxes and other home-related reoccurring expenses, plus whatever amount is required to service all outstanding household debts.	1
Employment	Quantitative / Qualitative	It can be quantified through parameter employment rates which is defined as a measure of the extent to which available labour resources (people available to work) are being used. They are calculated as the ratio of the employed to the working age population. Qualitatively, it can be analysed in increase or decrease of employment opportunities for population influenced by a certain structure/bridge.	1
Household income	Quantitative / Qualitative	Household income is a measure of the combined incomes of all people sharing a particular household or place of residence.	1

Economic performance indicator	Type of indicator	Definition of indicator	IRL
Corruption	Quantitative / Qualitative	In general, corruption is a form of dishonesty or criminal activity undertaken by a person or organization entrusted with a position of authority, often to acquire illicit benefit. The Corruption Perceptions Index (CPI) is an index published annually by Transparency International since 1995 which ranks countries "by their perceived levels of corruption, as determined by expert assessments and opinion surveys." It is not easy to quantify corruption, especially in a context of analysis of a bridge, but it can be used in a qualitative manner when assessing different alternatives of construction or maintenance.	1
Fair competition	Qualitative	Competition based on the factors of price, quality, and service; not on the abuse of near-monopoly powers, competitor bashing, predatory pricing, etc. Like the previous indicator, it is not easily quantifiable, but can be used in a qualitative assessment.	1
Dumping	Quantitative / Qualitative	Dumping, in economics, is a kind of injuring pricing, especially in the context of international trade. It occurs when manufacturers export a product to another country at a price below the normal price with an injuring effect. The objective of dumping is to increase market share in a foreign market by driving out competition and thereby create a monopoly situation where the exporter will be able to unilaterally dictate price and quality of the product.	1
Economic indicators listed in this table, when used in the analysis of a structure/bridge, are used as an indicator presenting influence of existence of a structure or maintenance works causing interruptions or costs (different maintenance alternatives-different costs) to the population. Population can be local community or wider, depending on the importance of a structure (local road, regional road, highway, international corridor...) or its influence on traffic flows.			

Table 13. Proposed Societal Research Performance Indicators

Societal performance indicator	Type of indicator	Definition of indicator	IRL
Population growth	Quantitative / Qualitative	Population growth refers to change in the size of a population —which can be either positive or negative—over time, depending on the balance of births and deaths. In the context of bridges this PI can be analysed in a sense that a bridge, the existence of a bridge or a good condition of a bridge, influences the willingness of the local community to stay, live and start families in the analysed area. Areas population is affected, either positive or negative, by the structure/bridge due to good/bad traffic conditions.	1
Urban population growth rate	Quantitative / Qualitative	The same as previous RPI, population growth, with the difference that analysed population is urban population.	1
Aesthetics	Qualitative	Aesthetics generally means a set of principles concerned with the nature and appreciation of beauty. In the context of RPI for bridges it should be considered as a mean that a structure/bridge which is more appealing will bring e.g. more tourism in the area, causing economic development leading to other beneficial aspects.	1
Preservation of way of life	Qualitative	This performance indicator can influence the population growth (local or wider) since when way of life changes due to certain external influences one of the most often consequences is the outflow of people. If a new traffic route is opened which diverge traffic from a certain area or a bridge becomes unusable due to its bad condition economic circumstances of the affected area are changed.	1
Dumping	Quantitative / Qualitative	Dumping, in economics, is a kind of injuring pricing, especially in the context of international trade. It occurs when manufacturers export a product to another country at a price below the normal price with an injuring effect. The objective of dumping is to increase market share in a foreign market by driving out competition and thereby create a monopoly situation where the exporter will be able to unilaterally dictate price and quality of the product. In bridge management dumping is analysed in the process of construction or maintenance of a bridge.	1

B.8. RPI'S RELATED TO RESILIENCE

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B.8.1. THE CONCEPT OF RESILIENCE

Resilience is the capability of a system to anticipate a possible disruption to reduce failure probabilities; to resist, absorb and respond in an effective way to decrease the consequences of an incident; to perform recovery activities in order to mitigate the future disruptions (Bruneau, et al., 2003) (Cutter, et al., 2008) (NAS, 2012) (Faber, et al., 2017) (Faber, 2018). Generally, the system characteristics robustness, redundancy, resourcefulness and rapidity are associated with resilience. Robustness may be associated with the system ability to endure an unexpected disruption without severe loss of function/service. Redundancy is the ability to fulfil functional requirements in the disruptive event. Resourcefulness is the capacity to determine priorities as well as to be able to mobilize required material and human resources. Rapidity is the capacity to meet priorities in a timely manner (Bruneau, et al., 2003) (Giuliani , et al., 2016). These properties can be designated to event phases, e.g. robustness may be seen to be effective in the during event phase, rapidity is important in the after-event phase and resourcefulness is essential both in the before and after event phases.

Following the framework for the probabilistic modelling of systems by the JCSS (Faber, 2008), the exposure of a bridge might be associated with damages and failures caused to aging, earthquake loads and any operational or environmental loads acting on the structure. The indicators related to the exposure (Faber, 2008) might include use/functionality, location, environment, design life and societal importance. There are two types of consequences of exposure, namely direct and indirect consequences. Direct consequences are associated with the damages on the system which are induced by failures of the elements of the system. While indirect consequences are related with the functionality losses. The vulnerability and lack of robustness are associated with direct and indirect consequences respectively.

Indicators are the instruments providing specific information on the state or condition of the constituents of systems. As seen in Figure 3, indicators are used to provide information on direct and indirect consequences of hazardous events which may damage both environment and negatively affect safety. The knowledge and information gathered through indicators are also essential in decision making for built and organisational system.

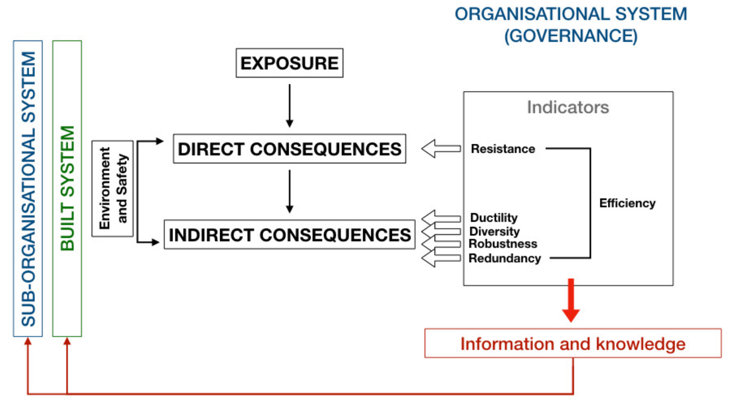


Figure 5. Representation of Resilient Organisational System at different levels of hazard scenario, adapted from (Faber, 2008)

Moreover, the information flow which provides information, data and knowledge to decision makers is demonstrated between structure and organisation, shown in Figure 4. Herein, Organisation includes both the operational management of bridge structures and first responders.

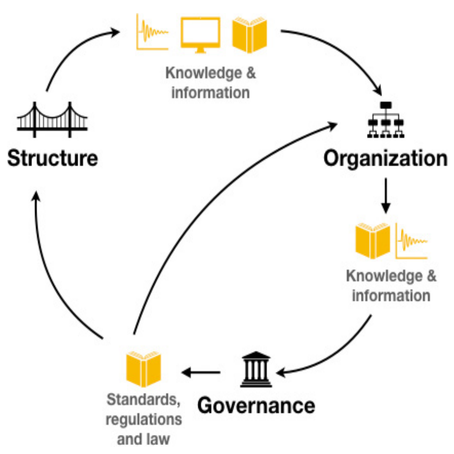


Figure 6. Figure 5 Representation of Resilient Organisational System at different levels of hazard scenario, adapted from (Faber, 2008)

Any observable and measurable property of the system or its constituents which contain information about the risk may be associated with the term risk indicators (Faber, 2008). Resilience indicators, which comprise information about the resilience level of a system, have been set by authors as redundancy and robustness, as in (Bruneau, et al., 2003) (Giuliani , et al., 2016); resistance and ductility as in structural engineering; efficiency and diversity to present the variety and complementarity properties of each system. The selected indicators differ for structure, information flow and organisation in each event phase.

However, to build a resilient system all indicators should be considered together for the structure (and for organization and information flow, in itself) in three event phases. Additionally, indicators should be taken account for the structure and organisation, together, in each event phase. In the following section the Partial Indicators of Resilience are described with reference to different capacities of the system: structural capacity, organizational capacity and information capacity.

B.8.2. PARTIAL INDICATORS OF RESILIENCE

Resistance is a system characteristic indicating a system's ability to meet ultimate demands with respect to service provision. The resistance of a system might be limited by physical and organisational constraints. Typically, a system's resistance is associated with the ultimate level of demands the system can meet. A system can be designed to fulfil criteria with respect to its resistance, e.g. in accordance with design codes.

- The resistance of a mechanical system might be characterized by its ultimate load carrying capacity.
- The resistance of an organisational system might be characterized by its maximum level of provision of intended services.
- The resistance of knowledge/information systems might be characterized by ultimate amount of data which can be transmitted between systems.

Redundancy is a system characteristic indicating a system's ability to meet demands by internal distribution and sharing of functions. The redundancy of a system might be limited by physical and organisational constraints. A system can be designed to meet criteria with respect to redundancy.

- The redundancy of a mechanical system might be characterized by its degree of static indeterminateness.
- The redundancy of an organisational system might be characterized by the number of organisational units which can provide the same functionalities and services.
- The redundancy of knowledge/information systems might be characterized by the availability of same data that can be provided with different techniques.

Ductility is a system characteristic, which indicates the system's ability to continue service provision after its ability to meet demands has been exhausted. A systems ductility might be limited by physical and/organisational constraints. A system can be designed to meet criteria for ductility.

- The ductility of a mechanical system might be characterized by its ability to carry load after reaching its ultimate load bearing capacity (resistance). If the loading at this stage is maintained or increased on the mechanical system this will cause increasing strains. The ductility of a mechanical systems might be limited by strain. A mechanical system which has infinite strain capacity is often referred to as perfect plastic or perfectly ductile. A mechanical system which has no strain capacity after reaching its ultimate load bearing capacity is referred to as perfect brittle and has no ductility.
- The ductility of an organisational systems might be characterized by its ability to maintain its operations and provide its intended services after the point where demands have reached or exceeded its capacity. The ductility of organisational systems might be limited by stress which in turn might depend on the time duration and level of exceedance of its capacity.
- The ductility of knowledge/information systems might be characterized by the ability to provide data/information after reaching to ultimate level of demands. The ductility of information systems can be limited with the available and accessible data against demands in timely manner.

Diversity is a system characteristic indicating the ability of a system to meet different demands and to meet demands by different means. The diversity of a system might be limited due to physical and organisational constraints. The diversity of a system can be designed to meet requirements to diversity.

- The diversity of a mechanical system might be characterized by its ability to carry loads of different sources, multiple natural hazards, etc. or to provide multiple alternatives for the distribution of internal demands.
- The diversity of organisational systems might be characterized by the number of different services provided and the number of provided alternative ways to which these services may be provided.
- The diversity of knowledge/information systems might be characterized by the numerous ways of gathering data and using this data in different scenarios.

Robustness is a system characteristic indicating a system's ability to provide of maintain service provision in situations of unintended or unexpected demands. The robustness of a system might be limited by physical and organizational constraints and a generally functions of capacity, redundancy, ductility and diversity. The robustness of a system can be designed to meet requirements.

- The robustness of mechanical systems might be characterized by its ability to reduce losses in case of accidents or provide services under unintended/unexpected demands.
- The robustness of knowledge/information systems might be characterized by its ability to reduce losses of data in case of accidents or provide transmission of information under unintended/unexpected demands.
- The robustness of organisational systems might be characterized by their ability to reduce loss of services in cases where parts of the systems have exceeded their capacities or provide services meeting unintended/unexpected demands.

Efficiency is a system characteristic indicating a systems ability to meet demands relative to the resources required by the system to do this. The efficiency of a system might be limited by physical and organisational constraint – and what is often referred to as best practices. A system can be designed to optimize efficiency.

- The efficiency of a mechanical system might be characterized by the costs and consumption of natural resources it implies to meet demands.
- The efficiency of organisational systems might be characterized by costs and time it requires to meet demands.
- The efficiency of an information system might be characterized by the costs and availability of data flow it implies to meet demands.

Each of these partial indicators describes an aspect of being resilience. They should be somehow combined to describe the resilient system. Even if several of these indicators may reach the highest level of maturity, their combination to quantify the resilience of a system is still the subject of investigations therefore the Indicator Readiness Level is herein assigned as **IRL=1**. (Faber, 2015) (Faber, et al., 2007) (Faber & Qin, 2016).

C. INNOVATION ON TECHNOLOGIES

To minimize intrusion in the transport flow, bridge inspection and monitoring methods should be non-destructive, minimally invasive. They should be capable of yielding rapid and accurate inspection results allowing an adequate response from the asset manager. Research aims at including autonomously operating equipment (e.g. robotics), non-intrusive (remote or proximity) observation techniques, or other methods that ensure quality and performance control of the roadway bridges in time, more safely, more quickly and/or to a higher degree of accuracy and precision. During the past few years, several European projects or actions (IMAC, COST action F3, COST action TU 1402, Sustainable Bridges, Arches, Bridgemon, Infrastar, Infravation…) or US projects as the FHWA's Long-Term Bridge Performance (LTBP) have dealt with innovations on technologies. Within these cooperative programs, different techniques, methods, sensor technologies, monitoring systems etc. have been developed.

This chapter investigates novel condition monitoring and sensing technologies for the assessment of structural serviceability and safety of existing structures. Advanced, integrated, cost-effective and reliable instrumentation solutions, techniques and concepts that can be used to compute innovative performance indicators are looked at (Orcesi, et al., 2019). First, one explores innovation in non destructive testing (section C.1). Second, one presents some example of innovative structural health monitoring solutions (section C.2).

The aim of the following sections is to summarize some innovations on technologies as an answer to the current needs of bridge owners. It is highlighted that (i) in its current form, the innovations on technologies presented in this chapter cannot be considered as exhaustive and this work should more serve as a starting point for future discussions and research activities, (ii) the maturity level rating of these technologies (qualitatively or quantitatively using TRL) has to be considered as a dynamic processes that need to be updated constantly based on new, or previously unknown, research outcomes.

C.1. INNOVATION IN NON DESTRUCTIVE TESTING (NDT)

Most NDT methods aim to achieve the highest quality of visual imaging of the relevant internal and/or external features of structures (Forde, 2010) (Forde, 2013). However, no single method can detect all types of defects in concrete structures as well as the traditional inspection combination of visual and sounding inspections (Hiasa, et al., 2018). (Hung, et al., 2009) classify common NDT techniques into seven major categories:

- visual (e.g., visual inspection using borescope (Allgaier, et al., 1993)),
- penetrating radiation (e.g., X-ray (Nagarkar, et al., 2002), and neutron imaging ((Michaloudaki, et al., 2005)),
- magnetic-electrical (e.g., magnetic particle (Lovejoy, 1993), and Eddy current (Udpa & Moore, 2002)),
- mechanical vibration (e.g., ultrasonic (Birks, et al., 1991)),
- acoustic emission (Miller & McIntire, 1987), and tapping (Birks, et al., 1991)),
- chemical/electrochemical (e.g., chemical spot testing),
- thermal (e.g., infrared thermography (Maldague, 1993) and other optical methods (e.g., Moire´ interferometry, holography, and shearography (Hung, 1996), (Hung, 1999), (Hung, et al., 2000), (Hung, et al., 2000)).

Research efforts are being directed at developing and perfecting NDT techniques capable of monitoring (1) materials production processes, (2) material integrity during transportation, storage and fabrication and (3) the amount and rate of degradation during service (Hung, et al., 2009).

The following sections present NDT techniques, some of them already conventional, and other still under development with need for innovative solutions.

C.1.1. NON-DESTRUCTIVE TESTING FOR REINFORCED CONCRETE

Contributors: Géraldine Villain, Odile Abraham, Xavier Derobert

C.1.1.1. RELATIONS BETWEEN INDICATORS AND MEASUREMENTS – GRADIENTS

NDT techniques are more and more used for the quantitative Non Destructive Evaluation (NDE) of concrete mechanical, physical and chemical durability indicators for mechanical characterization, durability diagnosis and degradation monitoring. The advances and recommendations for concrete NDE can be found in (Balayssac & Garnier, 2017) (Balayssac & Garnier, 2018). The ultrasonic, electromagnetic and electrical methods are described and recommendations about accuracy and statistical evaluation are given.

C.1.1.1.1. CAPACITY PROBES

Capacitive probes were developed in the 70's and used for permittivity measurements then water content estimation (Dérobert, et al., 2008). As several electrode sets of different sizes can be applied on the concrete structure surface, the apparent permittivity of different concrete volumes can be evaluated. Then, after inversion of the results and calibration, the water content profiles can be evaluated (Fares, et al., 2016).

For capacitive probes as well as electromagnetic methods, let us note that the calibration on cores of the same concrete mix design is mandatory to evaluate precisely the water content because the components properties influence the measured permittivity or resistivity (Villain, et al., 2018).

C.1.1.1.2. USE OF ELECTROMAGNETIC WAVES IN THE RADIOFREQUENCY RANGE

The monitoring and durability diagnosis of hydraulic concrete structures by means of NDT has become an increasingly important topic (Ihamouten, et al., 2012). Water penetrating into the concrete pore network contributes to the transfer of degrading and corrosive agents such as chloride. Electromagnetic (EM) waves in the radiofrequency range are found to be sensitive to concrete and mortar mix design, water content (WC), porosity, chloride, etc. Some results have been recently presented from an experimental study conducted on concrete slabs (and corresponding core cylinders) in a controlled laboratory environment (Villain, et al., 2015), (Xiao, et al., 2017).

C.1.1.1.3. SURFACE WAVE MEASUREMENTS

Mechanical and chemical attacks can degrade concrete starting at its surface. Such degradations include microcracks and porosity increases. Surface waves offer an effective means for evaluating in a non-destructive manner the mechanical properties of cover concrete and thus serve to tackle the problem of assessing in situ durability Indicators (Villain, et al., 2009). A variation in water content with depth will also impede SW propagation. (Abraham, et al., 2012) presented two automated devices for conducting non-contact ultrasonic measurements (with air coupling and laser interferometry) for the purpose of recovering mechanical properties (porosity) on a series of concrete slabs using surface waves. Surface wave amplitude measurements have also been applied to concrete in order to characterise surface opening cracks (Song, et al., 2003) using the crack filtering effect on SW: the crack affects high frequencies travelling near the surface differently from low frequencies that are less affected by the crack and that travel below it (Goueygou, et al., 2008).

C.1.1.1.4. COMBINATION OF NDT DATA

The idea behind data fusion is to combine information from multiple sensors to improve overall performance of damage detection and quantification. (Sbartaï, et al., 2012) mention that the measurement of NDT physical parameters such as velocity of ultrasonic waves, electrical resistivity or GPR (Ground Penetrating Radar) wave attenuation is disturbed by uncertainties due to various causes: (a) The lack of accuracy and repeatability of the measurement process at a given point of the tested material, (b) The variability of the material at different scales: in a limited volume assumed to be homogeneous, between samples of the same concrete mixture, or between two successive batches of the same mixture, (c) On site, variability can also be induced in an originally homogeneous concrete by some contrast in environmental exposure conditions such as a variation of moisture or temperature.

To directly evaluate one concrete property, (Sbartaï, et al., 2012) indicate another notable problem when using only one NDT technique. For instance, GPR technology is sensitive to water saturation but also slightly sensitive to porosity (Sbartaï, et al., 2006), ultrasound is able to evaluate the modulus of elasticity but it is also sensitive to moisture and density (Ohdaira & Masuzawa, 2000), (Popovics, 2005), and so on. For these reasons, some researchers have proposed combining several techniques for concrete strength evaluation (Kheder, 1999), (Soshiroda, et al., 2006), (Qasrawi, 2000), (RILEM Draft Recommendation, Essais Non Destructifs Combines, DU Beton, 1993) or for detection and visualization in concrete structures (Kohl, et al., 2005), (Kohl & Streicher, 2006), or the combination of several NDT parameters obtained with the same technique (Sbartaï, et al., 2012), (Zaid, et al., 2004), (Hugenschmidt & Kalogeropoulos, 2009)) in an attempt to confirm the diagnosis or to reduce the measurement noise. In particular, (Villain, et al., 2012) have developed and checked a data fusion model based on 2D cross-correlation models by comparison with results obtained on the same slabs. These results have shown that a combination of techniques based on different kinds of wave propagation (GPR and impact-echo) and different frequency ranges (capacitive and GPR techniques) serves to reduce uncertainty in durability parameter evaluation.

C.1.1.1.5. ELECTRICAL RESISTIVITY TOMOGRAPHY

Electrical resistivity is very sensitive to water content and is recommended by RILEM (Polder, 2001). It is also advised to use this NDE method in combination with surface concrete permeability estimation (Andrade, et al., 2007), (Torrent & Fernández Luco, 2007), (Bonnet & Balayssac, 2018). Electrical resistivity tomography was developed for geotechnical and geophysical mapping (Loke & Barker, 1996). More recently, measurement systems were adapted to concrete and inversion was developed for imaging the concrete resistivity 1D or 2D distributions (Du Plooy, et al., 2013), (Du Plooy, et al., 2015), (Lecieux, et al., 2015). Thanks to calibration, the gradients of water content or chloride content in saturated concrete can be evaluated (Fares, et al., 2018) (Figure 7).

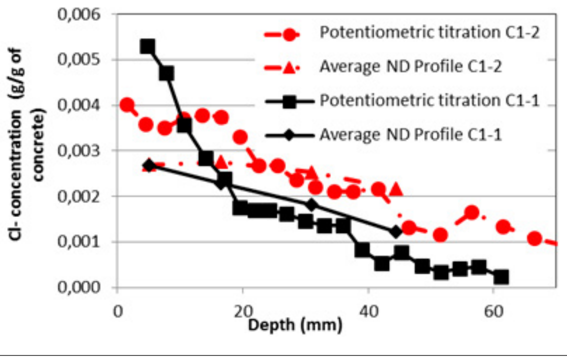


Figure 7. Evolution of profiles of free chloride content for concrete C1 after 18 weeks (slab C1-1) and 40 weeks of chloride diffusion (slab C1-2) evaluated by ERT or by destructive titration (Fares, et al., 2018).

C.1.1.1.6. IMPACT ECHO

This method is based on the analysis of the wave spectrum of a semi-infinite concrete structure with parallel faces. If the slab thickness is known, the compressive wave velocity can be evaluated (Sansalone & Carino, 1988), (Sansalone & Streett, 1997). More recently, (Gibson & Popovics, 2005) show that the empirical coefficient used by Sansalone is related to the Poisson's coefficient. So, it makes it possible to evaluated the dynamic Young's modulus and Poisson's coefficient of concrete. After calibration, global compressive strength, static Young's modulus and concrete porosity can be evaluated on slabs (Villain, et al., 2011) or on real structures (Villain, et al., 2012). As the water content influences the results, it is recommended to evaluate the concrete moisture conditions (water content or degree of saturation) and mechanical properties in the same time.

C.1.1.2. DURABILITY OF REINFORCED CONCRETE

C.1.1.2.1. ULTRASONIC PULSE ECHO

Ultrasonic techniques can be used to detect faults in concrete such as delaminations and voids. However, traditional ultrasonic techniques cannot be used when access to structures is limited to a single side. Moreover, traditional ultrasonic techniques do not provide depth information about the location of internal defects, unless a huge amount of measurements are made and sophisticated software is used to create tomograms. (Corbett, et al., 2018) proposed an ultrasonic pulse echo technique that enables users to collect this information even when access is limited to a single side. Recent research carried out has sought to use intelligent algorithms to improve speed and precision of large-scale scanning, and also to rely on artificial intelligence for object recognition in a scan. Furthermore, the method proposed by (Corbett, et al., 2018) uses information collected with other technologies to increase depth accuracy.

C.1.1.2.2. HIGH DEFINITION IMAGE-BASED TECHNOLOGIES USING INFRARED THERMOGRAPHY

Among several NDE technologies, image-based bridge inspection technologies, consisting of high-definition (HD) imaging and infrared thermography (IRT), are promising candidates to conduct rapid and in-depth bridge inspections by attaching these cameras on vehicles, such as cars and unmanned aerial vehicles (UAVs). Actually, the combination of IRT and HD imaging system has been applied for some bridges at normal driving speeds for several years, although the ASTM standard suggests an IRT data collection speed of no greater than 16 km/h (ASTM, 2014). Capturing the temperature difference between sound and defective parts under ambient conditions is key for infrared thermography (IRT) on concrete bridges. Effective utilization of the IRT on civil engineering structures also depends on the application time window on structures (Hiasa et al. 2018).

C.1.1.2.3. CODA WAVE INTERFEROMETRY (CWI)

The idea behind CWI is to take benefit of the multiple diffusion of ultrasonic waves due to the heterogeneity of concrete (when the wavelengths have a size similar to that of the aggregates) and/or of multiple reflections at the structures boundaries (Abraham, et al., 2018). The late part of the ultrasonic signal, which is called the coda, looks like noise but it is repetitive and very sensitive to small changes in the material due to the long path ultrasonic waves have travelled. In the literature, applications of CWI in small scale concrete specimen, typically meter size or smaller, are reported (Planès & Larose, 2013). Some CWI sensors have recently been successfully installed inside a 25 meters long pre-tensioned reinforcement concrete beam in the frame of the BAM thematic project BLEIB on bridge monitoring (Wang & Niederleithinger, 2018). In this experiment conducted within the European Union's Horizon 2020 Infrastar (http://infrastar.eu), the dynamic test showed that CWI method has a high sensitivity (10-3 %) in wave propagation velocity change. The static test showed that CWI method is sensitive to load effect. When the loads pass through an area, before the cracks appear, the bending tensile stress increased and the velocity change decreased, on the contrary, in the area where is on the opposite side of the structure, the compression stress increased and velocity change increased (Wang & Niederleithinger, 2018) (Figure 8).

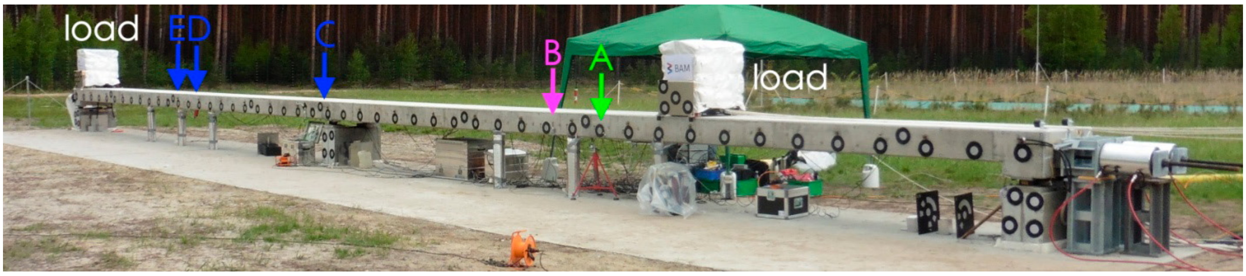


Figure 8. BLEIB structure tests using CWI sensors (Wang & Niederleithinger, 2018).

C.1.1.2.4. RADAR TECHNIQUE

The radar technique (also referred to as Ground Penetrating Radar – GPR) is well known for its ability to detect and localize steel reinforcements in concrete (Lai, et al., 2018), however in recent years, it has been shown that this technique can also be used for the characterization of concrete; primarily its water content ((Klysz & Balayssac, 2007); (Dérobert & Villain, 2016)). For different positions of the antennas, the transmitter radiates electromagnetic pulses which propagates into the surveyed medium. The receiving antenna records the successive echoes coming from embedded objects (or layers) in time signals.

Some calibration is required to obtain the EM velocity of the medium to transform travel times into depth units. As the EM velocity and attenuation are mainly water (and/or chloride) content, GPR measurements can be indicative of moisted areas.

C.1.1.2.5. DEVELOPMENT OF ROBOTIC SYSTEMS FOR NDE OF CONCRETE STRUCTURES

Contributor: Ali Maher

The Federal Highway Administration's (FHWA's) Long-Term Bridge Performance (LTBP) program developed a new automated bridge inspection tool called RABIT™ (Robot-Assisted Bridge Inspection Tool™) (Figure 7). (Gucunski, et al., 2015) describe its development and implementation for data collection using multiple NDE technologies. The system is designed to characterize three most common deterioration types in concrete bridge decks: rebar corrosion, delamination, and concrete degradation. It implements four NDE technologies: electrical resistivity (ER), impact echo (IE), ground-penetrating radar (GPR), and ultrasonic surface waves (USW) method. The technologies are used in a complementary way to enhance the interpretation. In addition, the system utilizes advanced vision to complement traditional visual inspection. Finally, the RABIT collects data at a significantly higher speed than it is done using traditional NDE equipment. The robotic system is complemented by an advanced data interpretation (Gucunski, et al., 2015).

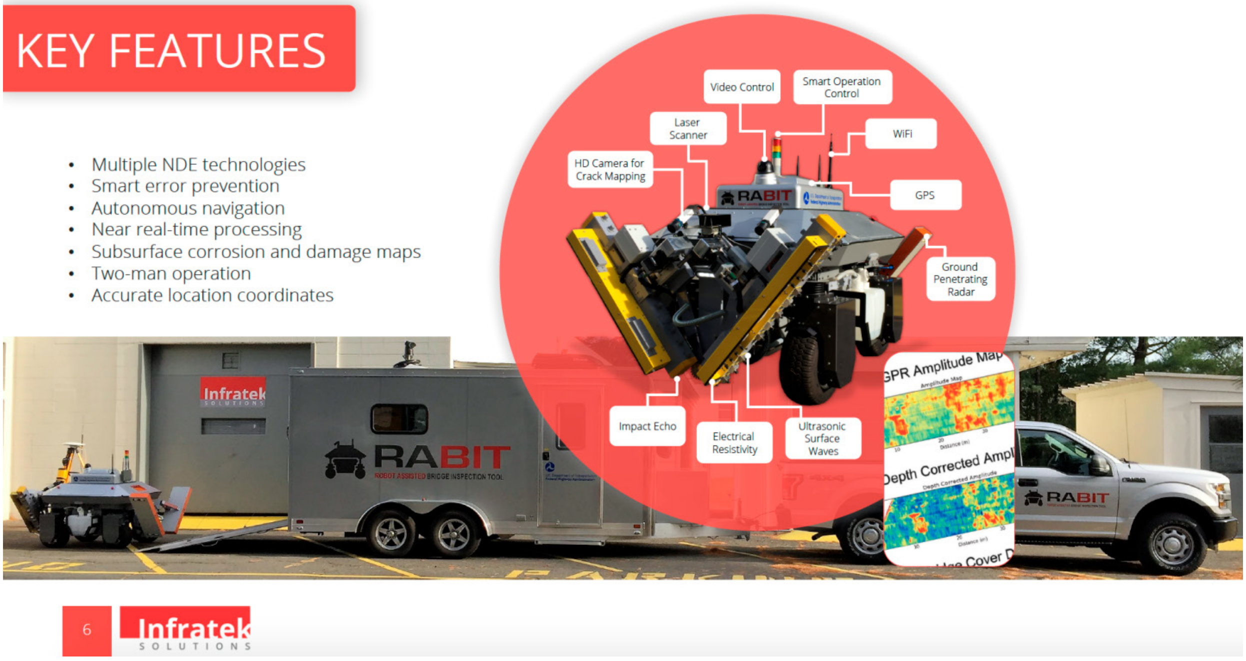


Figure 9. Key features of the RABIT™ robotic system.

C.1.2. NDT FOR BRIDGE CABLES, ROPES AND PRESTRESSED CONCRETE ELEMENTS

Contributors: Laurent Gaillet, Bruno Godart, François Lenoir

C.1.2.1. GAMMA- OR X-RAY RADIOGRAPHY FOR DETECTION OF VOIDS IN TENDON DUCTS

The detection of voids in grouted tendon ducts in concrete structures is crucial. Commonly used for decades, the gamma- or X-ray radiography technique provides a reliable diagnosis of the quality of the grouting (Abraham & Côte, 2002). In some cases, one can detect anomalies like abnormal deformations, wire or strands failures, as well as distensions of wires (Dérobert, et al., 2002). This powerful technique unfortunately has the disadvantage of requiring a large security zone for radiation protection of the operator as well as the people in the vicinity (Abraham & Côte, 2002). Besides, it is an expensive technique for a relatively limited length of tested tendon. The examinations are local (30 cm × 40 cm picture) and need an important gamma-ray exposure duration which is a function of the thickness of the investigated structure. The numerical radiography is developing.

C.1.2.2. IMPACT ECHO TECHNIQUE AS AN ALTERNATIVE TO GAMMA- OR X-RAY RADIOGRAPHY?

Since the end of the 1980s, the impact-echo method has also been proposed ((Abraham & Côte, 2002) (Abraham & Côte, 2002) (Abraham & Côte, 2002), (Dérobert, et al., 2002)(Dérobert, et al., 2002)(Dérobert, et al., 2002), (Abraham, et al., 2000)(Abraham, et al., 2000)(Abraham, et al., 2000)). It is based on a frequential analysis of the time response of a structure excited by a shock. Shifts in the frequencies due to changes of the duct depth and of the slab thickness have been used as indicators of voids in ducts (Abraham, et al., 2000)(Abraham, et al., 2000)(Abraham, et al., 2000). The advantages of impact echo lie in the simplicity of experimental set-up that requires access to only one side (Dérobert, et al., 2002)(Dérobert, et al., 2002)(Dérobert, et al., 2002).

Two major phenomena are indicative of the presence of a void in the impact echo method: a decrease of the thickness resonance frequency f_e and the apparition of a higher frequency, named f_{void} in the literature. The first observable alone might be in a number of case misleading since a frequency shift has been sometime observed in tendon that are diagnosed as fully grouted by gammagraphy. The addition of the second observable (f_{void}) that confirms the existence of a void can ascertain the diagnosis (Abraham, et al., 2009)(Abraham, et al., 2009)(Abraham, et al., 2009).

Some device is commercialized for void detection in prestress ducts but still requires some research developments (Dérobert, et al., 2002). In particular, the high frequency content of the impact echo signal is difficult to record, first because sending energy in this frequency range is difficult, second due to the existence of scattering and damping, and third because traditional contact transducer is not optimal in this frequency range. (Abraham, et al., 2009) recently considered the use of a laser interferometer which has a large bandwidth and which offers the possibility to record simultaneously both observables.

C.1.2.3. THE CAPACITY PROBE FOR DETECTING VOIDS AND WHITE PASTE IN EXTERNAL HDPE DUCTS

This technique was developed to detect voids or the presence of white paste in an external tendon in a HDPE duct filled by a cement grout (Bore, et al., 2009). The device consists of two electrodes constituting a capacitor whose capacity depends on the dielectric characteristics of the materials existing inside the HDPE duct. It allows detecting grouting defects. Its depth of investigation is between 1 and 3 cm. The device is light to handle along the duct, but the technique cannot detect the corrosion of strands or the presence of liquid inside the duct.

C.1.2.4. THE US TOMOGRAPHY FOR DETECTING VOIDS IN PRESTRESSING TENDONS

This technology (Terzioglu, et al., 2018) consists of simultaneously sending and receiving ultrasonic waves through a material, using several transmitters/transducers at frequencies ranging from 25 to 75 kHz. A software integrated in the device analyses the travel times of these waves making possible the detection of heterogeneities in the tested material (Figure 10). The restitution is a coloured cartography visualizing the different zones. This method is effective for the detection of voids in prestressing cables and as such can be an alternative to the use of radiography when it is too restrictive.

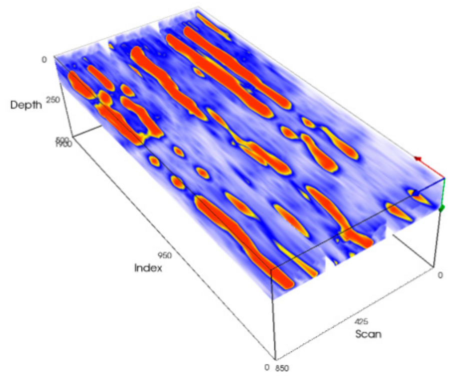


Figure 10. Example of images obtained using Ultrasonic Tomography

C.1.2.5. ELECTROMAGNETIC TECHNIQUES TO DETECT CORROSION OF CABLES

Electromagnetic method uses magnetic inductance scanning. Cable elements, being metallic, are highly susceptible to electromagnetic fields. In this method, a strong magnetic field is generated around the steel cable with the help of an electromagnetic coil. Corrosion defects are changing the boundary conditions of the field and therefore causing a field disturbance that can be detected by the device (Zahn & Bitterli, 1995).

C.1.2.6. MAGNETIC FLUX LEAKAGE (MFL) FOR BROKEN WIRES DETECTION

Magnetic flux leakage is a NDT method used to detect defects and corrosion in steel structures, especially for stay cables with large diameters. The basic principle is that a powerful magnet is used to magnetize the steel (Xu, et al., 2012). At areas where there is corrosion or missing metal, the magnetic field "leaks" from the steel. By placing a magnetic detector between the poles of the magnet, one can then detect the leakage field and, indirectly, the flaws of the cables. However, MFL method can only detect the local fault near the surface: hidden damage at the inside of large cable will not be detected (Kim, et al., 2014).

C.1.2.7. ULTRASONIC AND ACOUSTIC TECHNIQUES FOR INACCESSIBLE AREAS

Among the major suspension bridge components, cables and anchorage zones are critical locations. Indeed, cables are primarily undergoing environmental aggressions and the anchorage zone, often an inaccessible part of the bridge located at the bottom of the retaining cables, provides an ideal location for water accumulation that promotes the development of corrosion (Kharrat & Gaillet, 2015).

The acoustic emission (AE) technique is based on the release of stored elastic energy as elastic waves due to sudden micro-fracturing in a rigid body. In the last decades several developments focused on the detection of wire rupture of suspension and stay cables and prestressing tendons ((Sluszka, et al., 2006), (Kurz, et al., 2013), (Zejli, et al., 2006)). The ultrasonic guided waves method is another technique, particularly suitable for anchorage zones. It is used to inspect each individual wire of strands and allows detecting net breaks of wires close to the anchorage area, along the guiding direction. It is potentially attractive for the inspection of anchorages embedded in massive concrete blocks. This technique is already used in industrial applications and is continuously improved by an active community in the world ((Kurz, et al., 2013), (Laguerre, et al., 2004), (Laguerre & Treyssède, 2011), (Laguerre, et al., 2018)).

The Acousto-Ultrasonic technique is relatively new in civil engineering for defect detection. This technique combines aspects of acoustic emission signal analysis with ultrasonics assessment methods. The transmitter transducer generates a specific ultrasonic waveform that propagates through the specimen before being collected by a receiver transducer. Signals resulting from multiple reflections and interactions with the microstructure of the material are treated in the same way as acoustic emission signals. (Kharrat & Gaillet, 2015) compared the Acousto-Ultrasonic technique with classical ultrasonics testing method for damage detection in anchorage zones. The former is useful for evaluating the global structural health of the anchorage zones, whereas the latter would be rather suitable for detecting local and crack-like defects.

C.1.3. NON-DESTRUCTIVE CRACK DETECTION METHODS OF WELDED STEEL STRUCTURES

Contributors: Marjorie Bourquencier, Bruno Godart, Christian Tessier, André Orcesi

For steel bridges, the weld joints generally constitute the most critical sites which determine the overall fatigue resistance. The unfavorable situation at the weld joint is caused by the interaction of several negative factors as high stress concentrations at the weld toe, complex residual stresses and flaws in the weld seam (Ummenhofer & Medgenberg, 2009). The strong localization of the early damage processes and the inhomogeneous properties of the weld seam pose significant problems with respect to the experimental quantification and analysis of the damage evolution. This section describes the development of sensitive techniques, some of them being considered as already "conventional" (commonly used for welded joint inspections) and others still under development for detection of defects of steel welded bridges. Most of them need special equipment and highly trained inspectors. They are very time consuming and cannot be applied to a whole structure, but only on some aimed zones (the inspection must focus on the most critical details).

C.1.3.1. CONVENTIONAL METHODS

Conventional non-destructive detection methods (excluding visual examination) of welded joints are as follows: liquid penetrant (bleeding), magnetic crack detection, Eddy current, ultrasonic and radiography based methods, as described below (MIKTI, 2010).

C.1.3.1.1. BLEEDING (STANDARD NF EN 571-1)

The liquid penetrant method consists of introducing a colored or fluorescent impregnation liquid on the surface to be inspected. The liquid penetrates inside small cracks or discontinuities in the material. Once the cavities have been filled, excess liquid present at the surface is eliminated by cleaning. This action leads to breaking the capillary equilibrium created between liquid and solid. The liquid contained in the cavities tends to rise to the surface until capillary equilibrium has been restored. This liquid rise serves to visually detect the existence of a "large-sized" crack; small cracks typically remain invisible to the naked eye. For this reason, a so-called "developer" liquid is systematically used for the purpose of increasing method sensitivity. When applied, the developer forms a very thin film that acts like a "sponge" and absorbs the penetrating liquid contained in the imperfections. This absorbed liquid spreads over the developer film and enhances defect visibility. This technique is often used for control during construction and can detect only surface emerging defects, so coating must be removed before use. It is very simple to apply and cheap.

C.1.3.1.2. MAGNETIC PARTICLES CRACK DETECTION (STANDARDS NF EN 1290 AND 1291, A 09-590);

The magnetic particles detection method enables localizing surface discontinuities as well as a number of internal discontinuities lying close to the surface of ferromagnetic elements (MIKTI, 2010). Magnetic control consists of submitting the part (or a portion of it) to a magnetic field. The existence of a discontinuity within the inspected zone causes local interruption or distortion in the field, which in turn leads to the appearance of magnetic « leaks » (known as the point effect). These zones (and hence the discontinuities) are identified thanks to the use of very fine ferromagnetic particles applied onto the surface. Discontinuities within the magnetic field attract and then retain the particles. This group of magnetically-held particles indicates the localization, shape and dimensions of the discontinuity.

This technique is highly sensitive and serves to locate very small surface cracks invisible to the naked eye. It also offers the possibility of detecting internal discontinuities, provided they lie close to the surface (about 3 mm). The magnetic crack detection method is capable of detecting the following types of discontinuities: cracks; absence of weld penetration; lack of weld fusion; excessive porosity at the surface; gas pockets located near the surface. Generally, there is no need to remove the paint. This method however displays the following limitations:

- use restricted to ferromagnetic materials (it does not work on steel having more than 13 % of Chromium),
- steel must be cleaned before the test (paint, corrosion,...),
- requirement that the magnetic field lie in a direction intercepting the principal discontinuity plane, which might necessitate performing more than one magnetization sequence,
- the risk of « burning » surfaces at the electrical contact points,
- the potential need for demagnetization following inspection.

C.1.3.1.3. EDDY CURRENT

Eddy current testing (Figure 11) is one of the most extensively used non-destructive techniques for inspecting electrically conductive materials at very high speeds that does not require any contact between the test piece and the sensor (García-Martín, et al., 2011).

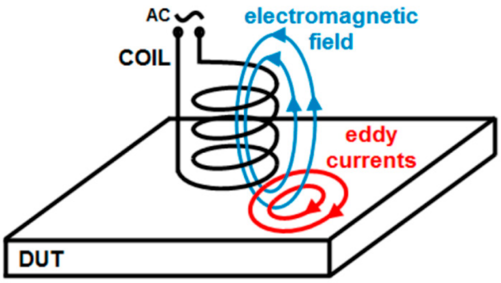


Figure 11. Eddy Current principle

The principle of the eddy current technique is based on the interaction between a magnetic field source and the test material (Eddy current generated by an applied magnetic field). This interaction induces eddy currents in the test piece (Janousek, et al., 2008). Scientists can detect the presence of very small cracks by monitoring changes in the eddy current flow (magnitude and phase of the induced current) (Hashizume, et al., 1992).

Eddy current testing permits crack detection in a large variety of conductive materials, either ferromagnetic or non-ferromagnetic, whereas other non-destructive techniques such as the magnetic particle method are limited to ferromagnetic metals. Another advantage of the Eddy current method over other techniques is that inspection can be implemented without any direct physical contact between the sensor and the inspected piece (García-Martín, et al., 2011).

One of the advantages of this technique is that no paint removal before test is mandatory.

It is noted that Eddy current testing is based on inducing electromagnetic currents in the object being inspected while observing the interaction between those currents and the object. Therefore, credible Eddy current testing requires a high level of inspector training and awareness (Droubi, et al., 2017).

C.1.3.1.4. ULTRASONIC (STANDARDS NF EN 1712 - 1714, STANDARD NF EN 583 - PARTS 1 TO 6, THE IS US 319-21 DOCUMENT PUBLISHED IN JUNE 1995);

Inspection using ultrasound is a method that enables detecting surface and internal discontinuities in both ferromagnetic and non-ferromagnetic materials; it enjoys widespread use, especially for weld verification. This method can also be used to determine the thickness of metallic parts.

This method is based on introducing, via an emitter, a beam of high-frequency sound waves into the component under inspection (MIKTI, 2010). The sound waves travel through the component and get reflected by the interfaces. The degree of wave reflection depends primarily on the physical state of the materials composing the interface and, to a lesser extent, on the wave characteristics. The cracks, cavities, intrusions, porosities and other discontinuities are what produce these reflective interfaces.

The reflected wave beam can then be analyzed to detect the presence of the targeted defects. Inspection by ultrasound is performed at frequencies between 0.1 and 25 MHz (i.e. well beyond the range of human perception, which extends from 20 Hz to 20 kHz). In the case of steel, the amplitude of vibrations induced during an ultrasonic inspection produces very low stresses that generate no damage.

This technique offers the following advantages:
- very high power of penetration, which serves to identify discontinuities located deep inside the material,
- strong sensitivity, which enables detecting minutely-sized discontinuities,
- greater accuracy than other non-destructive inspection methods in determining discontinuity position, dimension, orientation and origin,
- need for just a single element surface to be accessible for inspection,
- instantaneous output of results,
- no risk to personnel adjacent to the controlled zone.

Nonetheless, the technique does present a number of limitations, namely:
- the need for highly-qualified personnel,
- the difficulty involved in inspecting zones that are either rough in texture, irregularly shaped or too thin or constituted by a superposition of several steel plates,
- the need for prealable knowledge of the damage type assessed and its global location,
- the need for paint removal before test.

C.1.3.1.5. RADIOGRAPHY (STANDARDS NF P 22-471, EN 1435, EN 444).

RRadiographic inspection is a non-destructive control method that relies on the penetrating power of electromagnetic radiation inside the inspected element; it enables detecting and characterizing both internal and surface discontinuities of metallic and non-metallic materials (MIKTI, 2010). For a long time, radiography enjoyed the most widespread use among non-destructive control procedures and today remains one of the main techniques.

Radiography basically depends on the radiation absorption capacity by the inspected component. This capacity is correlated with material composition, its thickness, and the existence or not of discontinuities inside the material. When a radiation beam passes through the targeted element, a portion of the radiation energy is absorbed by the element, resulting in lower beam intensity. The beam intensity variations (due to absorption variations within the material) are then recorded on a film sensitive to unabsorbed radiation crossing the component.

The technique offers the following advantages:
- a very high penetrating power, which enables locating discontinuities placed deep inside the component,
- the capacity to detect variations in component composition,
- detection of a large number of defects associated with poor welding practices,
- generation of a permanent register of control records available for consultation, in order to verify a posteriori the initial conclusions.

This method however also displays the following drawbacks:
- high cost,
- lower method sensitivity in the detection of surface discontinuities (i.e. very small discontinuities are not detectable),
- impact of discontinuity orientation on method sensitivity,
- need to halt all other activities in the vicinity of the targeted control zone in order to avoid the risk of radiating personnel,
- both element sides need to be accessed,
- on site, this method needs restriction or stop of traffic (radiation protection).

C.1.3.2. UNCONVENTIONAL METHODS

C.1.3.2.1. TOFD (TIME OF FLIGHT DIFFRACTION)

As mentioned by (Yeh, et al., 2018), the Ultrasonic Time-of-Flight Diffraction (TOFD) was first reported by (Silk & Lidington, 1975) as a method that focus on diffracted waves, bringing many advantages in flaw assessment towards conventional ultrasonic techniques, based mostly on reflected waves (Figure 12).

Over the years the conventional TOFD technique proved to be a sensitive and accurate method for through thickness sizing of discontinuities such as weld defects, absence of penetration and fatigue cracks for a variety of materials, geometries and applications (Yeh, et al., 2018) (Figure 12).

Figure 12. TOFD equipment and results

Main advantages are as follows:
- fast process,
- strong sensitivity,
- reliable results unsensitive to operator skills,
- recording of test results,
- determination of flaw depth and height (more difficult or even impossible with conventional US techniques),
- access to the thickness of the specimen.

The technique does present a number of limitations, namely:
- difficult to locate flaws transversely,
- only adequate for butt joint welding,
- difficult to test welds of paint elements or under another kind of anticorrosion system,
- the need for highly-qualified personnel.

Although the conventional TOFD technique is powerful, this technique is restricted due to inspection targets' shape complexity and ultrasonic noise. The conventional TOFD weaknesses then inspired the development of alternative TOFD techniques. Among them are the "one-skip" TOFD method (Ido, et al., 2004), the "TOFDW" method (Chi & Gang, 2013), shear wave TOFD (S-TOFD) (Baskaran, et al., 2006) and immersion TOFD (I-TOFD) (Subbaratnam, et al., 2011). (Yeh, et al., 2018) recently presented a new method based on a specific mode-converted diffraction scheme, effective to measure defects located near to the inspection surface, bottom edges of internal cracks and cracks under compressive stress. Also, advances in post-processing of signals improved reliability of the technique. Examples include methods for de-noising weak diffractions, enhancement of signal time resolution, automatic pattern recognition and classification with images and Artificial Neural Network (Yeh, et al., 2018).

C.1.3.2.2. PHASED ARRAY

An ultrasonic phased-array probe (Figure 13) is composed of multiple elements, usually between 32 and 128, each of which can act as a single ultrasonic transducer. The pattern in which the elements may be arranged offer a variety of options, the simplest of which is a linear array. The ultrasonic wavefronts can be excited by pulsing the elements individually or as a group. The combination of these wavefronts generates the beam profile. The beam profile can be modified by varying the amplitude and timing of the excitation of each element. The focal laws are used to control the amplitude and time delay for each element.

Advantages: due to imaging by B-, C-, S- and L-scans the evaluation and documentation is easy according to (Deutsch & Kierspel, 2012). The inspection speed is high because of fast scanning in combination with electronic beam steering. A connected position encoder simplifies the defect localization. The continuous data recording enables a later interpretation of the results with a PC.

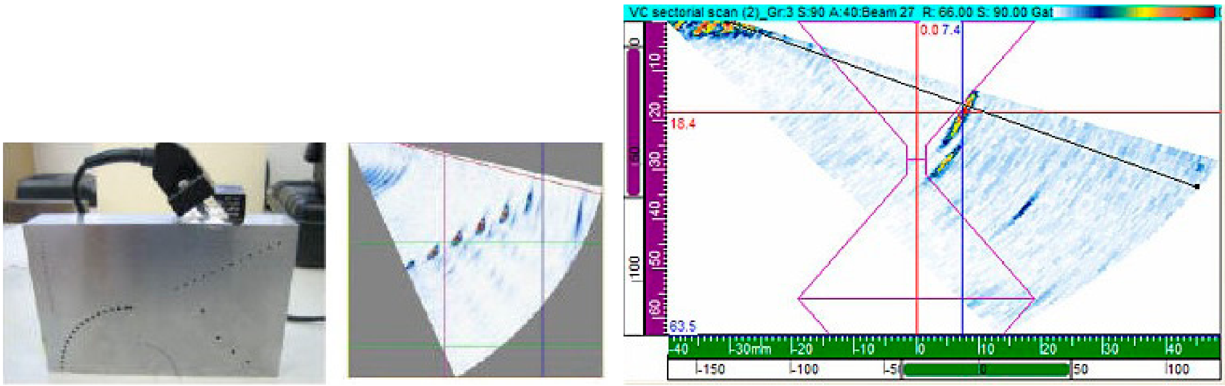


Figure 13. Phased array technology

Drawbacks: the setup for a phased array system is considerably more difficult, which may result in any errors. The larger phased array probes can cause coupling and space problems. A careful preparation and probe positioning is required in order to benefit from coordinate related imaging. Standards for the application of phased arrays and acceptance criteria are still under development. Special trainings of the operators are necessary and finally the start-up investment as well as the operating costs (for instrument, probes and wedges) are higher compared to the application of conventional equipment.

C.1.3.2.3. ACFM (ALTERNATING CURRENT FIELD MEASUREMENT)

The alternating current field measurement (ACFM) technique has been widely used in the oil, gas, mining and petrochemical industries to detect cracks and size surface breaking defects in a wide range of structural materials (LeTessier, et al., 2002) (Figure 14).

ACFM is an electromagnetic inspection method that does not require electrical contact with the surface under inspection. ACFM inspection can be carried out through coatings of up to 10 mm in thickness. Generally ACFM requires little or no surface preparation prior to inspection.

Figure 14. ACFM technique

The technique involves the use of a hand-held probe containing two magnetic field sensors and an electric current induction system. The probe is connected directly to the ACFM system electronics, with all control and data collection/storage provided by a control PC.

(LeTessier, et al., 2002) report the significant advantages of ACFM over traditional NDT techniques:
- ability to test through coated materials,
- reduced requirement for pre-cleaning,
- full data records are kept, allowing data to be reviewed by another operator,
- no instrument calibration is required,
- significantly faster than other conventional NDT methods,
- ability to test materials at elevated temperatures,
- detection and sizing in one instrument.

They also detail the following limitations of ACFM technique:
- sensitive to surface breaking defects only,
- depth sizing models are based on isolated semi elliptical defects,
- may provide misleading results when testing within areas containing multiple defects,
- probes are sensitive to gross geometry changes,
- sensitivity reduces with increasing coating thickness,
- test specimen must be of an electrically conductive material,
- high cost.

C.1.3.2.4. EMAT (ELECTRO-MAGNETIC-ACOUSTIC TRANSDUCERS)

Electro-magnetic-acoustic transducers (EMATs) consist of a magnet and coil and do not require mechanical contact with the specimen under test ((Isla & Cegla, 2017), (Jian, et al., 2006)). Electro-magnetic acoustic transducers (EMATs) generate ultrasonic waves in metals through an electromagnetic coupling mechanism ((Shujuan, et al., 2010), (Noorian & Sadr, 2010), (Aliouane, et al., 2000)). EMATs are widely used in non-destructive evaluation (NDE) to speed up the inspection process because ultrasonic couplant is not required and coatings thinner than a couple of millimetres do not need to be removed (Figure 15Figure 15). However, EMATs produce less intense signals in comparison to piezoelectric elements and therefore more attention to their design optimization is required as well as the use of signal processing techniques, such as pulse-compression, to increase the signal-to-noise ratio (SNR) (Isla & Cegla, 2017).

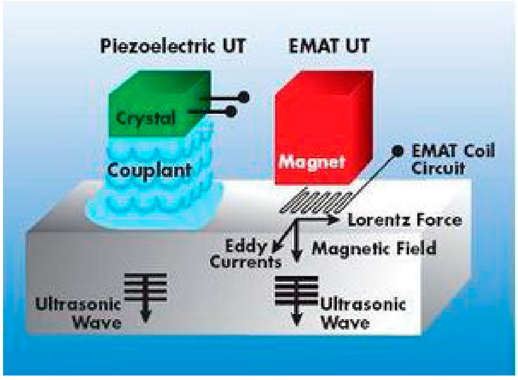


Figure 15. EMAT principle

C.1.3.2.5. TOWARDS EMAT PHASED ARRAY?

(Isla & Cegla, 2017) recently reported that EMAT phased arrays would be advantageous in general over standard monolithic transducers because, as any ultrasonic phased array, they can generate different electrically-controlled ultrasonic fields and rapidly visualise the internal structure of the specimen through focused images. However, the fact that EMATs are inherently poor transmitters and receivers and therefore have to be excited using powers in excess of 1 kW has hindered the development of bulk-wave phased arrays. (Isla & Cegla, 2017) recently proposed two recent developments in EMAT technology, namely a methodology for increasing the bias magnetic field of shear wave EMATs and a new type of coded excitation for pulse-echo mode operation to design a pulse-echo EMAT array, which can be used to image defects.

C.1.3.2.6. ACOUSTIC EMISSION

Acoustic emission (AE) monitoring is a useful technique for distinguishing active and extinguished cracks in steel bridges, particularly because AE energy is released under normal service loads (Kosnik, et al., 2011).So only active damage can be detected, and this technique cannot detect all existing cracks. AE can be a valuable technique for detecting fatigue cracking. It is based on real time detection and can be useful for remote monitoring. Simple observation of changes in count rate can be a useful metric. Even though these events may come from either crack extension itself, localized plastic deformation around the crack tip, or from fretting of surfaces in the crack wake, the exact mechanism is not of great importance to the structural engineer — knowledge that a crack is propagating or extinguished is sufficient (Kosnik, et al., 2011).

(Ranganayakulu, et al., 2014) utilized AE to study two weld defects, lack of penetration and lack of side fusion, in nuclear grade stainless steel materials. The results were analysed in the time domain where energy, counts and amplitude of acquired AE signals were evaluated. They observed that both defects can be identified by using such different AE parameters where “counts vs. amplitude” parameter was found to give the widest distinction with respect to the type of defects.

(Droubi, et al., 2017) presented a systematic investigation into the application of AE technique in detection and identification of weld defects and showed that AE parameters were influenced by defect presence. They concluded that AE offers the potential to detect and identify different weld defects and thus assess the overall structural health of welded structures. Among all AE parameters tested in this study, AE energy, root mean square and peak amplitude were found to be key parameters in detecting a presence of a weld defect. This was due to these three parameters showing the largest percentage differences from the no-defect values. Each AE sensor can monitor a limited zone, so the global location of the possible defaults must be defined. These sensors can be affected by near environment perturbations.

C.1.3.2.7. INFRARED THERMOGRAPHY

Infrared thermography is a contact-less optical imaging technique for detecting invisible infrared radiation (Hung, et al., 2009). Infrared thermography can be used to assess and predict the structure or behavior beneath the surface by measuring the distribution of infrared radiation and converting the measurements into a temperature scale. Infrared thermography is generally divided into two main streams: passive infrared thermography (PIT) and active infrared thermography (AIT).

In PIT, abnormal temperature profiles indicate suspicious problem. The technique is mainly used for qualitative inspection to pinpoint the anomalies. For reducing the sensitivity to variations in ambient condition and obtaining more accurate measurement, AIT was developed to provide more accurate information by considering the amount of thermal radiation and heat transfer. The common thermal stimulation techniques in AIT are: transient pulse, step heating (long pulse), periodic heating (lock-in), and thermal mechanical vibration. Active thermography has been successfully employed for flaws inspection in many industrial applications, such as aircraft and automotive components inspection, adhesive bonding and spot welding investigations, and pressure vessel and pipeline inspections (Hung, et al., 2009).

Infrared thermography can be used for thermographic investigations of thermomechanical coupling phenomena. The two most important thermomechanical effects during mechanical loading of an elastically–plastically deformable solid body as ductile steel are thermoelastic coupling and plastic dissipation (Ummenhofer & Medgenberg, 2009). Thermoelastic coupling causes a temporal variation of the temperature field during mechanical loading. For materials with a positive coefficient of thermal expansion – this includes steel – a temperature drop during tension and a temperature rise during compression can be observed. For cyclic loading in the elasticity domain the mean temperature of the body is not affected by thermoelasticity and remains constant.

Since some of the material properties depend on stress and/or temperature thermoelastic coupling becomes nonlinear. Plastic dissipation occurs if the body is stressed in some area above its yield limit. In this case a major portion of the plastic work is released as heat. Since the plastic dissipation is always positive, it causes a continuous mean temperature rise for cyclic loading until the heat transfer with the surrounding equals the bulk heat generation within the loaded specimen. (Huss, 1994) showed that the appearance of cracks causes significant nonlinearities in the thermoelastic temperature variations. Their analysis allows for the detection of cracks at and near to the surface by analysis of higher harmonics in the TSA signal. The nonlinearities have mainly been attributed to the crack opening and closure whereas the effects of crack tip plasticity has been found to be of minor importance.

C.1.3.2.8. GUIDED WAVES

Recent studies investigated the interaction of weld-guided waves with defects located in the material adjacent to the weld, with application interest to non-destructive evaluation. The idea came from an experimental observation on a large butt-welded plate which found that the weld can concentrate and guide the energy of a guided wave travelling along the direction of the weld (Sargent, 2006), (Fan & Lowe, 2012)). Therefore, instead of seeing features as a problem, they could be used to focus the energy of guided waves in order to detect defects in or near them.

Also, (Pahlavan & Blacquière, 2016) have proposed a baseline-free quantitative sizing methodology utilizing ultrasonic guided waves for fatigue cracks under welded stiffeners in steel bridge deck. Having conducted experiments on a test bridge deck subject to fatigue loading, it has been demonstrated that the crack profile can be estimated from the reflection coefficients obtained. In comparison with the reference measurements, the maximum crack depth estimation error turned out to be about 20%.

C.1.3.2.9. MAGNETIC BARKHAUSEN NOISE (MBN) TECHNIQUE

The MBN signal contains information which is closely related to the microstructure of an examined ferromagnetic material. Thus, any change in grains configuration, due to the presence of stresses or of lattice distortions, results in rearrangement of the magnetic domains' configuration. The fore mentioned dependence of the MBN to the material's intrinsic properties makes it a potential tool of non-destructive techniques (NDT), for the evaluation of metallurgical, microstructural, mechanical and micromagnetic parameters (Vourna, et al., 2015).

Welding introduces high heat input to the material being welded. As a result of this, non-uniform heat distributions, plastic deformations and phase transformations occur on the material. These changes generate different residual stress patterns for weld region and in the heat affected zone (HAZ) (Yelbay, et al., 2010). Residual stresses can be measured by destructive methods such as hole-drilling, sawcutting, sectioning, and layer removal; or by non-destructive methods like X-ray diffraction, neutron diffraction, ultrasonic, magnetic and a relatively new technique Raman spectrum method (Ju, et al., 2003). As a non-destructive method Magnetic Barkhausen Noise (MBN) technique is applicable to ferromagnetic materials, which are composed of small order magnetic regions called magnetic domains.

C.1.3.2.10. SHEAROGRAPHY

Shearography is an interferometric technique for surface deformation measurement (displacement and displacement-derivatives) (Hung, et al., 2009). The technique reveals flaws by searching flaw-induced deformation anomalies after the object is stressed. Three versions of shearography recording methods are used: (1) photographic recording, (2) thermoplastic recording, and (3) digital recording. The photographic version is relatively high cost, laborious and time consuming, using a wet photographic processes and a portrait camera with high-resolution photographic emulsion as the recording media.

In the thermoplastic version, the photographic plate is replaced by a reusable thermoplastic plate. Digital shearography (DISH) uses video sensors such as a charge coupled device (CCD) as the recording media for data acquisition and employs digital processing techniques for image analysis (Hung, et al., 2009).

C.1.3.2.11. USE OF LOW-FREQUENCY MAGNETORESISTIVE SENSORS

Methods such as eddy current testing, or magnetic flux leakage testing (MFL), have a long history in non destructive testing.

As mentioned above, Eddy current methods are commonly used to detect flaws at the surface or subsurface of conductive materials, because the eddy current strength decreases with depth due to the skin effect. Recently, low frequency eddy current testing using magnetic sensors, such as a magneto-resistive sensor (MR) (Tsukada, et al., 2006), Hall sensor (Kosmas, et al., 2005) or Superconducting Interference Device (SQUID), which can measure low frequency magnetic signals instead of a search coil, have been developed to detect deeper flaws (Jenks, et al., 1997) (Tsukada, et al., 2010).

The magnetic flux leakage method is usually used for ferromagnetic material structures such as steel pipelines, storage tanks, bridges, etc. If a corrosion pit or crack is present in a steel pipe, magnetic flux leakage occurs outside the pipe wall when an applied magnetic field penetrates the wall in a direction parallel to the wall surface. In general, MFL uses a strong magnetization system to yield a measurable magnetic flux leakage from the specimen.

The applied magnetic field strength is usually the saturation region of the B–H curve, so that a reduction in the material wall thickness will cause a large flux to leak. However, it is difficult to detect anomaly thick-wall, since it requires a stronger magnetization system to attain saturation.

Recently, a magnetic method using a magnetic sensor instead of a search coil has been applied to not only surface flaws but also to deep flaws since it is a low-frequency operation with high performance and high sensitivity (Tsukada, et al., 2011), (Tsukada, et al., 2013). MFL, which is useful for ferromagnetic material structures, detects the magnetic flux leakage from the specimen surface when a magnetic flux is induced.

By using a highly sensitive magnetic sensor, a low- magnetization operation that does not need a strong magnetic field to attain the saturation region of the B–H curve of a specimen can be realized.

C.1.4. NON-DESTRUCTIVE TESTING FOR MASONRY

C.1.4.1. RADAR TECHNIQUE

Ground penetrating radar (GPR) is extremely useful to determine the interface between the old and the modern parts of structures constructed at different periods of time, to identify older constructions embedded inside walls or buried under the building structures (Lai, et al., 2018). GPR has also been used to assess the efficacy of cement grouting in historical building, as well as in-fill of cracks/voids (Dérobert, et al., 2019).

C.1.4.2. STEP-FREQUENCY RADAR

One of the most promising techniques used to reconstruct images consists of exploiting the entire radar signal, through using a Full Wave Form inversion technique. The current application exploits a 1D reconstruction of the medium's permittivity vs. depth, with prior calibration enabling permittivity values to be correlated to water content values.

This type of inversion technique requires accurate modelling of radar wave propagation in a tabular multi-layer medium (Lambot & Andre, 2014), simulating concrete with a water content gradient.

Air-coupled radar antennas generally operate as dipole behavior, which enables analytic modelling of wave propagation and reflection in a multi-layer medium, based on Green functions. Associated iterative inversion techniques therefore enable to compare and minimize differences between the radar signal measured and the signal reconstructed using multi-layer configuration, in which permittivities are the desired parameters.

(Guan, et al., 2017) developed this method in order to monitor the rise of the freshwater soaking front in a porous limestone block (tuffeau stone from the Loire Valley, France) 20 cm thick. The configuration studied in this experiment is bi-layer (dry and saturated limestone), unknown parameter is the height of the soaking front in the material.

A step frequency radar system is used, which is designed based on a network analyzer and an ultra-broadband Vivaldi antenna, and the frequency bandwidth studied is [0.8-3 GHz].

C.1.4.3. ULTRASONICS: TOMOGRAPHY AND IMPULSE ECHO APPLICATIONS

The European research project ONSITEFORMASONRY (On-site investigation techniques for the structural evaluation of historic masonry buildings) investigated the development and optimisation of non-destructive testing (NDT) and minor destructive testing (MDT) systems. Among others, one main aim of the development of an ultrasonic prototype was to investigate historic masonry with ultrasonic tomography. This equipment should enable the user to localise voids and delaminations in masonry elements, when they are accessible from at least two sides. A second aim was to adapt ultrasonic echo techniques for masonry in order to localise delaminations and to determine crack depths.

C.1.4.4. PHOTOGRAMMETRY

Photogrammetry is the process of generating a 3D model from 2D images. The resulting model can be scaled and used to measure distances between objects. (Napolitano & Glisic, 2019) mention photogrammetry a good candidate for NDT, and more precisely diagnostic tool for crack pattern identification, as it is a low-cost, accurate method (Mandelli, et al., 2017) (Malihi, et al., 2018) which is widely utilized in the fields of architectural heritage (Sapirstein, 2014) (Patias & Santana Quintero, 2009) and more specifically historic structures damage assessment (Percy, et al., 2015) (Shrestha, et al., 2017).

C.1.4.5. USE OF DISTRIBUTED STRAIN AND ACOUSTIC EMISSION SENSORS

(Verstrynge, et al., 2018) tested several novel types of strain and acoustic emission sensors were investigated for crack monitoring in masonry structures. The applied techniques were integrated optical fibres with distributed fibre Bragg grating sensors (FBGs), stereo-vision digital image correlation (DIC) without the use of a speckle pattern, optical fibre sensors for acoustic emission sensing (AE-FOS), piezo-electric transducers for acoustic emission sensing (AE-PZT) and LVDTs.

While the latter two were applied as reference techniques, the former three were under investigation as novel application. In particular, (Verstrynge, et al., 2018) indicate that for crack measurements in large (historical) masonry elements, a combination of resonance-type AE sensors and optical fibres equipped with FBGs provides optimal crack detection.

The optical fibre with semi-distributed FBGs can cover larger distances and performs excellently for crack width quantification, provided that temperature compensation is foreseen. When a crack tip is identified, the AE sensors positioned within a region of several decimetres around the crack tip are able to register the moment of occurrence of crack growth and friction-related AE events, and to detect unstable crack growth well in advance.

C.1.4.6. THERMOGRAPHY

Infrared thermography (IRT), as a remote imaging system, represents a powerful tool to be used for quick periodic inspection. (Meola, 2007) investigated the nondestructive evaluation of masonry structures using a Focal Plane Array infrared camera. Tests were carried out in laboratory on specimens, which simulated one- and two-layer structures, with defects of different geometry and nature and located at different depths.

More recently, it was also used to investigate the internal structural condition of concrete masonry walls (Khan, et al., 2014) (Khan, et al., 2015a). (Khan, et al., 2015b) mention (i) that masonry is an inherently composite material given the presence of different components such as bricks and mortar joints, and (ii) that further research is needed to predict infrared thermography results by finite element modeling in partially grouted concrete masonry walls. They propose an outlier Analysis was implemented to allow an enhanced representation of internal voided regions in partially grouted concrete masonry walls by leveraging temperature time histories extracted from active thermography.

C.1.5. NON-DESTRUCTIVE TESTING FOR BRIDGE TOWER BOLTS

The wind, vibration and other sources generates forces on towers and causes strain to tower foundations. Tower Base Bolts are subjected to high forces therefore, in order to ensure sustainable and safe service life, performance of tower base bolts should be monitored. Use of torque, torque and angle or tensioning may not be adequate, whereas ultrasonic bolt elongation measurement is able to provide accuracy equal to stain gauging and allows the user to return at any time and re-verify the level of tension in each bolt during its service life. An ultrasonic measuring method is quick and accurate; a measurement is taken before the bolt is tightened and then after the bolt is tightened. The difference between the readings is the bolt elongation.

The system can be calibrated to measure load, stress and strain. If corrosion takes place, the properties of the bolt will be changed and this can be measured. The ultrasonic bolt elongation measurement technique offers the following advantages: greater accuracy than use of torque or tensioning, reliable and cost effective, minimises the requirement for extensive operator training, easy output of results and verification, no risk to personnel adjacent to the controlled zone.

Osmangazi Bridge is a suspension bridge with two towers seating at -40.00 meter deep of İzmit Bay. In each tower leg there are 84 number of M110x10100 EN 14399 tower base bolts (Dia 11 cm., lenth 10.1 metes) connecting steel tower bases to reinforced concrete foundation plinths. Ultrasonic Bolt Meter device named “NORBAR USM-3” was used in measurement of high strenth bolts during construction and after completion for: Tower base bolts, Cable clamp bolts, Saddle bolts.

Figure 16 shows the place of anchor bolts at the tower base connecting tower to reinforced concrete foundation.

Figure 16. Anchor bolts connecting Steel Tower Block to RC Plinth.

Technical specification Vol.7 item 8.5.4 states that "Preloading force shall be checked at completion of bridge. If the preloading is less than 95% of the design preloading value, the rods shall be retightened." On november 2016, the preloading on selected anchor rods at all tower legs were measured and preloading force conformed to the requirement in technical specification. The first in-service check of preloading force shall be done 5 years after completion. If the preloading force is less than 90% of the design preloading the rods shall be retightened.

The system measures the elongation, stress and local load in fasteners, quickly and accurately, and displays the result on a easy to read screen (Figure 17). The measurement is achieved by determining the change in the transit time of an ultrasonic shock wave along the length of the fastener as the fastener is tightened (Figure 18).

Figure 17. Ultrasonic Stress Meter-NORBAR USM-3.

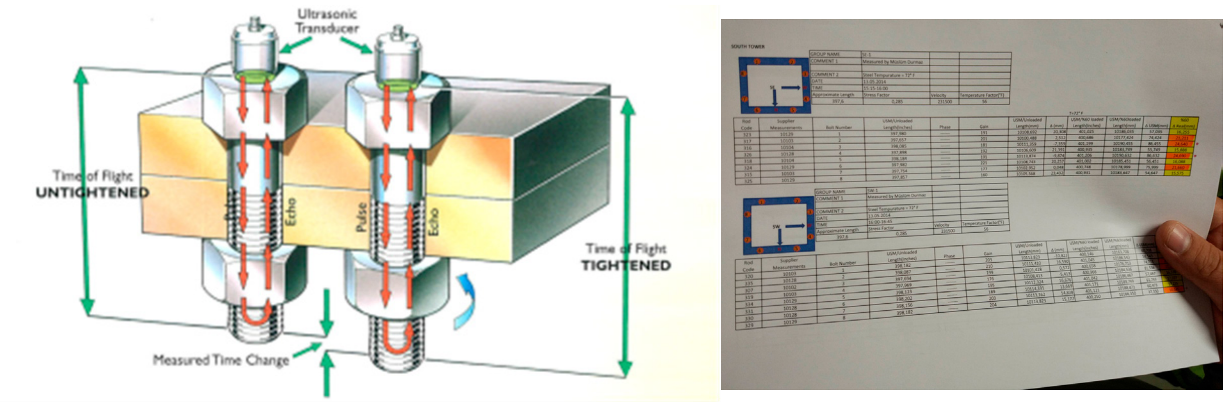


Figure 18. Measuring principal and measurement readings

An ultrasonic bolt elongation measurement system is an easy, reliable technology used in measurement, recording and monitoring of bolt tensions.

C.2. INNOVATION IN STRUCTURAL HEALTH MONITORING (SHM)

C.2.1. HIGH RESOLUTION PHOTOGRAPHY

Contributor: Nicolas Manzini (source SITES: Pierre Carreaud, Bertrand Collin, Géraldine Camp).

Large civil engineering structures, especially those dedicated to transportation and networks such roadway or railway bridges and tunnels, require constant monitoring to ensure their safety and serviceability. The scale of these elongated structures, coupled with often complex and/or dangerous access to elements to inspect, tends to limit the frequency of on-site interventions. Moreover, most of these structures have a significant contribution to local or regional economic development, therefore minimum service interruption and disturbance to users are required. In this context, efficiency of inspection and processing should be optimized while guaranteeing both accuracy and repeatability. Solutions based on high resolution camera acquisition (Collin, et al., 2016) may help meet these objectives.

Very high resolution digital cameras, with specific lens and calibration (according to the light conditions and distance) are used to generate semi-automatically panoramic views of the structure that are used for an off-site inspection. Report of observed features and anomalies are performed manually afterwards. The use of high definition hardware allows high accuracy in the inspection and detection of defects. This allows the user to monitor the appearance and the opening of cracks over time, and to map them with accuracy (Figure 19). These data about crack density and severity allow to describe the health state of the structure.

Figure 19. VHRPI application on a viaduct in Oran (Left: Operator, Up Right: HR picture, Down Right: cracks mapping)

To increase efficiency of interventions, solutions to automate data acquisition and mapping of anomalies have been developed. One developed tool is the ScanSites solution. It consists of a combination of Very High Resolution camera mounted on robotized head, with GIS (Geographic Information System) software. This solution permits real-time observation, detection and localization of defects. With progressive upgrades, the system now has the ability to detect cracks down to 0.1mm at 200m distance, and to estimate their position with centimetric accuracy, this tool is very efficient for large structures like road-bridges. These automated acquisition tools can be combined with Lidar and/or photogrammetric processing in order to produce referenced multi-layer numerical models with both cracks and deformation information (Camp, et al., 2013).

Table 14. TRL scale for Very High Resolution Photographic Inspection on road bridges applications

Ranking	YES/NO	Explanations
1	YES	Camera acquisition have sufficient resolution to be used for damage assessment on large scales
2	YES	Combining inspection tools with GIS allows mapping of features identified on a structures
3	YES	An automated system is conceived combining software tools, automated acquisition and GIS
4	YES	A prototype with a robotized video inspection head is proposed
5	YES	Prototype is validated
6	YES	Operational system is used on cooling towers
7	YES	Operational system is used to monitor a concrete arcg dam.
8	YES	Operational system performance is evaluated on a concrete arcg dam, and could be applied to monitor large concrete bridges
9	YES*	System is operational in commercial applications

The inspection of inner areas, such as tunnels or the interior of bridges, can be tackled with 360° panoramic tools ScanTubes, which is equipped with a number of calibrated high definition cameras that allow to maximise the inspection speed of tunnels. The system allows for high precision visual inspection, but also relies on photogrammetric processing in order to produce a high definition 3D model of the structure (Figure 20). In railway tunnel applications, it allows defects detection down to 0.05mm, and was proven able to scan up to 6km of tunnels in less than 4h. An original application of the 360° panoramic tool that overcomes limitations related to human inspections, consists in using it to inspect vertical structural components (Carreaud & Collin, 2017). This enables for example both 3D modelling and defect inspection of hollow viaduct piers.

Figure 20. Left: Inspection head, Center: suspended ScanTube system, Right: 3D model of an hollow pier

Table 15. TRL scale 360° high resolution photographic inspection on road bridges applications

Ranking	YES/NO	Explanations
1	YES	Georeferenced photographic data acquisition can be automatized to ease surveys in restrained/dangerous/complex acquisition sites
2	YES	Combining 360° acquisitions with georeferencing and/or lidar can allow easy feature identification and localisation
3	YES	Combination algorithms exist in classical photogrammetry applications
4	YES	A prototype has been validated using Lidar
5	YES	A prototype has been validated using only photographic acquisition, with better accuracy and faster processing than lidar-based systems
6	YES	System has been used in railway and road tunnels
7	YES	System has been demonstrated in a vertical ventilation shaft of tunnel and inner side of piles of viaducts
8	YES	System has been qualified in a vertical ventilation shaft, and could be used for applications inside piers of bridges
9	YES	System is operational in commercial applications

Further developments of this technology focus on the increase of acquisition speed, the reduction of noise, and the application of big data processing software. Moreover, while cracks mapping gives an insight about the structural health, a clear definition of a “crack index” transferable from one structure to another is not yet available. The achievement of a precise operational definition will require further research efforts.

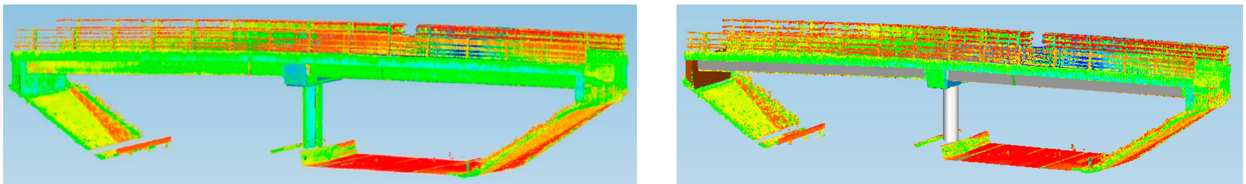
C.2.2. CONCRETE-EMBEDDED OPTIC FIBER

Contributor: Linh Truong-Hong

Laser scanning known as Light Detection And Ranging (LiDAR) is a non-contact measurement method acquiring a massive amount of three-dimensional (3D) topographic data of visible surfaces of structural components highly accurately and quickly. The laser sensors can be integrated into various platforms to capture objects from air (airborne and unmanned aerial vehicle, UAV), mobile (e.g. car, boat and train) or ground, and can offer various flexible data capture systems to maximize data coverage of a bridge structure (Truong-Hong Linh & Laefer. 2014, Riveiro et al. 2013, Laefer 2013). An aerial laser scanning (ALS), in which the laser sensors were integrated into a fixed wing plane or helicopter often captures vertical side surfaces and bridge’s deck with low density, while a UAV with LiDAR can capture the bridge from different field of views.

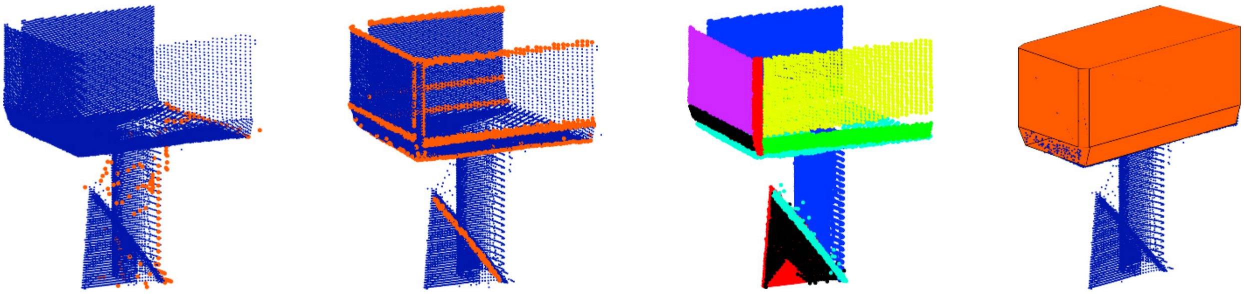
Additionally, the mobile laser scanning (MLS) only collects a point cloud of the structures’ surfaces along a routine, while the terrestrial laser scanning (TLS) has the flexibility to capture the structures from the different ground locations at millimeter accuracy for a single scan. Regarding to data coverage, both the UAV-LiDAR and TLS can be used for acquiring geometric data of the bridge. However, as the accuracy of the UAV-LiDAR is still at centimeter, the point clouds from this unit is limited in reconstructing 3D models. With high level of the data capture flexibility and accuracy, the TLS is to be a prominent unit used for geometric capture for bridge inspection, which can span from geometric modelling to measurement of structural deformation to detection of structural surface deficiencies.

Geometric modelling: 3D models of bridge structures are often created by fitting point clouds of the bridge components to known generic surfaces or volumes (Figure 21). This process can be done by using Computer Aided Drawing (e.g. Revit) or a point cloud processing software (e.g. Leica cyclone) or semi-automatic method (Figure 22).



a) b)

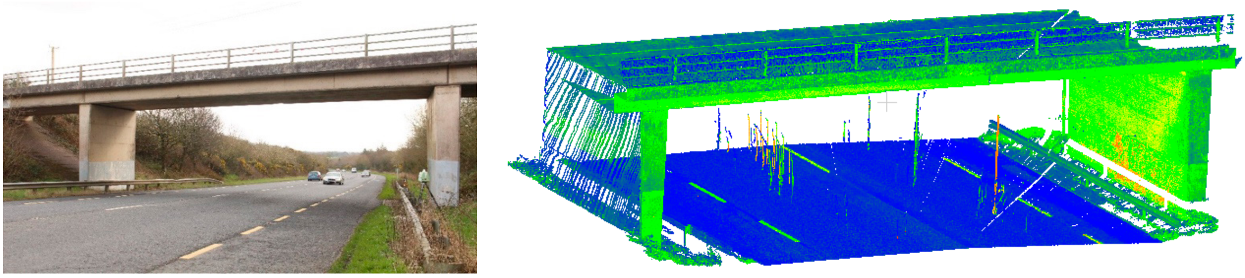
Figure 21. 3D model of parts of the bridge manually created within a Leica cyclone V9.1.3; a) A point cloud; b) Fitting objects created by using built-tools within Leica cyclone V9.1.3 (Truong-Hong and Laefer, 2019)



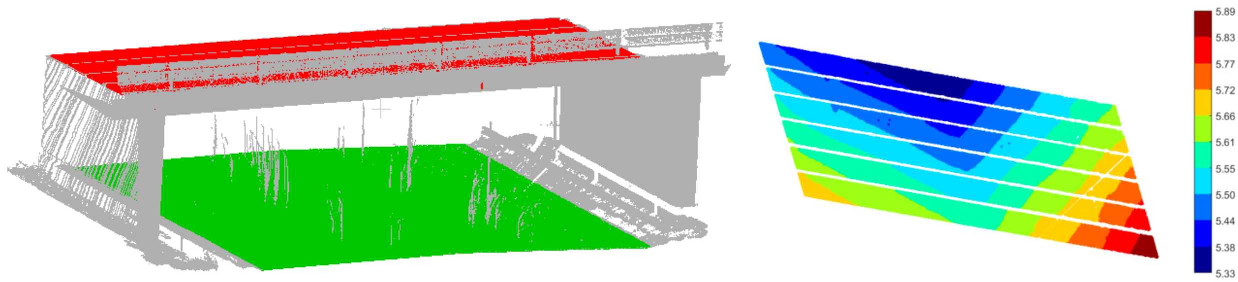
a) b) c) d)

Figure 22. Semi-automatic method to create a 3D model of a pier cap; a) A point cloud of a pier cap; b) Edge point extraction by users; c) Point clouds of individual surfaces; d) A complete 3D model (Walsh et al. 2013)

Vertical clearance and deformation: Laser scanning can be used to measure overall displacements of a bridge (Lichti et al. 2002, Zogg & Ingensand 2008, Lovas et al. 2008), vertical clearance (Riveiro et al. 2013, Liu et al. 2012), and deformations/distortions of each member (Truong-Hong & Laefer 2015a). As the measurement of a structure's deformation requires millimeter accuracy, the TLS and MLS point clouds are commonly to used. However, MLS integrated into a vehicle is limited in acquiring data points with respect to the field of view of the scanner along the vehicle's routine. Thus, MLS data is often restricted for use in establishing in-situ vertical clearance (Figure 23).



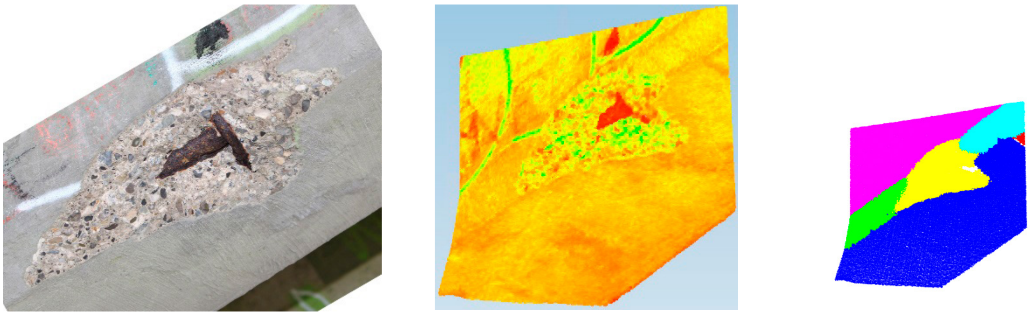
a) Photo of the bridge b) Point cloud of bridge and road



c) Point clouds of bottom surfaces of girders and road surface d) Vertical clearance at each data point

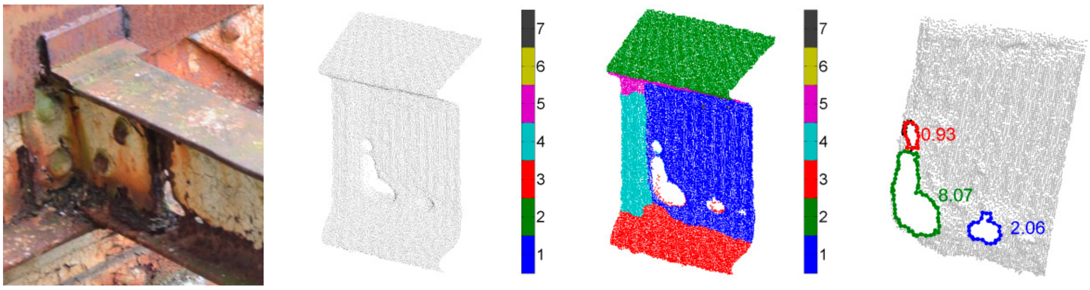
Figure 23. Vertical clearance values of the bridge (Truong-Hong and Lindenbergh, 2019)

Spalling, scaling, disintegration and surface loss: At the location of damage, (e.g. spalling), the local surface at those locations differs in appearance significantly its surroundings. Thus, the local surface features can be indicators for extracting damage or section loss due to corrosion (Figure 24 and Figure 25).



a) b) c)

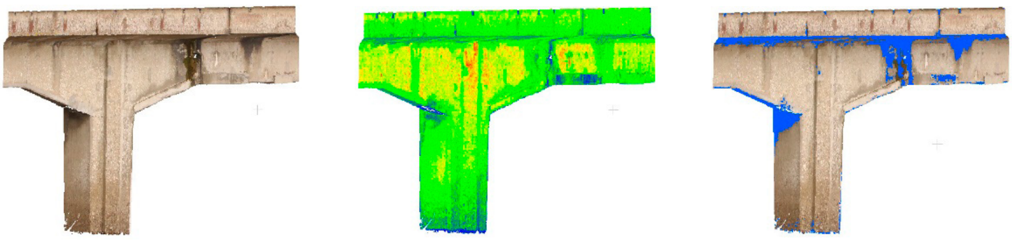
Figure 24. A hierarchical method for identify a point cloud of damaged surfaces; a) Photo of the girder's section containing spalling; b) Point cloud of the surface; c) Resulted damage detection (Truong-Hong and Laefer, 2019)



a) b) c) d)

Figure 25. Determine surface loss area: a) Photo of the surface loss due to corrosion; b) A point cloud of the section of interest; c) Segments of the input data; d) Boundary points and hole areas (unit in cm2) (Truong-Hong and Laefer, 2015)

Chemical attack, leading water bleed and corrosion: At the locations subjected to water bleed, chemical attack or corrosion, significant geometric changes may not be presented. A machine learning method, likely SVM is a prominent solution to extract damaged surfaces directly from attributes of the point cloud (Figure 26).



a) b) c)

Figure 26. Illustration of damage prediction by a support vector machine (SVM): a) A structure rending by RGB, b) A point cloud of the bridge parts rending by intensity, c) Results of damage extraction (Truong-Hong and Laefer, 2019)

Crack: The crack width and length can be estimated by combining geometric coordinates and RGB colors of the point cloud (Figure 27).

a) b) c) d)

Figure 27. Crack detection by incorporating high-resolution image and point cloud: a) Photo of a crack, b) Point cloud rendering by intensity, c) Point cloud rendering by RGB from the image, d) Identification of crack's width and length (Truong-Hong et al., 2016)

Table 16. TRL scale for concrete-embedded bragg sensors

Ranking	YES/NO	Explanation
1	YES	Fiber bragg gratings are a commonly used high performance tools for deformation, temperature, acceleration or pressure.
2	YES	Embedding sensors directly in concrete is expected to allow for a better monitoring of the segments.
3	YES	Embedding sensors directly in concrete is expected to allow for a better monitoring of the segments.
4	YES	Multiple tests of the system and its components have been realized
5	YES	Multiple tests of the system and its components have been realized
6	YES	Fiber bragg sensors have been embedded inside segmental concrete linings (metallic fiber reinforced concrete).
7	YES	Bragg sensors have been embedded inside segmental concrete linings. Sensors show good correlation between applied pressure, cracks opening, and traction constraints
8	NO*	Further evaluations are being undergone to qualify the performances of the system over time. No reference yet for applications on road bridges.
9	NO*	No reference yet for applications on road bridges.

C.2.3. GNSS SOLUTIONS

Contributor: Joan R. Casas

Due to the novelty of this technology, the use of DOFS in civil engineering infrastructures SHM is still a relatively recent practice. These sensors share the same advantages of the other optical fiber sensors, but present the unique advantage of enabling the monitoring over greater length extents of the infrastructure and with a very short distance between each measuring point, in the order of millimeters. These sensors can be externally bonded or embedded to the structure to be monitored and when temperature or strain variations occur, these changes are transmitted from the material to the sensor, which then generates a deviation of the scattered signal, which is being reflected within the fiber cable core. This scattering phenomenon is the basis behind the distributed optical fiber sensing, and defined by the interaction between the emitted light and the physical optical medium.

There are three different scattering processes that occur and can be used in order to obtain distributed strain and/or temperature measurements, namely the Raman, Brillouin and Rayleigh scattering (Barrias, Casas, and Villalba 2016). Raman scattering is highly dependent of temperature variations, which has led to some applications in civil engineering SHM but has been more typically used in other fields (Barrias et al. 2018a). Brillouin, on the other hand has been the most studied and used scattering technique for distributed sensing in civil engineering structures monitoring. This is a consequence of its extended measurement range capability, which allows its use in applications where monitoring distances in the order of kilometers are present. This technique was initially introduced with the use of optical time domain reflectometry (OTDR) which has an inherent low spatial resolution (of around 1 m). More recently, this has been improved through the development of the Brillouin optical time domain analysis (BOTDA) which relatively enhances the achievable spatial resolution.

Finally, the Rayleigh scattering based DOFS are currently limited to a sensing range of 70 m but provide a significant high spatial resolution of 1 mm, making it ideal for the use of damage monitoring detection application such as crack detection in concrete structures. This is achieved through the use of swept wavelength interferometry (SWI) to measure the Rayleigh backscatter as a function of length. The optical backscattered reflectometry (OBR) system based on the Rayleigh OFDR (Figure 28) is composed by an active part which sends a laser light through an optical fiber, and a passive part, where the light is reflected by the intrinsic variations along the fiber length (Samiec 2012). This pattern of the reflections and the corresponding time of flight of the light is measured and stored, acting as a unique fingerprint for each fiber. When an external stimulus (like strain or temperature variation) happens, a temporal and spectral shift in the local Rayleigh backscatter pattern occurs. This new reflection pattern is then stored and compared with the original one providing through cross-correlation the variation and evolution of generated strains along the entire length of the fiber due to this external stimulus (Grave et al 2015). More detailed information of this technology can be consulted in (Barrias, Casas, and Villalba 2016).

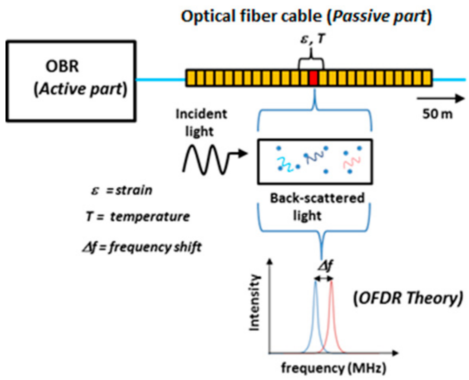


Figure 28. Measuring process by OBR system

Apart from being applied in several tests in the laboratory (Villalba and Casas 2012, Rodríguez et al.2015, Barrias et al. 2018a,c, Barrias et al. 2019) with excellent performance, some relevant applications in real structures have been also carried out (Barrias et al. 2018b, Barrias et al. 2016)

Table 17. TRL scale for GNSS-network on road bridges applications

Ranking	YES/NO	Explanations
1	YES	Multiple-frequency GNSS is an available versatile tool
2	YES	GNSS (with multiple-frequency) performances are evaluated as compatible for some SHM applications
3	YES	GNSS (with multiple-frequency) performances are evaluated as compatible for some SHM applications,
4	YES	Post-processing network solutions are validated for various applications
5	YES	Post-processing network solutions are validated for various applications
6	YES	Post-processing network solutions are validated on applications such as surveys, ground deformation, slow displacements monitoring
7	YES	GNSS networks have been deployed on various bridges, confirming the ability to monitor both static deformation and dynamic response with proper processing
8	YES	GNSS network performances on various bridges have been evaluated and qualified, using a local reference station or a CORS station
9	YES	Multiple commercial solutions are available, and a high-end GNSS network is becoming a more common tool in monitoring scenario for slender structures

C.2.4. USE OF NETWORK OF GNSS STATIONS USING RTK PROCESSING TO MONITOR BRIDGE DYNAMIC RESPONSES

Contributor: Manzini N. (sources SITES: Mikhael De Mengin, Yohann Blanco, Pierre Brouillac, Bertrand Collin ; sources Bouygues TP : Florence Duchêne, Samir Renai, Laurent Dabet)

Fiber bragg sensors have been embedded inside segmental concrete linings (Metallic Fiber Reinforced Concrete, MFRC) using "fakir boards" technique. This experimented was realized in order to demonstrate that MFRC linings performed in accordance with the requirements of the contract specifications. Data from the bragg sensors provide deformation and temperature information that can be correlated with pressure/charge applied on the concrete linings to monitor their behaviour during the operation of the tunnel boring machine (Figure 29).

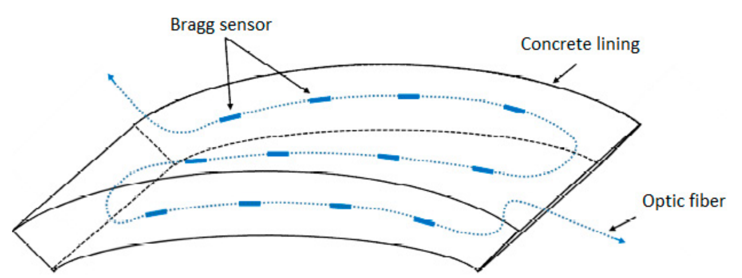


Figure 29. Insertion of Bragg sensors inside concrete linings (Source: (De Mengin, et al., 2017))

Table 18. TRL scale for RTK network on road bridges applications

Ranking	YES/NO	Explanations
1	YES	RTK is an operational technique mostly used in survey applications
2	YES	GNSS RTK performances are evaluated as compatible for some SHM applications
3	YES	GNSS RTK performances are evaluated as compatible for some SHM applications
4	YES	NRTK solutions are validated for various applications
5	YES	NRTK solutions are validated for various applications
6	YES	NRTK solutions are validated on applications such as slope monitoring,
7	YES	A GNSS NRTK system has been deployed on a structure and confirmed the ability to qualify firsts modal frequencies
8	YES	A GNSS NRTK system performances on a long span bridge have been evaluated and qualified, using a local reference station
9	NO	NRTK commercial solutions are available, but the use of this type of solution to detect a change in dynamic behaviour of a bridge is yet to be confirmed

C.2.5. USE OF NETWORK OF LOW-COST SINGLE FREQUENCY GNSS STATIONS

Contributor: Manzini N. (source: (Yu, et al., 2016))

C.2.5.1. USE OF NETWORK OF MULTIPLE FREQUENCY GNSS SOLUTIONS

GNSS positioning enables 3D positioning almost anywhere on the globe with no measurement drift. A GNSS station is able to use satellite signals and its internal clock to estimate pseudo-ranges with each GNSS satellite. Time series of coordinates in a reference frame are computed by using those pseudo-ranges and orbit parameters of the satellites (Figure 30). With multiple frequency signals, and phase-based computation, post-processing of satellite observations allow accuracy up to a few millimeters. One (or more) reference station is used to mitigate propagation errors of the signals in the atmosphere, by using double differences technique. This ensures both high precision positioning and long-term stability of obtained coordinates. The use of high-end solutions permits acquisitions up to 100Hz and high protection against parasitic signals (multipath).

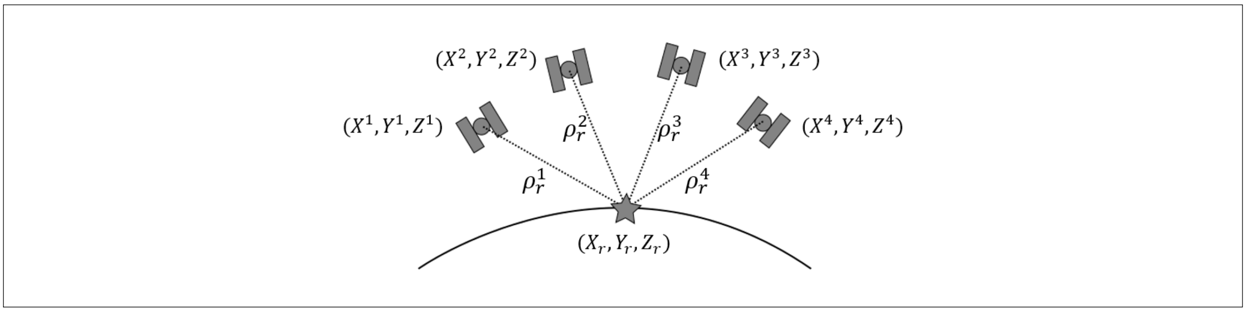


Figure 30. Basic principles of GNSS positioning by using estimated pseudo ranges

Table 19. TRL scale for GNSS-network on road bridges applications

Ranking	YES/NO	Explanation
1	YES	Multiple-frequency GNSS is an available versatile tool
2	YES	GNSS (with multiple-frequency) performances are evaluated as compatible for some SHM applications
3	YES	GNSS (with multiple-frequency) performances are evaluated as compatible for some SHM applications,
4	YES	Post-processing network solutions are validated for various applications
5	YES	Post-processing network solutions are validated for various applications
6	YES	Post-processing network solutions are validated on applications such as surveys, ground deformation, slow displacements monitoring
7	YES	GNSS networks have been deployed on various bridges, confirming the ability to monitor both static deformation and dynamic response with proper processing
8	YES	GNSS network performances on various bridges have been evaluated and qualified, using a local reference station or a CORS station
9	YES	Multiple commercial solutions are available, and a high-end GNSS network is becoming a more common tool in monitoring scenario for slender structures

C.2.5.2. USE OF NETWORK OF GNSS STATIONS USING RTK PROCESSING TO MONITOR BRIDGE DYNAMIC RESPONSES

RTK positioning uses static reference station with known coordinates near the structure, transmitting correction data in real time via radio transmission (or optic fiber links) to a network of monitoring GNSS stations. This system allows for accuracy up to 1cm in real time, with high acquisition rate (up to 50Hz). Such data allows the system to monitor both long term absolute displacements and structure dynamics in real time (Figure 31).

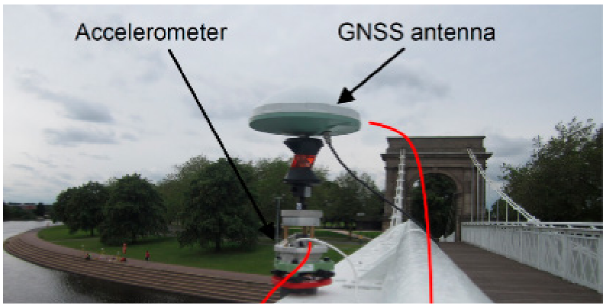


Figure 31. High end GNSS antenna installed on Wilford suspension bridge from (Yu, et al., 2016)

Table 20. TRL scale for RTK network on road bridges applications

Ranking	YES/NO	Explanations
1	YES	RTK is an operational technique mostly used in survey applications
2	YES	GNSS RTK performances are evaluated as compatible for some SHM applications
3	YES	GNSS RTK performances are evaluated as compatible for some SHM applications
4	YES	NRTK solutions are validated for various applications
5	YES	NRTK solutions are validated for various applications
6	YES	NRTK solutions are validated on applications such as slope monitoring,
7	YES	A GNSS NRTK system has been deployed on a structure and confirmed the ability to qualify firsts modal frequencies
8	YES	A GNSS NRTK system performances on a long span bridge have been evaluated and qualified, using a local reference station
9	NO	NRTK commercial solutions are available, but the use of this type of solution to detect a change in dynamic behaviour of a bridge is yet to be confirmed

C.2.5.3. USE OF NETWORK OF LOW-COST SINGLE FREQUENCY GNSS STATIONS

Another field of innovation regarding GNSS applications consists in the use of low-cost hardware solutions (Figure 32). With simpler hardware, GNSS stations require a more restricted post-processing, because of inferior number and quality of satellite observations.

This limits applications to static or semi-static monitoring, but with proper processing conditions and parameters, this type of hardware can achieve nevertheless accuracy up to a few millimeters in good conditions.

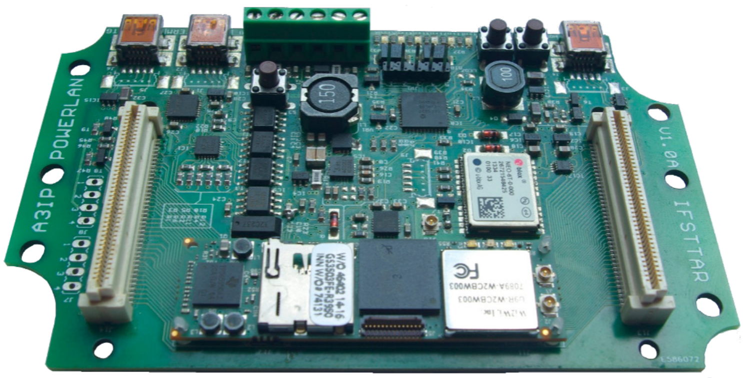


Figure 32. Pegase board, developed by Ifsttar team I4S and equipped with a low-cost GNSS receiver

C.2.6. MONITORING DYNAMIC DISPLACEMENT USING GROUND-BASED RADAR INTERFEROMETRY (GB-SAR)

Contributors: Nicolas Manzini, Vincent Baltazart (source: (Zhang, et al., 2018))

Radar (RAdio Detection And Ranging) is a detection system that uses radio waves to determine the range, angle, or velocity of objects (Figure 33). A radar antenna is installed next to a structure, and radar pictures of it are collected. Interferometry uses the phase of back propagated signals to construct a phase difference between two (or more) consecutive acquired images (e.g. (Monserrat, et al., 2014)).

While the number of complete wave cycles is unknown between two radar images, the detection of at least one stable point and the use of phase unwrapping algorithms allows for the construction of a relative displacement map. The displacements are obtained in the line of sight of the sensor. The use of microwave and high acquisition rates allows dynamic monitoring of structures, including vibrations-based applications (modal analysis).

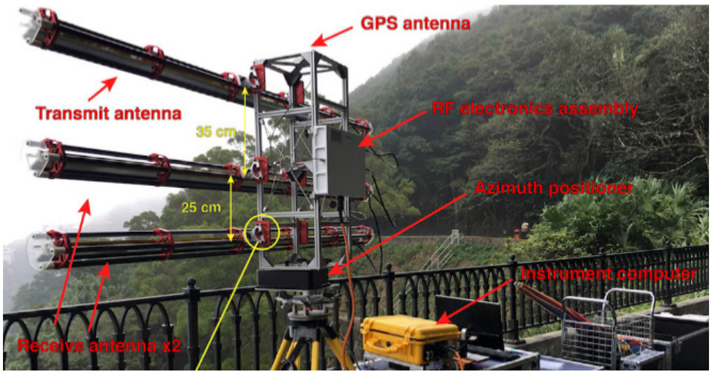


Figure 33. Left: GPRI microwave solution from (Zhang, et al., 2018); Right: HYDRA-G solution from IDS Georadar

Table 22. TRL scale for ground based interferometry on road bridges applications

Ranking	YES/NO	Explanations
1	YES	Radar interferometry allows dense dataset and high accuracy displacement maps
2	YES	On ground microwave radar interferometry can be used to monitor bridge dynamic displacements
3	YES	On ground microwave radar interferometry can be used to monitor bridge dynamic displacements
4	YES	Interferometry validated on lab experiments
5	YES	Interferometry validated on relevant environment
6	YES	Interferometry validated on relevant environment
7	YES	A prototype is demonstrated on two long span bridges with success
8	YES*	Commercial solutions are available, but may lack academic references
9	YES*	Commercial solutions are available, but may lack academic references

C.2.7. VIDEO CAMERA-BASED VIBRATION MEASUREMENTS

Contributor: Vincent Baltazart

Vision-based sensors have recently emerged for performing vibration analysis of simple structures. Selected pixels within the image are supposed acting as remote sensors and then substituting conventional in situ sensor gauges which usually provide the modal identification of the structure. The processing within the image is focused on the pixels attached to either some artificial targets (coded paper targets) or natural targets on the structure (e.g., edges). The grey level of those pixels are supposed to vary with time when mechanical vibrations occurs on the structure.

There exists different computer vision techniques to extract subpixel displacements for both target and target-less configurations or to amplify small imperceptible changes in the images, e.g., (Mas, et al., 2014), (Zhang, et al., 2016), (Chen, et al., 2016). Besides, specific processing technique, namely, the motion magnification in (Chen, et al., 2016), enables to display the deflection shapes of the different vibration modes within the field of view of the camera.

Table 23. TRL scale for video camera-based vibration measurements

Ranking	YES/NO	Explanations
1	YES	Video-based sensor allows dense dataset
2	YES	?
3	YES	Video-based sensor can be used to monitor vibrations of simple structures
4	YES	Vision-based sensor validated on lab experiments
5	YES	Vision-based sensor validated on simple structures
6	YES	Video-based sensor validated on relevant environment
7	NO	?
8	NO	Lack of comparison with other sensor data and between the different processing methods
9	NO	

C.2.8. DIGITIZED MEASUREMENT OF THE CRACKING INDEX

Contributor: Vincent Baltazart

The internal swelling of concrete is revealed by the cracking at the material surface. The manual survey method which has been used in France consists in estimating the average elongation of the material along a 1 m2 surface facing of concrete from the cumulative crack width measurements along four directions (Fasseu & Michel, 1997). To overcome some drawbacks of the manual method, a semi-automated technique has been developed to measure the cracking index from digital images with the help of devoted image processing tools (Moliard, et al., 2016).

Moreover, an Android application has been embedded on smartphone for supporting non-contact crack width measurement and then, substituting the pocket size crack-width comparator gauge which is conventionally used by the operators.

C.2.9. RELATIVE DEFORMATION MONITORING WITH PERSISTENT SCATTERER INTERFEROMETRIC SPACE-BORN SYNTHETIC APERATURE RADAR (PSINSAR)

Contributor: Nicolas Manzini, Vincent Baltazart (source: (Ferretti, et al., 2001))

Radar (RAdio Detection And Ranging) is a detection system that uses radio waves to determine the range, angle, or velocity of objects. Radar images are constantly acquired by space-born satellites, covering large surfaces at constant weekly or monthly intervals. Interferometry is used to compute phase differences between pictures, and the use of stable points and phase unwrapping algorithms allows for the construction of a relative displacement map.

These displacements are obtained in the line of sight of the satellite. Datasets are also corrected from the effects of relief, by using a reference interferogram or an artificial interferogram built with DEM data. Finally, using a pile of radar images coupled with selection of points with stable properties (Persistent Scatterers, (Ferretti, et al., 2001)) allows for an accuracy up to a few milliters for displacements monitoring over months/years, e.g., ((Crosetto, et al., 2016); (Ferretti, et al., 2001); (Ferretti, et al., 2005)).

Table 24. TRL scale for Persistent Scatterers InSAR on road bridges applications

Ranking	YES/NO	Explanation
1	YES	Differential InSAR techniques are used to monitor land deformation on large scales
2	YES	The use of Persistant Scatterers in a pile of differential interferograms is formulated as an efficient tool to produce smoother time series over urban areas
3	YES	Selection of PS is formulated, and confirmed the ability to extract linear and non-linear components
4	YES	Radar interferometry is validated with on-ground sensors
5	YES	Radar interferometry is validated with satellite missions
6	YES	PSInSAR performances are validated on seismic deformation applications
7	YES	PSInSAR performances are demonstrated with the monitoring of urban subsidence
8	YES	PSInSAR performances show coherent results with model of a structure and shows good correlation with temperature measurements
9	NO	PSInSAR lacks direct comparison with other traditional sensors, and is yet to be confirmed for prevention applications

C.2.10. USE OF UNMANNED AERIAL SYSTEMS (DRONES)

Contributors: Philippe Chrobocinski Nicolas Manzini (source: AEROBI project, Pierre Carreaud, (Khaloo, et al., 2018))

C.2.10.1. THE EUROPEAN PROJECT AEROBI

Drones can be considered in a broad sense i.e. Unmanned Aerial Systems, including autonomous and remotely piloted systems. One major goal is to have an easy access to infrastructures (especially high infrastructures that would necessitate scaffolding or specific instrumentation) for an efficient, reliable and secure (taking into account cybersecurity) collection, distribution (including wireless transmission) and automatic processing of data (on ground and on board). In recent years, there has been a great boost in robotic technologies, such as, unmanned aerial robotics with multiple-joint arms, intelligent control, sensing, computer vision and machine learning that can provide the required elements for aerial robotic in-depth inspection of bridges.

The AEROBI project (http://www.aerobi.eu) recently increased industry-academia cross-fertilisation by bringing together civil engineers and bridge operators with robotics experts from academia and system integrators from Industry. An innovative, integrated, low flying, robotic system with a specialised multi-joint arm was developed to scan concrete beams and piers in a bridge to detect defects from all types (cracks, swelling, spalling, etc.) and to measure cracks with an innovative ultrasonic sensor, estimate re-bars diameter reduction and also measure the deflection of the beams (Figure 34).

Figure 34. Bridge inspection with camera: UAV reaching the take-off waypoint (left) and the bridge entry approaching waypoint (right)

C.2.10.2. USE OF UAVS FOR VISUAL INSPECTION OF BUILDING (CRACKS, GENEREAL STATE) WITH PHOTOGRAPHY

Unmanned aerial vehicles (UAV) allow for easier operations in areas with difficult or dangerous access. UAVs can lift enough weight to allow high-definition cameras to be carried on-flight. Data acquired by such camera can be used for traditional visual inspection.

Table 25. TRL scale for UAV visual inspection on road bridges applications

Ranking	YES/NO	Explanations
1	YES	UAVs are able to carry enough payload for visual inspection
2	YES	UAV-mounted cameras require specific stabilization and may require processing for motion blur
3	YES	Hardware is available
4	YES	UAVs with cameras are functional in lab experiments
5	YES	UAVs with cameras are operational
6	YES	UAVs with cameras are operational for visual inspection of various buildings
7	YES	UAVs have been used for crack inspection on bridges
8	YES	Performance of UAV-based visual inspection for bridges has been confirmed with proper motion blur processing
9	YES*	Various UAV-based system are operational (academic reference missing)

C.2.10.3. UAV-BASED PHOTOGRAMMETRY (3D MODELLING)

This section illustrates the use of UAVs to collect photographic data in order to produce 3D model of a bridge's elements. Data acquired by UAV-mounted cameras can be used for photogrammetric applications. Photogrammetry uses high-definition images acquired from various angles in order to reconstruct 3D numerical models of a structures, allowing to perform measurements and obtain the exact positions of surface points.

Table 26. TRL scale for UAV photogrammetry on road bridges applications

Ranking	YES/NO	Explanations
1	YES	UAVs are able produce data with enough accuracy for visual inspection, and are able to fly close to structures, thus requiring less performant hardware for photogrammetry applications
2	YES	UAV-mounted cameras require stabilization devices. Position is estimated using INS.
3	YES	Hardware is available, 3D reconstitution can be achieved with light-weight cameras
4	YES	UAV-based photogrammetry has been achieved in off-site experiments
5	YES	UAV-based photogrammetry has been achieved in urban environment
6	YES	UAV-based photogrammetry is proven operational on man-made structures
7	YES	UAV-based images are used to compute 3D models of bridges successfully
8	YES	An operational system is evaluated in comparison with other UAV and non-UAV based acquisitions
9	YES*	Various UAV-based system are operational (academic reference missing)

C.2.10.4. USE OF UAV-MOUNTED LIDAR TO CREATE 3D MODELS

Unmanned aerial vehicles (UAV) allow for easier operations in areas with difficult or dangerous access. UAVs can lift enough weight to allow small Lidar sensors to be carried on-flight. Lidar (Light Detection And Ranging) measures distance to a target by illuminating the target with pulsed laser light and measuring the back-propagated reflected pulses. Differences in laser return times and wavelengths are used to generate a 3D numerical model of the target.

Table 27. TRL scale for UAV lidar on road bridges applications

Ranking	YES/NO	Explanation
1	YES	UAVs are able to carry small high-accuracy lidar
2	YES	UAVs are able to carry small high-accuracy lidar
3	YES	Hardware is available, 3D reconstitution can be achieved with light-weight lidar
4	YES	UAV-based lidar data has been used in lab experiment
5	YES	UAV-based lidar data can be used to generated 3D models of large structures
6	YES	UAV-based photogrammetry is proven operational on man-made structures
7	YES	UAV-based lidar data is used to compute 3D models of bridges successfully
8	YES	An operational system is evaluated in comparison with other UAV and non-UAV based acquisitions. Limitations in accuracy are pointed out
9	NO	Lack of academic reference for fully-operational applications

C.2.11. AUTOMATED PROCESSING DEDICATED TO ANALYZE LARGE DATASETS

C.2.11.1. STRUM CRACK DETECTION

Contributors: Nicolas Manzini (source: (Prasanna, et al., 2016))

The STRUM (spatially tuned robust multifeature) crack detection algorithm in (Prasanna, et al., 2016) is based on machine learning classification to eliminate the need for manually tuning threshold parameters. The algorithm uses robust curve fitting to spatially localize potential crack regions, independently from the presence of noise.

Table 28. TRL scale based on (Prasanna, et al., 2016)

Ranking	YES/NO	Explanations
1	YES	Classifier can be used to achieve cracks detection, avoiding user-dependant bias.
2	YES	A robust classifier is designed to automatically detect objects and recognize cracks in large sets of pictures.
3	YES	A robust classifier is designed to automatically detect objects and recognize cracks in large sets of pictures.
4	YES	Automated detection of bolts and cracks has been experimented on steel beams in lab conditions
5	YES	Successfully applied on bridge deck for concrete cracks
6	YES	Successfully applied on bridge deck for concrete cracks
7	YES	Prototype demonstrated on bridge deck for concrete cracks, with 95% accuracy
8	NO	
9	NO	

C.2.11.2. OPTICAL DIGITAL IMAGE CORRELATION FOR STRAIN AND DISPLACEMENT MONITORING

Contributors: Nicolas Manzini (source: (Nonis, et al., 2013))

Digital Image Correlation (DIC) is a technique that can be used for measuring displacement fields by tracking artificially applied random speckle patterns. 3-dimensional DIC is a non-contact, full field, optical measuring technique that uses digital cameras to measure surface geometry, displacement, and strain. DIC can be used for monitoring by imaging a bridge periodically and computing strain and displacement from images recorded at different dates or operating conditions.

Table 29. TRL scale based on (Nonis, et al., 2013)

Ranking	YES/NO	Explanations
1	YES	DIC can be used to measure displacement fields on a set of pictures
2	YES	DIC can be combined with photogrammetry: DIC is used as a the measurement tool and photogrammetry is used for scaling
3	YES	An experimental processing workflow is established
4	YES	Technique has been proved efficient to track the crack mouth opening displacement, and monitor fracture toughness on steel
5	YES	Technique was confirmed on concrete applications
6	YES	Technique was applied successfully on lower section of a bridge
7	NO	No operational application referenced with a refined prototype
8	NO	No operational application referenced with a refined prototype
9	NO	No operational application referenced with a refined prototype

C.2.12. WIRE BREAK DETECTION BY ACOUSTIC MONITORING

Contributors: Christophe Boulemar, Gilles Hovhanessian, Stephane Joye

The detection of wire breaks in cable structures such as pre-stressed concrete (internal or external), stay cable or suspension cable can highlight structural weaknesses to be addressed.

With an acoustic sensor (Figure 35), it is possible to record continuously the acoustic signal generated by a structure. In the case of a wire break a specific acoustic signature can be detected. Using a mesh of sensors, it is also possible to detect and locate single wire failures in structures even in noisy traffic environment. At the moment it is an efficient technique allowing continuous detection and localization of a single wire break in a non-destructive manner.

This technique has been applied for the monitoring of many cable structures through the world. The reliability of the method has been proven in some studies involving blind tests procedures and destructive test to confirm the results provided by the acoustic monitoring.

Table 30. TRL scale for wire break detection by acoustic monitoring.

Ranking	YES/NO	Explanation
1	YES	Acoustic sensors are a commonly used in acoustic emission NDT applications.
2	YES	Acoustic sensors allow to detect wire break acoustic emission signature.
3	YES	Events similar to wire breaks were detected in laboratory at early proof of concept stage.
4	YES	Single wire breaks were detected in laboratory
5	YES	Multiple tests have been performed including blind test
6	YES	Test ran in harsh and noisy environment confirmed the ability to detect wire breaks is noisy traffic environment
7	YES	Prototype test ran successfully over several structures
8	YES*	Several bridges in the world are currently continuously monitored with acoustic sensors. However, for some specific cases another type of sensors might be required
9	NO*	The technology is not supplied on the shelf, but as a custom solution tailored for each use case. At the moment it requires human data processing along with automated processing.

Acoustic Sensor

Acoustic sensor installed on pre-stressed concrete wall

Acoustic sensor installed on suspension cable clamp

Acoustic sensor installed on a stay cable lower anchorage

Figure 35. Acoustic monitoring.

C.2.13. SCOUR MONITORING SYSTEMS

Contributors: Franziska Schmidt, Christophe Chevalier

Scour is considered as one main cause of bridge damage and accounts for nearly half of all bridge collapses in the USA. In order to anticipate this risk, it is important to determine the actual scour depth at bridge supports, namely the piers and abutments. In literature, many empirical formulas are proposed to evaluate scour depth. However, most of them usually lead to an overestimation of its value. Many monitoring devices already exist and are used in the field such as: float-out , radar, sonar, time domain reflectometry, magnetic sliding collar, fiber optic and electrical conductivity devices.

Therefore, recent studies attempt to suggest more accurate and practical monitoring techniques to evaluate scour at bridge foundations. A new emergent technique is based on the dynamic response of the structure. Recent works have demonstrated the feasibility of indirect monitoring of scouring process, by sensing the dynamic behavior of bridge piles (Boujia, et al., 2017) (Florens, 2018) or rods installed in the riverbed (Boujia, et al., 2018) (Chevalier, et al., 2017).

C.2.13.1. MONITORING OF DYNAMIC BEHAVIOR OF PILE

(Prendergast, et al., 2013) studied the effect of scour on the dynamic response of a single pile. The experimental laboratory set-up consisted on a pile placed in a block of sand. Scour was simulated with the progressive extraction of a layer of the soil. For every scour depth, an impact was applied and the dynamic response of the pile recorded with an accelerometer placed on the top. The test showed that the first natural frequency decreases with the increase of the depth of the scour hole. The same experimental protocol was applied in situ to a 8.76m in length pile and showed the same results. To establish a relation between the first frequency and the scour depth, a spring-beam finite element model was developed and validated.

Unlike (Zarafshan, et al., 2012) who used the vibration response of the sensor to determine the stiffness of the springs k, (Prendergast, et al., 2013) used two geotechnical methods: the first one uses the small strain shear modulus determined with in-situ test with Multi-channel analysis of surface waves (MASW) or Cone Penetration Test (CPT) and the second one used the American Petroleum Institute design code (API). However the API method showed significant difference with experimental results and was therefore not recommended. Both studies show that the first frequency of piles decreases with the increase of scour depth. However, the correlation between the frequency and the scour depth is not direct and requires the use of both a numerical model and experimental data to calibrate the spring stiffness.

C.2.13.2 - DEVELOPEMENT OF A SCOURING SENSOR

(Zarafshan, et al., 2012) proposed a device to estimate scour depth based on the change of the fundamental frequency of a rod partially embedded in the riverbed. The rod is equipped with a fiber-optic Bragg grating sensor that uses the strain response history in the time domain to identify the fundamental frequency. In order to correlate the frequency to the scour depth, a numerical model was developed based on the Winkler model of the soil. Once the rod is placed in the soil, its frequency is used to calculate the stiffness of the springs used in the model. Then the model can be used to measure the first frequency for different scour depths.

This sensor has been studied in detail by (Boujia, et al., 2018), where the dynamical response has been studied analytically, numerically and experimentally by acceleration (accelerometers) and deformation (fiber optics).

C.2.14. WEIGH IN MOTION SYSTEMS (WIM)

Contributors: Franziska Schmidt, Gérard Baron

C.2.14.1. UPDATING REMAINING FATIGUE SERVICE LIFE WITH WIM INFORMATION

WIM is the process of measuring the dynamic tire forces of a moving vehicle and estimating the corresponding tire loads of the static vehicle (ASTM, 2009).

Using recorded traffic data makes it possible to infer the consequences of traffic regulation changes for example on pavement (changes in the fatigue and rutting), on geometrical constraints (geometry of the road and ability to drive on the existing infrastructure) and on bridge loading. In a recent study (Schmidt & Jacob, 2010), the impact on the bridge lifetimes of the increase of the limit of 40 t to 44 t for GVW of 5 axle-trucks has been investigated. Weigh-in-motion data have been used in order to obtain an accurate description of the actual traffic volumes and loads (Figure 36). Such traffic information has then been modified in order to account for the regulations changes. The very simple and crude assumptions done were that (i) the gross weight of every heavy vehicle in the range between 36 and 44 tons was increased by 10%, the additional weight being uniformly distributed on all axes; (ii) all the other vehicles were assigned the same load as before (volume limitation or not enough freight to carry).The effects on bridges were calculated by applying these traffic loads, either directly as measured by WIM systems or simulated, on influence lines or surfaces. The increase of the extreme stresses was limited to 6.5 or 8.5%, on a 40 m simple supported span (bending moment at mid span effect), carrying three traffic lanes. The reduction of the lifetime, whatever the fatigue resistance (S-N class) and the bridge, remained below 20%.

As part of the Federal Highway Administration's (FHWA) Long-Term Bridge Performance (LTBP) Program's Technical Assistance Contract, a literature review of the state of the practice was performed for WIM systems installed in pavements and on bridges (Al-Qadi, et al., 2016). The literature review outlines important topic areas related to WIM systems and the research performed by various agencies.

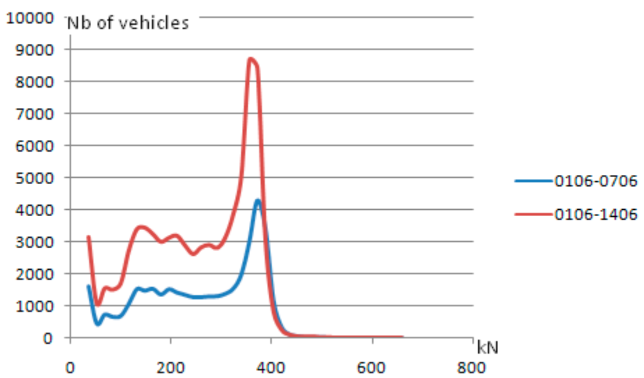


Figure 36. Gross weight distributions on the slow lane of the A9 motorway, 1 & 2 weeks in June 2009.

C.2.14.2. THE USE OF FIBRE OPTICAL STRANDS FOR WIM APPLICATIONS

This section describes a methodology gathering data concerning the measurement of live loads as well as the monitoring of the relevant effects on the structure itself. Within the context of a dramatic increase of the trucks cross border traffic and the number of overloaded convoys, bridge asset managers are more and more facing the need of accurate tools to measure the transiting loads on their networks.

Existing WIM (weigh in motion) systems are most of the time intrusive to road coating, costly and they must be removed during the pavement maintenance. The hereinafter technology, so called WIM+D, uses the bridge deck as a weighing platform (Tinawi, et al., 2017). Not intrusive to the pavement, it offers the possibility to be coupled with a SHM system, using the same sensors and a unique data acquisition unit. The OSMOS WIM+D application provides both weigh in motion measurements and strain monitoring with a single set of sensors and data acquisition unit. The SHM system measures strains on the bridge deck using long fibre optical strands (Figure 37). The sensors rely on micro wave technology based on light's continuous intensity modulation. The light attenuations are directly correlated to the deformation of the optical strand.

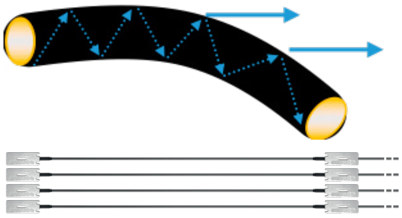


Figure 37. Optical strands

The system OSMOS WIM+D consists in installing optical strands under two consecutive spans to record the most significant strain variations due to the bending under the effect of the live loads. The data are processed with proprietary algorithms and the information given are: gross weight, speed, direction, maximal strain in the main elements, length of the truck, number of axles, axles distance, axle load for each axle. As mentioned, in addition to the weighing system the OSMOS WIM+D provides, with the same Optical strands technology, an efficient structural monitoring (Figure 38, Figure 39).

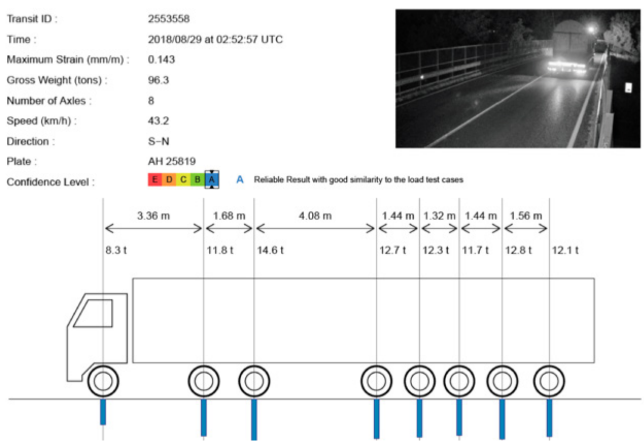


Figure 38. Passage data sheet

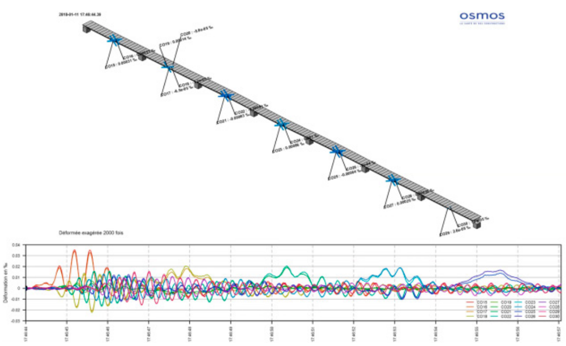


Figure 39. Strain monitoring on a prestressed bridge.

D. CONCLUSIONS

Bridge owner's main objective is the minimization of consequences, should they be societal, environmental or economic, caused by inadequate functioning or by unexpected structural failure, of a component, a system or an equipment. Issues are numerous and their difficulty is not least. Nevertheless, many opportunities should help meet up these challenges and sharing the knowledge on recent research developments was one goal in this report.

Also, digitalisation, which evolution is progressing rapidly, already presents promising research leads. The Internet of Things, mobility "as a service", artificial intelligence and autonomous vehicles are already present on infrastructures. These technologies will help develop both innovative and new solutions for bridges managers. We have never had so much data available. The means to acquire these data should, whether using automatic or semi-automatic tools, make it possible to improve their reliability and to diversify the type of data collected (thanks to new sensors). Big data and machine learning tools will also enable improving the use of data, to establish behavioural models of bridges.

Ultimately, advanced, integrated, cost-effective and reliable instrumentation solutions, techniques and concepts should be developed to provide data that can be used to compute reliable performance indicators enabling optimal management decisions.

This report summarizes the collaborative work carried out under the lead of the Innovation Subgroup of the COST TU Action 1406. The main goal of the work is to collect, describe and rate, based on their maturity level, some recent innovative scientific and technological developments in link with the Research Performance Indicators (RPIs) that is Performance Indicators that are still the object of scientific research and therefore not yet eligible as operational Indicators for quality checks of bridges. Developments of the technologies that enable to collect the observation needed to compute the Indicators have been documented as well in the report.

A survey was carried out in the network of COST TU1406 network to gather both Indicators and Technologies that are being currently investigated by researchers within and in connection with the network

A scale, defined Indicator Readiness Level (IRL), has been proposed to rank RPIs. The IRL rates these indicators based on their level of maturity in terms of their "proved" application demonstrated through published scientific results or real-world applications. This scale is meant to serve as a supporting tool for a twofold aim:

- to check the eligibility of a performance indicator for quality check and related decision making on roadway bridges based on its maturity,
- to select research needs on performance indicators that is to underpin the indicators on which more research is needed in order to bring them to the level of full applicability for quality checks.

Examples of the application of the IRL to research performance indicators have been discussed in this report and in a number of papers presented at international conferences (Limongelli & Orcesi, 2017) (Limongelli, et al., 2018) (Limongelli, et al., 2018a).

Innovative technological developments have been also detailed herein, with a focus on non-destructive testing tools and structural health monitoring solutions. Some results have been detailed at international conference (Orcesi, et al., 2019).

Information and results reported in this volume have the scope to describe the current state of Research on Performance Indicators and Monitoring Technologies based on the knowledge related to the COST TU1406 network. Due to the nature of the RPIs and of the Technologies described- that are the outcomes of research investigations - and due to way the database was gathered -based on the connections within and with the COST TU1406 and on the some expert opinion within the Action – they have to be considered dynamic processes that need to be updated constantly based on new research outcomes.

The COST Action TU1406 database and the IRL rating of the RPIs will be provided in excel format as a deliverable and will constitute a base for further integrations and updates.

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